**GEOTECHNICAL REVIEW/UPDATE
AND PRELIMINARY GEOTECHNICAL EVALUATION
±15-ACRE SITE, APN'S 338-150-029 AND 031
SUN CITY, RIVERSIDE COUNTY, CALIFORNIA**

FOR

**THE WOMBLE GROUP
P.O. BOX 3609
SEAL BEACH, CALIFORNIA 90740**

W.O. 5431-A-SC MAY 9, 2007

GeoSoils, Inc.

Geotechnical • Coastal • Geologic • Environmental

26590 Madison Avenue • Murrieta, California 92562 • (951) 677-9651 • FAX (951) 677-9301

May 9, 2007

W.O. 5431-A-SC

The Womble Group
P.O. Box 3609
Seal Beach, California 90740

Attention: Mr. Andrew Yim

Subject: Geotechnical Review/Update and Preliminary Geotechnical Evaluation, ±15-Acre Site, APN's 338-150-029 and 031, Sun City, Riverside County, California.

Dear Mr. Yim:

In accordance with your request and authorization, this report presents the results of GeoSoils, Inc.'s (GSI) geotechnical update review and preliminary geotechnical evaluation of the subject property. The purpose of the study was to evaluate the onsite soils and geologic conditions and their effects on the proposed site development from a geotechnical viewpoint.

EXECUTIVE SUMMARY

Based on our review of the available data (see Appendix A), as well as field exploration, laboratory testing, and geologic and engineering analysis, the proposed development of the property appears to be feasible from a geotechnical viewpoint, provided the recommendations presented in the text of this report are properly incorporated into design and construction of the project. The most significant elements of this study are summarized below:

- In general, the site may be characterized as being underlain by Pleistocene-age fan deposits, as mapped by Morton, et al. (2003). The fan deposits are slightly dissected, indurated, and are thus not generally susceptible to liquefaction. These units are mantled by ±2 to 3 feet of potentially compressible topsoil/alluvium. Removal and recompaction of the topsoil/alluvium, and weathered near-surface Pleistocene fan deposits will be required should settlement-sensitive improvements be proposed within their influence. For preliminary planning purposes, removal depths are estimated to generally range from ±3 to ±4 feet across the site, with localized deeper removals possible, if not removed by planned excavation. Actual depths of removals will be evaluated in the field during grading by the geotechnical consultant.

- It should be noted, that the Uniform Building Code/California Building Code ([UBC/CBC], International Conference of Building Officials [ICBO], 1997 and 2001), indicates that removals of unsuitable soils be performed across all areas to be graded, not just within the influence of the residential structure. Relatively deep removals may also necessitate a special zone of consideration, on perimeter/confining areas. This zone would be approximately equal to the depth of removals, if removals cannot be performed offsite. Thus, any settlement-sensitive improvements (perimeter walls, curbs, flatwork, etc.), constructed within this zone may require deepened foundations, reinforcement, etc., or will retain some potential for settlement and associated distress. This will require proper disclosure to all homeowners and any homeowners association, and all interested/affected parties, if complete removals cannot be preformed due to perimeter/confining conditions.

- According to the Riverside County Land Information System, the site has been indicated to lie mostly within a moderate liquefaction potential zone with a low liquefaction potential zone indicated in the extreme southwest corner of the site. Based on our research, regional groundwater at the site ranges from ±65 feet to ±68 feet below the ground surface. Perched groundwater, however, was encountered during our site exploration at a depth of ±39 feet. Earth materials underlying the site are generally medium dense to dense/very stiff where encountered. Such materials are <u>not</u> generally prone to liquefaction. Due to their nature, the beds and lenses are typically interfingered and discontinuous, both horizontally and vertically, within the site area. Thus, as part of our liquefaction screening analysis, GSI has concluded that liquefaction potential does not constitute a significant risk to site development, provided our recommendations are properly implemented.

- Our review of site conditions indicates regional seismic shaking, ranging from moderate to moderately high, may occur on the site associated with nearby and/or regional faults. Accordingly, the proposed structures and foundations should be designed to resist seismic forces in accordance with the criteria contained in the UBC/CBC (ICBO, 1997 and 2001), for Seismic Zone 4. Based on our site specific seismic hazard analysis, seismic design parameters are provided herein.

- We assume that planned fill depths are proposed to be less than about 10 feet after remedial removals. Based on the maximum fill depth and our settlement evaluation, the footings and/or slabs should be preliminarily designed to accommodate a differential settlement of 1 inch (i.e., at least 1 inch in a 40-foot span), in accordance with the structural engineer. Post-construction settlement of the fill should be mitigated by conventional or post-tension design, provided the design parameters presented herein are properly utilized in design of foundation systems by the structural engineer/slab designer. In addition to the above, the structural engineer/slab designer should also consider estimated settlements due to short duration seismic loading and applicable load combinations, as required by the County and/or the UBC/CBC (ICBO, 1997 and 2001). An updated settlement analysis should be performed during the 40-scale plan review phase.

- Cut and fill slopes are assumed at gradients of 2:1 (horizontal to vertical [h:v]), or flatter, up to about ±10 feet in height. Significant height cut slopes are not anticipated. Based on our field mapping, and our evaluation, the anticipated 2:1 cut and fill slopes are generally considered grossly and surficially stable; however, this should be further evaluated at the 40-scale grading plan stage and during future grading. Keyways for fills over cut, and daylight cuts, should be properly constructed, as depicted in the General Earthwork and Grading Guidelines section of this report (Appendix E). If adverse geologic conditions are encountered during grading, stabilization fills may be recommended.

- Based on our laboratory testing, and for preliminary planning purposes, the expansion potential of the onsite soils generally have a low to medium Expansion Index ([E.I.] from 21 to 90). However, high (E.I. from 91 to 130) expansive soils may not be precluded from occurring onsite. Foundations should be designed in accordance with Section 1815 and/or Section 1816 of the UBC/CBC (ICBO, 1997 and 2001). Based on the general parameters outlined above, updated preliminary foundation recommendations for conventional and post-tension design are provided in this report. Additional E.I. and plasticity testing should be conducted during, or shortly after, site grading to further evaluate the preliminary test results obtained. The E.I. testing frequency for as-graded conditions should be evaluated in the field based on the conditions exposed during site grading. One E.I. test per three lots is generally considered the industry standard, with 258 lots anticipated.

- Representative samples of onsite materials were collected for soluble sulfate testing. Sulfate testing results indicate a sulfate content of 0.0053 percentage by weight. This result indicates that site soils are in the negligible range for sulfate exposure per the UBC (ICBO, 1997). Based upon the soluble sulfate test results, sulfate-resistant, concrete is not required; however, higher sulfate contents may exist onsite. In addition, pH and resistivity (saturated) testing was performed. Testing results indicate a pH of 8.0, which is moderately alkaline, and a saturated resistivity of 2,760 ohm-centimeters. This would generally be considered moderately corrosive to ferrous metals (between 2,000 to 10,000 ohm-cm is considered moderately corrosive). It is our understanding that standard concrete cover over reinforcing steel is usually appropriate for these conditions; however, consulting a qualified corrosion engineer is recommended to provide specific recommendations for foundations and piping, etc. The sulfate/corrosion testing frequency for as-graded conditions should be evaluated in the field based on the conditions exposed during site grading, generally one sulfate/corrosion test per three lots is generally considered the industry standard, with 258 units anticipated.

- Groundwater was encountered in our exploratory boring B-3, perched at a depth of ±39 feet below the ground surface. Based on historic water well data acquired (California Department of Water Resources [CDWR], 2005), regional groundwater levels in other nearby wells were previously measured at depths between ±65 feet to ±68 feet below the ground surface. Perched groundwater onsite may occur in the Pleistocene fan deposits, or along jointing and/or fractures due to migration from adjacent drainage areas and developments during and/or after periods of above normal or heavy precipitation or irrigation. Thus, perched groundwater conditions exist now, may occur in the future after development, and should be anticipated. This potential increases on cut lots in dense native materials. Owing to the general lack of suitable cover (i.e., less than 10 feet of fill), lack of a suitable flowline gradient in removal bottoms, and lack of a suitable outlet, subdrains are generally not anticipated (based on the available data), but nonetheless may not be precluded. The need for, and locations of subdrainage systems should be evaluated during site grading as subsurface conditions are exposed. This potential for seepage and/or perched groundwater to occur after site development, should be disclosed to all homeowners and any homeowners association, and all interested/affected parties. Should manifestations of perched conditions (i.e., seepage) develop in the future, GSI could assess the conditions and provide mitigative recommendations, as necessary.

- Our review indicates no known active faults are crossing the site area, and the site is not within an Alquist-Priolo Earthquake Fault Zone.

- Adverse geologic features that would preclude project feasibility were not encountered.

- The recommendations presented in this report should be incorporated into the design and construction considerations of the project.

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to contact the undersigned.

Respectfully submitted,

GeoSoils, Inc.

John P. Franklin
Engineering Geologist, CEG 1340

David W. Skelly
Civil Engineer, RCE 47857

Daniel P. Walsh
Engineering Geologist, CEG 2413

DPW/DWS/JPF/jk/ps

Distribution: (6) Addressee

TABLE OF CONTENTS

SCOPE OF SERVICES 1

SITE LOCATION/CONDITIONS 1

PROPOSED DEVELOPMENT 3

BACKGROUND/PREVIOUS STUDIES 3

FIELD STUDIES 3

GEOLOGY 3
- Local Geology and Site Earth Materials 5
  - Artificial Fill - Undocumented (Map Symbol - Afu) 5
  - Topsoil/Alluvium (Unmapped) 5
  - Pleistocene-age Fan Deposits (Map Symbol - Qof) 5

FAULTING AND REGIONAL SEISMICITY 6
- Seismic Shaking Parameters 8

GROUNDWATER 9

OTHER GEOLOGIC HAZARDS 9
- Liquefaction 9
- Subsidence 11
- Hydrocollapse 12
- Mass Wasting 12

LABORATORY TESTING 13
- General 13
- Moisture-Density 13
- Laboratory Standard 13
- Expansion Potential 13
- Atterberg Limits 14
- Consolidation Testing 14
- Corrosion/Sulfate Testing 14
- R-value 14

EMBANKMENT FACTORS 15

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS 15
- General 15
- General Grading 17
- Demolition/Grubbing 17
- Treatment of Existing Ground 18
- Fill Placement 18
- Transition Areas/Overexcavation 19

SUBDRAINS 19

SLOPE CONSIDERATIONS AND SLOPE DESIGN 19

PRELIMINARY FOUNDATION DESIGN 20
Bearing Value 20
Lateral Pressure 20

FOUNDATION CONSTRUCTION 21
Expansion Index - Very Low to Low (E.I. 0 to 50, and P.I. <15) 21

POST-TENSIONED SLAB SYSTEMS 22

SOIL MOISTURE CONSIDERATIONS 24

PRELIMINARY FOUNDATION SETTLEMENTS 26

PRELIMINARY WALL DESIGN PARAMETERS CONSIDERING EXPANSIVE SOILS . . 26
Conventional Retaining Walls 26
Restrained Walls 26
Cantilevered Walls 27
Retaining Wall Backfill and Drainage 27
Wall/Retaining Wall Footing Transitions 31

TOP-OF-SLOPE WALLS/FENCES/IMPROVEMENTS AND EXPANSIVE SOILS 31
Expansive Soils and Slope Creep 31
Top of Slope Walls/Fences 32

EXPANSIVE SOILS, DRIVEWAY, FLATWORK, AND OTHER IMPROVEMENTS 33

PRELIMINARY PAVEMENT DESIGN 35

PRELIMINARY PAVEMENT GRADING RECOMMENDATIONS 36
General 36
Subgrade 36
Aggregate Base Rock 36
Paving 36
Drainage 37
Additional Considerations 37

DEVELOPMENT CRITERIA 37
Slope Deformation 37
Slope Maintenance and Planting 38
Drainage 38

Toe of Slope Drains/Toe Drains 39
Erosion Control 40
Landscape Maintenance 40
Gutters and Downspouts 43
Subsurface and Surface Water 43
Site Improvements 43
Tile Flooring 43
Additional Grading 44
Footing Trench Excavation 44
Trenching/Temporary Construction Backcuts 44
Utility Trench Backfill 44

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING 45

OTHER DESIGN PROFESSIONALS/CONSULTANTS 46

PLAN REVIEW 47

LIMITATIONS 47

FIGURES:
Figure 1 - Site Location Map 2
Figure 2 - Excavation Location Map 4
Figure 3 - California Fault Map 7
Detail 1 - Typical Retaining Wall Backfill and Drainage Detail 28
Detail 2 - Retaining Wall Backfill and Subdrain Detail Geotextile Drain 29
Detail 3 - Retaining Wall and Subdrain Detail Clean Sand Backfill 30
Detail 4 - Schematic Toe Drain Detail 41
Detail 5 - Subdrain Along Retaining Wall Detail 42

ATTACHMENTS:
Appendix A - References . . . . Rear of Text
Appendix B - Boring Logs . . . . Rear of Text
Appendix C - Laboratory Data . . . . Rear of Text
Appendix D - EQFAULT, EQSEARCH, and FRISKSP . . . . Rear of Text
Appendix E - General Earthwork and Grading Guidelines . . . . Rear of Text

GEOTECHNICAL REVIEW/UPDATE
AND PRELIMINARY GEOTECHNICAL EVALUATION
±15-ACRE SITE, APN'S 338-150-029 AND 031
SUN CITY, RIVERSIDE COUNTY, CALIFORNIA

SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of available geologic maps, data, and previous soils and geologic reports, including aerial photography, for the site area (Appendix A).

2. Geologic and geomorphic site reconnaissance and mapping.

3. Performance of a photo-lineament analysis for the site and vicinity.

4. Subsurface exploration, consisting of five hollow-stem auger borings, to evaluate the near-surface and deeper soils as well as geologic and hydrogeologic conditions (Appendix B).

5. Laboratory testing of representative soil samples collected during our subsurface exploration program (see Appendix C).

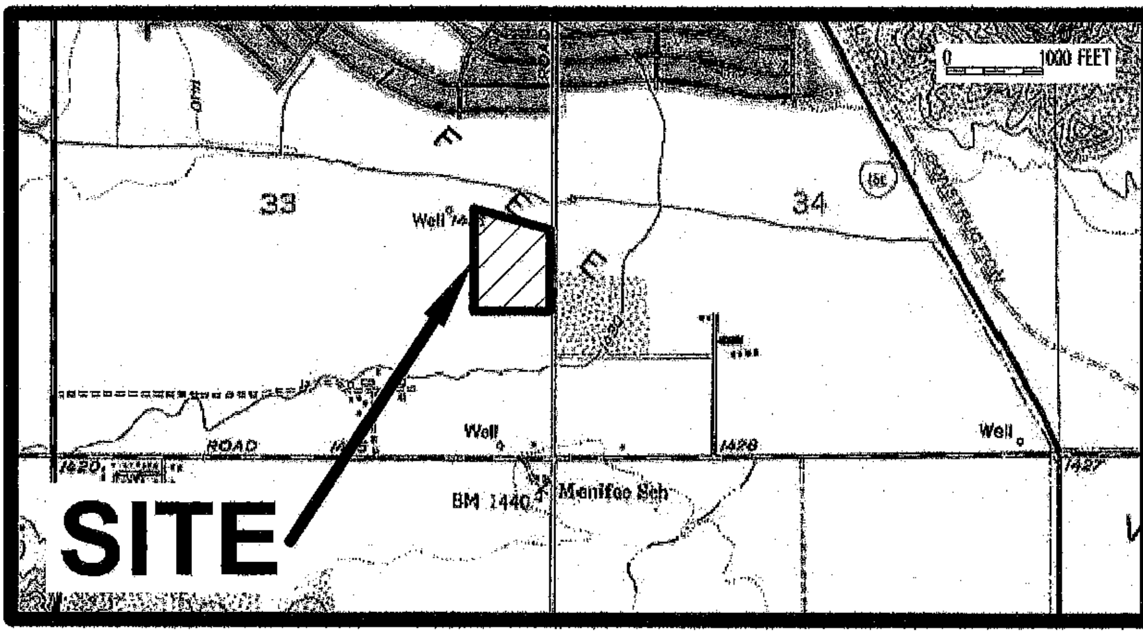
6. Site specific seismic hazard analysis (see Appendix D).

7. Appropriate engineering and geologic analysis of data collected.

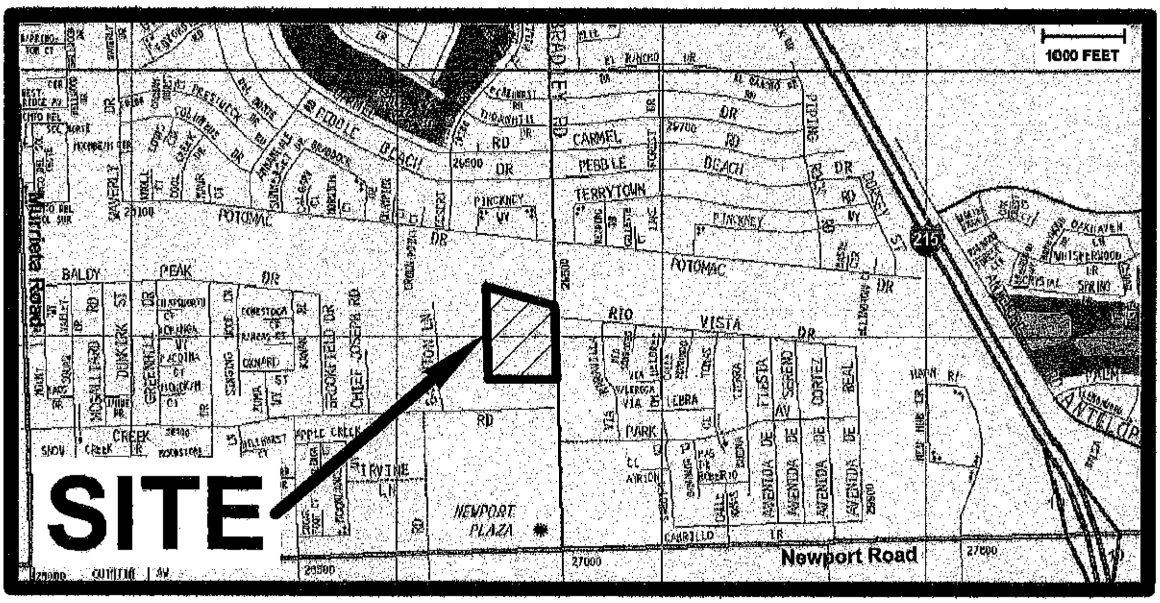
8. Preparation of this geologic/geotechnical report, and attachments, including test pit logs, settlement evaluation, regional seismic data, preliminary pavement design, preliminary foundation design, general earthwork factors, and recommendations for site grading.

SITE LOCATION/CONDITIONS

The nearly rectangular-shaped property consists of vacant, undeveloped land to the west of Bradley Road, and south and east of the Salt Creek Flood Control Channels in Sun City, Riverside County, California. Topographically, the site is relatively flat-lying, generally ranging in elevation between ±1,422 feet Mean Sea Level (MSL) to ±1,417 feet MSL. Drainage across the site is primarily to the north toward Salt Creek by sheetflow (see Figure 1, Site Location Map).



Base Map: TOPO!® ©2003 National Geographic, U.S.G.S Romoland Quadrangle, California – Riverside Co., 7.5 Minute, dated 1976, current, 1979.



Base Map: The Thomas Guide, Riverside County, Street Guide and Directory, 2005 Edition, by Thomas Bros. Maps, page 883.

N

GeoSoils, Inc.	W.O. 5431-A-SC
SITE LOCATION MAP	
Figure 1	

PROPOSED DEVELOPMENT

According to the preliminary site plans (Morris Designs, undated), rough grading will create building pad areas for 258 residential units. Maximum cut and fill depths, not including remedial grading, are not anticipated to exceed ±10 feet. Cut slopes and proposed fill slopes are assumed not to exceed ±10 feet in height, designed at inclinations of 2:1 (horizontal:vertical [h:v]). As mentioned above, it is our understanding that typical cut and fill grading techniques would be utilized to prepare the site for construction of 258 senior residences, and associated infra-structure improvements. It is also our understanding that the buildings would be one- and/or two-story structures, utilizing typical wood-frame construction with slabs-on-grade. Building loads are assumed to be typical for this type of relatively light construction. It is assumed that sewage disposal would be accommodated by tying into the regional system. The need for import of fill soils is unknown at this time.

BACKGROUND/PREVIOUS STUDIES

Previous geotechnical site work was completed in 2004, in the northernmost ±10-acres of the site, by Zeiser Kling Consultants, Inc (ZKCI). Their findings, conclusions, and recommendations were presented in a geotechnical report dated March 29, 2004 (ZKCI, 2004). ZKCI (2004) provided a liquefaction analysis and conservatively and reasonably concluded that the sediments underlying the site were not susceptible to liquefaction.

FIELD STUDIES

Field investigations conducted during our evaluation of the property for this study consisted of geologic reconnaissance mapping, and performance of five hollow-stem auger borings for evaluation of the near-surface soil, and geologic conditions of the site. The test pits were logged by a geologist from our firm. Representative bulk and in-place samples were taken for appropriate laboratory testing. Logs of the borings are presented in Appendix B. The approximate locations of the borings are presented on Figure 2, Excavation Location Map.

GEOLOGY

The property is situated in Menifee Valley within the Perris Block of the Peninsular Range province of California. The Peninsular Ranges province is characterized by steep, elongated ranges and valleys that trend northwesterly. This province is typified by plutonic and metamorphic rocks (bedrock) which comprise the majority of the mountain masses, with relatively thin volcanic and sedimentary deposits discontinuously overlying the

LEGEND

Artificial fill - undocumented

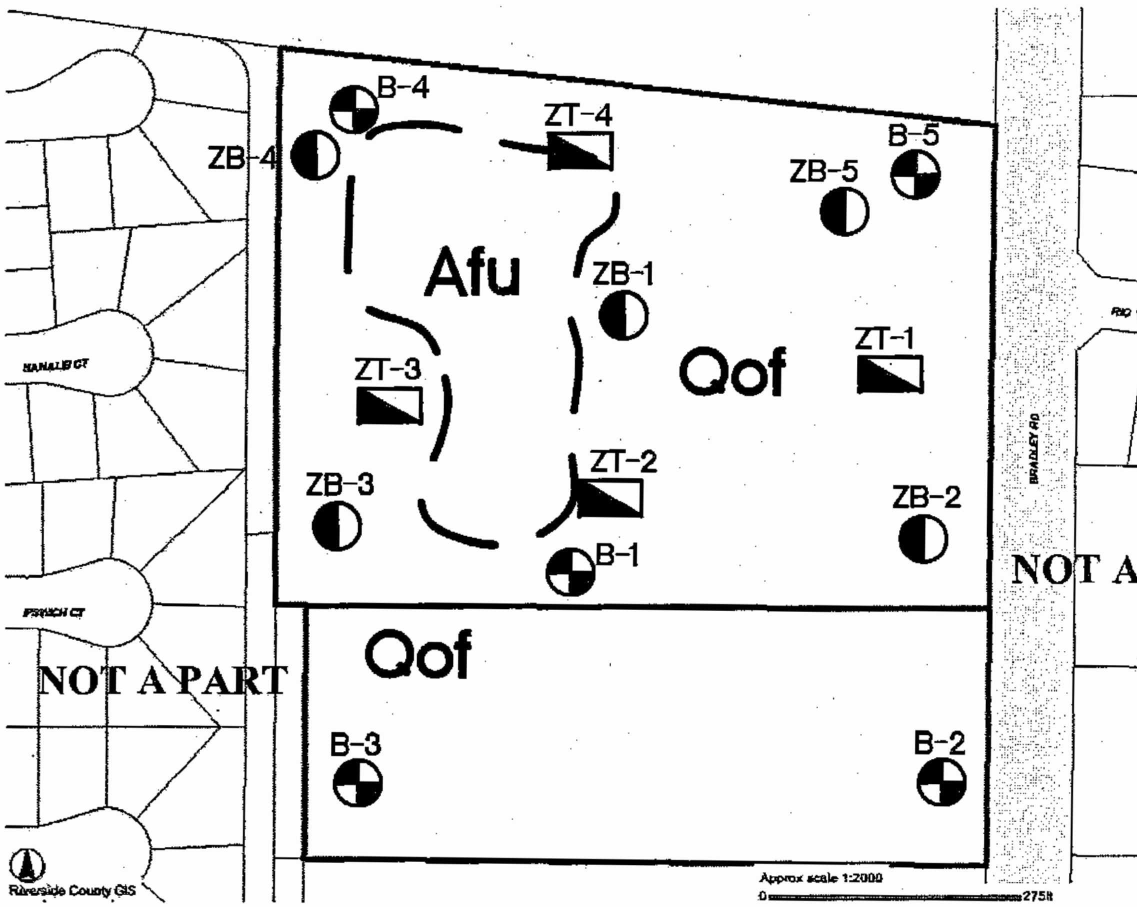
Quaternary Pleistocene-age fan deposits

Approximate location of geologic contact

Approximate location of exploratory boring (this study)

Approximate location of exploratory boring (ZKCI, 2004)

Approximate location of exxploratory test pit (ZKCI, 2004)



ALL LOCATIONS ARE APPROXIM

ap From Riverside County
formation System, May 1, 2007

N

GeoSoils, Inc.

RIVERSI
ORANC
SAN DIEC

EXCAVATION LOCAT MAP

W.O. 5431-A-SC	DATE 05/07	SCALE

bedrock, and with early to middle Pleistocene fan deposits filling in the valleys and younger alluvium filling in the incised drainages. The fan deposits are derived from the water borne deposition of the products of weathering and erosion of the bedrock.

Local Geology and Site Earth Materials

As mapped by Morton, et al. (2003), the site is underlain by relatively flat-lying, Pleistocene fan deposits. As encountered within the project site during our exploration, a ±2- to ±3-foot layer of topsoil/alluvium mantles the site. Mappable geologic units are shown on Figure 2. These units are described as follows, from youngest to oldest:

Artificial Fill - Undocumented (Map Symbol - Afu)

The undocumented fill, locally ±2 to ±4 feet in thickness (stockpiles), was encountered in the greater northwest quadrant the site. Abundant trash and debris was observed within the stockpiles, including concrete and metal. Due to the potentially compressible nature of these soils/materials, they are considered unsuitable for support of structures and/or improvements in their existing state. Clean fill materials may be reused for compacted fills provided the trash and debris have been removed from the site, and they have been approved by the geotechnical engineer prior to fill placement. Any remaining trash, debris, and other deleterious materials, will need to be removed, prior to site grading.

Topsoil/Alluvium (Unmapped)

The site is mantled by a relatively thin layer of topsoil/colluvium. Thicknesses of ±3 to ±4 feet was encountered at all exploratory test pit locations, but may vary locally across the site. The topsoil/colluvium was generally observed to be yellowish brown, dry to moist, loose to medium dense, silty sand. These surficial soils contained locally abundant roots and rootlets. In general, and based on our laboratory testing and observations, these materials typically have a very low to low expansion potential. Due to the potentially compressible nature of these surficial soils, they are considered unsuitable for support of structures and/or improvements in their existing state. Therefore, these soils will be need to be removed and recompacted, if not removed during planned excavation, should settlement sensitive improvements be proposed within their influence.

Pleistocene-age Fan Deposits (Map Symbol - Qof)

Pleistocene-age fan deposits were encountered underlying the topsoil/alluvium on the majority of the site. These sediments were generally yellowish brown, silty sand, and some occasional gravels. These deposits were generally dry to moist, well-dissected, well-indurated, and generally medium dense to dense where encountered. Based on our laboratory testing, these deposits typically have a low to medium expansion potential; however, highly expansive soils may not be entirely precluded. The near-surface weathered fan deposits will require some removal and recompaction, should settlement sensitive improvements be proposed within their influence.

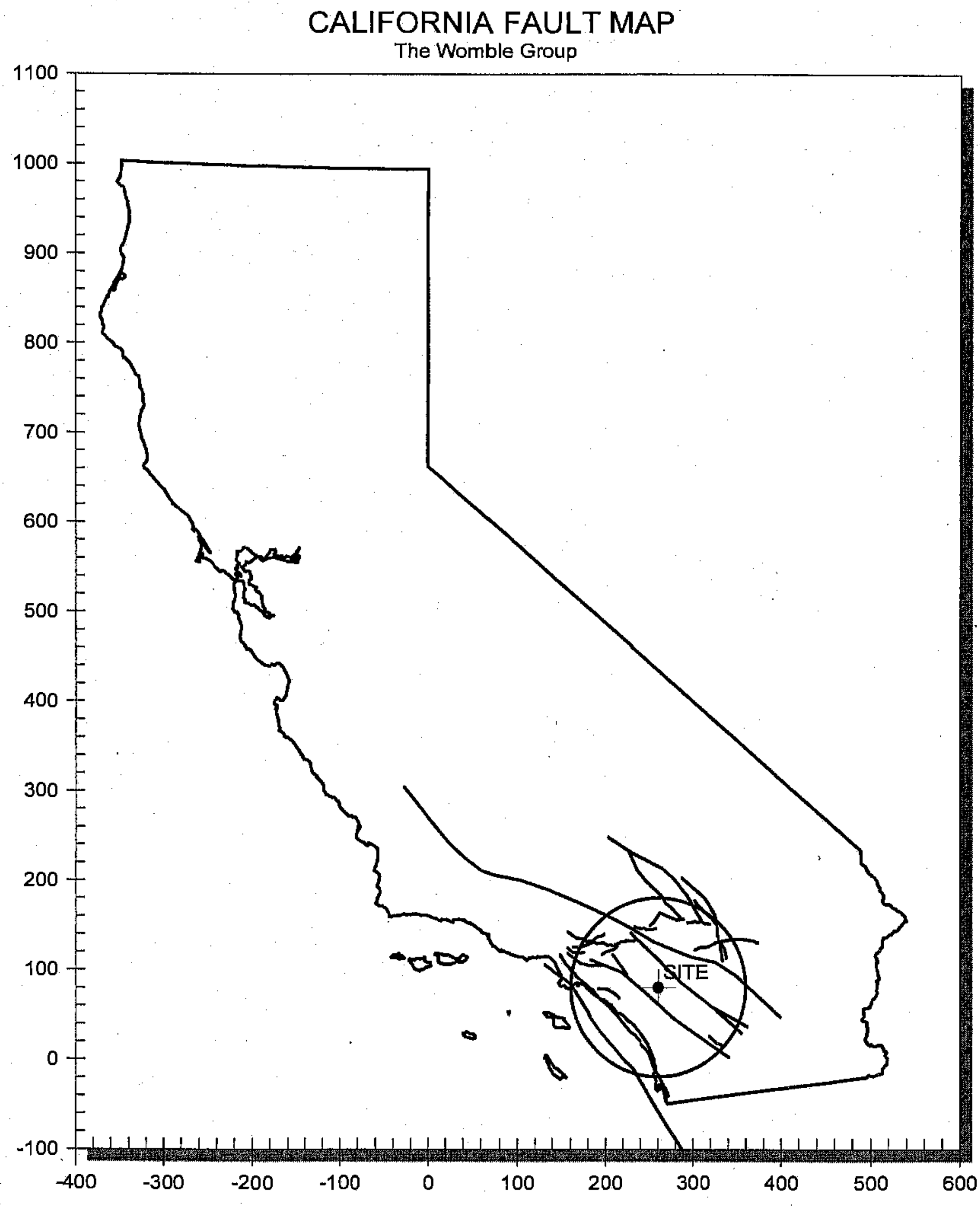
FAULTING AND REGIONAL SEISMICITY

The site is situated in an area of active as well as potentially-active faults. The nearby San Jacinto and Elsinore fault zones are considered active and are included within an Alquist-Priolo Earthquake Fault Zone. Our review indicates that there are no known active faults crossing the site, and the site is not within a Fault-Rupture Hazard Zone (Hart and Bryant, 1997). During our review of aerial photographs (USDA, 1980), we did not observe photolineaments or other features specifically indicative of faulting crossing the site.

The following table lists the major faults and fault zones in southern California that could have a significant effect on the site should they experience activity.

ABBREVIATED FAULT NAME	APPROX. DISTANCE MILES (KM)	ABBREVIATED FAULT NAME	APPROX. DISTANCE MILES (KM)
Burnt Mountain	49.0 (78.8)	North Frontal Fault Zone (West)	37.8 (60.9)
Chino - Central Avenue (Elsinore)	24.2 (39.0)	Palos Verdes	51.5 (82.9)
Clamshell - Sawpit	55.4 (89.1)	Pinto Mountain	37.0 (59.5)
Cleghorn	40.6 (65.3)	Puente Hills Blind Thrust	42.4 (68.2)
Coronado Bank	51.3 (82.5)	Raymond	56.8 (91.4)
Cucamonga	38.7 (62.3)	Rose Canyon	41.5 (66.8)
Earthquake Valley	49.6 (79.9)	San Andreas - 1857 Rupture	45.0 (72.5)
Elsinore - Glen Ivy	9.8 (15.8)	San Andreas - Coachella	44.4 (71.5)
Elsinore - Julian	24.0 (38.6)	San Andreas - San Bernardino	28.4 (45.7)
Elsinore - Temecula	8.8 (14.2)	San Jacinto - Anza	16.2 (26.0)
Eureka Peak	52.2 (84.0)	San Jacinto - Coyote Creek	42.4 (68.3)
Helendale/South Lockhardt	50.5 (81.3)	San Jacinto - San Bernardino	22.8 (36.7)
Johnson Valley (Northern)	60.2 (96.9)	San Jacinto - San Jacinto Valley	13.9 (22.3)
Landers	54.7 (88.0)	San Joaquin Hills	29.9 (48.1)
Lenwood/Lockhart/Old Wmn. Spgs.	55.9 (89.9)	San Jose	41.6 (66.9)
Newport - Inglewood (L.A. Basin)	42.7 (68.7)	Sierra Madre	44.4 (71.4)
Newport - Inglewood (Offshore)	34.9 (56.1)	Upper Ellysian Park Blind Thrust	58.5 (94.1)
North Frontal Fault Zone (East)	43.4 (69.8)	Whittier	28.1 (45.3)

The possibility of ground shaking at the site may be considered similar to the southern California region as a whole. The relationship of the site location to these major mapped faults is indicated on the California Fault Map (see Figure 3). Our field observations and review of readily available geologic data indicate that known active faults do not cross the site.



CALIFORNIA FAULT MAP
The Womble Group
1100
1000
900
800
700
600
500
400
300
200
100
0
-100
-400
-300
-200
-100
0
100
200
300
400
500
600
SITE

Figure 3

The acceleration-attenuation relations of Bozorgnia, Campbell, and Niazi (1999), and Campbell and Bozorgnia (1994 and 1997), have been incorporated into EQFAULT (Blake, 2000a). For this study, peak horizontal ground accelerations anticipated at the site were determined based on the random mean and random mean plus 1 - sigma attenuation curves developed by those authors. These acceleration-attenuation relations have been incorporated in EQFAULT, a computer program by Thomas F. Blake (2000a), which performs deterministic seismic hazard analyses using digitized California faults as earthquake sources. The program estimates the closest distance between each fault and a user-specified file. If a fault is found to be within a user-selected radius, the program estimates peak horizontal ground acceleration that may occur at the site from the upper bound ("maximum credible") earthquake on that fault. Site acceleration (g) is computed by user-selected acceleration-attenuation relations that are contained in EQFAULT. Based on the above, peak horizontal ground accelerations from an upper bound event may be on the order of 0.25g to 0.42g.

Historical site seismicity was evaluated with the acceleration-attenuation relations of Bozorgnia, Campbell, and Niazi (1999), and the computer program EQSEARCH (Blake, 2000b, updated to June 2006). This program performs a search of historical earthquake records for magnitude 4.0 to 9.0 seismic events within a 100-mile radius, between the years 1800 through December 2006. Based on the selected acceleration-attenuation relationship, a peak horizontal ground acceleration is estimated, which may have effected the site during the specific event listed. Based on the available data and the attenuation relationship used, the estimated maximum repeatable peak site acceleration during the period 1800 through June 2006. was 0.33g. In addition, site specific probability of exceeding various peak horizontal ground accelerations and a seismic recurrence curve are also estimated/generated from the historical data. Printouts from EQSEARCH are provided in Appendix D.

A probabilistic seismic hazards analysis was performed using FRISKSP (Blake, 2000c) which models earthquake sources as three-dimensional planes and evaluates the site specific probabilities of exceedance for given peak acceleration levels or pseudo-relative velocity levels. Based on a review of these data, and considering the relative seismic activity of the southern California region, a peak horizontal ground acceleration of 0.42g was calculated. This value was chosen as it corresponds to a 10 percent probability of exceedance in 50 years (or a 475-year return period). Printouts from FRISKSP are also included in Appendix D.

Seismic Shaking Parameters

Per Chapter 16 of the UBC/CBC (ICBO, 1997 and 2001), the following updated seismic design parameters are provided:

UBC/CBC TABLE/FIGURE DESIGNATION	FAULT PARAMETERS
Seismic zone (per Figure 16-2*)	4
Seismic zone factor Z (per Table 16-I*)	0.40
Soil Profile Types (per Table 16-J*)	S_D
Seismic Coefficient C_a (per Table 16-Q*)	0.44 N_a
Seismic Coefficient C_v (per Table 16-R*)	0.64 N_v
Near Source factor N_a (per Table 16-S*)	1.0
Near Source factor N_v (per Table 16-T*)	1.0
Distance to Seismic Source (San Jacinto - San Jacinto Valley)	8.8 mi. (14.2 km)
Seismic Source Type (per Table 16-U*)	B**
Upper Bound Earthquake (Elsinore - Temecula)	M_W 6.8**
PHSA (10% probability of exceedance in 50 years)	0.42g
* Figure and table references from Chapter 16 of the UBC/CBC (ICBO, 1997 and 2001). ** ICBO (1998).	

GROUNDWATER

Perched groundwater was encountered in our onsite exploratory Boring B-3 at a depth of ±39 feet below the ground surface. Based on historic water well data acquired from the California Department of Water Resources (CDWR, 2005), "Water Data Library," regional groundwater levels in other nearby wells were previously measured at depths between ±65 feet to ±68 feet below the ground surface. However, shallower, perched groundwater may occur onsite in the Pleistocene fan deposits, or along jointing and/or fractures due to migration from adjacent drainage areas and developments during and/or after periods of above normal or heavy precipitation or irrigation. Thus, perched groundwater conditions may occur in the future, and should be anticipated. This potential increases on cut lots in dense native materials. This potential should be disclosed to all interested/affected parties, including owners and any owners association. Should manifestations of perched conditions (i.e., seepage) develop in the future, GSI could assess the conditions and provide mitigative recommendations, as necessary.

OTHER GEOLOGIC HAZARDS

Liquefaction

Seismically-induced liquefaction is a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in soils. The soils may thereby acquire a high degree of mobility, and lead to lateral movement, sliding, sand boils, consolidation and settlement of loose sediments, and other damaging deformations.

This phenomenon occurs only below the water table; but after liquefaction has developed, it can propagate upward into overlying, non-saturated soil as excess pore water dissipates. Typically, liquefaction has a relatively low potential at depths greater than 45 feet and is virtually unknown below a depth of 60 feet.

Liquefaction susceptibility is related to numerous factors and the following conditions should be concurrently present for liquefaction to occur: 1) sediments must be relatively young in age and not have developed a large amount of cementation; 2) sediments generally consist of medium- to fine-grained relatively cohesionless sands; 3) the sediments must have low relative density; 4) free groundwater must be present in the sediments; and 5) the site must experience a seismic event of a sufficient duration and magnitude, to induce straining of soil particles.

The condition of liquefaction has two principal effects. One is the consolidation of loose sediments with resultant settlement of the ground surface. The other effect is lateral sliding. Significant permanent lateral movement generally occurs only when there is significant differential loading, such as fill or natural ground slopes within susceptible materials. No such loading conditions exist on the site. In the site area, we found there is a potential for seismic activity and a regional groundwater table that ranges from ±39 feet to ±68 feet below the ground surface. The Pleistocene fan deposits encountered across the site were generally medium dense to dense where encountered.

According to the Riverside County Land Information System, the site has been indicated to lie mostly within a moderate liquefaction potential zone with a low liquefaction potential zone indicated in the extreme southwest corner of the site. As indicated earlier, ZKCI (2004) performed a liquefaction analysis on the site, and concluded that the site was generally not susceptible to liquefaction.

It should be noted that throughout our site observations and borings, there was no evidence of upward-directed hydraulic force that was suddenly applied, and was of short duration, nor were there any features commonly caused by seismically-induced liquefaction, such as dikes, sills, vented sediment, lateral spreads, or soft-sediment deformation. These features would be expected if the site area had been subject to liquefaction in the past (Obermeier, 1996). Inasmuch as the future performance of the site with respect to liquefaction should be similar to the past, excluding the effects of urbanization (irrigation), GSI concludes that the site generally has not been subject to liquefaction in the geologic past, regardless of the depth of the regional water table.

Since two of these five required concurrent conditions discussed above do <u>not</u> have the potential to affect the site, evidence of paleoliquefaction features was not observed, relatively dense Pleistocene fan deposits underlie the site, our evaluation indicates that the potential for liquefaction and associated adverse effects within the site is low, even with a seasonal rise in groundwater levels.

In accordance with the procedures in the referenced Special Publication 117, as well as the Recommended Procedures for Implementation of DMG Special Publication 117 (SP117), Guidelines for Analyzing and Mitigating Liquefaction Hazards in California, this report should be considered a "screening investigation," as provided for in Chapter 3 of SP117 and Section 4 of the Guidelines for Analyzing and Mitigating Liquefaction Hazards in California. Those guidelines state the following: *"The purpose of the screening investigations for sites within zones of required study is to filter out sites that have no potential or low potential for liquefaction."*

In summary, based on our research, regional groundwater at the site ranges from ±39 feet to ±68 feet below the ground surface. Earth materials underlying the site are generally medium dense to dense/very stiff where encountered. Such materials are <u>not</u> generally prone to liquefaction. Due to their nature, the beds and lenses are typically interfingered and discontinuous, both horizontally and vertically, within the site area. Thus, GSI has concluded that liquefaction potential does not constitute a significant risk to site development, provided our recommendations are implemented.

Accordingly, it is our opinion that the standards outlined in SP117 have been met and sufficient data exists to support, and is consistent with, the stated conclusions regarding the low potential hazard from liquefaction. Further, it is our opinion that no further investigation or analyses is necessary or warranted, provided our recommendations are properly implemented.

Subsidence

The effects of areal subsidence generally occur at the transition or boundaries between low-lying areas and adjacent hillside terrain, where materials of substantially different engineering properties (i.e., topsoil/colluvium vs. bedrock) are present. Based on the available data, bedrock generally underlies the Pleistocene fan deposits at the site at depth, and the site is not known to be situated at, or near the boundaries of an actively subsiding basin. Therefore, the potential for this phenomena to affect the site is considered low.

In addition, our review of available data, as well as stereoscopic aerial photographs (USDA, 1980), showed no features generally associated with areal subsidence (i.e., radially-directed drainages flowing into a depression(s), linearity of depressions associated with mountain fronts, etc.). Ground fissures are generally associated with excessive groundwater withdrawal and associated subsidence, or active faulting. Our review did not reveal any information that active faulting or excessive groundwater withdrawal, or ground fissures, or hydroconsolidation in the specific site vicinity, is occurring at this time. Therefore, the potential for areal subsidence or ground fissures is deemed low.

Local ground subsidence may occur over the site because of equipment working (vibrations). Such subsidence depends upon the equipment used and on the dynamic effects of the equipment. Given that the site is underlain by the dense fan deposits, the amount of such subsidence would be minimal. We estimate that ground subsidence due to vibration/loading would be less than 0.10 to 0.15 feet across the site.

Hydrocollapse

Based on our evaluation of the relatively dense Pleistocene fan deposits that underlie the site, and due to the elevation of perched groundwater encountered during our field investigation at ±39 feet below ground surface (b.g.s.), the potential for hydrocollapse of the soil materials between the anticipated remedial removal bottom and the current perched groundwater elevation is considered low. In addition, our review (see Appendix A) did not reveal any information that excessive groundwater withdrawal, ground fissures, or hydroconsolidation in the specific site vicinity, is occurring at this time. Therefore, the potential for hydrocollapse is deemed low, and should not be any greater than approved projects that bound the site. However, given the anticipated strong ground shaking during a design seismic event, some seismic densification (above the water table) may occur onsite. Mitigative measures include removal and recompaction of near-surface low density earth materials, geotechnical seismic design criteria (including estimated site accelerations), and the utilization of post-tension/mat foundations systems, etc.

Mass Wasting

Mass wasting refers to the various processes by which earth materials are moved down slope in response to the force of gravity. Examples of these processes include slope creep, surficial failures, rockfall, deep-seated landslides, etc. Creep is the slowest form of mass wasting and generally involves the outer 5 to 10 feet of the slope surface. During heavy rains, such as those in 1969, 1978, and 1980, 1983, 1993, 1998, 2004, and 2005, creep-affected materials may become saturated, resulting in a more rapid form of down slope movement (i.e., landslides, surficial failures, and/or rockfall). The subject site consists of relatively flat terrain and indications of mass wasting phenomena on the site were not observed during our review of stereoscopic photographs of the area (USDA, 1980) or during our site reconnaissance.

Due to the sedimentary nature of the materials that exist on portions of the site, caving and sloughing should be anticipated in all subsurface excavations and trenching. Appropriate safety considerations for potential caving and sloughing, such as shoring or layback cuts, should be incorporated into the construction design details. Therefore, the potential for mass wasting phenomena to effect the site is considered low, provided our recommendations are properly implemented. Likewise, the potential for seismically-induced landsliding is considered low to nil.

LABORATORY TESTING

General

Laboratory tests were performed on representative samples of the onsite earth materials in order to evaluate their physical characteristics. The test procedures used and results obtained are presented below.

Moisture-Density

The field moisture content and dry unit weight were determined for each undisturbed sample of the soils encountered in the borings. The dry unit weight was determined in pounds per cubic foot (pcf), and the field moisture content was determined as a percentage of the dry weight. The results of these tests are shown on the Boring Logs (see Appendix B).

Laboratory Standard

The maximum dry density and optimum moisture content was determined for the major soil types encountered in the trenches, in general accordance with ASTM D-1557. The moisture-density relationship obtained for this soil is shown below:

DESCRIPTION	SAMPLE LOCATION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
Sandy CLAY, Dark Brown	B-2 @ 0'-5'	116.0	15.5

Expansion Potential

Expansion testing was performed on a representative samples of site soil in general accordance with UBC Standard 18-2. The results of expansion testing are presented in the following table:

SAMPLE LOCATION	EXPANSION INDEX	EXPANSION POTENTIAL
B-3 @ 2-7	81	Medium
B-4 @ 0' - 5'	24	Low

Atterberg Limits

Atterberg limits testing was performed on the sample exhibiting a medium expansion potential to evaluate the liquid limit, plastic limit and plasticity index in general accordance with ASTM D-4318-64. These tests results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The test results and classification are presented in Appendix C.

Consolidation Testing

Consolidation testing was performed on a relatively undisturbed soil samples in general accordance with ASTM Test Method D-2435-90. The consolidation test results are presented in Appendix C.

Corrosion/Sulfate Testing

GSI conducted sampling of onsite materials for sulfate content and soil corrosivity on the subject project. Laboratory test results were completed by M.J. Schiff & Associates (consulting corrosion engineers). The data should be utilized by the project structural engineer (or corrosion engineer) in their evaluation of site corrosivity mitigation measures (see Appendix C).

Representative samples of onsite materials were collected for soluble sulfate testing. Sulfate testing results indicate a sulfate content range of 0.0053 percentage by weight. This result indicates that site soils are in the negligible range for sulfate exposure per the UBC (ICBO, 1997). Based upon the soluble sulfate test results, sulfate-resistant, concrete is not required; however, higher sulfate contents may exist onsite.

In addition pH and resistivity (saturated) testing was performed. Testing results indicate a pH of 8.0, which is moderately alkaline, and a saturated resistivity of 2,760 ohm-centimeters.. This would generally be considered moderately corrosive to ferrous metals (between 2,000 to 10,000 ohm-cm is considered moderately corrosive). It is our understanding that standard concrete cover over reinforcing steel is usually appropriate for these conditions; however, consulting a qualified corrosion engineer is recommended to provide specific recommendations for foundations and piping, etc.

R-value

A representative sample was collected for R-value testing. The R-value was evaluated in general accordance with the California Materials Method No. 301. Test results indicate an R-value of 10, also included in Appendix C.

EMBANKMENT FACTORS

Embankment factors (shrinkage) for the site have been estimated based upon our field and laboratory testing, visual site observations, and experience. It is apparent that shrinkage would vary with depth and with areal extent over the site, based on previous site use. Variables include vegetation, weed control, and discing. However, all these factors are difficult to define in a three-dimensional fashion. Therefore, the information presented below represents average shrinkage/bulking values, using certain earthwork assumptions as follows:

Maximum Density = 116.0
Relative Compaction = 92%

Material	% Range
Topsoil/Alluvium	15 to 22 (shrinkage)
Pleistocene Fan Deposits	6 to 12 (shrinkage)

Subsidence due to dynamic compaction from equipment routes, etc., is estimated to be 0.10 feet. An additional shrinkage factor item would include the removal of root systems of individual large plants or trees. These plants and trees vary in size, but when pulled, they may generally result in a loss of ½ to 1½ cubic yards, to locally greater than 3 cubic yards of volume, respectively. This factor needs to be multiplied by the number of significant plants, trees, or tree roots present to determine the net loss. Also, it should be noted that the grading contractor also controls shrinkage by the degree of relative compaction attained in the field (i.e., 92 vs. 94 percent compaction, etc.). Accordingly, a shrinkage factor for relative compaction variances may be on the order of 2 or 3 percent additional, either way. The above facts indicate that earthwork balance for the site would be difficult to define and flexibility in design is essential to achieve a balanced end product.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

General

Based on our supplemental field exploration and geotechnical engineering analysis, it is our opinion that the site appears suitable for the proposed development from a geotechnical engineering and geologic viewpoint, provided that the recommendations presented in the following sections are incorporated into the design and construction phases of site development. The primary geotechnical concerns with respect to the proposed development are:

- Depth to suitable bearing strata.
- Expansion and corrosion potential of site soils.
- Ongoing potential for perched water to occur after site development.
- Regional seismic activity.

The recommendations presented herein consider these, as well as other aspects of the site. The engineering analyses performed concerning site preparation and the recommendations presented herein, have been completed using the information provided by the client and obtained during our field work.

The conclusions and recommendations contained in this report shall not be considered valid unless the proposed development is reviewed and the recommendations of this report verified or modified in writing by this office. Foundation design parameters are considered preliminary until the foundation design, layout, and structural loads are provided to this office for review.

1. Soil engineering, observation, and testing services should be provided during grading to aid the contractor in removing unsuitable soils and in his effort to compact the fill.

2. Geologic observations should be performed during grading to verify and/or further evaluate geologic conditions. Although unlikely, if adverse geologic structures are encountered, supplemental recommendations and earthwork may be warranted.

3. Existing topsoil/alluvium, and weathered Pleistocene-age fan deposits are typically porous, loose, and subject to settlement and considered unsuitable for the support of settlement-sensitive structures in their present condition, and therefore should be removed and recompacted, based on current industry standards. Laboratory testing and visual observations made during this investigation indicate these materials are not uniform, are potentially compressible in their present condition, and may be subject to adverse differential settlement, if not mitigated during site grading operations. Remedial removals are estimated to range from ±3 to ±4 feet across the site, with localized deeper removals possible, if not removed by planned excavation.

4. It should be noted, that the UBC/CBC (ICBO, 1997 and 2001) indicates that removals of unsuitable soils be performed across all areas to be graded, not just within the influence of the structures. Relatively deep removals may also necessitate a special zone of consideration, on perimeter, confining areas, such that this potential zone is approximately equal to the depth of removals, if removals cannot performed offsite. Thus, any settlement-sensitive improvements (perimeter walls, curbs, flatwork, etc.), constructed within this zone may require deepened foundations, reinforcement, etc., or will retain some potential for settlement and associated distress.

5. In general and based upon the available data to date, regional groundwater is generally not anticipated to affect site development, providing that the recommendations contained in this report are incorporated into final design and construction, and that prudent surface and subsurface drainage practices are incorporated into the construction plans. Perched groundwater conditions along

zones of contrasting permeabilities were observed and should be anticipated in the future due to site irrigation, poor drainage conditions, or damaged utilities. Should perched groundwater conditions develop, this office could assess the affected area(s) and provide the appropriate recommendations to mitigate the observed groundwater conditions.

6. Our laboratory test results and experience on nearby sites related to expansion potential indicate that soils with low to medium expansion indices underlie the site. The possibility that soils with high expansion potentials are present also exists. This should be considered during project design. Foundation design and construction recommendations are provided herein for low to medium expansion potential classifications. Final foundation designs will be based on testing of finish grade materials at the conclusion of grading.

7. The seismicity-acceleration values provided herein should be considered during the design of the proposed development

8. General Earthwork and Grading Guidelines are provided at the end of this report as Appendix E. Specific recommendations are provided below.

General Grading

All grading should conform to the guidelines presented in the UBC (ICBO, 1997), the County of Riverside, and Appendix F (this report), except where specifically superceded in the text of this report. When code references are not equivalent, the more stringent code should be followed. During earthwork construction, all site preparation and the general grading procedures of the contractor should be observed and the fill selectively tested by a representative of GSI. If unusual or unexpected conditions are exposed in the field, they should be reviewed by this office and if warranted, modified and/or additional recommendations will be offered. All applicable requirements of local and national construction and general industry safety orders, the Occupational Safety and Health Act, and the Construction Safety Act should be met.

Demolition/Grubbing

1. Existing structures, vegetation, and any miscellaneous debris should be removed from the areas of proposed grading.

2. Any previous foundations, irrigation lines, cesspools, septic tanks, leach fields, or other subsurface structures uncovered during the recommended removal should be observed by GSI so that appropriate remedial recommendations can be provided.

3. Cavities or loose soils remaining after demolition and site clearance should be cleaned out and observed by the soil engineer. The cavities should be replaced

with fill materials that have been moisture conditioned to <u>at least</u> optimum moisture content and compacted to at least 90 percent of the laboratory standard.

Treatment of Existing Ground

1. All undocumented artificial fill, topsoil/alluvium, and near-surface weathered Pleistocene fan deposits should be removed to competent fan deposits, as defined herein (i.e., removal bottoms should be greater than, or equal to, 85 percent saturation, and/or greater than, or equal to, 105 pcf dry density for in-place native materials), which has been demonstrated and proven to the controlling authorities as acceptable in the past to mitigate the potential for hydroconsolidation. For preliminary planning purposes, these depths are estimated to range from ±3 to ±4 feet across the site, with localized deeper removals possible, if not removed by planned excavation. Variations from these thicknesses should be anticipated. Actual depths of removals will be evaluated in the field during grading by the soil engineer.

2. Subsequent to the above removals, the upper 12 inches of the exposed subsoils should be scarified, brought to <u>at least</u> optimum moisture content, and recompacted to a minimum relative compaction of 90 percent of the laboratory standard.

3. Existing undocumented fill, topsoil/alluvium, and removed natural ground materials may be reused as compacted fill provided that major concentrations of oversized materials, vegetation, and miscellaneous debris are removed prior to or during fill placement.

4. Localized deeper removal may be necessary due to buried drainage channel meanders or dry porous materials. The project soils engineer/geologist should observe all removal areas during the grading.

Fill Placement

1. Subsequent to ground preparation, fill materials should be brought to <u>at least</u> optimum moisture content, placed in thin 6- to 8-inch lifts and mechanically compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard.

2. Fill materials should be cleansed of major vegetation and debris prior to placement.

3. Any oversized rock materials greater than 8 inches in diameter should be placed under the recommendations and supervision of the soils engineer and/or removed from the site. Should significant amounts of oversize rock be encountered, recommendations for rock fill placement will be provided by GSI. As per UBC/CBC (ICBO, 1997 and 2001) requirements, no oversize materials greater that 12 inches in diameter should be placed within 10 feet of finish grade.

4. Any import materials should be observed and determined suitable by the soils engineer prior to placement on the site. Foundation designs may be altered if import materials have a greater expansion value than the onsite materials encountered in this investigation.

Transition Areas/Overexcavation

In order to reduce the potential for differential settlements between cut and fill materials, materials of differing expansion potentials, the entire cut portion of cut/fill transitions within proposed lots (per the UBC/CBC [ICBO, 1997 and 2001]) should be overexcavated to a minimum depth of 3 feet below finish grade, and/or a maximum ratio of fill thickness on the lot of 3:1 (h:v), whichever is greater, and replaced with compacted fill. Cut lots should be overexcavated a minimum of 3 feet, and the subgrade sloped to drain to the street.

SUBDRAINS

Subdrains may be necessary where local seepage along the contact between the proposed fill and underlying natural materials exist, or within the proposed fill along contrasting zones of permeabilities (e.g., sand vs. silt, or clay). Owing to the generally low gradient nature of the site, lack of significant canyons, lack of gradient at flowline elevations, and the lack of suitable cover (i.e., less than about 10 feet of fill), subdrains are generally not anticipated during grading, but may not be entirely precluded. If required, where removals are below the subdrain flowline, the removal materials may be reused as compacted fill provided they are granular, and at a moisture content of at least 2 percent over optimum moisture content (or 1.2 times optimum moisture content, whichever is greater). If necessary, actual locations of subdrains will be further evaluated during the ro- or 100-scale stage, and actual locations provided during grading, based on exposed conditions. The project civil engineer should locate and map any subdrain and outlet systems.

SLOPE CONSIDERATIONS AND SLOPE DESIGN

All slopes should be designed and constructed in accordance with the minimum requirements of the UBC/CBC (ICBO, 1997 and 2001), County of Riverside, the recommendations presented in Appendix F, and the following:

1. Fill slopes should be designed at a 2:1 (h:v) gradients, or flatter, and should not exceed about ±10 feet in height. Fill slopes should be properly built and compacted to a minimum relative compaction of 90 percent throughout, including the slope surfaces. Guidelines for slope construction are presented in Appendix E.

2. Cut slopes should be designed at gradients of 2:1 and should not exceed about ±10 feet in height. While stabilization of such slopes is not anticipated, locally

adverse geologic conditions (e.g., daylighted joints/fractures or severely weathered material) may be encountered which may require remedial grading or laying back of the slope to an angle flatter than the adverse geologic condition.

3. Local areas of highly weathered materials may be present. Should these materials be exposed in cut slopes, the potential for long-term maintenance or possible slope failure exists. Evaluation of cut slopes during grading would be necessary in order to identify any areas of severely weathered fan materials. Should any of these materials be exposed during construction, the soils engineer/geologist, would assess the magnitude and extent of the materials and their potential affect on long-term maintenance or possible slope failures. Recommendations would then be made at the time of the field inspection.

4. Loose debris and fines remaining on the face of the cut slopes should be removed during grading. This can be accomplished by utilizing a slope board or by hand scaling, as warranted.

5. Where loose materials are exposed on the cut slopes, the project's engineering geologist would require that the slope be cleaned as described above prior to making their final inspection. Final approval of the cut slope can only be made subsequent to the slope being fully cut and cleaned.

6. Cut slopes should be mapped by the project engineering geologist during grading to allow amendments to the recommendations should exposed conditions warrant alternation of the design or stabilization.

PRELIMINARY FOUNDATION DESIGN

Bearing Value

1. An allowable soil bearing pressure of 1,500 pounds per square foot (psf) may be used for the design of continuous footings 12 inches wide and 12 inches deep. This value may be increased by 20 percent (per code) for each additional 12 inches in depth to a maximum value of 2,500 psf.

2. The bearing pressure may be increased by one-third for seismic or other temporary loads.

Lateral Pressure

1. For lateral sliding resistance, a 0.35 coefficient of friction may be utilized for a concrete to soil contact when multiplied by the dead load.

2. Passive earth pressure may be computed as an equivalent fluid having a density of 225 pcf with a maximum earth pressure of 2,500 psf.

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

4. All footings should maintain a minimum 7-foot horizontal distance the base of the footing and any adjacent descending slope, and minimally comply with the guidelines depicted on Figure No. 18-I-1 of the 1997 UBC (Setback Dimensions).

FOUNDATION CONSTRUCTION

The following foundation construction recommendations are presented as a minimum criteria from a soils engineering viewpoint. The expansion potential in the subject tract has been tested to be generally in the low to medium range (Expansion Index [E.I.] 21 to 90), however, soils with very low expansion potential are also anticipated on the site. Accordingly, the following foundation construction recommendations assume that the soils in the top 7 feet of finish grade will have a very low to medium expansion potential. Post-tensioned foundations will likely be recommended for lots where the E.I. exceeds 50 (E.I. >50). In addition, post-tensioned foundations may also be required where final expansion testing indicates an E.I. >20, and a Plasticity Index (P.I.) above 15, as per the Section 1815 and/or Section 1816 of the UBC/CBC (ICBO, 1997 and 2001). The site structural engineer should be informed of this to aid in preliminary foundation designs. Foundation design criteria for very low to medium expansion potentials are presented for planning, design, and budgetary considerations. These recommendations are not intended to preclude the transmission of water vapor through the foundations and slabs, which should be disclosed to all homeowners and/or other interested parties. Recommendations by the project's design-structural engineer, per section 1815 and/or 1816 of the 1997 UBC/CBC (ICBO, 1997 and 2001), or architect, which may exceed the soils engineer's recommendations, should take precedence over the following minimum requirements. Final foundation design will be provided based on the depth of fill, and expansion potential and plasticity index of the near-surface soils encountered during grading.

Expansion Index - Very Low to Low (E.I. 0 to 50, and P.I. <15)

1. Continuous exterior footings should be founded at a minimum depth of 18 inches below the lowest adjacent ground surface, for one- or two-story floor loads, and in accordance with the minimum requirements of the latest edition of the UBC. The structural engineer should review and approve these recommendations. Continuous interior footings may be founded at a minimum depth of 12 inches below the lowest adjacent ground surface. Footings should be a minimum of 12 inches wide, or as determined by the structural engineer. All footings should have one No. 4 reinforcing bar placed at the top and one No. 4 reinforcing bar placed at the bottom of each footing.

2. Isolated column footings and piers should be founded at a minimum depth of 18 inches, excluding the landscape zone (top 6 inches), and the column footings and piers should be tied together in one direction.

3. A grade beam reinforced as above, and at least 12 inches by 12 inches, should be provided across the garage entrances. The base of the grade beam should be at the same elevation as the adjoining footings.

4. Concrete slab underlayment should consist of 2 inches of sand (S.E. $\geq$30), underlain by a 10- to 15-mil vapor retarder (ASTM E-1745 - Class A or B type) to be installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.). The manufacturer shall provide instructions for lap sealing, including minimum width of lap, method of sealing, and either supply or specify suitable products for lap sealing (ASTM E-1745).

5. A minimum slab thickness of 4 inches is recommended. The design engineer should provide the actual thickness of concrete slabs based upon proposed loading and use.

6. Concrete slabs, including garage areas, should be reinforced with No. 3 rebars at 18 inches on center, each way. All slab reinforcement should be supported to ensure proper positioning at mid-height in the slab during placement of concrete.

7. Garage slabs should be poured separately from living area footings. A positive separation should be maintained with expansion joint material to permit relative movement.

8. Pre-moistening and/or presaturation of the slab areas is recommended for these soils conditions on a preliminary basis. The moisture content of the subgrade soils should be equal to or greater than optimum moisture to a depth equivalent to the exterior footing depth in the slab areas. Pre-moistening and/or presaturation should be evaluated by the soils engineer 72 hours prior to visqueen placement.

9. As an alternative to the above, an engineered post-tension foundation system may be used.

POST-TENSIONED SLAB SYSTEMS

Based on our preliminary evaluation, post-tensioned foundations will likely be recommended for lots where the E.I. exceeds 50 (E.I. >50, if encountered). In addition, post-tensioned foundations may also be required where final expansion testing indicates an E.I. >20, and a P.I. above 15, as per the Section 1815 and/or Section 1816 of the UBC/CBC (ICBO, 1997 and 2001). The site structural engineer should be informed of this to aid in preliminary foundation designs. Post-tensioned foundation recommendations are also provided for very low expansion conditions in the event such a system is desired. The recommendations presented below should be followed in addition to those contained in the previous sections, as appropriate. The information and recommendations presented in this section are not meant to supercede design by a registered structural engineer or civil engineer familiar with post-tensioned slab design. Upon request, GSI can provide additional data/consultation regarding soil parameters as related to post-tensioned slab design.

From a soil expansion/shrinkage standpoint, a common contributing factor to distress of structures using post-tensioned slabs is fluctuation of moisture in soils underlying the perimeter of the slab, compared to the center, causing a "dishing" or "arching" of the slabs. To mitigate this possibility, a combination of soil presaturation and construction of a perimeter cut-off wall should be employed.

Perimeter cut-off walls should be a minimum of 12 inches deep for low expansive soils, and 18 inches deep for medium to high expansive soils. The cut-off walls may be integrated into the slab design and should be a minimum of 6 inches wide. The vapor retarder should be covered with a 2-inch layer of sand to aid in uniform curing of the concrete; and it should be lapped adequately to provide a continuous water-proof barrier under the entire slab. If medium to high expansive soils are present, an additional 2 inches of sand should be placed on grade (4 inches total).

Specific soil premoistening or presaturation is required. The moisture content of the subgrade soils should be 100 or 120 percent of the soils' optimum moisture content to a depth of 12, or 18 inches below grade, for very low to low (100 percent), or medium expansive soils (120 percent), respectively.

Post-tensioned slabs should be designed using sound engineering practice and be in accordance with local and/or national code requirements. Soil related parameters for post-tensioned slab design are presented below:

Allowable surface bearing value	1,000 psf
Modulus of subgrade reaction	75 psi per inch
Coefficient of friction	0.35
Passive pressure	250 pcf

Post-Tensioning Institute Method: Post-tensioned slabs should have sufficient stiffness to resist excessive bending due to non-uniform swell and shrinkage of subgrade soils. The differential movement can occur at the corner, edge, or center of slab. The potential for differential uplift can be evaluated using the 1997 UBC Section 1816, based on design specifications of the Post-Tensioning Institute. The following table presents suggested minimum coefficients to be used in the Post-Tensioning Institute design method.

Thornthwaite Moisture Index	-20 inches/year
Correction Factor for Irrigation	20 inches/year
Depth to Constant Soil Suction	7 feet
Constant soil Suction (pf)	3.6
Modulus of Subgrade Reaction (pci)	75
Moisture Velocity	0.7 inches/month

The coefficients are considered minimums and may not be adequate to represent worst case conditions such as adverse drainage and/or improper landscaping and maintenance. The above parameters are applicable provided structures have positive drainage that is maintained away from structures. Therefore, it is important that information regarding drainage, site maintenance, settlements, and effects of expansive soils be passed on to future owners.

Based on the above parameters, the following values were obtained from figures or tables of the 1997 UBC, Section 1816. The values may not be appropriate to account for possible differential settlement of the slab due to other factors. If a stiffer slab is desired, higher values of y_m may be warranted.

EI OF SOIL SUBGRADE	VERY LOW E.I.	LOW E.I.	MEDIUM E.I.	HIGH E.I.
e_m center lift	5.0 feet	5.0 feet	5.5 feet	5.5 feet
e_m edge lift	2.5 feet	3.5 feet	4.0 feet	4.5 feet
y_m center lift	1.0 inch	1.7 inches	2.7 inches	3.5 inches
y_m edge lift	0.3 inch	0.75 inch	0.75 inch	1.2 inches

Deepened footings/edges around the slab perimeter must be used to minimize non-uniform surface moisture migration (from an outside source) beneath the slab. An edge depth of 12 inches should be considered a minimum. The bottom of the deepened footing/edge should be designed to resist tension per the structural engineer. Other applicable recommendations presented under conventional foundation and the California Foundation Slab Method should be adhered to during the design and construction phase of the project.

SOIL MOISTURE CONSIDERATIONS

Foundation systems and slabs shall not allow water or water vapor to enter into the structure so as to cause damage to another building component, or to limit the installation of the type of flooring materials typically used for the particular application (State of

California, 2006). Therefore, the following should be considered by the structural engineer/foundation/slab designer to mitigate the transmission of water or water vapor through the slab.

- Concrete slabs should be a minimum of 5 inches thick for very low expansive soil conditions, and be minimally reinforced as previously discussed. All slab reinforcement should be supported to provide proper mid-slab height positioning during placement of the concrete. "Hooking" of reinforcement is not an acceptable method of positioning. Increase of concrete slab thickness would tend to reduce moisture vapor transmission though slabs.

- Concrete slab underlayment should consist of a 10-mil to 15-mil vapor retarder, or equivalent, with all laps sealed per the UBC/CBC (ICBO, 1997 and 2001) and the manufacturer's recommendation. The vapor retarder should comply with the ASTM E-1745 Class A or B criteria and be installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.). The manufacturer shall provide instructions for lap sealing, including minimum width of lap, method of sealing, and either supply or specify suitable products for lap sealing (ASTM E-1745). In order to break the capillary rise of soil moisture, the vapor retarder should be underlain by 2 inches of fine or coarse, washed, clean gravel (80 to 100 percent greater than #4 sieve) and be overlain by at least 2 inches of clean, washed sand (SE $\geq$30) to aid in concrete curing.

- Concrete should have a maximum water/cement ratio of 0.50.

- Where slab concrete compressive strength is increased, admixtures used, and water/cement ratios are adjusted herein, the structural consultant should also make changes to the concrete in the grade beams and footings in kind so that the concrete used in the foundation and slabs are designed and/or treated for more uniform moisture protection.

- The use of a penetrating slab surface sealer may be considered in rooms where permeable floor tile or wood will be used. In all planned floorings, the waterproofing specialist should review the manufacturer's recommendations and adjust installation as needed. Homeowner(s) should be advised which areas are suitable for tile or wood floors.

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

Please be aware that the above should be implemented if the transmission of water or water vapor through the slab is undesirable. Should these recommendations not be implemented, then full disclosure of the potential for water or vapor to pass through the foundations and slabs and resultant distress should be provided to all interested parties, in writing. Regardless of the mitigation, some limited moisture/moisture vapor transmission through the slab should be anticipated.

PRELIMINARY FOUNDATION SETTLEMENTS

In addition to designing slab systems (post-tension or other) for the soil conditions described herein, the estimated settlement and angular distortion values that an individual structure (including walls, spas, pools, or other settlement-sensitive improvements, etc.), could be subjected to should be evaluated by a structural engineer. The levels of angular distortion were evaluated on a 40-foot length assumed as the minimum dimension of buildings; if, from a structural standpoint, a decreased or increased length over which the differential settlement is assumed to occur is justified, this change should be incorporated into the design.

Evaluation of potential primary settlement and secondary compression within the area under the purview of this report has been conducted, based on the available data. On a preliminary basis, the footings and/or slabs should be minimally designed to accommodate a differential settlement of 1 inch (i.e., at least 1 inch in a 40-foot span [1/480]). Any post-construction settlement of the fill should be readily mitigated by proper foundation design, provided the design parameters, provided in this report, are properly utilized in final design of the foundation systems. In addition to the above, the slab designer and/or structural engineer should also consider estimated settlements due to short duration seismic loading and applicable load combinations, as required by the County and/or the UBC/CBC (ICBO, 1997 and 2001). Foundation settlements will be re-evaluated once actual fill depths/thicknesses are known, and shown on the 40-scale plans.

PRELIMINARY WALL DESIGN PARAMETERS CONSIDERING EXPANSIVE SOILS

Conventional Retaining Walls

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

Restrained Walls

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

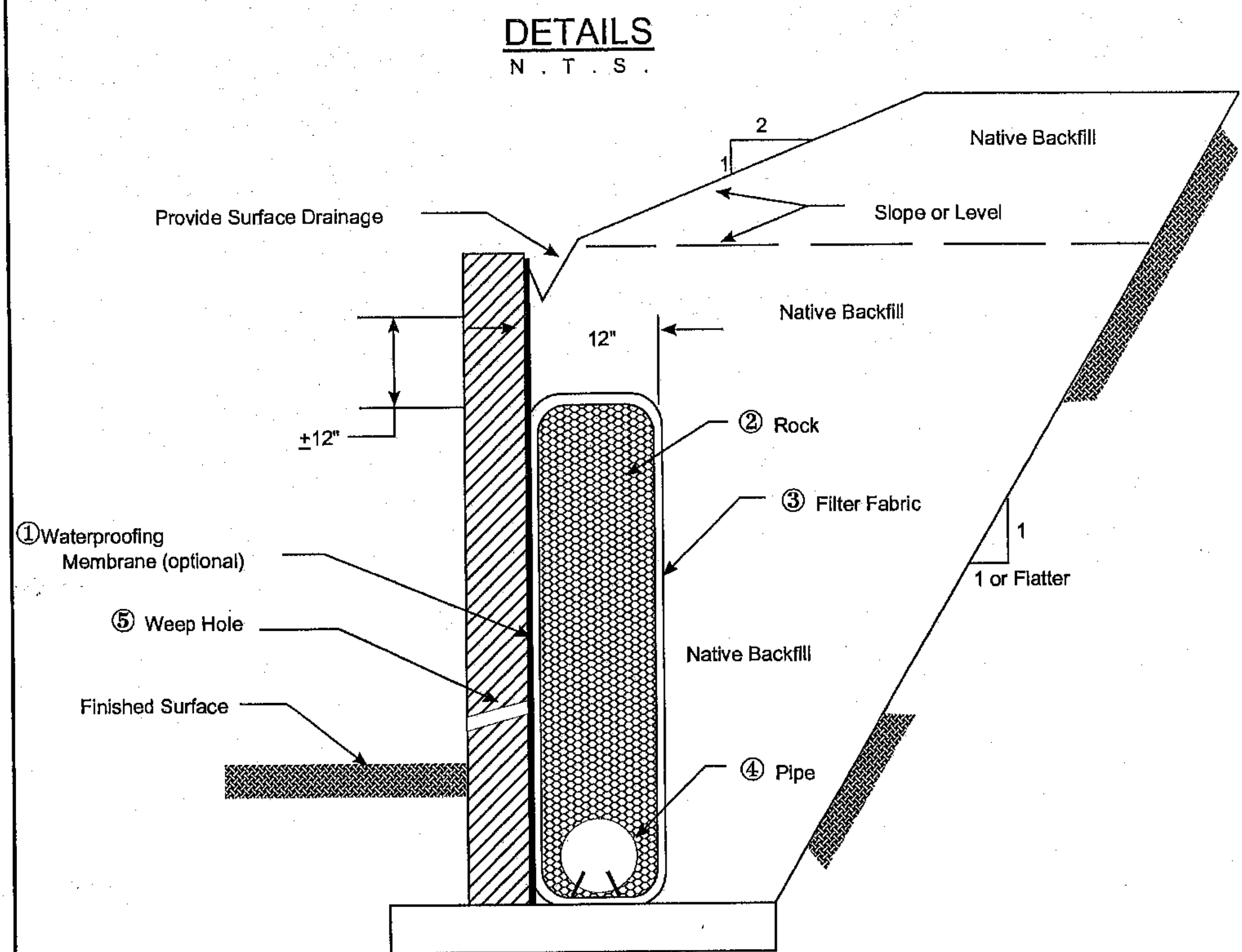
Cantilevered Walls

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

SURFACE SLOPE OF RETAINED MATERIAL (HORIZONTAL:VERTICAL)	EQUIVALENT FLUID WEIGHT P.C.F. (SELECT BACKFILL)	EQUIVALENT FLUID WEIGHT P.C.F. (NATIVE BACKFILL)
Level*	38	50
2 to 1	55	65
* Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall, where H is the height of the wall.		

Retaining Wall Backfill and Drainage

Positive drainage must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is considered necessary for retaining walls that are 2 feet or greater in height. Details 1, 2, and 3, present the backdrainage options discussed below. Backdrains should consist of a 4-inch diameter perforated PVC or ABS pipe encased in either Class 2 permeable filter material or ¾-inch to 1½-inch gravel wrapped in approved filter fabric (Mirafi 140 or equivalent). For low expansive backfill, the filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. For native backfill that has up to medium expansion potential, continuous Class 2 permeable drain materials should be used behind the wall. This material should be continuous (i.e., full height) behind the wall, and it should be constructed in accordance with the enclosed Detail 1 (Typical Retaining Wall Backfill and Drainage Detail). For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Retaining Wall Backfill and Subdrain Detail Geotextile Drain). Materials with an E.I. potential of greater than 90 should not be used as backfill for retaining walls. For more onerous expansive situations, backfill and drainage behind the retaining wall should conform with Detail 3 (Retaining Wall And Subdrain Detail Clean Sand Backfill).



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

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

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

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

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

GeoSoils, Inc.

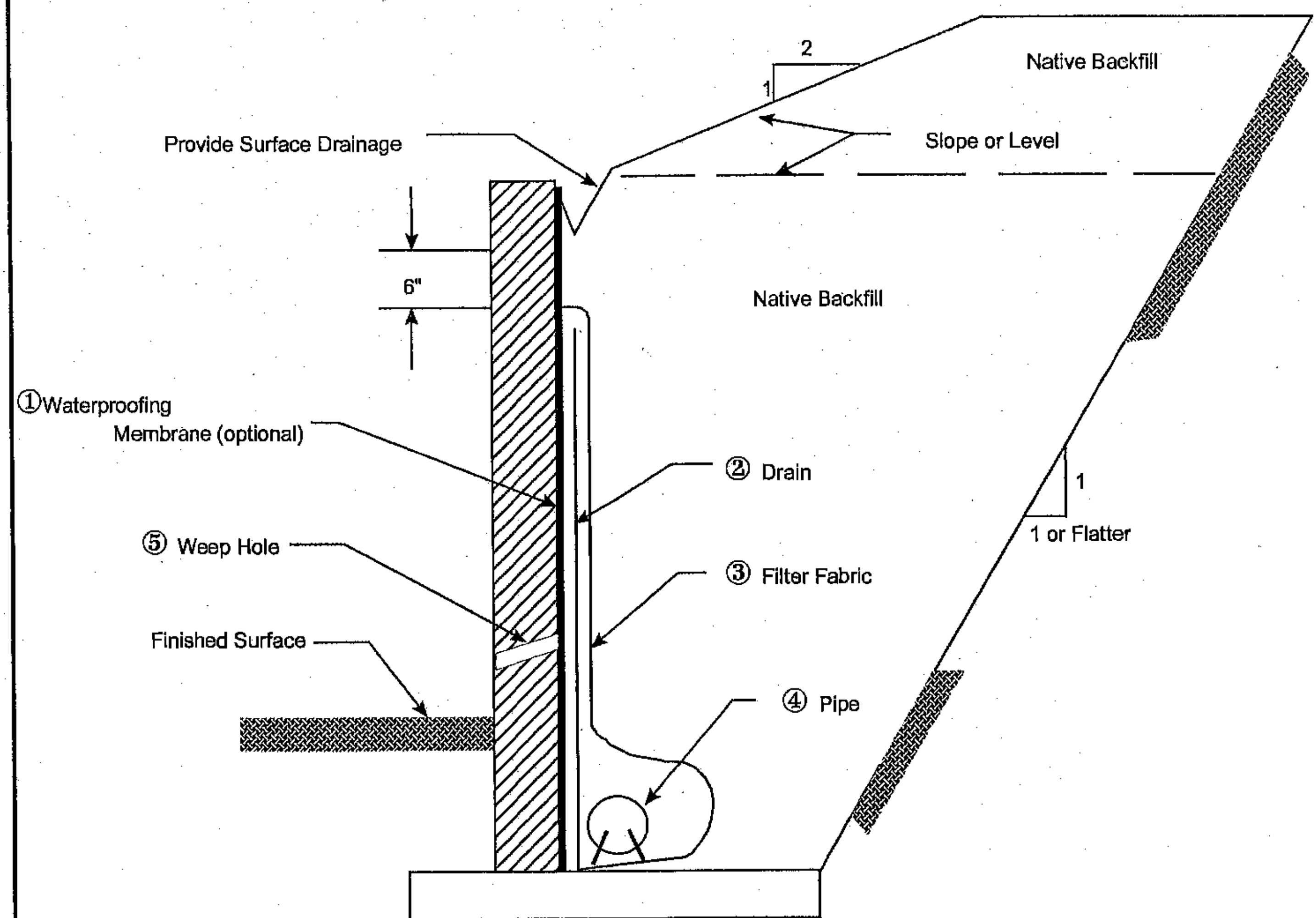
TYPICAL RETAINING WALL BACKFILL AND DRAINAGE DETAIL

DETAIL 1

Geotechnical • Coastal • Geologic • Environmental

DETAILS

N . T . S .



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

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

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

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

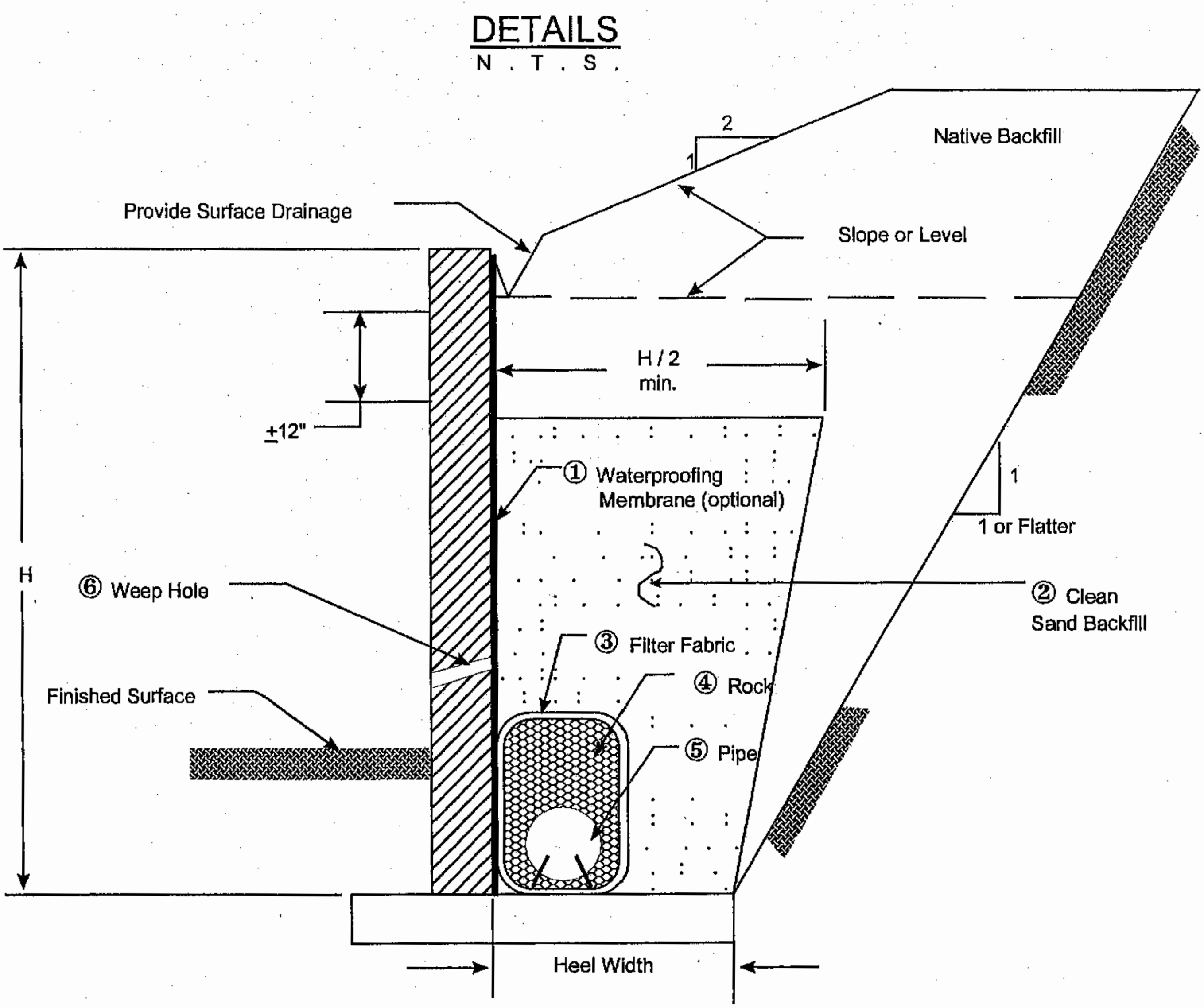
⑤ **WEEP HOLE:**
Minimum 2" (inches) diameter placed at 20' (feet) on centers along the wall, and 3" (inches) above finished surface. (No weep holes for basement walls.)

GeoSoils, Inc.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL GEOTEXTILE DRAIN

DETAIL 2

Geotechnical • Coastal • Geologic • Environmental



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

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

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

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

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

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

GeoSoils, Inc.

RETAINING WALL AND SUBDRAIN DETAIL
CLEAN SAND BACKFILL

DETAIL 3

Geotechnical • Coastal • Geologic • Environmental

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

Wall/Retaining Wall Footing Transitions

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

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

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

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

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

TOP-OF-SLOPE WALLS/FENCES/IMPROVEMENTS AND EXPANSIVE SOILS

Expansive Soils and Slope Creep

Soils at the site are likely to be expansive and therefore, become desiccated when allowed to dry. Such soils are susceptible to surficial slope creep, especially with seasonal changes in moisture content. Typically in southern California, during the hot and dry summer period, these soils become desiccated and shrink, thereby developing surface cracks. The extent and depth of these shrinkage cracks depend on many factors such as the nature and expansivity of the soils, temperature and humidity, and extraction of moisture from surface soils by plants and roots. When seasonal rains occur, water percolates into the cracks and fissures, causing slope surfaces to expand, with a corresponding loss in soil density and shear strength near the slope surface. With the passage of time and several moisture cycles, the outer 3 to 5 feet of slope materials experience a very slow, but progressive, outward and downward movement, known as slope creep. For slope heights greater than 10 feet, this creep related soil movement will typically impact all rear yard flatwork and other secondary improvements that are located within about 15 feet from the top of slopes, such as swimming pools, concrete flatwork, etc., and in particular top of slope fences/walls. This influence is normally in the form of detrimental settlement, and tilting of the proposed improvements. The dessication/swelling and creep discussed above continues over the life of the improvements, and generally becomes progressively worse. Accordingly, the developer should provide this information to any homeowners and homeowners association.

Top of Slope Walls/Fences

Due to the potential for slope creep for slopes higher than about 10 feet, some settlement and tilting of the walls/fence with the corresponding distresses, should be expected. To mitigate the tilting of top of slope walls/fences, we recommend that the walls/fences be constructed on a combination of grade beam and caisson foundations. The grade beam should be at a minimum of 12 inches by 12 inches in cross section, supported by drilled caissons, 12 inches minimum in diameter, placed at a maximum spacing of 6 feet on center, and with a minimum embedment length of 7 feet below the bottom of the grade beam. The strength of the concrete and grout should be evaluated by the structural engineer of record. The proper ASTM tests for the concrete and mortar should be provided along with the slump quantities. The concrete used should be appropriate to mitigate sulfate corrosion, as warranted. The design of the grade beam and caissons should be in accordance with the recommendations of the project structural engineer, and include the utilization of the following geotechnical parameters:

Creep Zone: 5-foot vertical zone below the slope face and projected upward parallel to the slope face.

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

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

Passive Resistance: Passive earth pressure of 300 psf per foot of depth per foot of caisson diameter, to a maximum value of 4,500 psf may be used to determine caisson depth and spacing, provided that they meet or exceed the minimum requirements stated above. To determine the total lateral resistance, the contribution of the creep prone zone above the point of fixity, to passive resistance, should be disregarded.

Allowable Axial Capacity:

Shaft capacity :	350 psf applied below the point of fixity over the surface area of the shaft.
Tip capacity:	4,500 psf.

EXPANSIVE SOILS, DRIVEWAY, FLATWORK, AND OTHER IMPROVEMENTS

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

1. The subgrade area for concrete slabs should be compacted to achieve a minimum 90 percent relative compaction, and then be presoaked to 2 to 3 percentage points above (or 125 percent of) the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. The moisture content of the subgrade should be proof tested within 72 hours prior to pouring concrete.

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

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

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

   In order to reduce the potential for unsightly cracks, slabs should be reinforced at mid-height with a minimum of No. 3 bars placed at 18 inches on center, in each direction. The exterior slabs should be scored or saw cut, ½ to ⅜ inches deep, often enough so that no section is greater than 10 feet by 10 feet. For sidewalks or narrow slabs, control joints should be provided at intervals of every 6 feet. The slabs should be separated from the foundations and sidewalks with expansion joint filler material.

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

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

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

8. Overhang structures should be supported on the slabs, or structurally designed with continuous footings tied in at least two directions.

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

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

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

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

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

PRELIMINARY PAVEMENT DESIGN

A representative sample was obtained and tested to determine the R-value. The material is thought to be typical and presumed to be representative of the existing soils within the subject site. Testing was performed in general accordance with the latest revisions to the Department of Transportation, State of California, Material & Research Test Method No. 301. The test results are presented in Appendix C.

The preliminary pavement sections presented in the following table are based on the R-value data obtained, assumed traffic indexes for the project, minimum pavement sections required by the County, and are in general conformance with the guidelines presented in the latest revision to the California Department of Transportation "Highway Design Manual" fifth edition. The following table presents the preliminary pavement sections.

RECOMMENDED MINIMUM PAVEMENT SECTIONS				
ROADWAY TYPE	T.I.	UNTREATED SUBGRADE R-VALUE	AC THICKNESS FEET (INCHES)	CLASS 2 BASE ROCK[1] THICKNESS FEET (INCHES)
Collector Street	7.0	10	0.33 (4.0)[2]	1.20 (14.0)
Local Street	5.5	10	0.25 (3.0)[2]	0.90 (10.8)

[1] Assumed R-values for base rock R=78 - Cal-Trans standard Class 2 base rock.
[2] Minimum required by the County of Riverside.

All pavement installation, including preparation and compaction of subgrade and placement and rolling of asphaltic concrete should be done in accordance with the City standard and under the observation and testing of the project geotechnical engineer and/or City.

The preliminary pavement sections provided above are intended as a minimum guideline. If thinner or highly variable pavement sections are constructed, or over-irrigation occurs, increased maintenance and repair should be expected. If the ADT or ADTT increases beyond that intended, as reflected by the T.I. used for design, increased maintenance and repair could be required for the pavement section. Consideration should be given to the increased potential for distress from overuse of paved street areas by heavy equipment and/or construction related heavy traffic (e.g., concrete trucks, loaded supply trucks, etc.), particularly when the final section is not in place (i.e., topcoat). Best management construction practices should be followed at all times, especially during inclement weather. Positive drainage should be maintained at all times, otherwise the subgrade will become wet or saturated, and may yield causing pavement and improvement distress.

PRELIMINARY PAVEMENT GRADING RECOMMENDATIONS

General

All section changes should be properly transitioned. If adverse conditions are encountered during the preparation of subgrade materials, special construction methods may need to be employed. A GSI representative should be present for the preparation of subgrade, aggregate base rock, and asphalt concrete.

Subgrade

Within driveway and parking areas, all surficial deposits of loose soil material should be removed and recompacted as recommended. After the loose soils are removed, the bottom is to be scarified to a depth of at least 12 inches, moisture conditioned as necessary and compacted to 95 percent of the maximum laboratory density or the County of Riverside minimum, as determined by ASTM test method D-1557.

Deleterious material, excessively wet or dry pockets, concentrated zones of oversized rock fragments, and any other unsuitable materials encountered during grading should be removed. The compacted fill material should then be brought to the elevation of the proposed subgrade for the pavement. The subgrade should be proof-rolled in order to ensure a uniform firm and unyielding surface. All grading and fill placement should be observed by the project soil engineer and/or his representative.

Aggregate Base Rock

Compaction tests are required for the recommended base section. Minimum relative compaction required will be 95 percent of the laboratory maximum density as determined by ASTM test designation D-1557. Base aggregate should be in accordance to the Caltrans Class 2 base rock (minimum R-value=78).

Paving

Prime coat may be omitted if all of the following conditions are met:

1. The asphalt pavement layer is placed within two weeks of completion of base and/or subbase course.

2. Traffic is not routed over completed base before paving.

3. Construction is completed during the dry season of May through October.

4. The base is kept free of debris prior to placement of asphaltic concrete.

If construction is performed during the wet season of November through April, prime coat may be omitted if no rain occurs between completion of base course and paving and the time between completion of base and paving is reduced to three days, provided the base is free of loose soil or debris. Where prime coat has been omitted and rain occurs, traffic is routed over base course, or paving is delayed, measures shall be taken to restore base course, and subgrade to conditions that will meet specifications as directed by the soil engineer.

Drainage

Positive drainage should be provided for all surface water to drain towards the area swale, curb and gutter, or to an approved drainage channel. Positive site drainage should be maintained at all times. Water should not be allowed to pond or seep into the ground. If planters or landscaping are proposed adjacent to paved areas, measures should be taken to minimize the potential for water to enter the pavement section, such as thickened edges, enclosed planters, etc. If thickened edges are not constructed, the potential for yielding subgrade and associated distress to improvements increases.

Additional Considerations

To mitigate perched groundwater and associated distress, consideration should be given to installation of subgrade separators (cut-offs) between pavement subgrade and landscape areas, although this is not a requirement from a geotechnical standpoint. Cut-offs, if used, should be 6 inches wide and at least 12 inches below the pavement subgrade contact or 12 inches below the aggregate base rock.

DEVELOPMENT CRITERIA

Slope Deformation

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

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

Slope Maintenance and Planting

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

Drainage

Adequate lot surface drainage is a very important factor in reducing the likelihood of adverse performance of foundations, hardscape, and slopes. Surface drainage should be sufficient to prevent ponding of water anywhere on a lot, and especially near structures and tops of slopes. Lot surface drainage should be carefully taken into consideration during fine grading, landscaping, and building construction. Therefore, care should be taken that future landscaping or construction activities do not create adverse drainage conditions.

Positive site drainage within lots and common areas should be provided and maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond and/or seep into the ground. In general, the area within 5 feet around a structure should slope away from the structure. We recommend that unpaved lawn and landscape areas have a minimum gradient of 1 percent sloping away from structures, and whenever possible, should be above adjacent paved areas. Consideration should be given to avoiding construction of planters adjacent to structures (buildings, pools, spas, etc.). Pad drainage should be directed toward the street or other approved area(s). Although not a geotechnical requirement, roof gutters, down spouts, or other appropriate means may be utilized to control roof drainage. Down spouts, or drainage devices should outlet a minimum of 5 feet from structures or into a subsurface drainage system. Areas of seepage may develop due to irrigation or heavy rainfall, and should be anticipated. Minimizing irrigation will lessen this potential. If areas of seepage develop, recommendations for minimizing this effect could be provided upon request.

Toe of Slope Drains/Toe Drains

Where significant slopes intersect pad areas, surface drainage down the slope allows for some seepage into the subsurface materials, sometimes creating conditions causing or contributing to perched and/or ponded water. Toe of slope/toe drains may be beneficial in the mitigation of this condition due to surface drainage. The general criteria to be utilized by the design engineer for evaluating the need for this type of drain is as follows:

- Is there a source of irrigation above or on the slope that could contribute to saturation of soil at the base of the slope?
- Are the slopes hard rock and/or impermeable, or relatively permeable, or; do the slopes already have or are they proposed to have subdrains (i.e., stabilization fills, etc.)?
- Are there cut-fill transitions (i.e., fill over native materials), within the slope?
- Was the lot at the base of the slope overexcavated or is it proposed to be overexcavated? Overexcavated lots located at the base of a slope could accumulate subsurface water along the base of the fill cap.
- Are the slopes north facing? North facing slopes tend to receive less sunlight (less evaporation) relative to south facing slopes and are more exposed to the currently prevailing seasonal storm tracks.
- What is the slope height? It has been our experience that slopes with heights in excess of approximately 10 feet tend to have more problems due to storm runoff and irrigation than slopes of a lesser height.

- Do the slopes "toe out" into a lot or a lot where perched or ponded water may adversely impact its proposed use?

Based on these general criteria, the construction of toe drains may be considered by the design engineer along the toe of slopes, or at retaining walls in slopes, descending to the rear of such lots. Following are Detail 4 (Schematic Toe Drain Detail) and Detail 5 (Subdrain Along Retaining Wall Detail). Other drains may be warranted due to unforeseen conditions, owner irrigation, or other circumstances. Where drains are constructed during grading, including subdrains, the locations/elevations of such drains should be surveyed, and recorded on the final as-built grading plans by the design engineer. It is recommended that the above be disclosed to all interested parties, including owners and any owners association.

Erosion Control

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

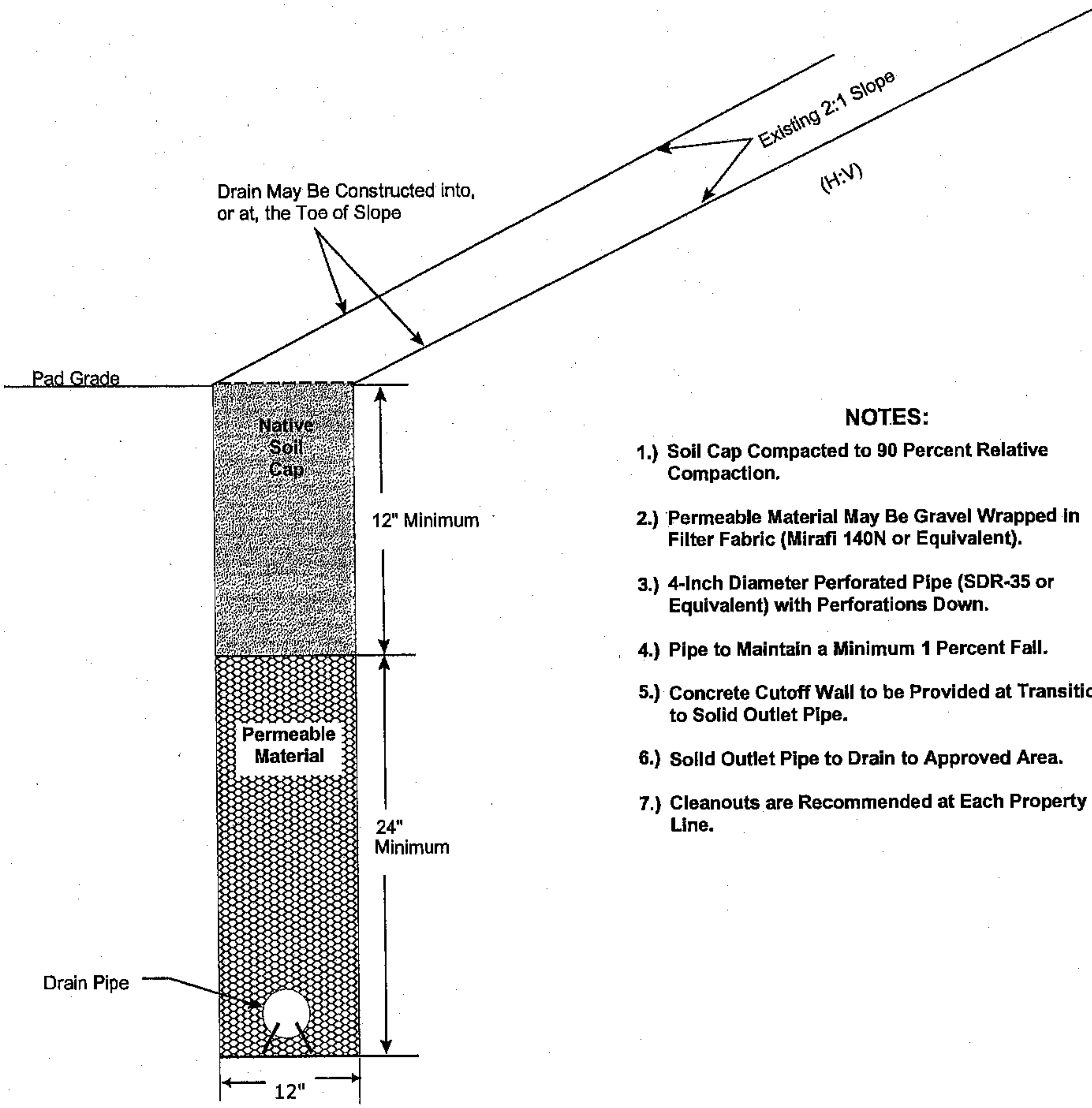
Landscape Maintenance

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

DETAILS

N . T . S .

SCHEMATIC TOE DRAIN DETAIL



NOTES:

1.) **Soil Cap Compacted to 90 Percent Relative Compaction.**

2.) **Permeable Material May Be Gravel Wrapped in Filter Fabric (Mirafi 140N or Equivalent).**

3.) **4-Inch Diameter Perforated Pipe (SDR-35 or Equivalent) with Perforations Down.**

4.) **Pipe to Maintain a Minimum 1 Percent Fall.**

5.) **Concrete Cutoff Wall to be Provided at Transition to Solid Outlet Pipe.**

6.) **Solid Outlet Pipe to Drain to Approved Area.**

7.) **Cleanouts are Recommended at Each Property Line.**

GeoSoils, Inc.

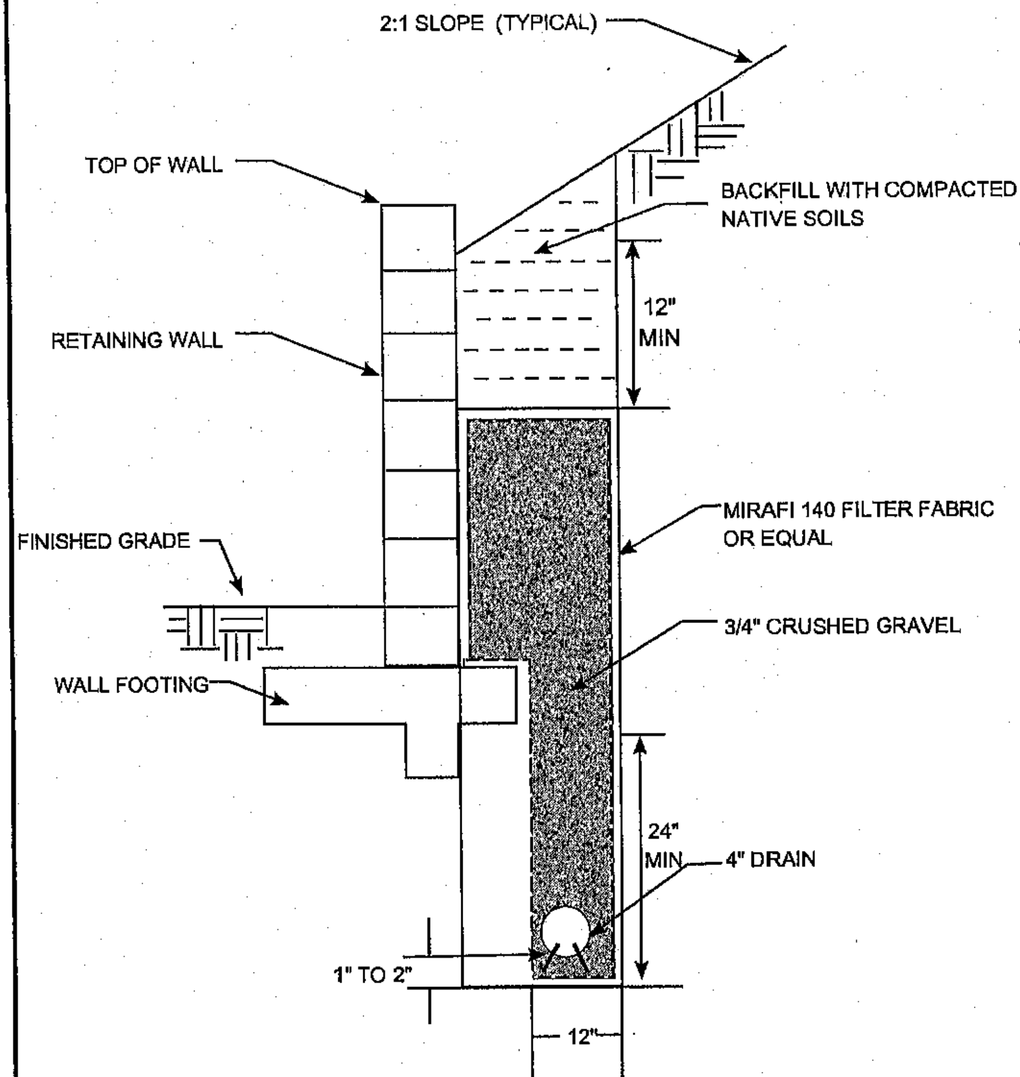
SCHEMATIC TOE DRAIN DETAIL

DETAIL 4

Geotechnical • Coastal • Geologic • Environmental

DETAILS

N . T . S .



NOTES:

1.) Soil Cap Compacted to 90 Percent Relative Compaction.

2.) Permeable Material May Be Gravel Wrapped in Filter Fabric (Mirafi 140N or Equivalent).

3.) 4-Inch Diameter Perforated Pipe (SDR-35 or Equivalent) with Perforations Down.

4.) Pipe to Maintain a Minimum 1 Percent Fall.

5.) Concrete Cutoff Wall to be Provided at Transition to Solid Outlet Pipe.

6.) Solid Outlet Pipe to Drain to Approved Area.

7.) Cleanouts are Recommended at Each Property Line.

8.) Compacted Effort Should Be Applied to Drain Rock.

SUBDRAIN ALONG RETAINING WALL DETAIL

NOT TO SCALE

GSI GeoSoils, Inc.

SUBDRAIN ALONG RETAINING WALL DETAIL

DETAIL 5

Geotechnical • Coastal • Geologic • Environmental

Gutters and Downspouts

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

Subsurface and Surface Water

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

Site Improvements

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

Tile Flooring

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

Additional Grading

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

Footing Trench Excavation

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

Trenching/Temporary Construction Backcuts

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

Utility Trench Backfill

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

2. Exterior trenches adjacent to, and within areas extending below a 1:1 plane projected from the outside bottom edge of the footing, and all trenches beneath hardscape features and in slopes, should be compacted to at least 90 percent of

the laboratory standard. Sand backfill, unless excavated from the trench, should not be used in these backfill areas. Compaction testing and observations, along with probing, should be accomplished to evaluate the desired results.

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

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

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING

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

- During grading/recertification.
- During excavation.
- During placement of subdrains, toe drains, or other subdrainage devices, prior to placing fill and/or backfill.
- After excavation of building footings, retaining wall footings, and free standing walls footings, prior to the placement of reinforcing steel or concrete.
- Prior to pouring any slabs or flatwork, after presoaking/presaturation of building pads and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (i.e., sand, pea-gravel, etc.), or vapor retarders (i.e., visqueen, etc.).
- During retaining wall subdrain installation, prior to backfill placement.
- During placement of backfill for area drain, interior plumbing, utility line trenches, and retaining wall backfill.
- During slope construction/repair.
- When any unusual soil conditions are encountered during any construction operations, subsequent to the issuance of this report.
- When any developer or owner improvements, such as flatwork, spas, pools, walls, etc., are constructed, prior to construction. GSI should review such plans prior to construction.

- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and clear documentation of site work, and/or to comply with code requirements.

- GSI should review project sales documents to owners/owners associations for geotechnical aspects, including irrigation practices, the conditions outlined above, etc., prior to any sales. At that stage, GSI will provide owners maintenance guidelines which should be incorporated into such documents.

OTHER DESIGN PROFESSIONALS/CONSULTANTS

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

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

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

PLAN REVIEW

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

LIMITATIONS

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

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

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

Blake, T.F., 2000a, EQFAULT, A computer program for the estimation of peak horizontal acceleration from 3-D fault sources; Windows 95/98 version.

_____, 2000b, EQSEARCH, A computer program for the estimation of peak horizontal acceleration from California historical earthquake catalogs; updated to June, 2006, Windows 95/98 version.

_____, 2000c, FRISKSP, A computer program for the probabilistic estimation of peak acceleration and uniform hazard spectra using 3-D faults as earthquake sources; Windows 95/98 version.

Bozorgnia, Y., Campbell, K.W., and Niazi, M., 1999, Vertical ground motion: Characteristics, relationship with horizontal component, and building-code implications; Proceedings of the SMIP99 seminar on utilization of strong-motion data, September, 15, Oakland, pp. 23-49.

California Department of Conservation, Division of Mines and Geology, 1996, Probabilistic seismic hazard assessment for the state of California, DMG Open-File Report 96-08.

California Department of Conservation, Division of Mines and Geology, 1997, Guidelines for evaluation and mitigating seismic hazards in California, CDMG Special Publication 117.

California Department of Transportation (Caltrans), 1995, Highway design manual, fifth edition (metric).

California Department of Water Resources, 2005, Water Data Library (www.well.water.ca.gov/).

Campbell, K.W. and Bozorgnia, Y., 1997, Attenuation relations for soft rock conditions; in EQFAULT, A computer program for the estimation of peak horizontal acceleration from 3-D fault sources; Windows 95/98 version, Blake, 2000a.

_____, 1994, Near-source attenuation of peak horizontal acceleration from worldwide accelerograms recorded from 1957 to 1993; proceedings, Fifth U.S. National Conference on Earthquake Engineering, Vol. III, Earthquake Engineering Research Institute, pp. 283-292.

Chen, Y., Chen, W., Wang, Y., Lo, T., Lui, T., and Lee, J., 2002, Geomorphic evidence for prior earthquakes: lessons from the 1999 Chichi earthquake in central Taiwan, *in* Geological Society of America, Geology, v. 30, no. 2. pp. 171-174.

County of Riverside, 2000, Technical guidelines for review of geotechnical and geologic reports.

CTL Thompson, 2005, Controlling moisture-related problems associated with basement slabs-on-grade in new residential construction.

Das, B. M., 1993, Principals of soil dynamics, Southern Illinois University at Carbondale, PWS-Kent Publishing Company, Boston.

Envicom Corporation, 1976, Seismic safety and safety elements technical report for the county of Riverside and the cities of Beaumont, Blythe, Coachella, Corona, Desert Hot Springs, Hemet, Indian Wells, Indio, Norco, Perris, Riverside, and San Jacinto.

Griffiths, D. H. and King, R. F., 1965, Applied geophysics for engineers and geologists, Pergamon Press, reprinted 1976.

Hart, E.W. and Bryant, W.A., 1997, Fault-rupture hazard zones in California, Alquist-Priolo earthquake fault zoning act with index to earthquake fault zones maps; California Division of Mines and Geology Special Publication 42, with Supplements 1 and 2, 1999.

International Conference of Building Officials, 2001, California building code, California code of regulations title 24, part 2, volume 1 and 2.

_____, 1998, Maps of known active fault near-source zones in California and adjacent portions of Nevada.

_____, 1997, Uniform building code: Whittier, California, vol. 1, 2, and 3.

Jennings, C.W., 1994, Fault activity map of California and adjacent areas, scale 1:750,000, California Division of Mines and Geology, California Data Map Series, Map No. 6.

Morris Designs, undated, Site Plan, Paseo Verde Development.

Morton, D.M., Bovard, K.R., and Morton, G., 2003, Geologic map of the Romoland 7.5-minute Quadrangle, Riverside County, California, USGS Open File Report OF-03-102.

Obermeier, S.F., 1996, Using liquefaction-induced features for paleoseismic analysis, Chapter 7, in Paleoseismology, McCalpin, J.P., ed., Academic Press, Inc., San Diego, California.

Southern California Earthquake Center, 1999, Recommended procedures for implementation of DMG Special Publication 117, guidelines for analyzing and mitigating liquefaction in California, pp. 28-32, dated March.

State of California, 2006, Civil Code, Sections 895 et seq.

United States Department of Agriculture, 1980, Aerial photographs, Project no. 615020, flight date August 10, Flight line 180, photos nos. 84, 85, and 86.

Zeiser Kling Consultants, Inc., 2004, Preliminary geotechnical investigation, Sun City multi-family residential development, Sun City, California, P.N. 04020-00, dated March 29.

APPENDIX B

BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

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

CONSISTENCY OR RELATIVE DENSITY

CRITERIA

Standard Penetration Test

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

Standard Penetration Test

Penetration Resistance N (blows/ft)	Consistency	Unconfined Compressive Strength (tons/ft²)
<2	Very Soft	<0.25
2 - 4	Soft	0.25 - .050
4 - 8	Medium	0.50 - 1.00
8 - 15	Stiff	1.00 - 2.00
15 - 30	Very Stiff	2.00 - 4.00
>30	Hard	>4.00

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

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

MOISTURE CONDITIONS

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

MATERIAL QUANTITY

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

OTHER SYMBOLS

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

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

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

BORING LOG

GeoSoils, Inc.

W.O. 5431-A-SC

PROJECT: WOMBLE
±15 Acres, Sun City

BORING B-1 SHEET 1 OF 2

DATE EXCAVATED 4-18-07

SAMPLE METHOD: Mod Cal Sampler & Spt, CME 75 HSA Rig

Approx. Elevation: 1,420' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
0				SM				**TOPSOIL/ALLUVIUM:** @ 0' SILTY SAND, brown, dry to damp, loose; fine.
3		Ring	53	CL	88.0	13.7	41	**PLEISTOCENE-AGE FAN DEPOSITS:** @ 3' CLAY, brownish gray, damp, hard; trace fine sand, carbonate development.
6		Ring	32	SP		1.2		@ 6' SAND, light brown, dry, medium dense; fine to coarse, trace silt.
9		Ring	57	SM	110.7	4.0	22	@ 9' SILTY SAND, brown, dry, dense; some sand layers.
12		Ring	47	SP	105.8	3.7	17	@ 12' SAND, light gray, dry, dense; fine, trace silt.
15		SPT	18			4.6		@ 15' As per 12', medium dense.
20		Ring	62		110.7	3.0	16	@ 20' As per 15', red, dense; fine to coarse.
25		SPT	7	CL		26.1		@ 25' SANDY CLAY, brownish gray, wet, medium stiff.

BORING LOG

GeoSoils, Inc.

W.O. 5431-A-SC

PROJECT: WOMBLE
±15 Acres, Sun City

BORING B-1 SHEET 2 OF 2

DATE EXCAVATED 4-18-07

SAMPLE METHOD: Mod Cal Sampler & Spt, CME 75 HSA Rig

Approx. Elevation: 1,420' MSL

Standard Penetration Test (cross-hatched)

Undisturbed, Ring Sample (hatched)

∇ *Groundwater*

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
30		Ring	76	SP	116.3	3.5	22	@ 30' SAND, light brown, dry, very dense; fine to coarse, trace silt.
35		SPT	33			4.1		@ 35' As per 30', dense.
39 ∇								@ 39' Perched water encountered.
40		Ring	63		122.1	11.8	88	@ 40' As per 35', wet; minor clay.
45		SPT	75			15.6		@ 45' As per 40', saturated, very dense; trace clay.
50		Ring	40/ 50-5"		108.2	19.4	97	@ 50' As per 45', some clay.
								Total Depth = 51' Groundwater Encountered @ 39' Backfilled 4-18-07 with Cuttings
55								

BORING LOG

GeoSoils, Inc.

W.O. 5431-A-SC

PROJECT: WOMBLE
±15 Acres, Sun City

BORING B-2 *SHEET* 1 *OF* 2

DATE EXCAVATED 4-18-07

SAMPLE METHOD: Mod Cal Sampler & Spt, CME 75 HSA Rig

Approx. Elevation: 1,422' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
0	Bulk			ML				**TOPSOIL/ALLUVIUM:** @ 0' SANDY SILT, brown, damp, medium stiff; fine.
3	Bulk	Ring	32	CL	83.4	17.0	46	**PLEISTOCENE-AGE FAN DEPOSITS:** @ 3' SANDY CLAY, brown, damp, very stiff; fine.
6		Ring	57	SP	117.0	4.5	29	@ 6' SAND, light red, damp, dense; fine to coarse, some silt layers.
9		Ring	38		106.1	1.9	9	@ 9' As per 6', dry; trace silt.
12		SPT	12			3.5		@ 12' As per 9', light gray, medium dense.
15		Ring	35		106.2	5.2	25	@ 15' As per 12', some silt.
20		SPT	19			1.9		@ 20' As per 9'.
25		Ring	30	ML	109.9	16.9	89	@ 25' SANDY SILT, brown, wet, very stiff; fine.

GeoSoils, Inc.

BORING LOG

W.O. 5431-A-SC

PROJECT: WOMBLE
±15 Acres, Sun City

BORING B-2 *SHEET* 2 *OF* 2

DATE EXCAVATED 4-18-07

SAMPLE METHOD: Mod Cal Sampler & Spt, CME 75 HSA Rig

Approx. Elevation: 1,422' MSL

Standard Penetration Test

Undisturbed, Ring Sample

∇ *Groundwater*

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
			28	SP/ML		7.9		@ 30' SAND w/SILT layers, light brownish gray, damp, medium dense/very stiff; fine.
								Total Depth = 31½' No Groundwater Encountered Backfilled 4-18-2007 with Cuttings
35								
40								
45								
50								
55								

GeoSoils, Inc.

BORING LOG

W.O. 5431-A-SC

PROJECT: WOMBLE
±15 Acres, Sun City

BORING B-3 *SHEET* 1 *OF* 2

DATE EXCAVATED 4-18-07

SAMPLE METHOD: Mod Cal Sampler & Spt, CME 75 HSA Rig

Approx. Elevation: 1,421' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
0				CL				**TOPSOIL/ALLUVIUM:** @ 0' SANDY CLAY, dark brown, damp, loose; fine.
2		Ring	22		89.7	15.1	47	@ 2' As per 0', wet.
3				CL				**PLEISTOCENE-AGE FAN DEPOSITS:** @ 3' SILTY CLAY, dark brown, damp, stiff; trace fine sand.
5		Ring	29		82.3	33.3	87	@ 5' As per 3', wet, very stiff.
7		Ring	50		107.9	20.1	100	@ 7' As per 5', saturated, hard.
10		Ring	57		119.9	14.4	100	@ 10' As per 7', trace fine to coarse sand, some carbonate.
15		SPT	21			28.1		@ 15' As per 10', very stiff.
20		Ring	26		99.8	22.8	92	@ 20' As per 15', wet.
25		SPT	12	SC		16.8		@ 25' CLAYEY SAND, reddish brown, moist, medium dense; fine to medium.

BORING LOG

GeoSoils, Inc.

W.O. 5431-A-SC

PROJECT: WOMBLE
±15 Acres, Sun City

BORING B-3 *SHEET* 2 *OF* 2

DATE EXCAVATED 4-18-07

SAMPLE METHOD: Mod Cal Sampler & Spt, CME 75 HSA Rig

Approx. Elevation: 1,421' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)		Description of Material
		▨	85	CL	98.0	26.0	100	▨	@ 30' SILTY CLAY, brownish gray, saturated, hard, trace sand.
									Total Depth = 31' No Groundwater Encountered Backfilled 4-18-2007 with Cuttings
35									
40									
45									
50									
55									

BORING LOG

GeoSoils, Inc.

W.O. 5431-A-SC

PROJECT: WOMBLE
±15 Acres, Sun City

BORING B-4 SHEET 1 OF 1

DATE EXCAVATED 4-18-07

SAMPLE METHOD: Mod Cal Sampler & Spt, CME 75 HSA Rig

Approx. Elevation: 1,417' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
0	Bulk			ML				**TOPSOIL/ALLUVIUM:** @ 0' SANDY SILT, light brown, dry, medium stiff; fine.
3	Bulk	Ring	70	ML	111.8	6.7	37	**PLEISTOCENE-AGE FAN DEPOSITS:** @ 3' SANDY SILT, light brown, damp, hard; fine.
6		Ring	69		119.8	13.0	90	@ 6' As per 3', reddish brown, wet; carbonate, trace clay.
9		Ring	35	SP	112.8	4.4	25	@ 9' SAND, red, dry, medium dense; fine to coarse, trace silt.
12		SPT	21			5.9		@ 12' As per 9'.
15		Ring	85		121.6	6.5	48	@ 15' As per 12', damp, very dense.
20		SPT	30			3.6		@ 20' As per 15', medium dense.

Total Depth = 21½'
No Groundwater Encountered
Backfilled 4-18-2007 with Cuttings

BORING LOG

GeoSoils, Inc.

W.O. 5431-A-SC

PROJECT: WOMBLE
±15 Acres, Sun City

BORING B-5 *SHEET* 1 *OF* 2

DATE EXCAVATED 4-18-07

SAMPLE METHOD: Mod Cal Sampler & Spt, CME 75 HSA Rig

Approx. Elevation: 1,419' MSL

Standard Penetration Test

Undisturbed, Ring Sample

∇ *Groundwater*

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
0				SM				**TOPSOIL/ALLUVIUM:** @ 0' SILTY SAND, red, dry, medium dense; fine.
2		Ring	37/ 50-5"	SM	113.0	9.9	57	**PLEISTOCENE-AGE FAN DEPOSITS:** @ 2' SILTY SAND, red, moist, dense to very dense; fine.
5		Ring	36	CL	113.8	16.2	95	@ 5' SANDY CLAY, reddish brown, wet, very stiff; fine.
7		Ring	34		111.7	16.3	90	@ 7' As per 5', light reddish brown.
10		Ring	16	SM	108.4	7.6	39	@ 10' SILTY SAND, brownish gray, damp, medium dense; fine to coarse.
15		SPT	9	CL		31.0		@ 15' SILTY CLAY, dark brown, moist, stiff.
20		Ring	13		94.7	25.1	89	@ 20' As per 15', wet.
25		SPT	18	SM		16.9		@ 25' SILTY SAND, brownish gray, moist, medium dense; very fine to fine.

BORING LOG

GeoSoils, Inc.

W.O. 5431-A-SC

PROJECT: WOMBLE
±15 Acres, Sun City

BORING B-5 *SHEET* 2 *OF* 2

DATE EXCAVATED 4-18-07

SAMPLE METHOD: Mod Cal Sampler & Spt, CME 75 HSA Rig

Approx. Elevation: 1,419' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
		SPT	17	SM		20.0		@ 30' As per 25'.
								Total Depth = 31½' No Groundwater Encountered Backfilled 4-18-2007 with Cuttings
35								
40								
45								
50								
55								

APPENDIX C

LABORATORY DATA

GeoSoils, Inc.

COMPACTION TEST

ASTM D 1557

Project Name: Womble — Date Tested: 4/24/07 — By: CH

Project Number: 5431-A-SC — Date Received: — By: DW

Boring Number: B-2 — Depth (ft.): 0-5

Sample Number: — Coarse Fraction (%): +#4: 0.0% +3/8": 0.0% +3/4": 0.0%

Sample Description: Dark Brown Sandy Clay

Preparation Method: [ ] Moist [X] Dry

Compaction Method: [X] Mechanical Rammer [ ] Manual Rammer

Mold Volume (ft.³): 4 inch 0.0333 — 6 inch 0.0750 — Rammer Weight: 10 lbs. — Drop: 18 inches

Water added (ml):	-50	0	50			
TEST NUMBER:	1	2	3	4	5	6
Weight of Soil and Mold (g)	3716	3813	3880			
Weight of Mold (g)	1858	1858	1858			
Weight of Soil (g)	1858	1955	2022			
Wet Soil and Tare (g)	200.0	200.0	200.0			
Dry Soil and Tare (g)	178.5	175.5	172.0			
Weight of Tare (g)	0.0	0.0	0.0			
Wet Density (pcf)	122.9	129.3	133.7			
Moisture Content (%)	12.0	14.0	16.3			
Dry Density (pcf)	109.7	113.5	115.0			

Maximum Dry Density (pcf) 116.0 — **Optimum Moisture Content (%)** 15.5

PROCEDURE

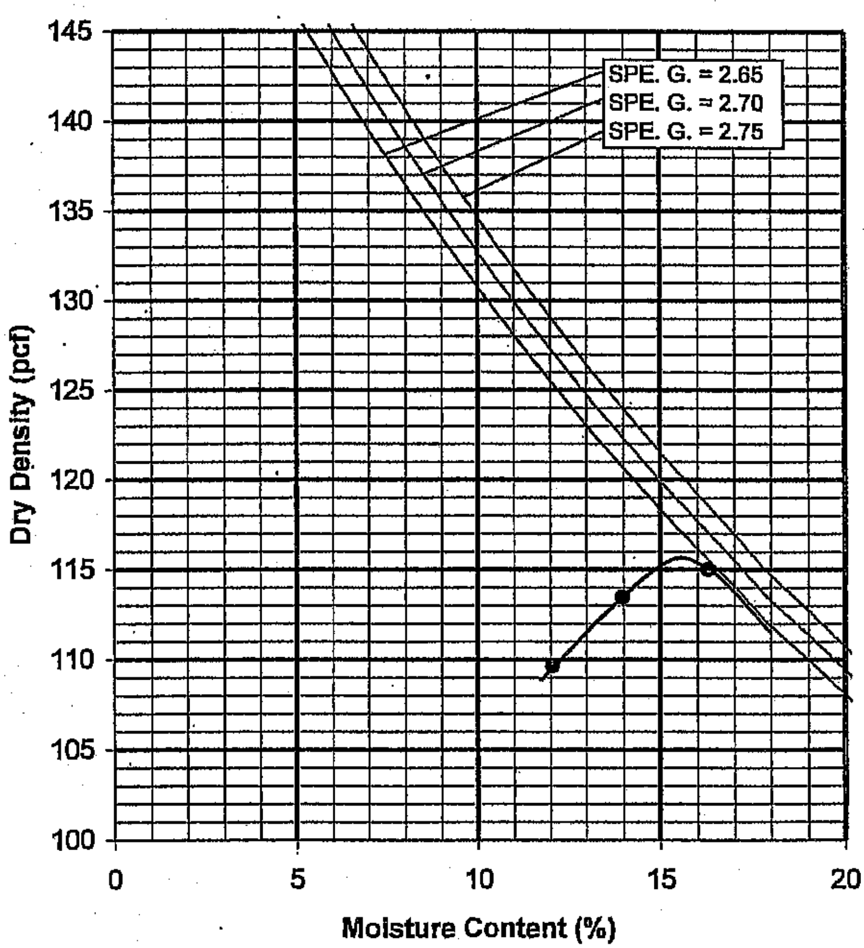
[X] **Procedure A**
Soil: Passing No. 4 (4.75mm) Sieve
Mold: 4 in. (101.6 mm) Diameter
Layers: 5 (five)
Blows per Layer: 25 (twenty-five)
May be used if 20% or less by weight of the material is retained on the No. 4 sieve.

[ ] **Procedure B**
Soil: Passing 3/8 in. (9.5 mm) Sieve
Mold: 4 in. (101.6 mm) Diameter
Layers: 5 (five)
Blows per Layer: 25 (twenty-five)
Shall be used if more than 20% by weight of the material is retained on the No. 4 sieve and 20% or less by weight is retained on the 3/8 in. sieve.

[ ] **Procedure C**
Soil: Passing 3/4 in. (19.0 mm) Sieve
Mold: 6 in. (152.4 mm) Diameter
Layers: 5 (five)
Blows per Layer: 56 (fifty-six)
Shall be used if more than 20% by weight of the material is retained on the 3/8 in. sieve and less than 30% by weight is retained on the 3/4 in. sieve.

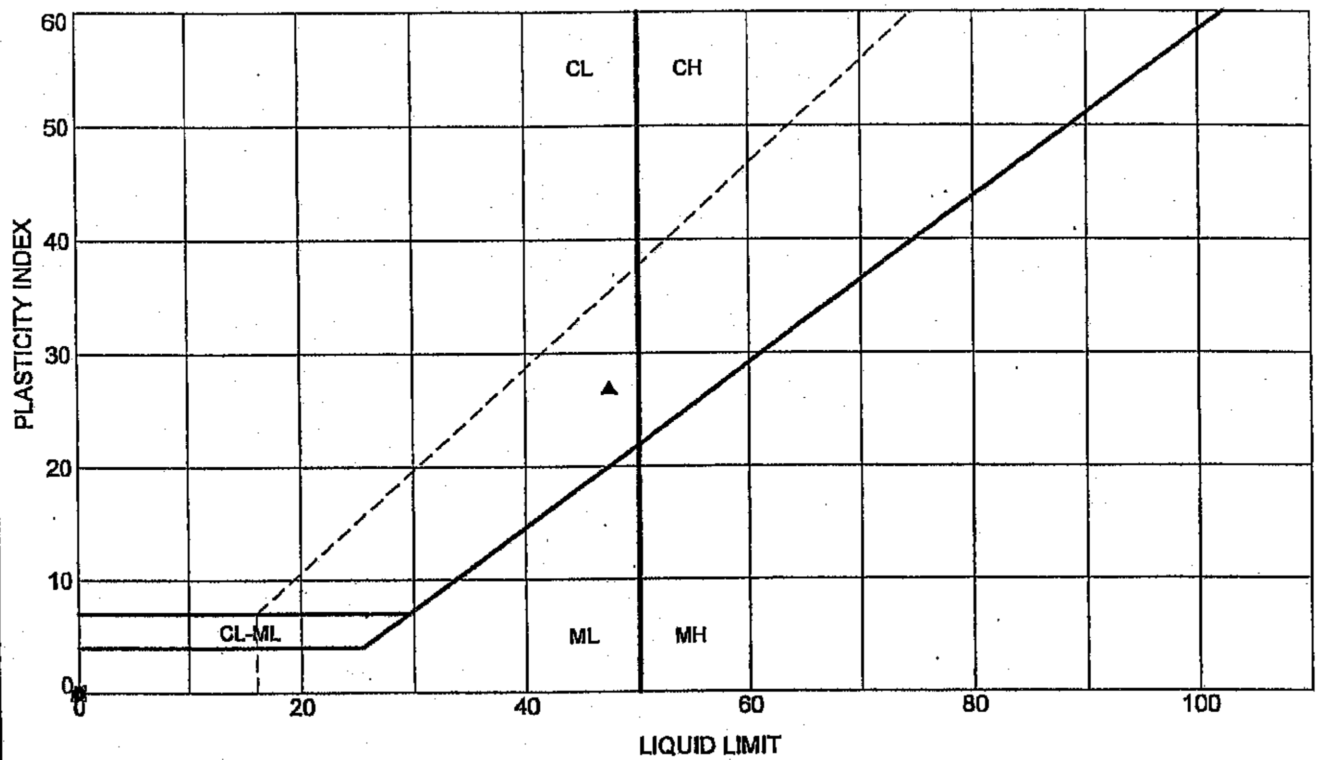
[ ] **Over 30% Retained on 3/4" Sieve**

Rock Correction needed: No



Z:\clint\[5431.xls]SHEAR — Rev. 01-05

Plate: C-1



	Sample	Depth/El.	LL	PL	PI	Fines	USCS CLASSIFICATION
●	B-1	30.0	NP	NP	NP	5	WELL-GRADED SAND with SILT(SW-SM)
☐	B-2	15.0	NP	NP	NP	9	Sand w/ Silt
▲	B-3	2.0	47	20	27		Clay w/ Sand

US ATTERBERG LIMITS 5431.GPJ US LAB.GDT 4/26/07

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

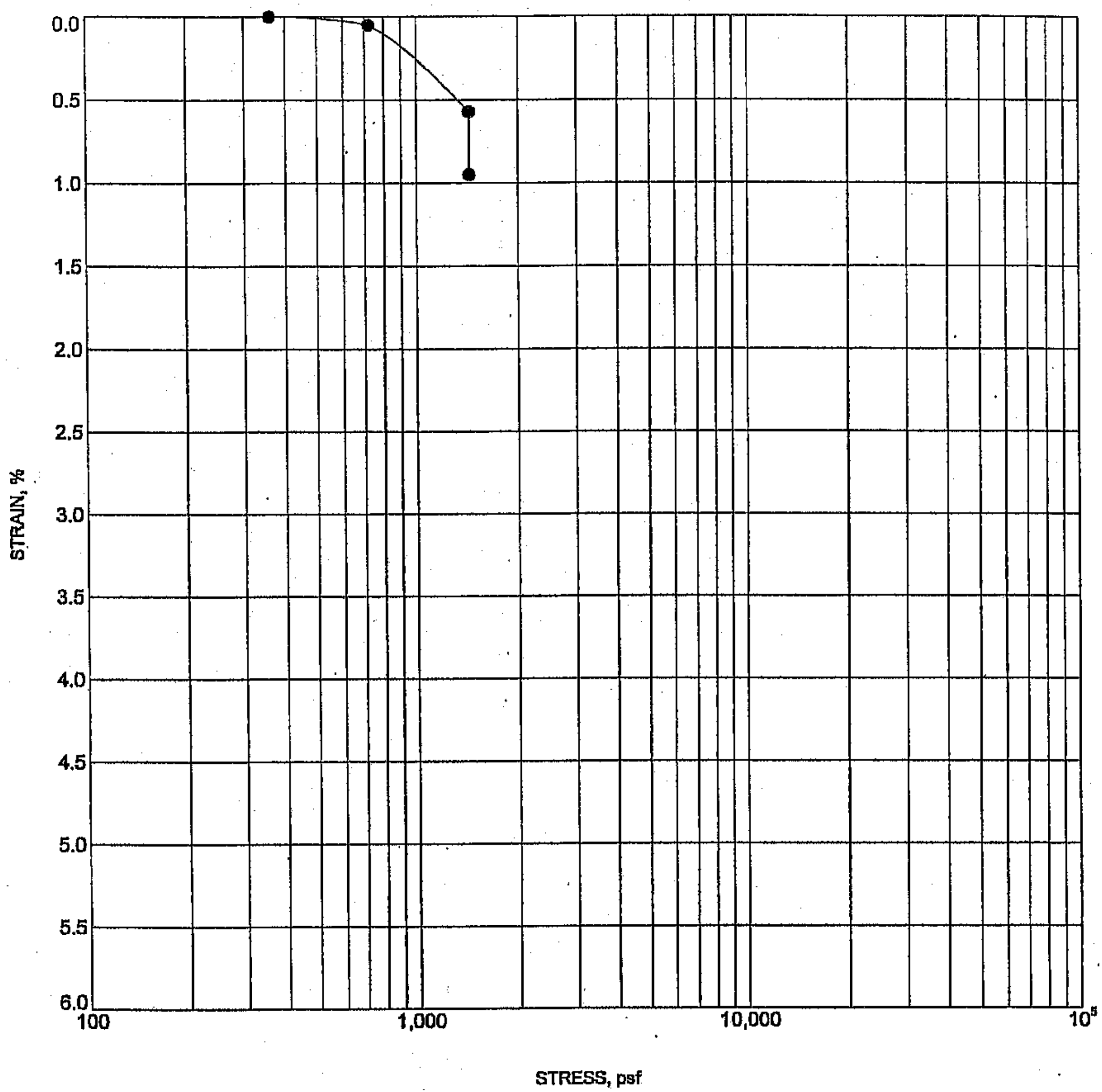
ATTERBERG LIMITS' RESULTS

Project: WOMBLE

Number: 5431-A-SC

Date: April 2007

Plate: C-2



	Sample	Depth/El.	Visual Classification	γ_d Initial	MC Initial	MC Final	H20
●	B-1	9.0	Sand w/ Silt	114.1	4.0	14.8	1440

Stress at which water was added: 1440 psf
Strain Difference: _____ 0.38%

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

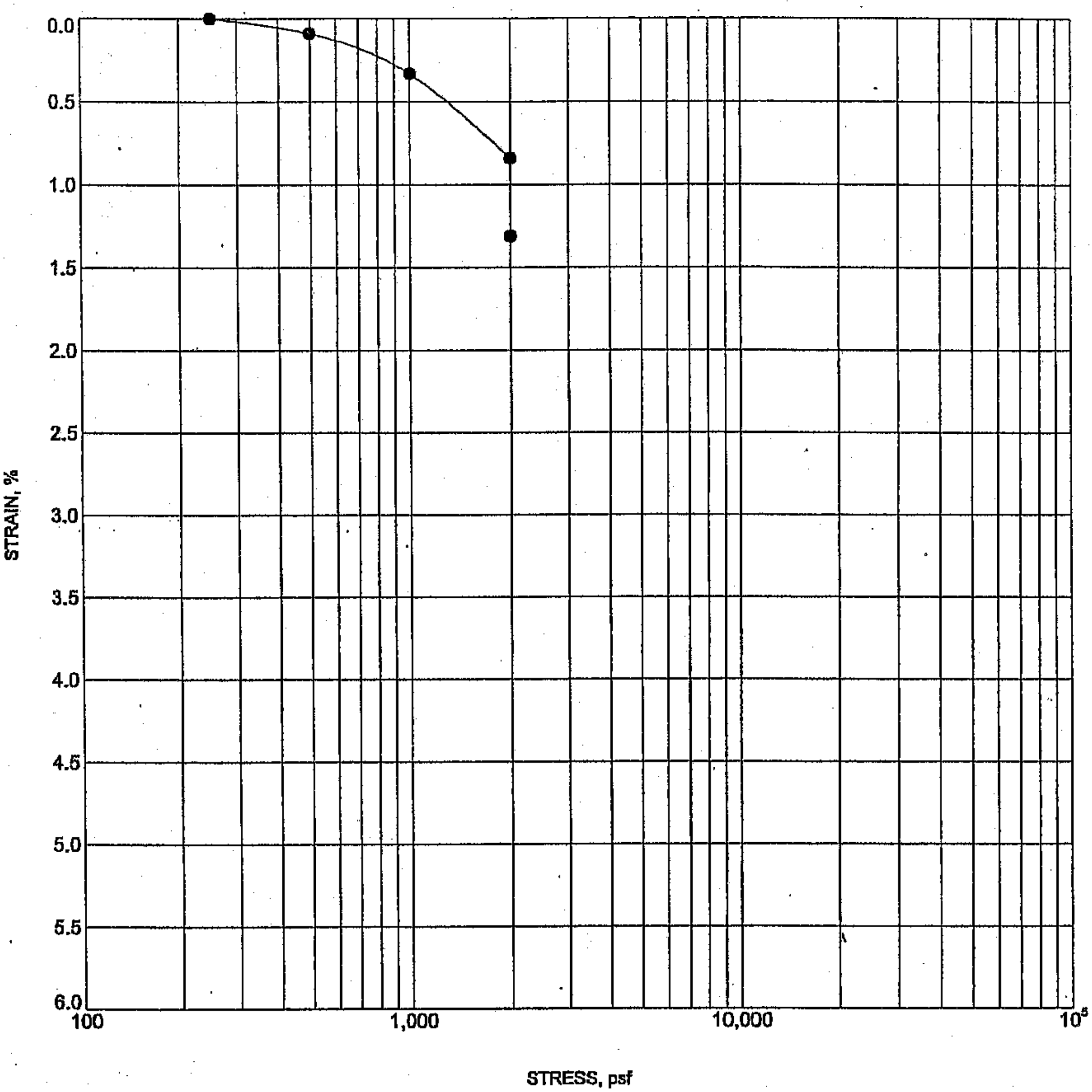
CONSOLIDATION TEST

Project: WOMBLE

Number: 5431-A-SC

Date: April 2007

Plate: C-3



	Sample	Depth/El.	Visual Classification	γ_d Initial	MC Initial	MC Final	H20
●	B-1	12.0	Sand w/ Silt	106.1	3.7	17.6	2000

Stress at which water was added: 2000 psf
Strain Difference: ______ 0.47%

US_CONSOL_STRAIN 5431.GPJ US_LAB.GDT 4/26/07

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

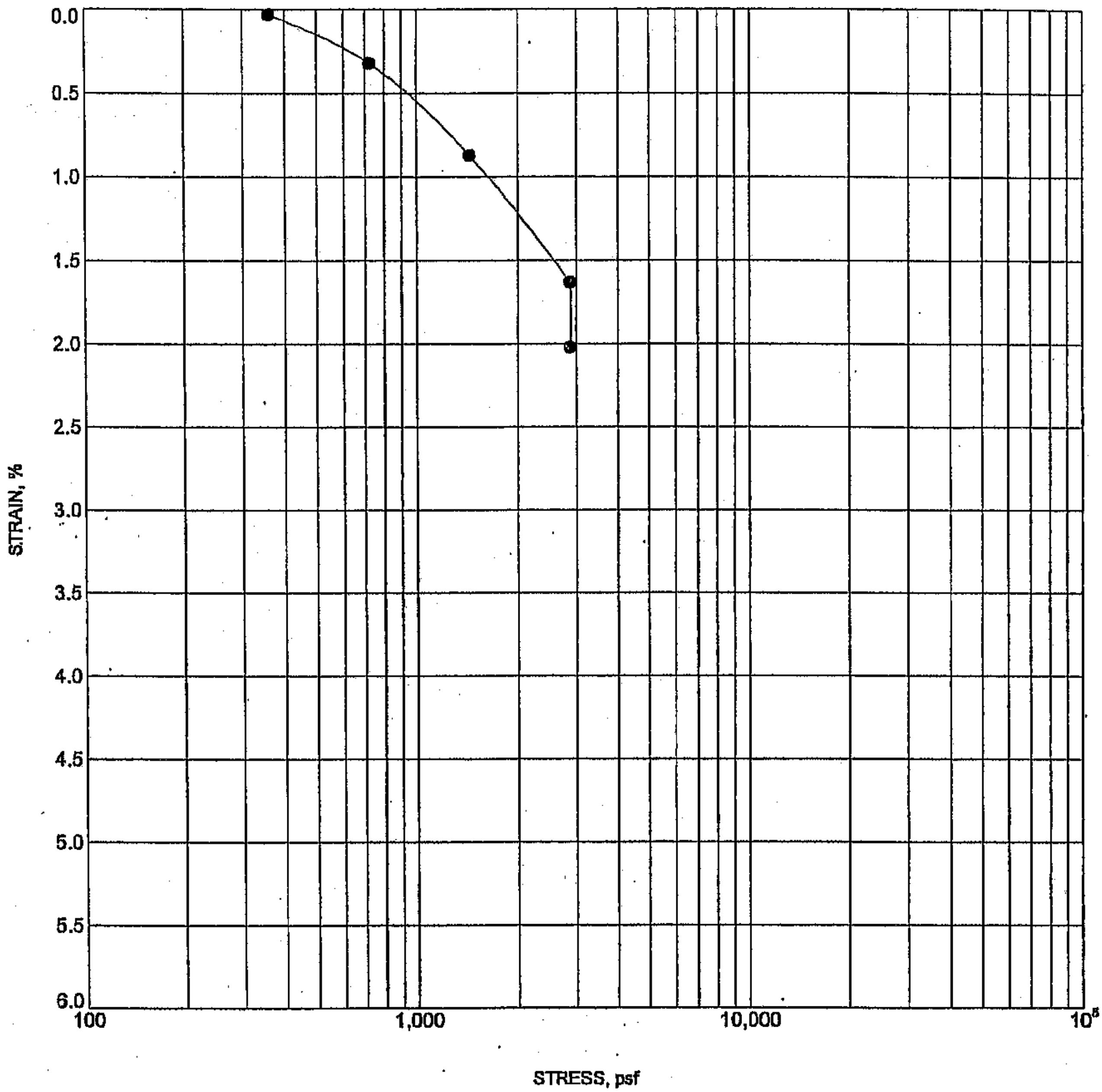
CONSOLIDATION TEST

Project: WOMBLE

Number: 5431-A-SC

Date: April 2007

Plate: C-4



	Sample	Depth/El.	Visual Classification	γ_d Initial	MC Initial	MC Final	H20
●	B-1	20.0	Sand w/ Silt	107.7	3.0	15.8	2880

Stress at which water was added: 2880 psf
Strain Difference: _____ 0.39%

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

CONSOLIDATION TEST

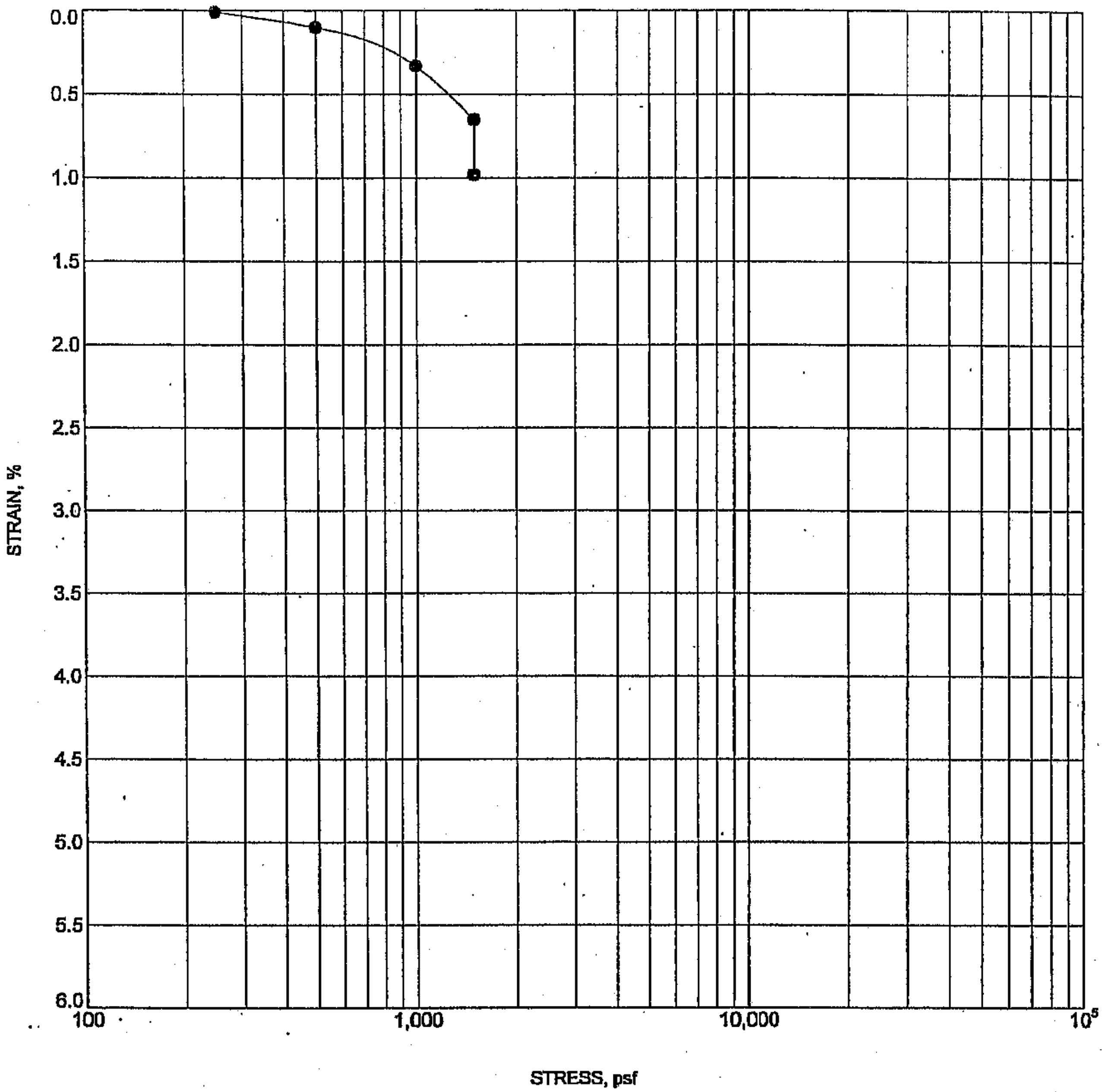
Project: WOMBLE

Number: 5431-A-SC

Date: April 2007

Plate: C-5

US CONSOL STRAIN 5431.GPJ US LAB.GDT 4/26/07



	Sample	Depth/El.	Visual Classification	γ_d Initial	MC Initial	MC Final	H20
●	B-2	9.0	Sand w/ Silt	105.5	1.9	14.1	1500

Stress at which water was added: 1500 psf
Strain Difference: _____ 0.33%

GeoSoils, Inc.
GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

CONSOLIDATION TEST

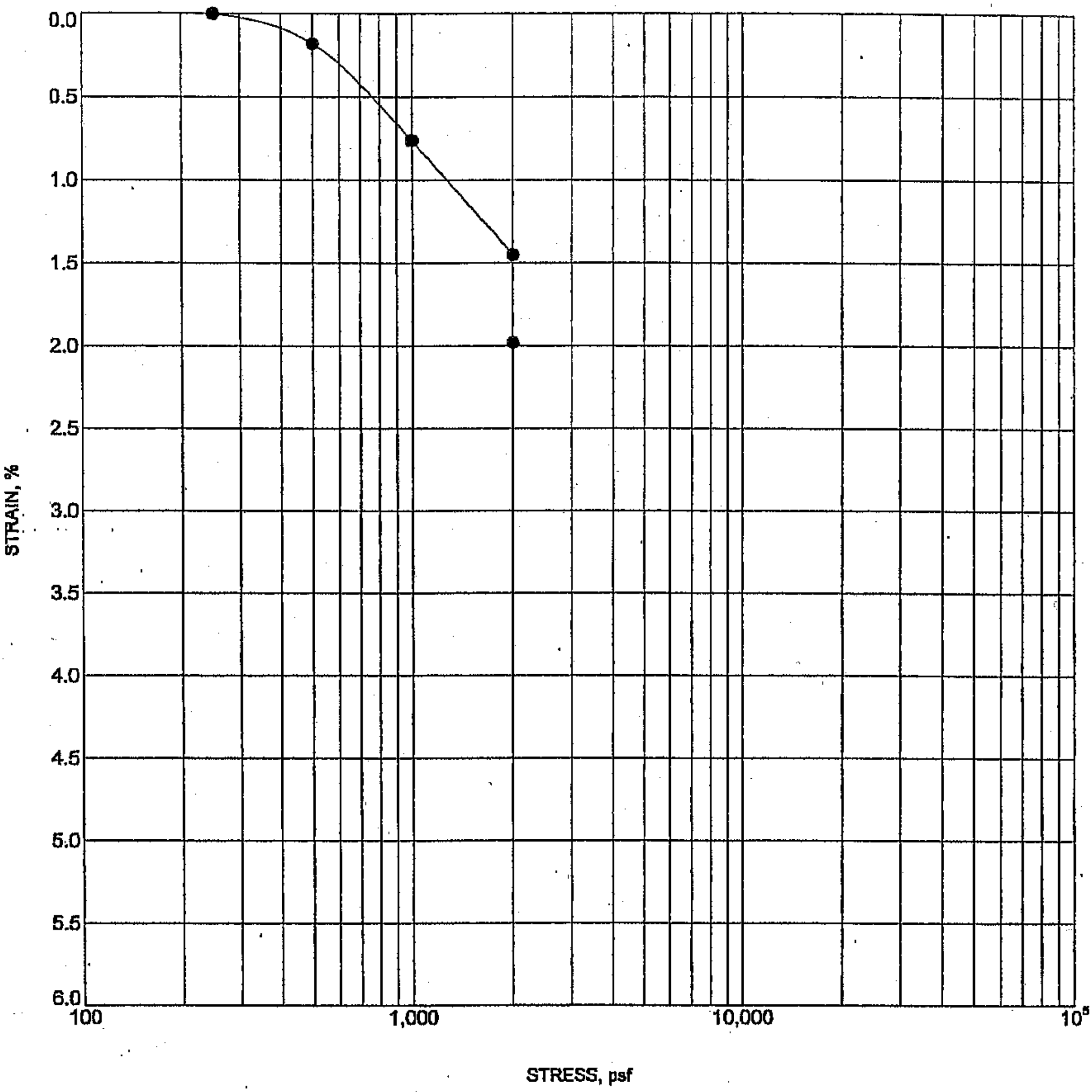
Project: WOMBLE

Number: 5431-A-SC

Date: April 2007

Plate: C-6

US_CONSOL_STRAIN 5431.GPJ US_LAB.GDT 4/26/07



	Sample	Depth/El.	Visual Classification	γ_d Initial	MC Initial	MC Final	H20
●	B-2	15.0	Sand w/ Silt	104.8	5.2	19.4	2000

Stress at which water was added: 2000 psf
Strain Difference: _____ 0.53%

US CONSOL STRAIN 5431.GPJ US LAB.GDT 4/26/07

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

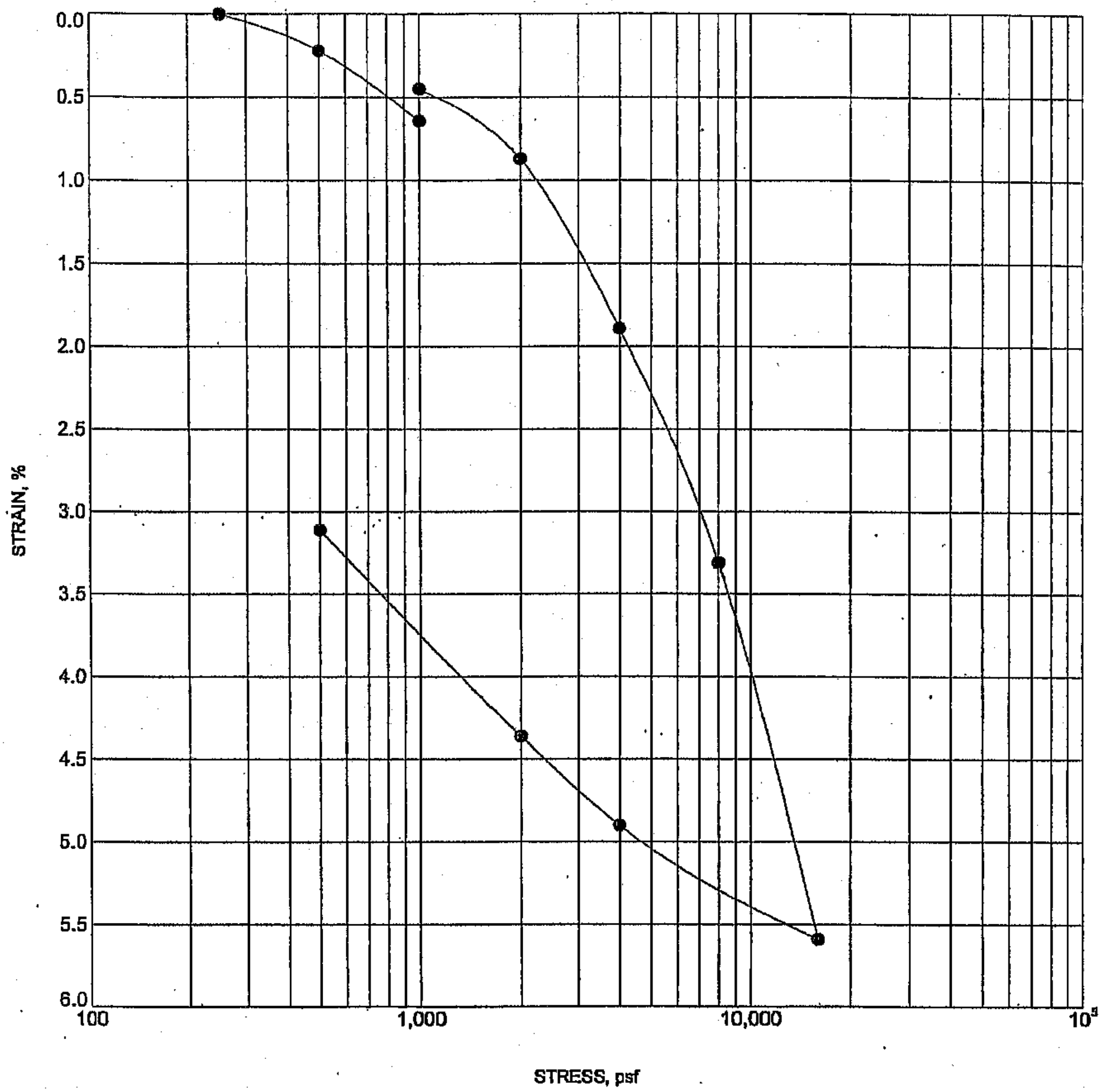
CONSOLIDATION TEST

Project: WOMBLE

Number: 5431-A-SC

Date: April 2007

Plate: C-7



	Sample	Depth/El.	Visual Classification	γ_d Initial	MC Initial	MC Final	H20
●	B-3	10.0	Sandy Clay	119.5	14.4	12.5	1000

Stress at which water was added: 1000 psf
Strain Difference: ____-0.19%

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

CONSOLIDATION TEST

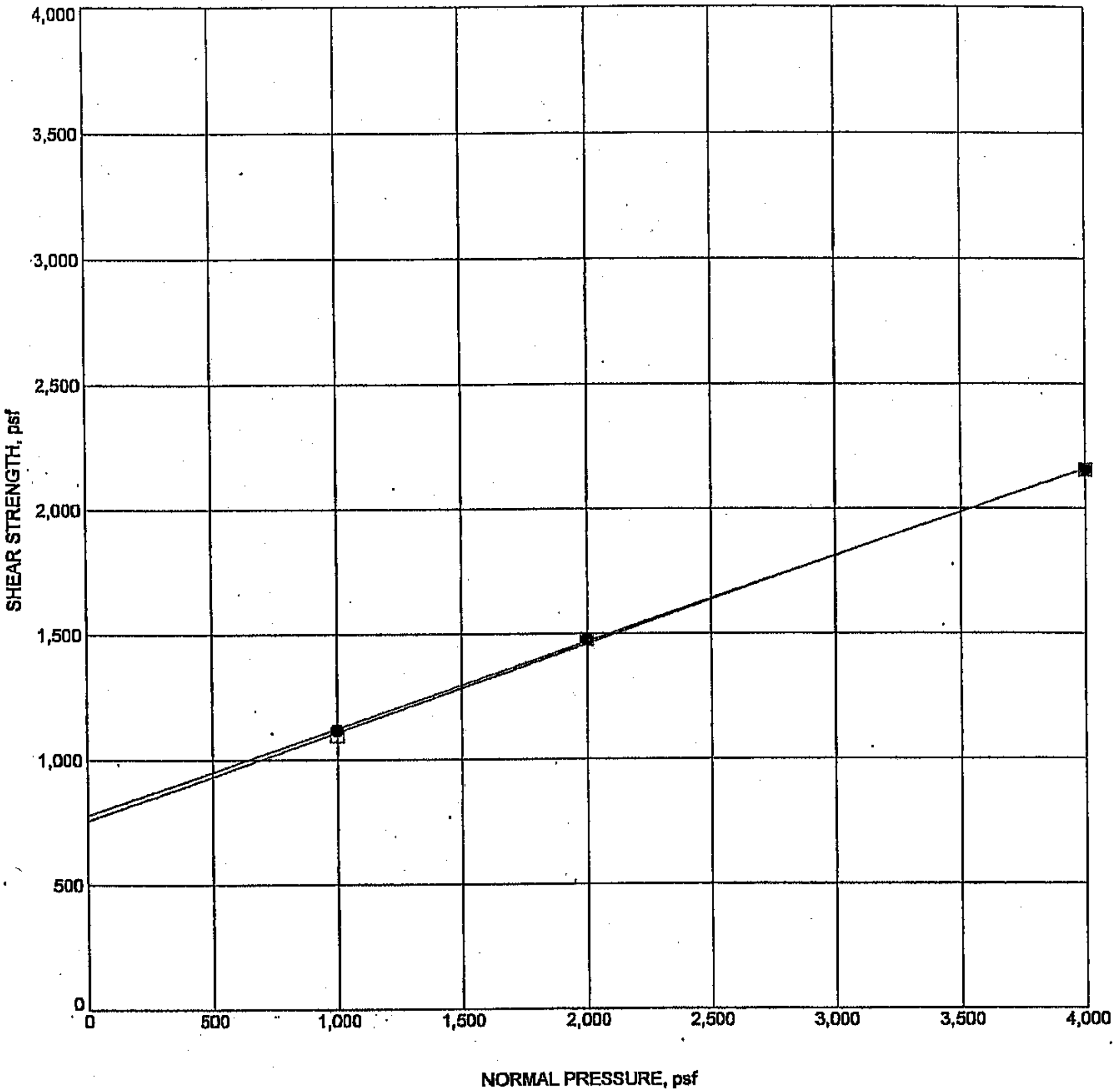
Project: WOMBLE

Number: 5431-A-SC

Date: April 2007

Plate: C-8

US CONSOL STRAIN 5431.GPJ US LAB.GDT 4/26/07



	Sample	Depth/El.	Range	Classification	Primary/Residual	Sample Type	γ_d	MC%	c	φ
●	B-2	0.0	0-5	Sandy Clay	Primary Shear	Remolded	104.4	15.5	779	19
□	B-2	0.0			Residual Shear	Remolded	104.4	15.5	757	19

Note: Sample Innundated prior to testing

US DIRECT SHEAR 5431.GPJ US LAB.GDT 4/26/07

GeoSoils, Inc.
GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

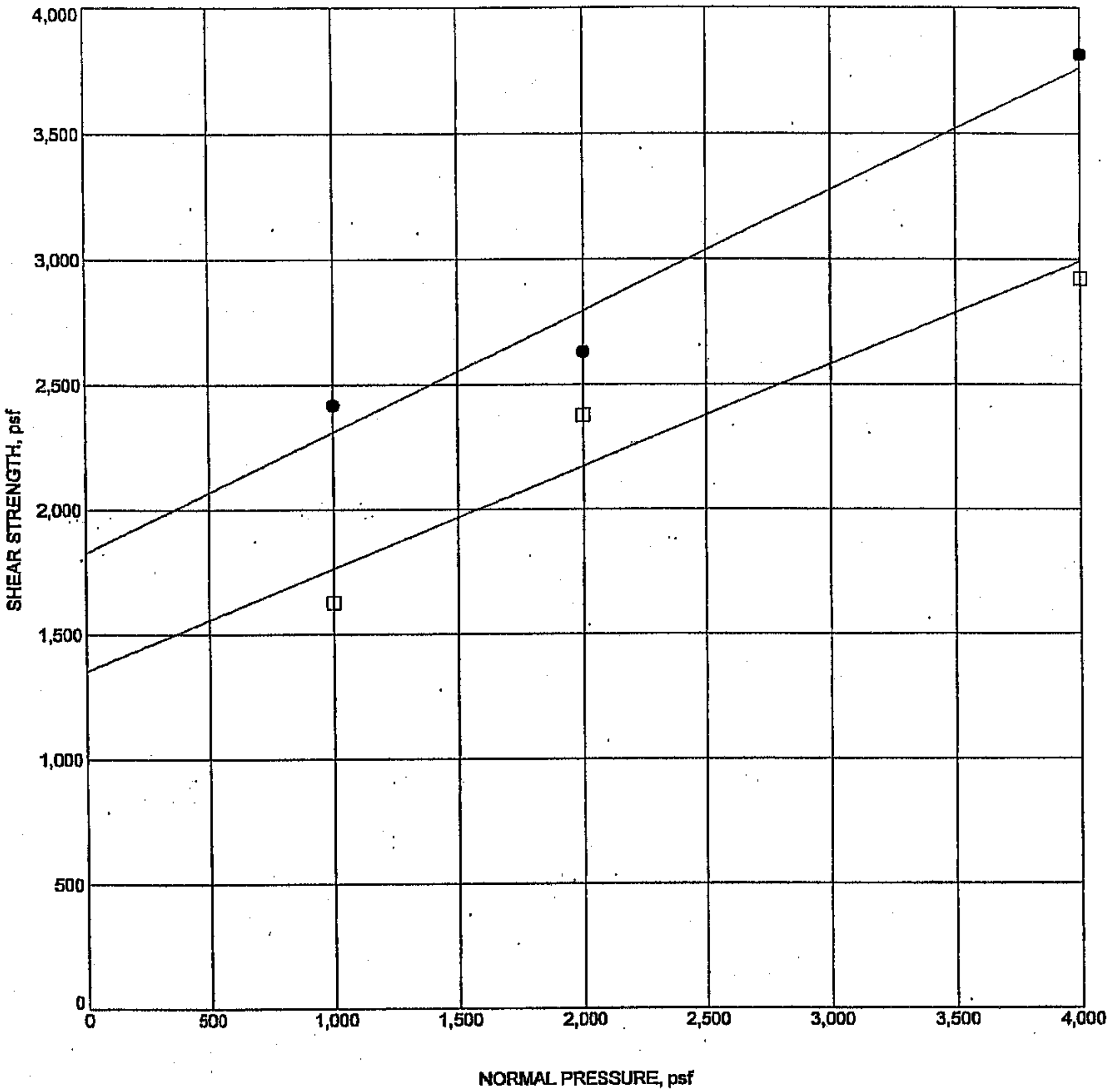
DIRECT SHEAR TEST

Project: WOMBLE

Number: 5431-A-SC

Date: April 2007

Plate: C-9



	Sample	Depth/El.	Range	Classification	Primary/Residual	Sample Type	γ_d	MC%	c	ϕ
●	B-5	5.0		Sandy Clay	Primary Shear	Undisturbed	112.9	16.2	1827	26
□	B-5	5.0			Residual Shear	Undisturbed	112.9	16.2	1353	22

Note: Sample Innundated prior to testing

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

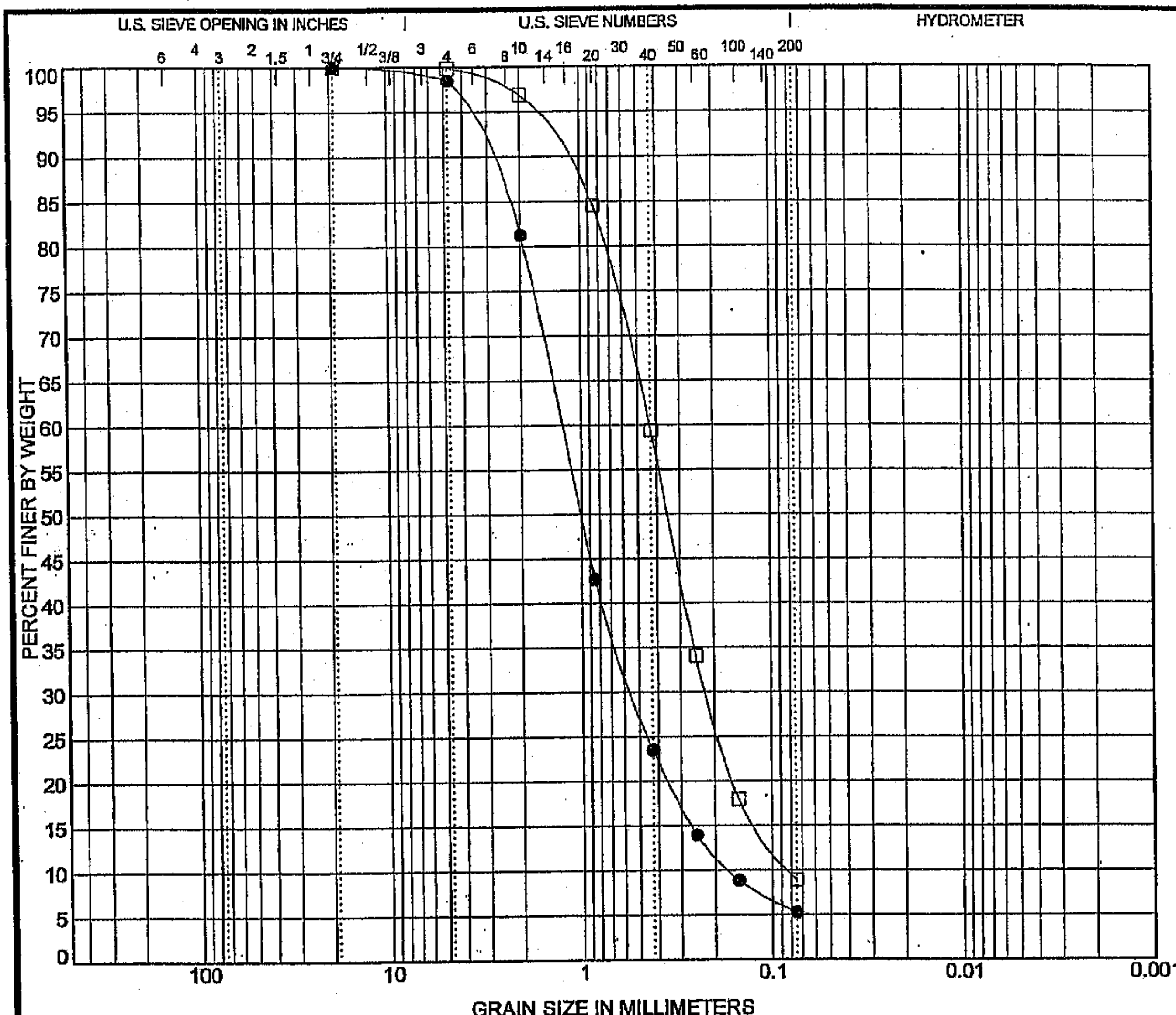
DIRECT SHEAR TEST

Project: WOMBLE

Number: 5431-A-SC

Date: April 2007

Plate: C-10



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth	Visual Classification/USCS CLASSIFICATION	LL	PL	PI	Cc	Cu
●	B-1	30.0	WELL-GRADED SAND with SILT(SW-SM)	NP	NP	NP	1.38	7.42
□	B-2	15.0	Sand w/ Silt	NP	NP	NP	1.37	5.30

	Sample	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1	30.0	19	1.248	0.537	0.168	1.5	93.2	5.3	
□	B-2	15.0	19	0.432	0.22	0.081	0.2	90.9	8.9	

US GRAIN SIZE 5431.GPJ US LAB.GDT 4/26/07

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

GRAIN SIZE DISTRIBUTION

Project: WOMBLE

Number: 5431-A-SC

Date: April 2007

Plate: C-11

TEST SPECIMEN		A	B	C	D
Compactor air pressure	PSI	250	110	50	
Water added	%	4.1	5.7	7.2	
Moisture at compaction	%	18.5	20.1	22.0	
Height of sample	IN	2.52	2.68	2.68	
Dry density	PCF	109.4	105.7	103.5	
R-Value by exudation		12	10	9	
R-Value by exudation, corrected		12	11	10	
Exudation pressure	PSI	690	355	277	
Stability thickness	FT	1.13	1.15	1.16	
Expansion pressure thickness	FT	2.30	1.23	0.43	

DESIGN CALCULATION DATA

Traffic index, assumed	5.0
Gravel equivalent factor, assumed	1.25
Expansion, stability equilibrium	1.15
R-Value by expansion	10
R-Value by exudation	10
R-Value at equilibrium	10

SAMPLE INFORMATION

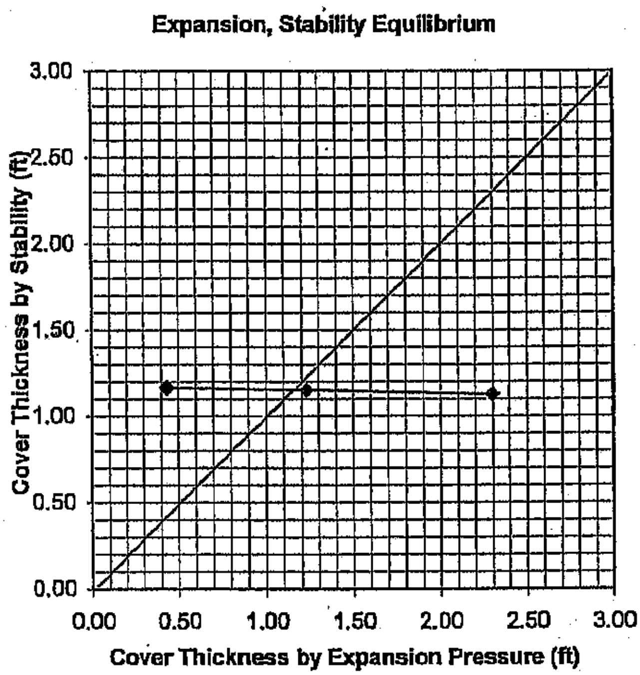
Sample Location: B-2 @ 0-5'

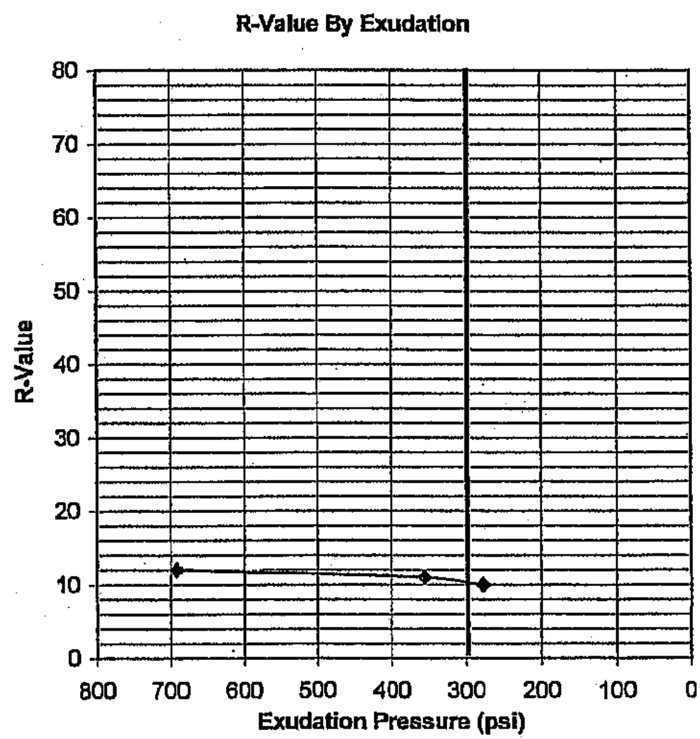
Sample Description: Brown Silty Clay

Notes:

0% Retained on 3/4 inch sieve

Test Method: Cal-Trans Test 301





GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

R - VALUE TEST RESULTS

Project: WOMBLE

Number: 5431-A-SC

Date: Apr-07

Plate: C-12

SCHIFF ASSOCIATES

www.schiffassociates.com
Consulting Corrosion Engineers – Since 1959

Table 1 - Laboratory Tests on Soil Samples

GenSoils, Inc.
Womble
Your #5431-A-SC, SA #07-0601LSD
26-Apr-07

Sample ID			B-1 @ 3-9 Mix
Resistivity		**Units**	
as-received		ohm-cm	48,000
saturated		ohm-cm	2,760
pH			8.0
Electrical Conductivity		mS/cm	0.16
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	93
magnesium	Mg^{2+}	mg/kg	10
sodium	Na^{1+}	mg/kg	87
potassium	K^{1+}	mg/kg	9.1
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	381
flouride	F^{1-}	mg/kg	4.8
chloride	Cl^{1-}	mg/kg	7.4
sulfate	SO_4^{2-}	mg/kg	53
phosphate	PO_4^{3-}	mg/kg	3.0
Other Tests			
ammonium	NH_4^{1+}	mg/kg	ND
nitrate	NO_3^{1-}	mg/kg	3.7
sulfide	S^{2-}	qual	na
Redox		mV	na

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

431 West Baseline Road · Claremont, CA 91711
Phone: 909.626.0967 · Fax: 909.626.3316

APPENDIX D

EQFAULT, EQSEARCH, AND FRISKSP

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

ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: W.I. 5431-A-SC

DATE: 04-19-2007

JOB NAME: The Womble Group

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

MAGNITUDE RANGE:
MINIMUM MAGNITUDE: 5.00
MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:
SITE LATITUDE: 33.6905
SITE LONGITUDE: 117.1898

SEARCH DATES:
START DATE: 1800
END DATE: 2007

SEARCH RADIUS:
62.0 mi
99.8 km

ATTENUATION RELATION: 17) Campbell & Bozorgnia (1994/1997) - Alluvium
UNCERTAINTY (M=Median, S=Sigma): S Number of Sigmas: 1.0
ASSUMED SOURCE TYPE: SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
SCOND: 0 Depth Source: A
Basement Depth: 5.00 km Campbell SSR: 0 Campbell SHR: 0
COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 3.0

EARTHQUAKE SEARCH RESULTS

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.329	IX	11.6(18.7)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.086	VII	11.6(18.7)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.083	VII	12.1(19.5)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.190	VIII	12.1(19.5)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.083	VII	12.1(19.5)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.228	IX	13.3(21.3)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.158	VIII	14.5(23.3)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.063	VI	15.3(24.6)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.079	VII	18.5(29.7)
DMG	34.0000	117.2500	07/23/1923	73026.0	0.0	6.25	0.125	VII	21.6(34.8)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.035	V	24.7(39.8)
DMG	33.9500	116.8500	09/28/1946	719 9.0	0.0	5.00	0.032	V	26.5(42.6)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.165	VIII	27.8(44.7)
MGI	34.1000	117.3000	07/15/1905	2041 0.0	0.0	5.30	0.037	V	29.0(46.6)
DMG	33.9760	116.7210	06/12/1944	104534.7	10.0	5.10	0.026	V	33.3(53.6)
DMG	33.9940	116.7120	06/12/1944	111636.0	10.0	5.30	0.030	V	34.5(55.5)
DMG	34.2000	117.1000	09/20/1907	154 0.0	0.0	6.00	0.054	VI	35.5(57.2)
DMG	34.1000	116.8000	10/24/1935	1448 7.6	0.0	5.10	0.023	IV	36.0(58.0)
DMG	34.1800	116.9200	01/16/1930	034 3.6	0.0	5.10	0.022	IV	37.2(59.8)
DMG	34.1800	116.9200	01/16/1930	02433.9	0.0	5.20	0.025	V	37.2(59.8)
DMG	34.2000	117.4000	07/22/1899	046 0.0	0.0	5.50	0.032	V	37.2(59.8)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.025	V	37.2(59.9)
GSP	34.1630	116.8550	06/28/1992	144321.0	6.0	5.30	0.026	V	37.8(60.9)
GSP	34.1950	116.8620	08/17/1992	204152.1	11.0	5.30	0.025	V	39.6(63.7)
PAS	33.9980	116.6060	07/08/1986	92044.5	11.7	5.60	0.032	V	39.6(63.8)
DMG	34.1000	116.7000	02/07/1889	520 0.0	0.0	5.30	0.025	V	39.8(64.1)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.020	IV	40.9(65.7)
GSN	34.2030	116.8270	06/28/1992	150530.7	5.0	6.70	0.083	VII	41.0(66.0)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.028	V	41.1(66.1)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.018	IV	41.8(67.3)
DMG	34.2670	116.9670	08/29/1943	34513.0	0.0	5.50	0.028	V	41.8(67.3)
GSP	34.1400	117.7000	02/28/1990	234336.6	5.0	5.20	0.021	IV	42.6(68.6)
GSP	34.2390	116.8370	07/09/1992	014357.6	0.0	5.30	0.022	IV	42.9(69.1)
GSP	34.2900	116.9460	02/10/2001	210505.8	9.0	5.10	0.018	IV	43.7(70.3)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.016	IV	44.1(70.9)
DMG	34.2700	117.5400	09/12/1970	143053.0	8.0	5.40	0.023	IV	44.8(72.0)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.052	VI	45.0(72.3)
DMG	34.0170	116.5000	07/25/1947	04631.0	0.0	5.00	0.016	IV	45.5(73.3)
DMG	34.0170	116.5000	07/25/1947	61949.0	0.0	5.20	0.019	IV	45.5(73.3)
DMG	34.0170	116.5000	07/24/1947	221046.0	0.0	5.50	0.025	V	45.5(73.3)
DMG	34.0170	116.5000	07/26/1947	24941.0	0.0	5.10	0.017	IV	45.5(73.3)
DMG	34.3000	117.5000	07/22/1899	2032 0.0	0.0	6.50	0.060	VI	45.7(73.5)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.018	IV	46.3(74.5)
GSG	34.3100	116.8480	02/22/2003	121910.6	1.0	5.20	0.018	IV	47.0(75.7)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.016	IV	47.8(76.9)
GSP	34.3400	116.9000	11/27/1992	160057.5	1.0	5.30	0.019	IV	47.8(76.9)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.019	IV	48.0(77.2)
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.057	VI	48.1(77.4)
DMG	34.3000	117.6000	07/30/1894	512 0.0	0.0	6.00	0.036	V	48.2(77.5)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.014	IV	48.9(78.7)
DMG	33.9330	116.3830	12/04/1948	234317.0	0.0	6.50	0.055	VI	49.2(79.2)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.022	IV	49.4(79.5)
GSP	34.3690	116.8970	12/04/1992	020857.5	3.0	5.30	0.018	IV	49.7(80.1)

EARTHQUAKE SEARCH RESULTS

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	33.7000	118.0670	03/11/1933	85457.0	0.0	5.10	0.015	IV	50.4(81.1)
DMG	33.7000	118.0670	03/11/1933	51022.0	0.0	5.10	0.015	IV	50.4(81.1)
MGI	34.0000	118.0000	12/25/1903	1745 0.0	0.0	5.00	0.013	III	51.1(82.3)
DMG	33.7500	118.0830	03/11/1933	230 0.0	0.0	5.10	0.015	IV	51.5(82.8)
DMG	33.7500	118.0830	03/11/1933	2 9 0.0	0.0	5.00	0.013	III	51.5(82.8)
DMG	33.7500	118.0830	03/13/1933	131828.0	0.0	5.30	0.018	IV	51.5(82.8)
DMG	33.7500	118.0830	03/11/1933	910 0.0	0.0	5.10	0.015	IV	51.5(82.8)
DMG	33.7500	118.0830	03/11/1933	323 0.0	0.0	5.00	0.013	III	51.5(82.8)
GSP	34.1390	116.4310	06/28/1992	123640.6	10.0	5.10	0.014	IV	53.4(85.9)
GSP	33.9610	116.3180	04/23/1992	045023.0	12.0	6.10	0.034	V	53.4(85.9)
GSP	34.1080	116.4040	06/29/1992	141338.8	9.0	5.40	0.018	IV	53.5(86.0)
DMG	34.2000	117.9000	08/28/1889	215 0.0	0.0	5.50	0.020	IV	53.8(86.5)
DMG	34.3700	117.6500	12/08/1812	15 0 0.0	0.0	7.00	0.076	VII	53.8(86.6)
GSP	33.9020	116.2840	07/24/1992	181436.2	9.0	5.00	0.013	III	54.0(86.9)
GSP	34.0640	116.3610	09/15/1992	084711.3	9.0	5.20	0.015	IV	54.1(87.0)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.026	V	54.2(87.2)
GSP	33.8760	116.2670	06/29/1992	160142.8	1.0	5.20	0.015	IV	54.5(87.7)
DMG	33.7830	118.1330	10/02/1933	91017.6	0.0	5.40	0.018	IV	54.5(87.7)
DMG	33.4000	116.3000	02/09/1890	12 6 0.0	0.0	6.30	0.040	V	55.0(88.5)
GSP	34.0290	116.3210	08/21/1993	014638.4	9.0	5.00	0.012	III	55.0(88.5)
DMG	34.0670	116.3330	05/18/1940	72132.7	0.0	5.00	0.012	III	55.6(89.4)
DMG	34.0670	116.3330	05/18/1940	55120.2	0.0	5.20	0.014	IV	55.6(89.4)
GSN	34.2010	116.4360	06/28/1992	115734.1	1.0	7.60	0.122	VII	55.7(89.7)
DMG	33.4080	116.2610	03/25/1937	1649 1.8	10.0	6.00	0.029	V	56.9(91.6)
PAS	34.0610	118.0790	10/01/1987	144220.0	9.5	5.90	0.026	V	57.0(91.8)
DMG	34.0830	116.3000	05/18/1940	5 358.5	0.0	5.40	0.017	IV	57.7(92.9)
PAS	34.0730	118.0980	10/04/1987	105938.2	8.2	5.30	0.015	IV	58.4(93.9)
GSP	34.3410	116.5290	06/28/1992	124053.5	6.0	5.20	0.013	III	58.7(94.5)
MGI	34.1000	118.1000	07/11/1855	415 0.0	0.0	6.30	0.036	V	59.3(95.5)
GSP	34.2680	116.4020	06/16/1994	162427.5	3.0	5.00	0.011	III	60.2(96.9)
GSP	34.3320	116.4620	07/01/1992	074029.9	9.0	5.40	0.015	IV	60.8(97.8)
GSP	34.2620	118.0020	06/28/1991	144354.5	11.0	5.40	0.015	IV	61.0(98.1)
DMG	33.7830	118.2500	11/14/1941	84136.3	0.0	5.40	0.015	IV	61.2(98.5)
PAS	34.3270	116.4450	03/15/1979	21 716.5	2.5	5.20	0.013	III	61.2(98.5)
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.011	III	61.7(99.3)

****

-END OF SEARCH- 88 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2007

LENGTH OF SEARCH TIME: 208 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 11.6 MILES (18.7 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.6

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.329 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
a-value= 1.210
b-value= 0.363
beta-value= 0.835

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	88	0.42308
4.5	88	0.42308
5.0	88	0.42308
5.5	29	0.13942
6.0	19	0.09135
6.5	8	0.03846
7.0	3	0.01442
7.5	1	0.00481

EARTHQUAKE EPICENTER MAP

The Womble Group

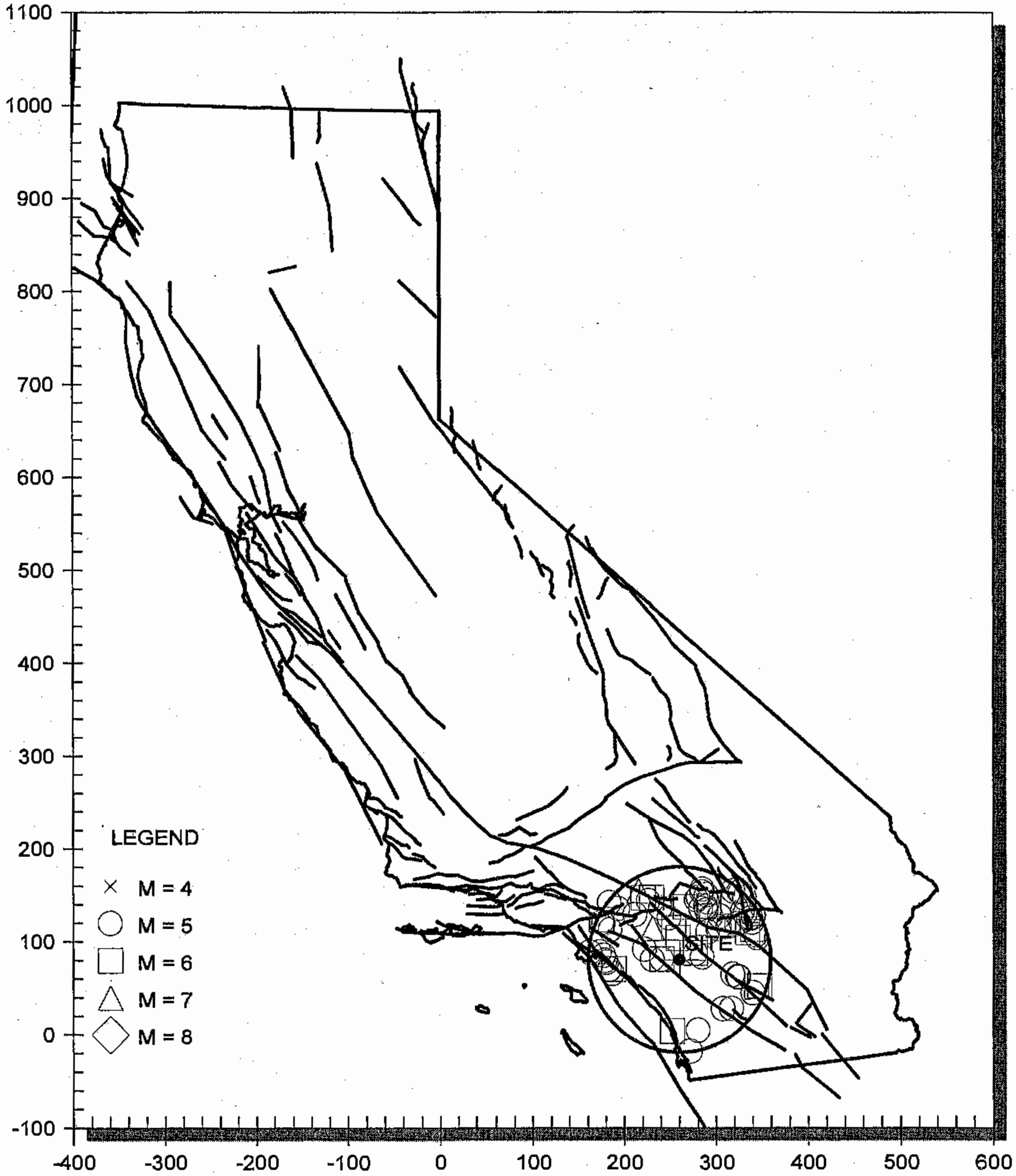
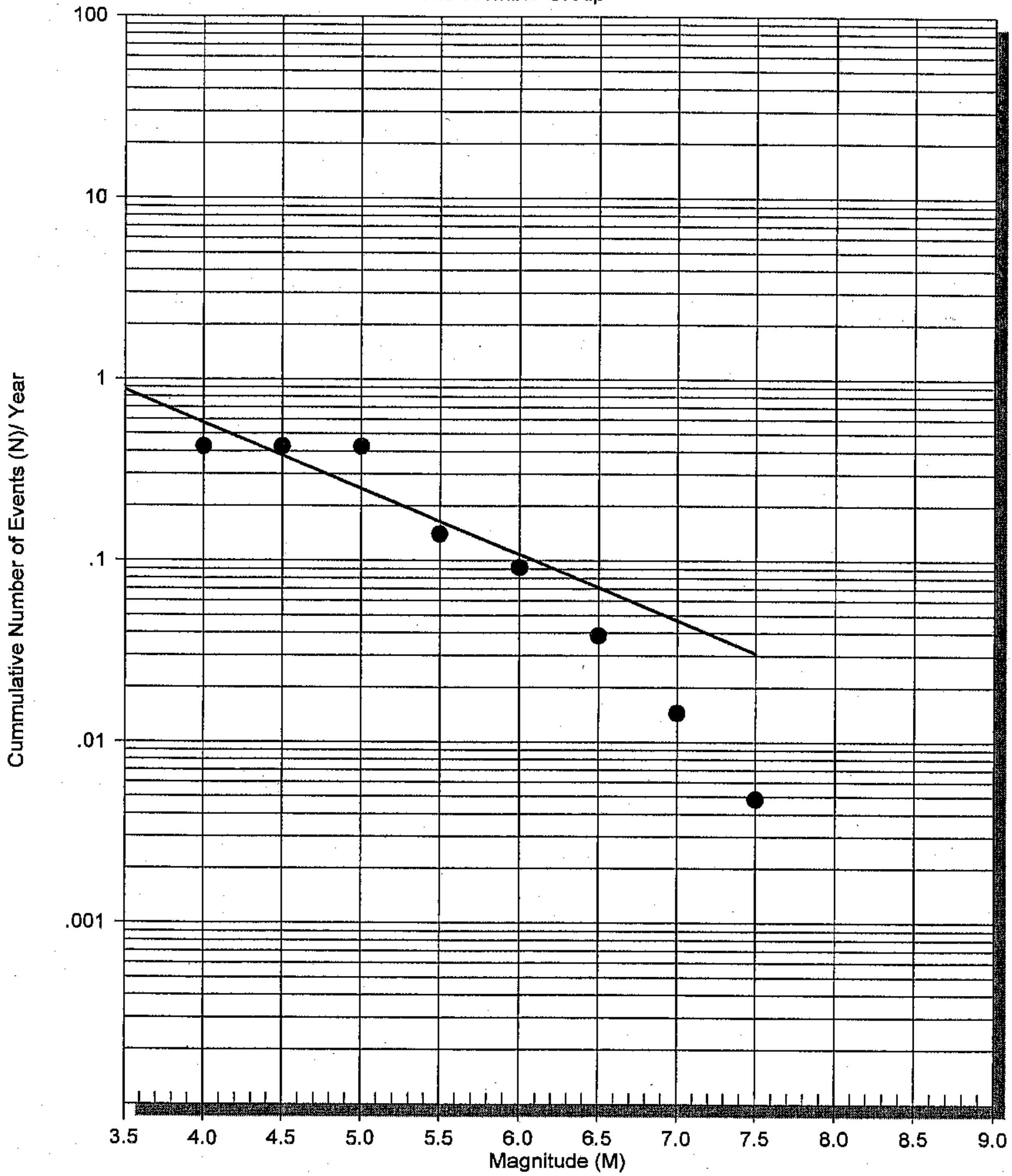
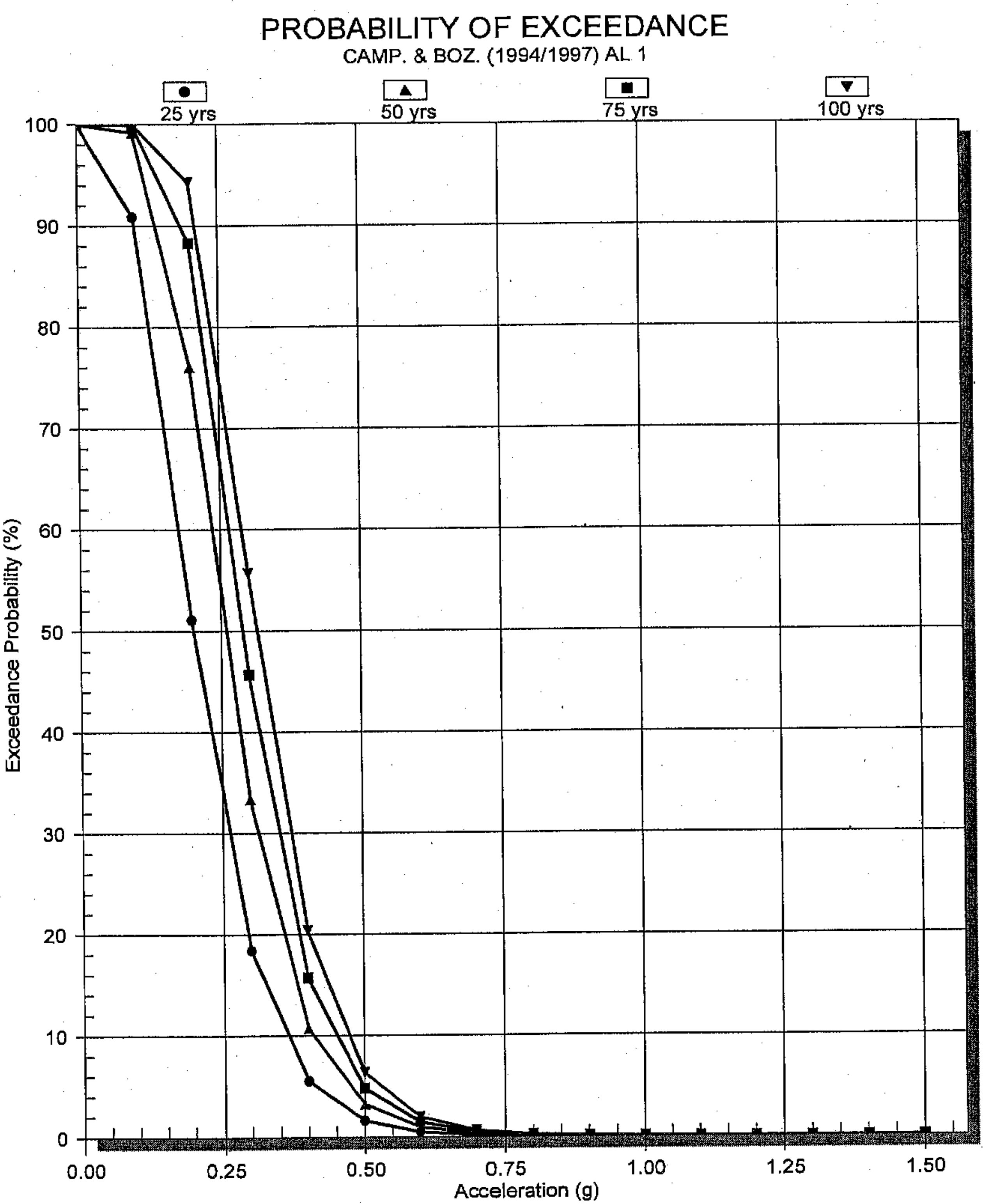


Plate: D-5

EARTHQUAKE RECURRENCE CURVE

The Womble Group

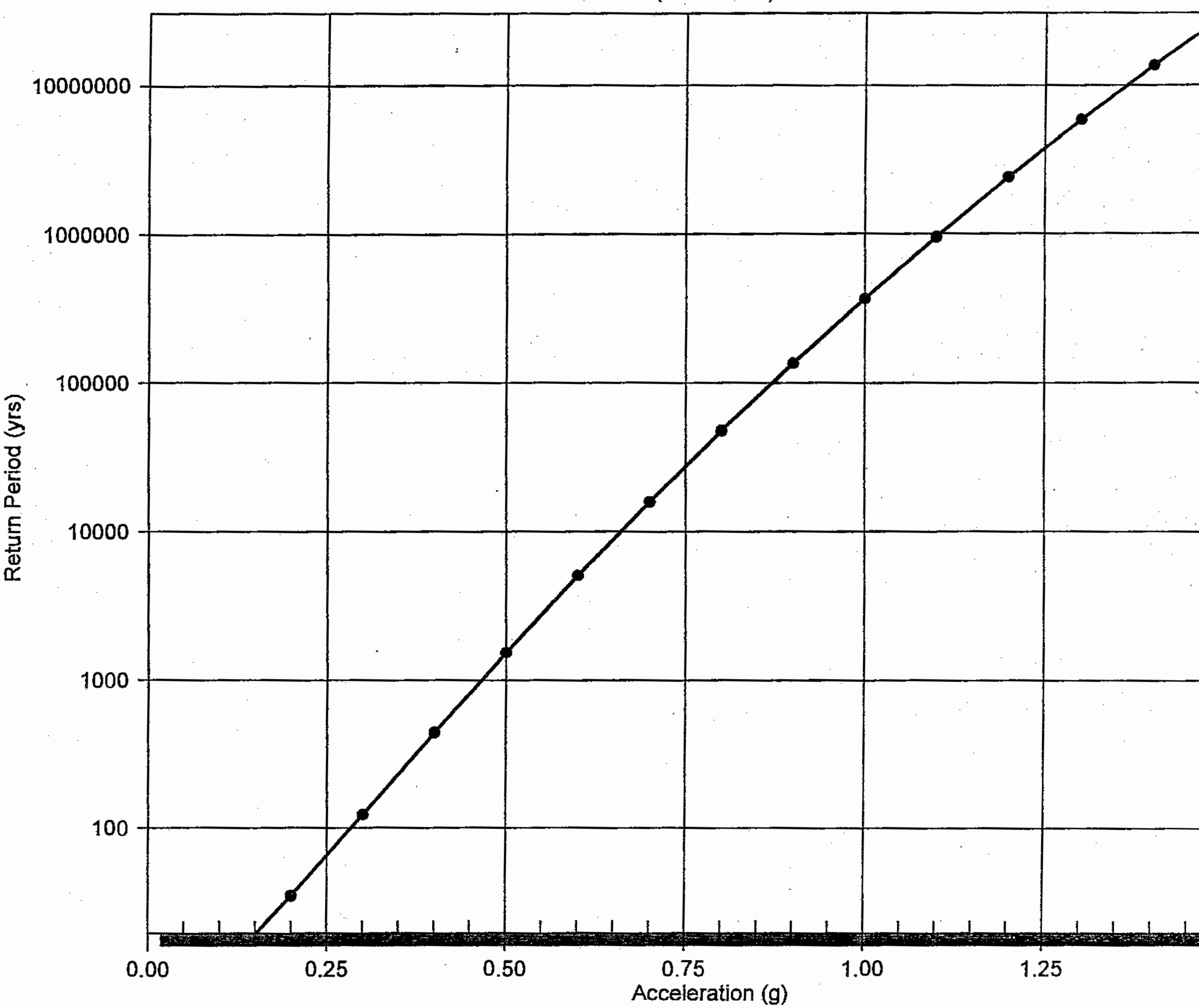




PROBABILITY OF EXCEEDANCE
CAMP. & BOZ. (1994/1997) AL 1
25 yrs
50 yrs
75 yrs
100 yrs
Exceedance Probability (%)
100
90
80
70
60
50
40
30
20
10
0
0.00
0.25
0.50
0.75
1.00
1.25
1.50
Acceleration (g)

RETURN PERIOD vs. ACCELERATION

CAMP. & BOZ. (1994/1997) AL 1



APPENDIX E

GENERAL EARTHWORK AND GRADING GUIDELINES

GENERAL EARTHWORK AND GRADING GUIDELINES

General

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

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

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

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

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

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

Laboratory and Field Tests

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

Contractor's Responsibility

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

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

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

SITE PREPARATION

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

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

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

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

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

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

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

COMPACTED FILLS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

SUBDRAIN INSTALLATION

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

EXCAVATIONS

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

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

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

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

COMPLETION

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

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

JOB SAFETY

General

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

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

Safety Meetings: GSI field personnel are directed to attend contractor's regularly scheduled and documented safety meetings.

Safety Vests: Safety vests are provided for, and are to be worn by GSI personnel, at all times, when they are working in the field.

Safety Flags: Two safety flags are provided to GSI field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.

Flashing Lights: All vehicles stationary in the grading area shall use rotating or flashing amber beacons, or strobe lights, on the vehicle during all field testing. While operating a vehicle in the grading area, the emergency flasher on the vehicle shall be activated.

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

Test Pits Location, Orientation, and Clearance

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

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

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

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

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

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

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

Trench and Vertical Excavation

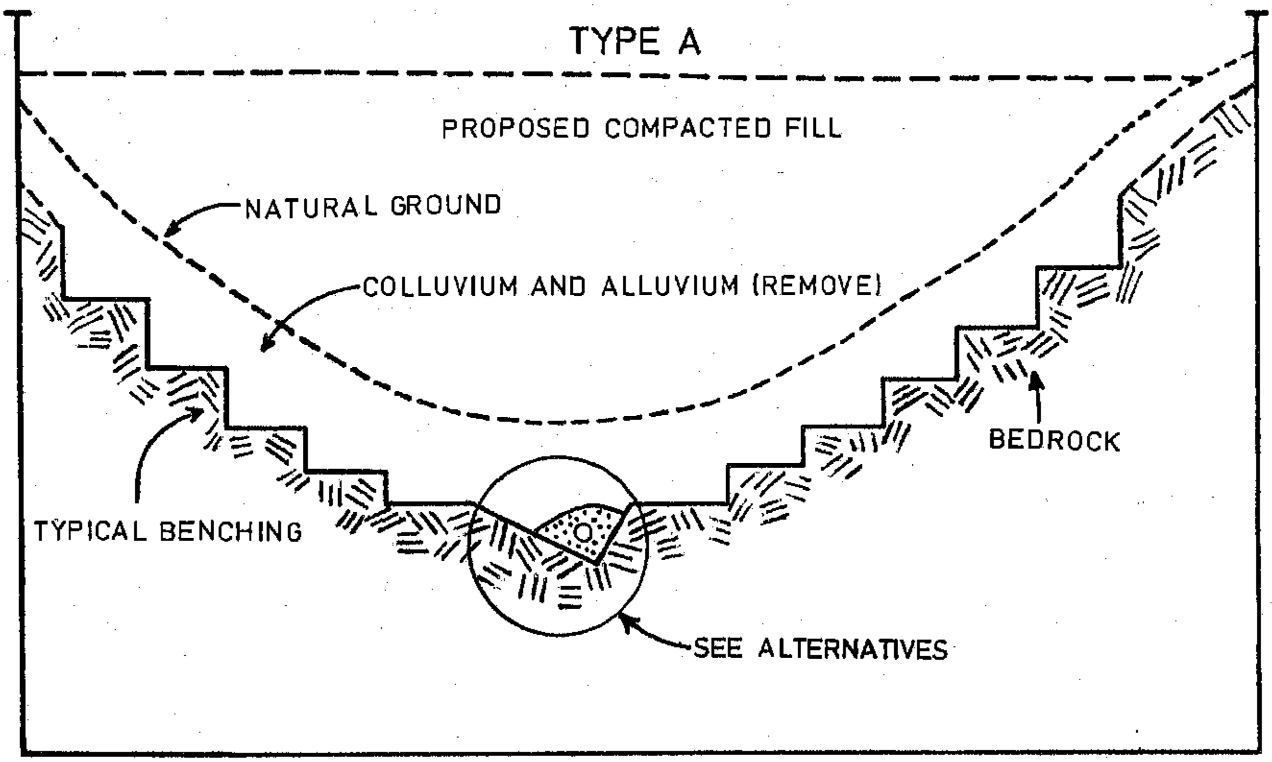
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or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

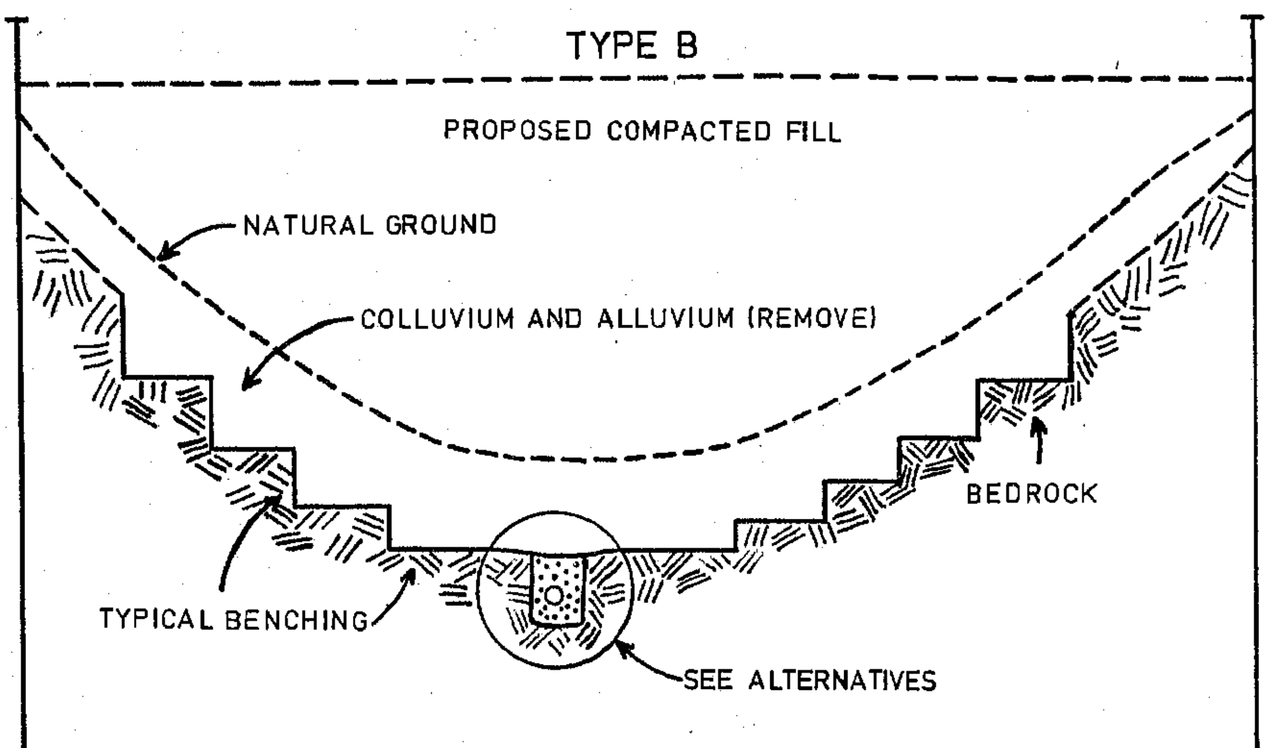
All trench excavations or vertical cuts in excess of 5 feet deep, which any person enters, should be shored or laid back. Trench access should be provided in accordance with CAL-OSHA and/or state and local standards. Our personnel are directed not to enter any trench by being lowered or "riding down"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

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

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

CANYON SUBDRAIN DETAIL



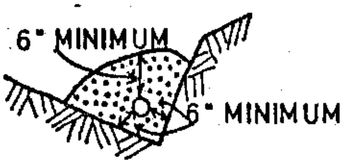


NOTE: ALTERNATIVES, LOCATION AND EXTENT OF SUBDRAINS SHOULD BE DETERMINED BY THE SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST DURING GRADING.

PLATE EG—1

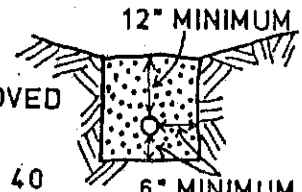
CANYON SUBDRAIN ALTERNATE DETAILS

ALTERNATE 1: PERFORATED PIPE AND FILTER MATERIAL



A-1

FILTER MATERIAL: MINIMUM VOLUME OF 9 FT.³ /LINEAR FT. 6" Ø ABS OR PVC PIPE OR APPROVED SUBSTITUTE WITH MINIMUM 8 (1/4" Ø) PERFS. LINEAR FT. IN BOTTOM HALF OF PIPE.
ASTM D2751, SDR 35 OR ASTM D1527, SCHD. 40
ASTM D3034, SDR 35 OR ASTM D1785, SCHD. 40
FOR CONTINUOUS RUN IN EXCESS OF 500 FT. USE 8" Ø PIPE

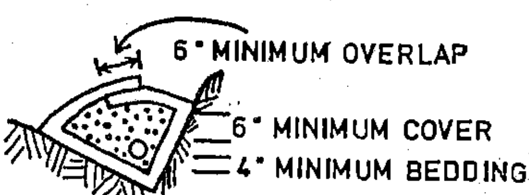


B-1

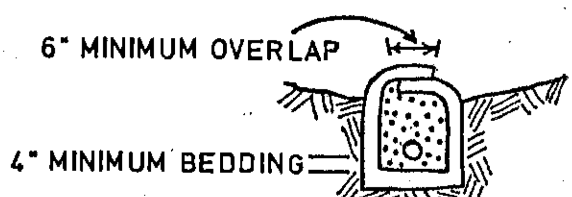
FILTER MATERIAL

SIEVE SIZE	PERCENT PASSING
1 INCH	100
3/4 INCH	90-100
3/8 INCH	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

ALTERNATE 2: PERFORATED PIPE, GRAVEL AND FILTER FABRIC



A-2



B-2

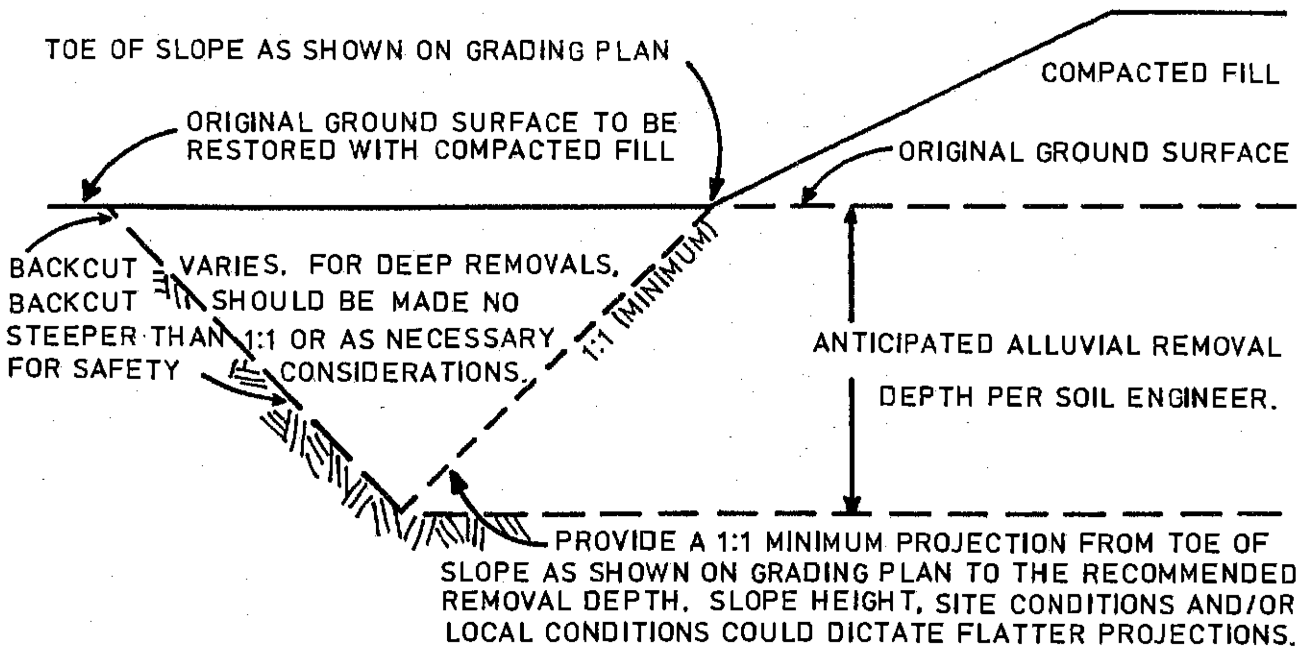
GRAVEL MATERIAL 9 FT³/LINEAR FT.

PERFORATED PIPE: SEE ALTERNATE 1

GRAVEL: CLEAN 3/4 INCH ROCK OR APPROVED SUBSTITUTE

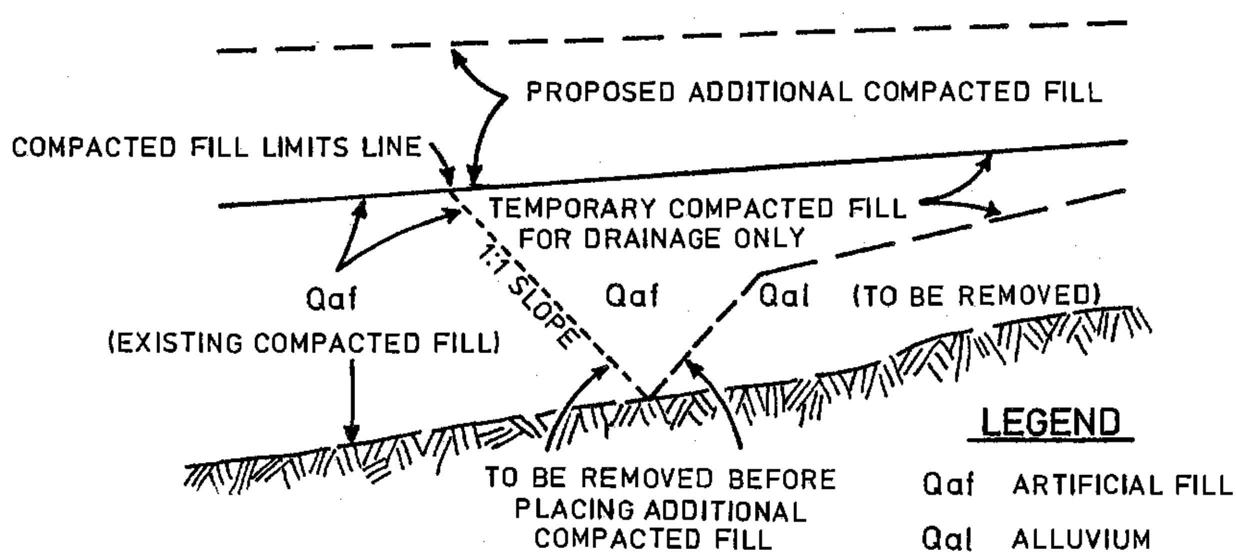
FILTER FABRIC: MIRAFI 140 OR APPROVED SUBSTITUTE

DETAIL FOR FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON

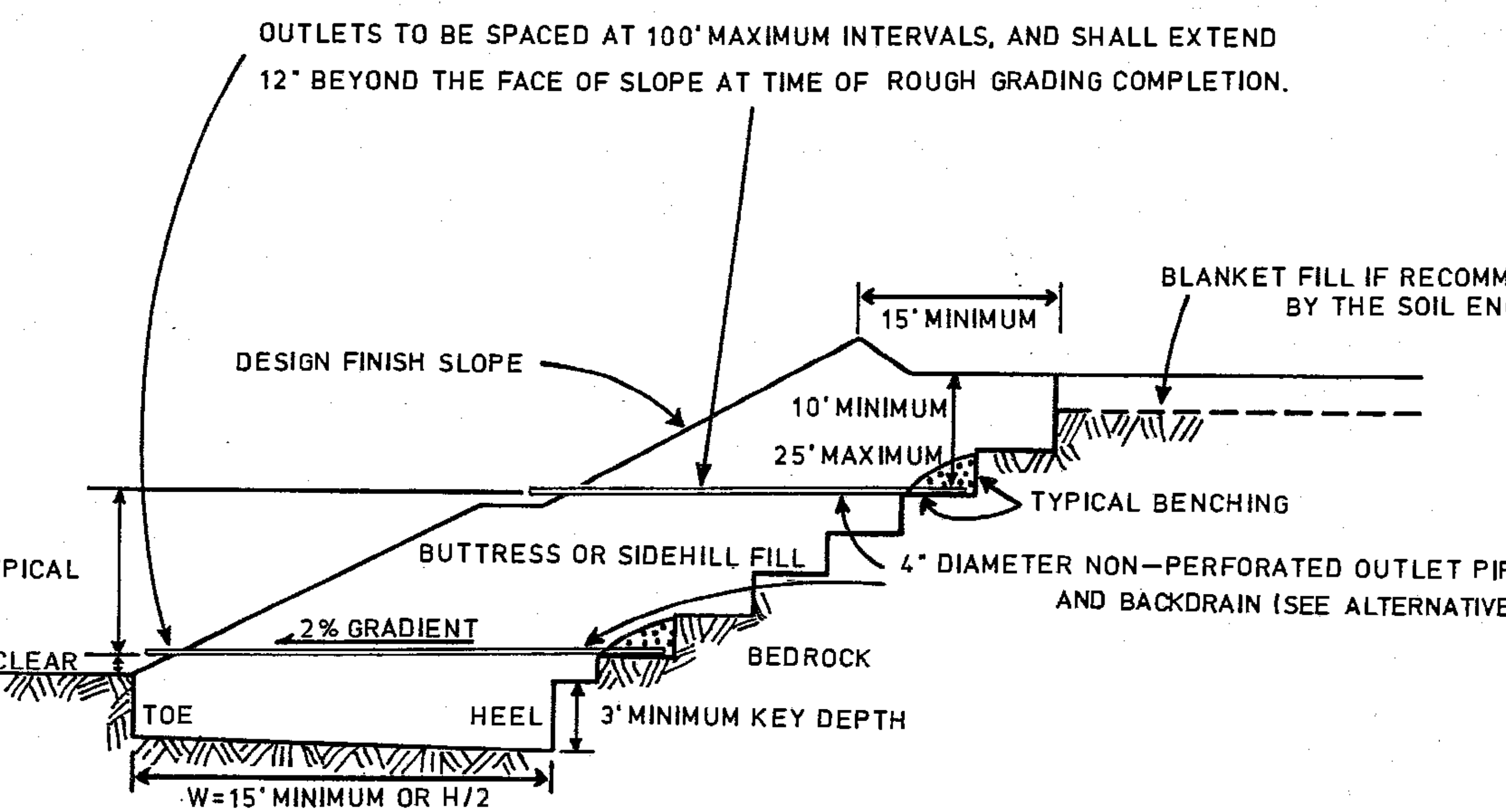


REMOVAL ADJACENT TO EXISTING FILL

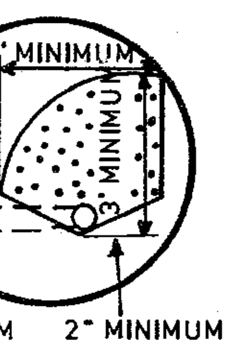
ADJOINING CANYON FILL

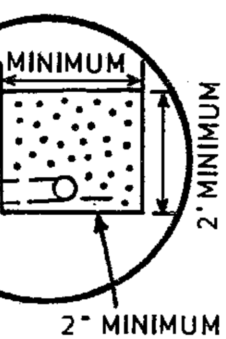


TYPICAL STABILIZATION / BUTTRESS FILL DETAIL



YPICAL STABILIZATION / BUTTRESS SUBDRAIN DETAIL





<u>FILTER MATERIAL:</u> MINIMUM OF FIVE Ft³/LINEAR Ft OF PIPF OR FOUR Ft³/LINEAR Ft OF PIPE WHEN PLACED IN SQUARE CUT TRENCH.

<u>ALTERNATIVE IN LIEU OF FILTER MATERIAL</u>: GRAVEL MAY BE ENCASED IN APPROVED FILTER FABRIC. FILTER FABRIC SHALL BE MIRAFI 140 OR EQUIVALENT. FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12" ON ALL JOINTS.

<u>MINIMUM 4" DIAMETER PIPE:</u> ABS—ASTM D—2751, SDR 35 OR ASTM D—1527 SCHEDULE 40 PVC—ASTM D—3034, SDR 35 OR ASTM D—1785 SCHEDULE 40 WITH A CRUSHING STRENGTH OF 1,000 POUNDS MINIMUM, AND A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS OF BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM END OF PIPE. SLOPE AT 2% TO OUTLET PIPE. OUTLET PIPE TO BE CONNECTED TO SUBDRAIN PIPE WITH TEE OR ELBOW.

NOTE: 1. TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON—SITE SOIL.

2. BACKDRAINS AND LATERAL DRAINS SHALL BE LOCATED AT ELEVATION OF EVERY BENCH DRAIN. FIRST DRAIN LOCATED AT ELEVATION JUST ABOVE LOWER LOT GRADE. ADDITIONAL DRAINS MAY BE REQUIRED AT THE DISCRETION OF THE SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST.

<u>FILTER MATERIAL</u> SHALL
THE FOLLOWING SPECIFI
OR AN APPROVED EQUIVA

SIEVE SIZE	PERCENT PA
1 INCH	100
3/4 INCH	90—10
3/8 INCH	40—10
NO. 4	25—40
NO. 8	18—33
NO. 30	5—15
NO. 50	0—7
NO. 200	0—3

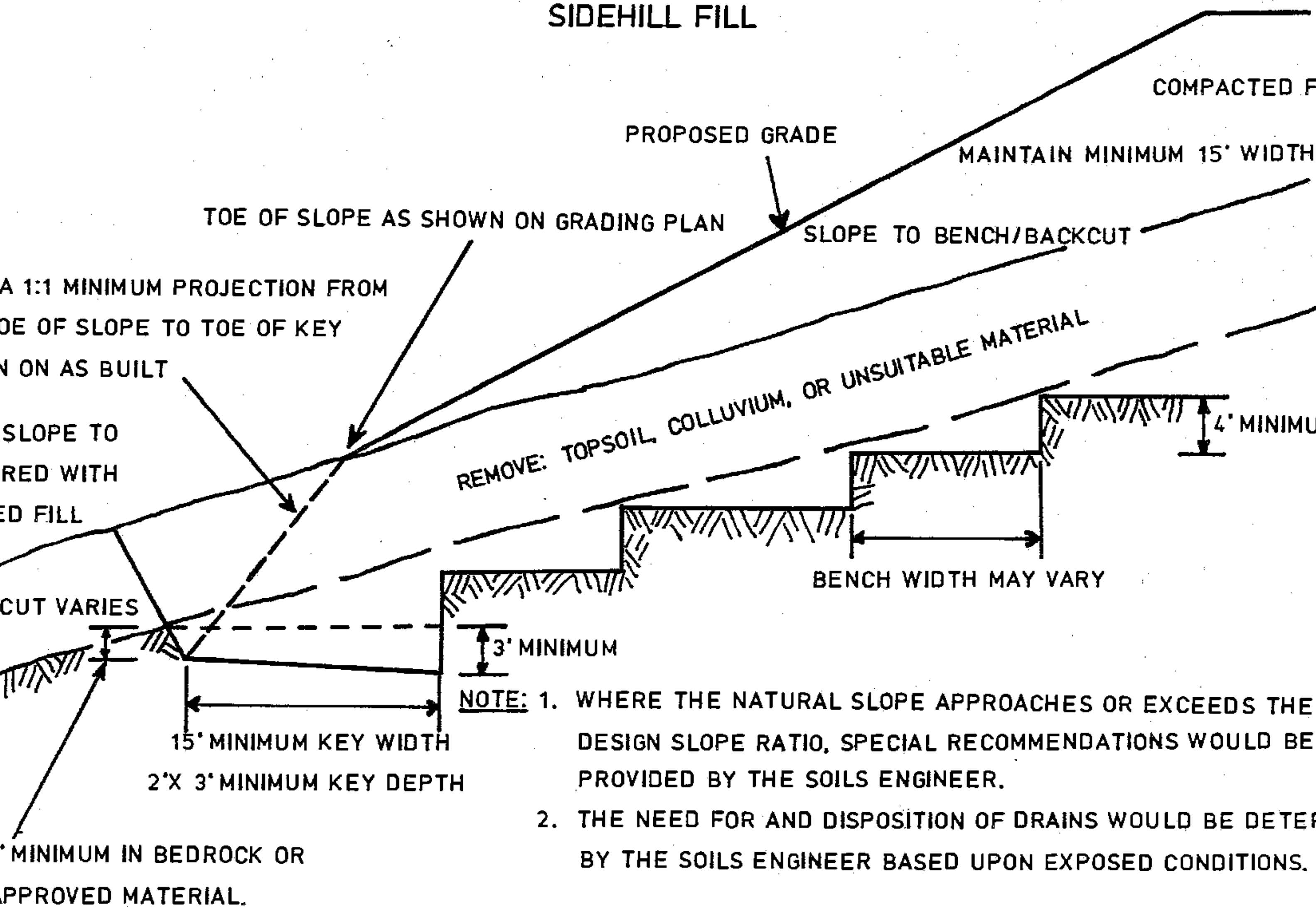
<u>GRAVEL</u> SHALL BE OF TH
FOLLOWING SPECIFICATI
AN APPROVED EQUIVALE

SIEVE SIZE	PERCENT PA
1 1/2 INCH	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT: MINIMU

FILL OVER NATURAL DETAIL

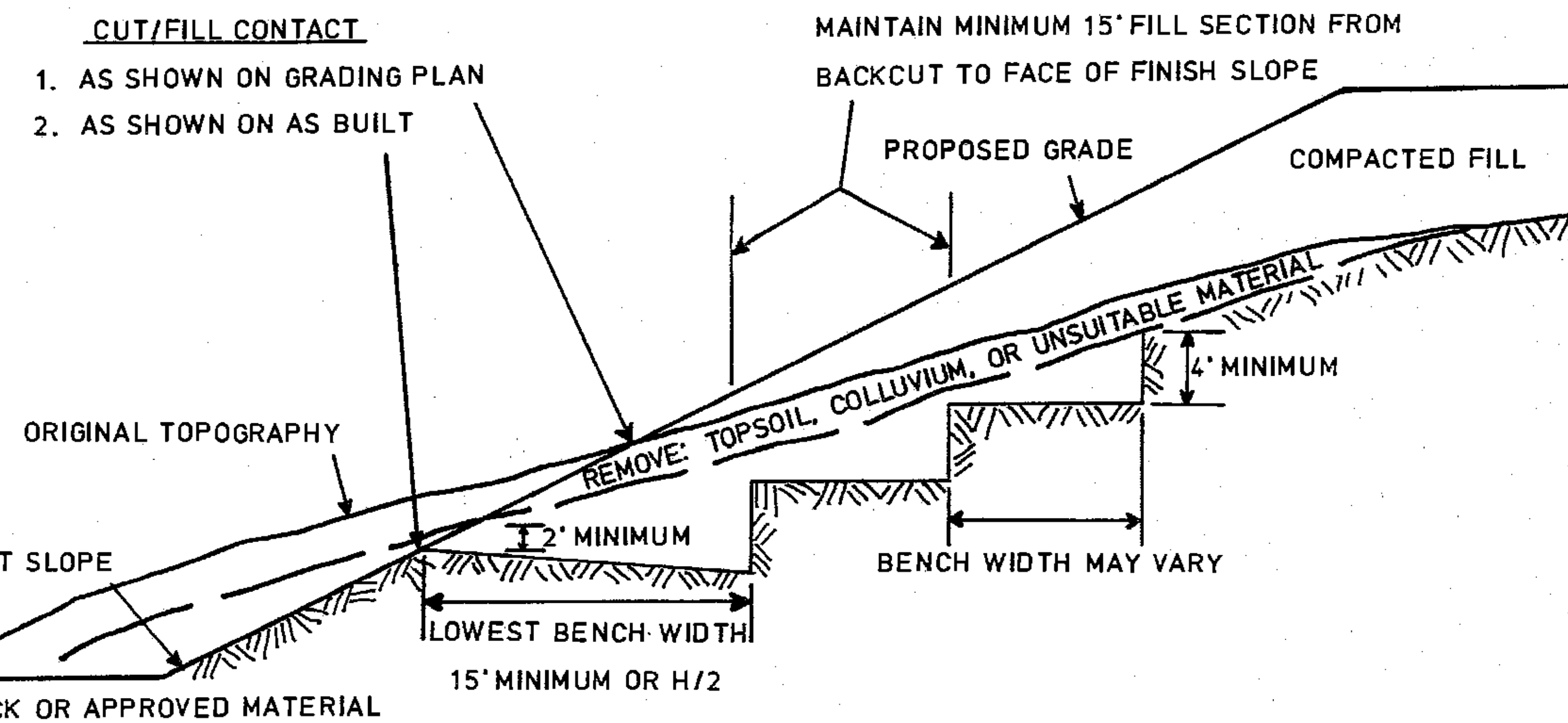
SIDEHILL FILL



NOTE: 1. WHERE THE NATURAL SLOPE APPROACHES OR EXCEEDS THE DESIGN SLOPE RATIO, SPECIAL RECOMMENDATIONS WOULD BE PROVIDED BY THE SOILS ENGINEER.

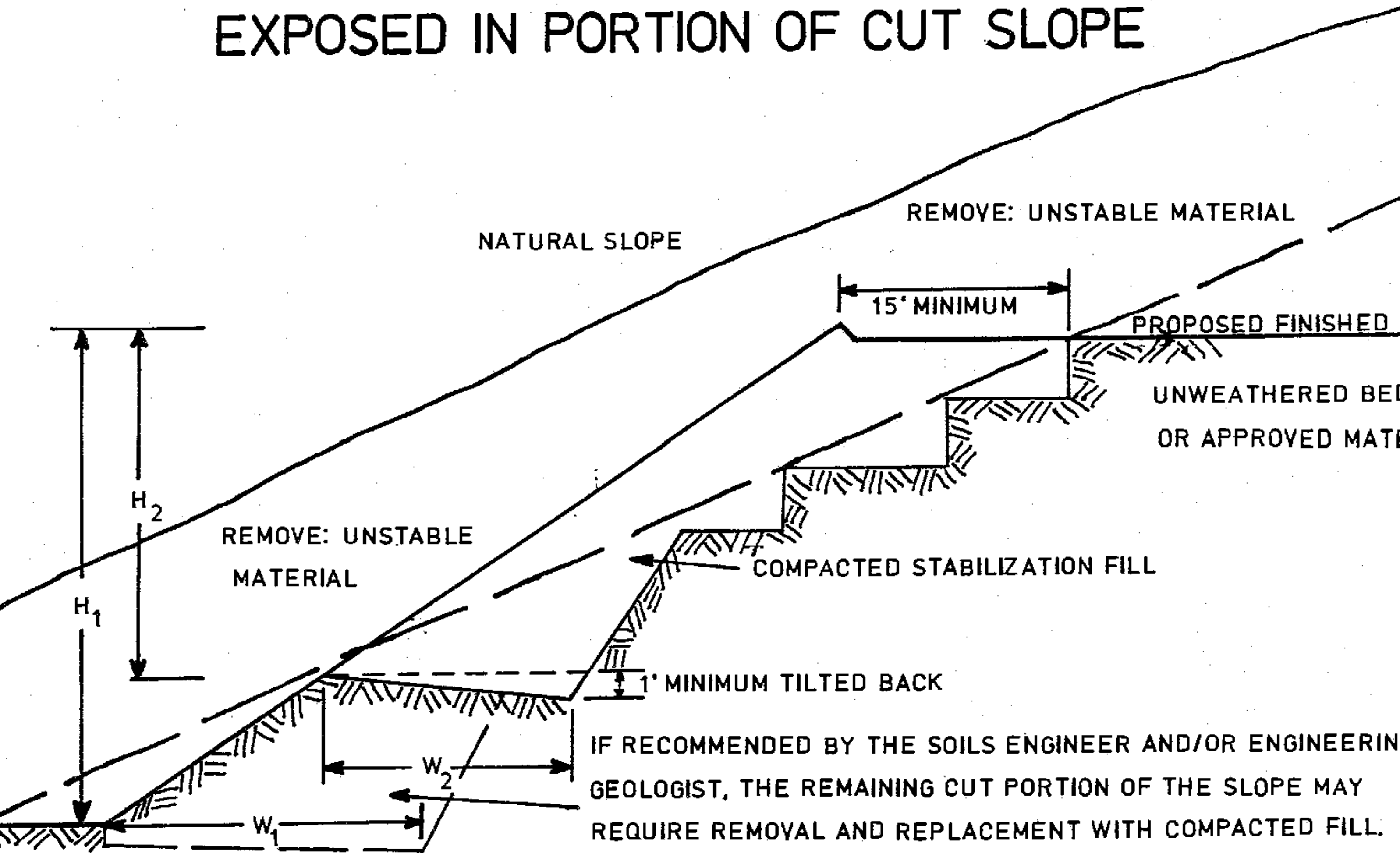
2. THE NEED FOR AND DISPOSITION OF DRAINS WOULD BE DETER BY THE SOILS ENGINEER BASED UPON EXPOSED CONDITIONS.

FILL OVER CUT DETAIL



NOTE: THE CUT PORTION OF THE SLOPE SHOULD BE EXCAVATE
EVALUATED BY THE SOILS ENGINEER AND/OR ENGINEERI
GEOLOGIST PRIOR TO CONSTRUCTING THE FILL PORTION.

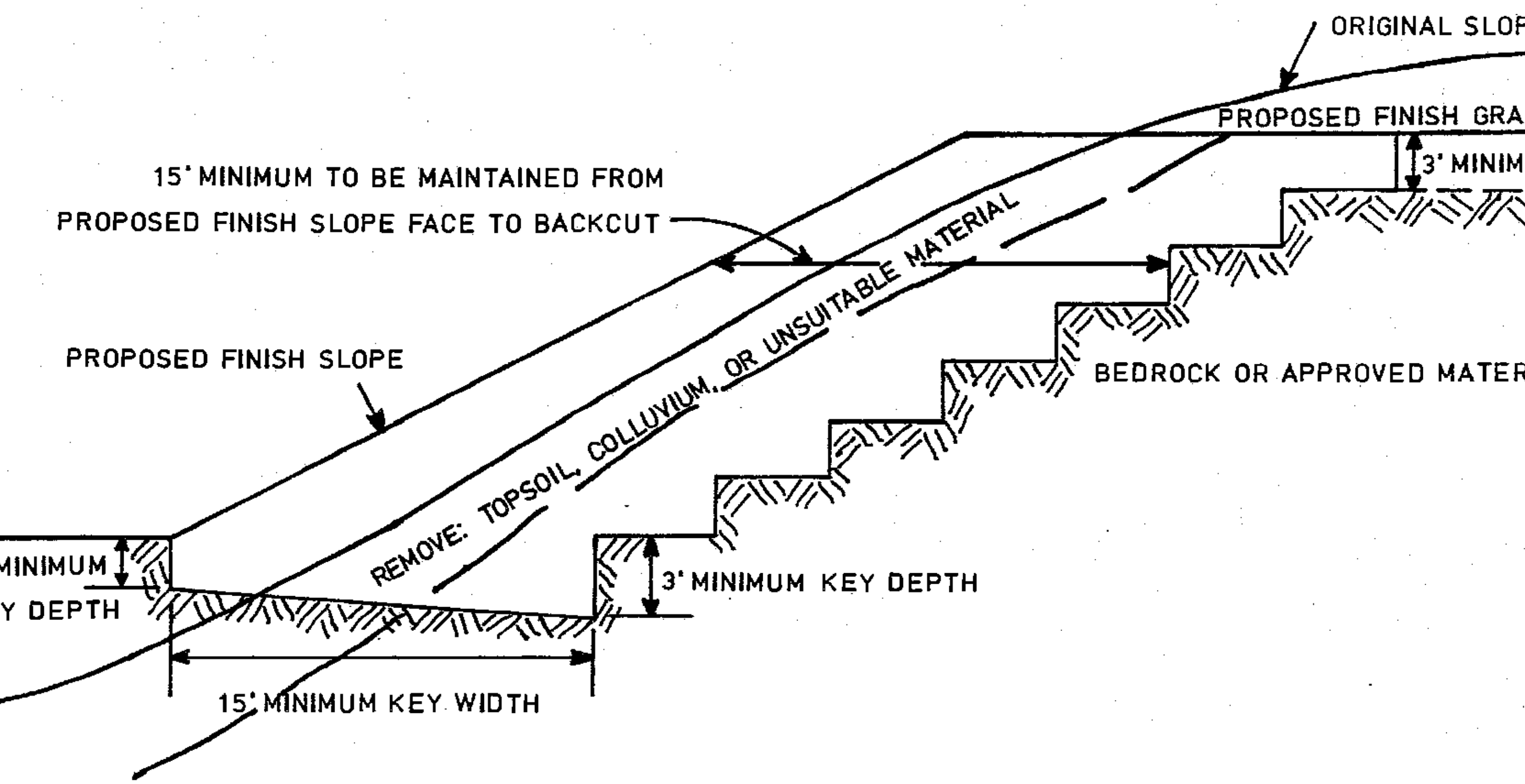
STABILIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN PORTION OF CUT SLOPE



OTE: 1. SUBDRAINS ARE NOT REQUIRED UNLESS SPECIFIED BY SOILS ENGINEER AND/OR ENGINEERING GEOLOG

2. "W" SHALL BE EQUIPMENT WIDTH (15') FOR SLOPE HEIGHTS LESS THAN 25 FEET. FOR SLOPES GREATE THAN 25 FEET "W" SHALL BE DETERMINED BY THE PROJECT SOILS ENGINEER AND /OR ENGINEERING GEOLOGIST. AT NO TIME SHALL "W" BE LESS THAN H/2.

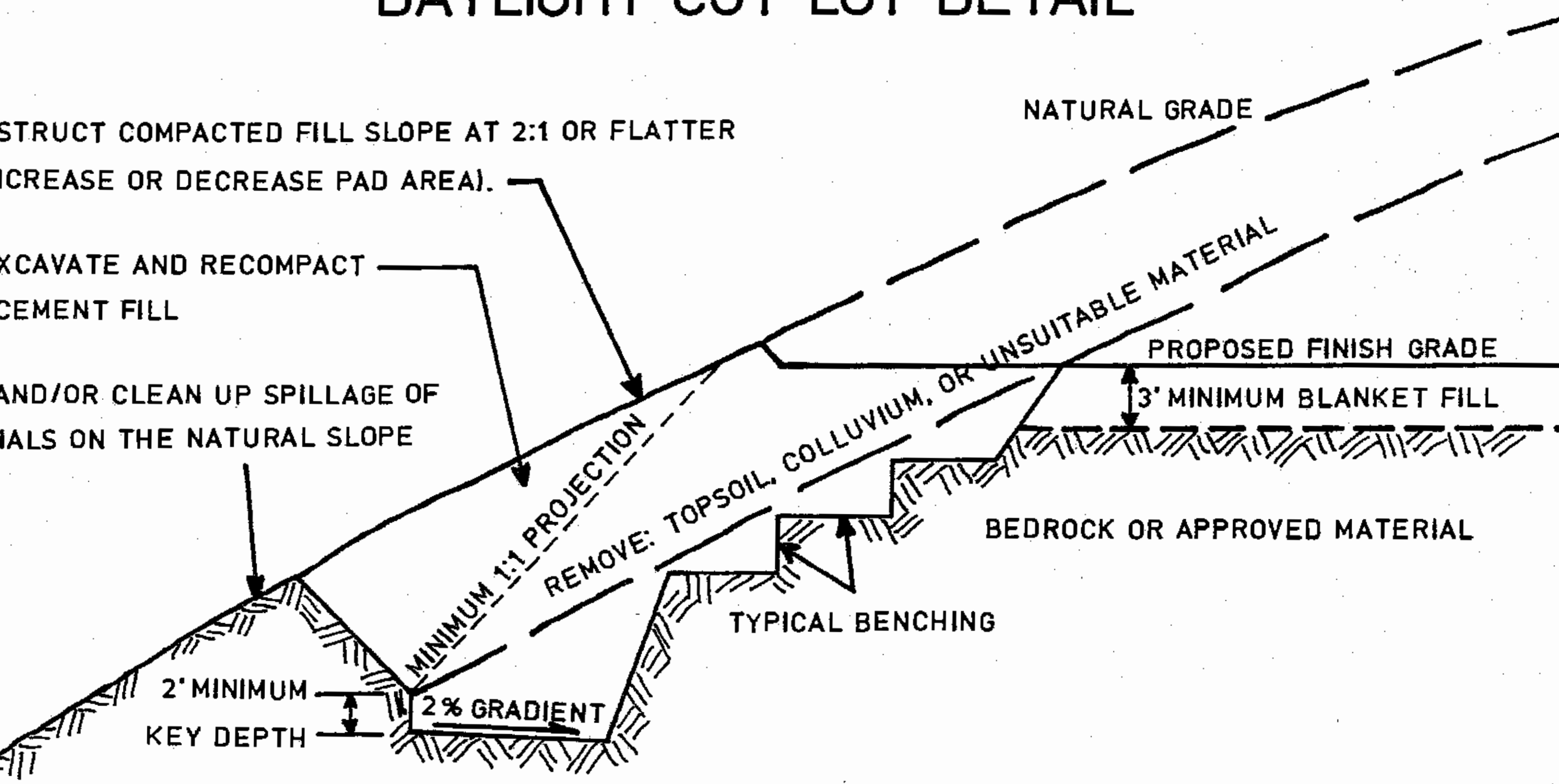
SKIN FILL OF NATURAL GROUND



NOTE: 1. THE NEED AND DISPOSITION OF DRAINS WILL BE DETERMINED BY THE SOILS ENGINEER AN ENGINEERING GEOLOGIST BASED ON FIELD CONDITIONS.

2. PAD OVEREXCAVATION AND RECOMPACTION SHOULD BE PERFORMED IF DETERMINED TO BE NECESSARY BY THE SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST.

DAYLIGHT CUT LOT DETAIL

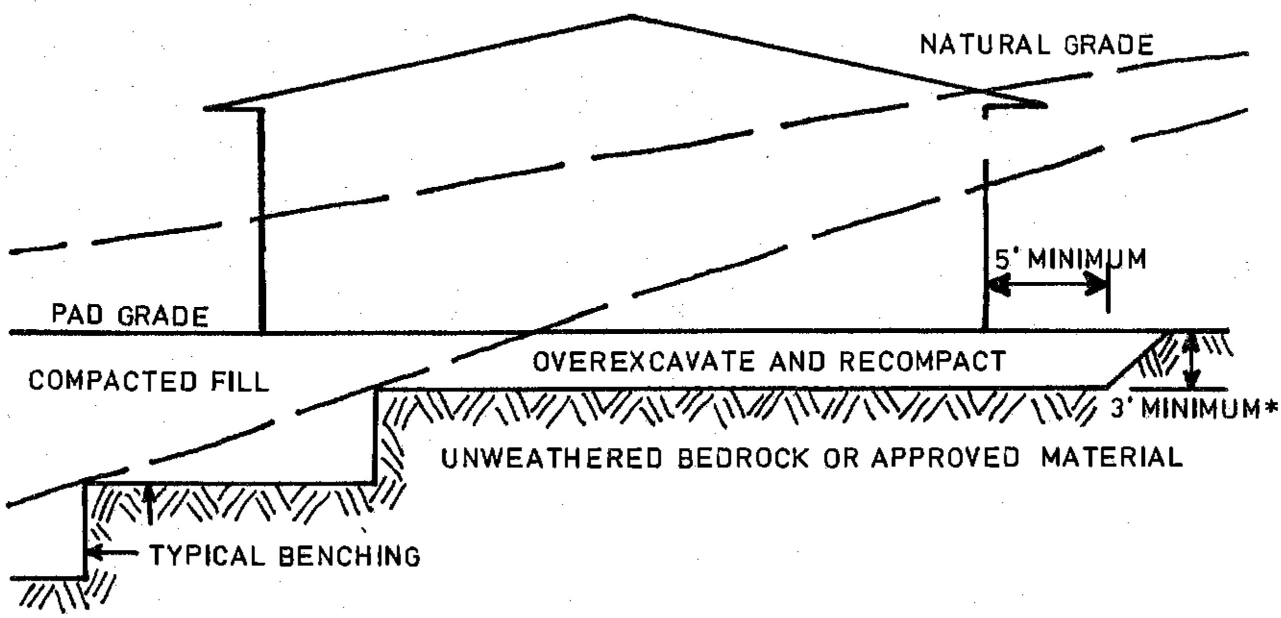


TE: 1. SUBDRAIN AND KEY WIDTH REQUIREMENTS WILL BE DETERMINED BASED ON EXPOSED SUBSURFACE CONDITIONS AND THICKNESS OF OVERBURDEN.

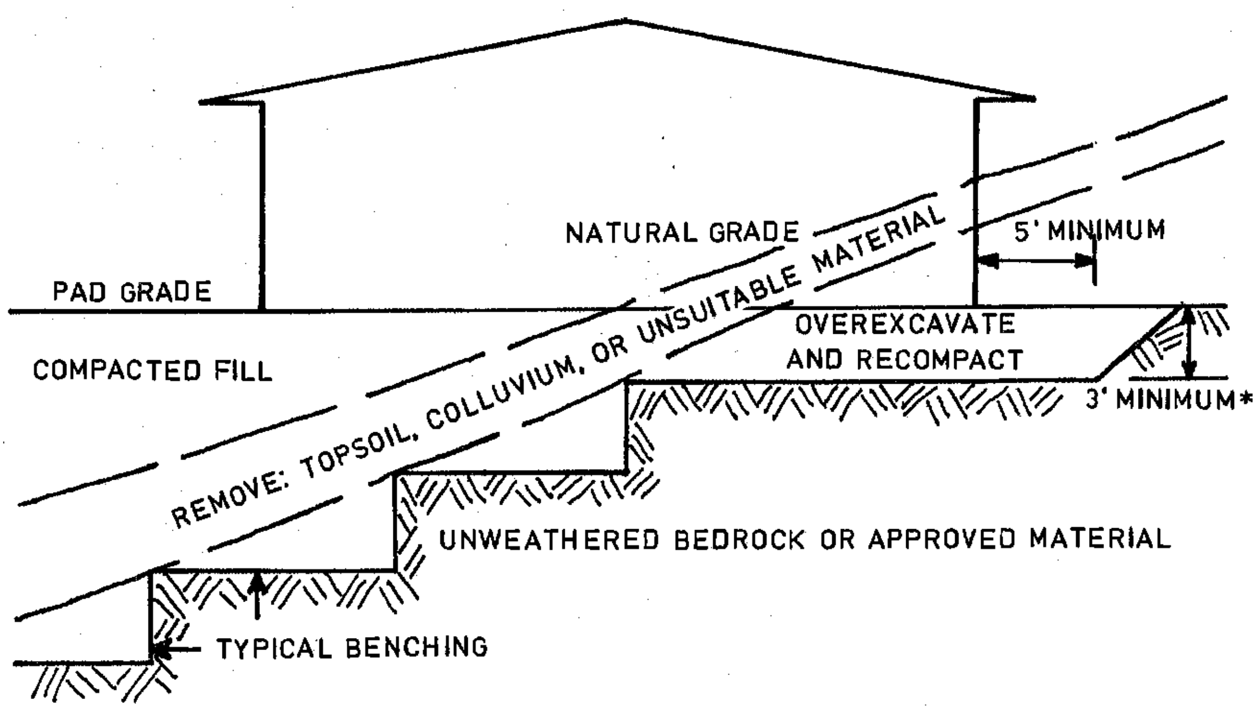
2. PAD OVER EXCAVATION AND RECOMPACTION SHOULD BE PERFORMED IF DETERMINED NECESSARY BY THE SOILS ENGINEER AND/OR THE ENGINEERING GEOLOGIST.

TRANSITION LOT DETAIL

CUT LOT (MATERIAL TYPE TRANSITION)

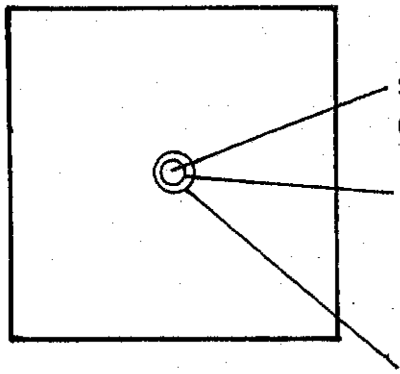


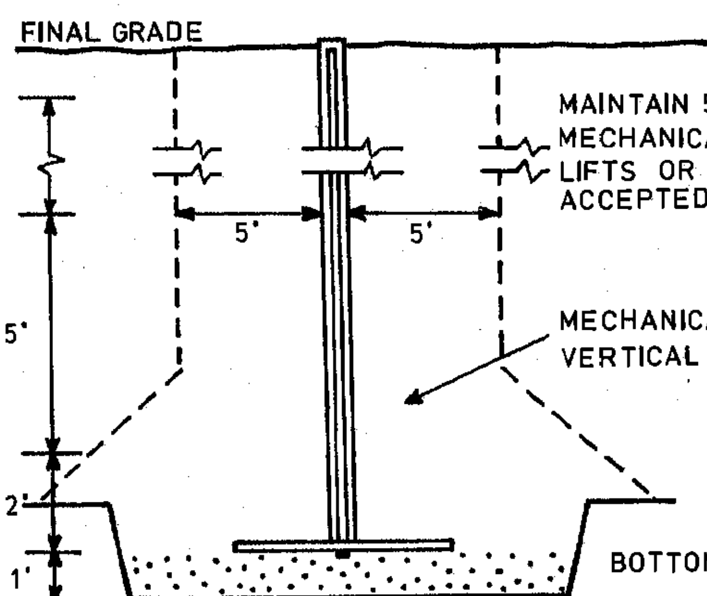
CUT–FILL LOT (DAYLIGHT TRANSITION)



NOTE: * DEEPER OVEREXCAVATION MAY BE RECOMMENDED BY THE SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST IN STEEP CUT–FILL TRANSITION AREAS.

SETTLEMENT PLATE AND RISER DETAIL

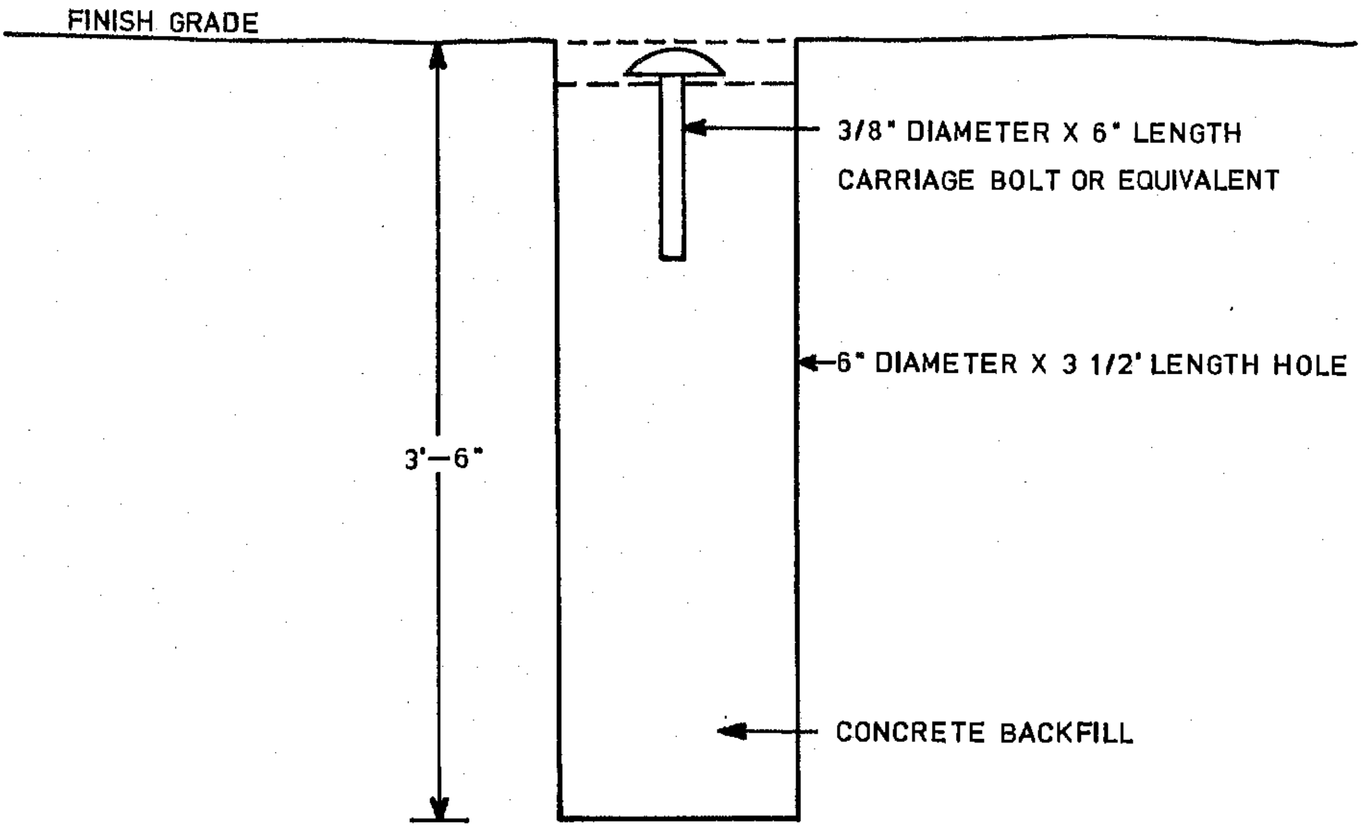




NOTE:

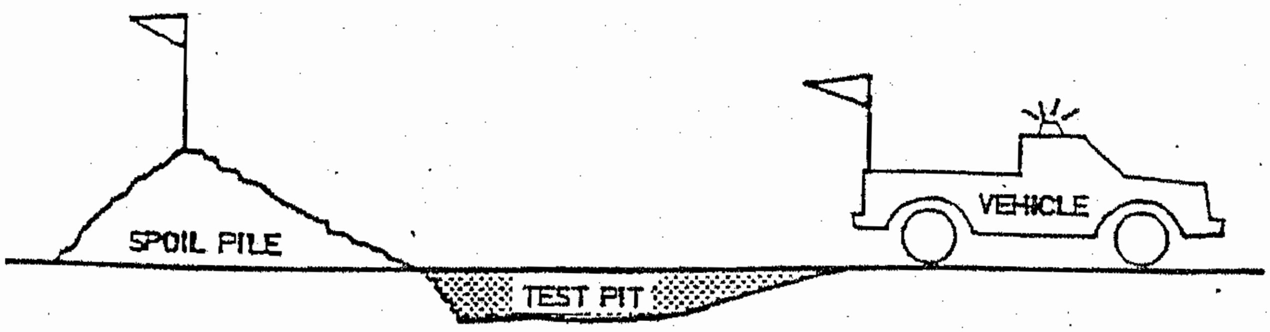
1. LOCATIONS OF SETTLEMENT PLATES SHOULD BE CLEARLY MARKED AND READILY VISIBLE (RED FLAGGED) TO EQUIPMENT OPERATORS.
2. CONTRACTOR SHOULD MAINTAIN CLEARANCE OF A 5' RADIUS OF PLATE BASE AND WITHIN 5' (VERTICAL) FOR HEAVY EQUIPMENT. FILL WITHIN CLEARANCE AREA SHOULD BE HAND COMPACTED TO PROJECT SPECIFICATIONS OR COMPACTED BY ALTERNATIVE APPROVED BY THE SOILS ENGINEER.
3. AFTER 5' (VERTICAL) OF FILL IS IN PLACE, CONTRACTOR SHOULD MAINTAIN A 5' RADIUS EQUIPMENT CLEARANCE FROM RISER.
4. PLACE AND MECHANICALLY HAND COMPACT INITIAL 2' OF FILL PRIOR TO ESTABLISHING THE INITIAL READING.
5. IN THE EVENT OF DAMAGE TO THE SETTLEMENT PLATE OR EXTENSION RESULTING FROM EQUIPMENT OPERATING WITHIN THE SPECIFIED CLEARANCE AREA, CONTRACTOR SHOULD IMMEDIATELY NOTIFY THE SOILS ENGINEER AND SHOULD BE RESPONSIBLE FOR RESTORING THE SETTLEMENT PLATES TO WORKING ORDER.
6. AN ALTERNATE DESIGN AND METHOD OF INSTALLATION MAY BE PROVIDED AT THE DISCRETION OF THE SOILS ENGINEER.

TYPICAL SURFACE SETTLEMENT MONUMENT



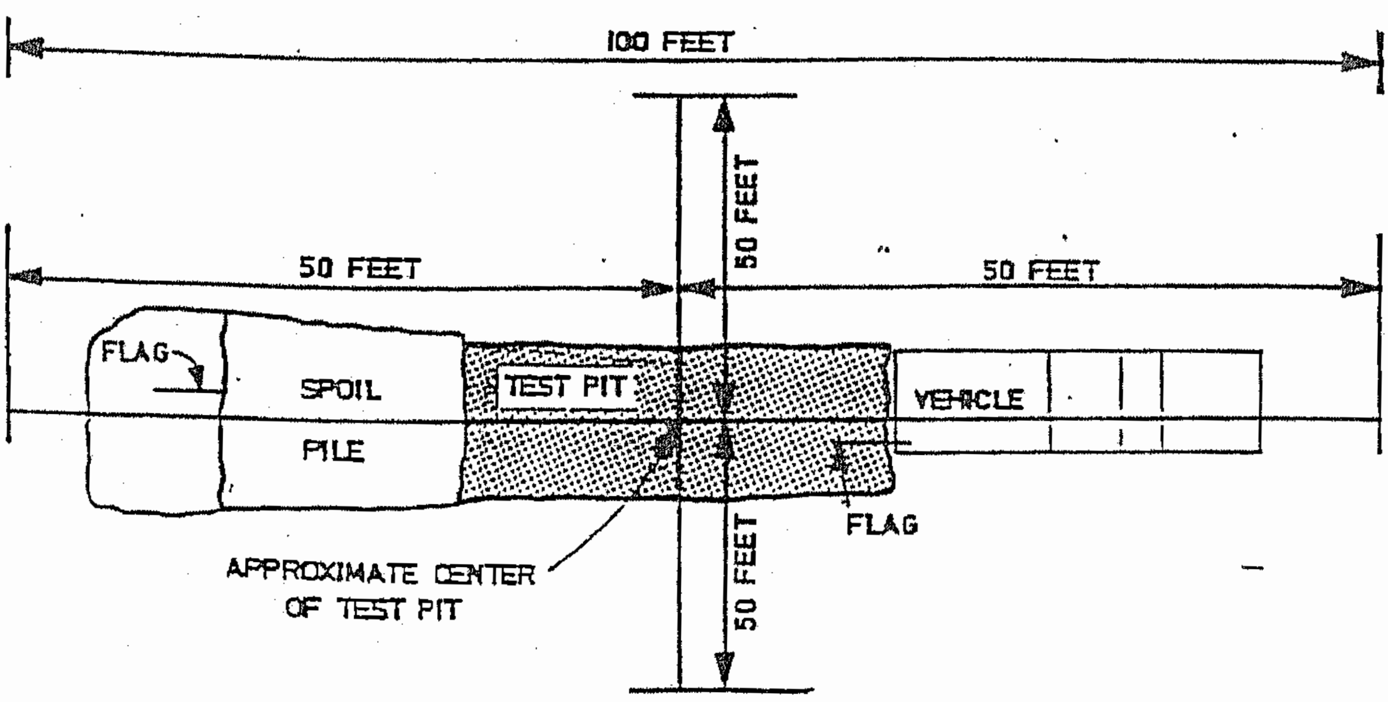
TEST PIT SAFETY DIAGRAM

SIDE VIEW



(NOT TO SCALE)

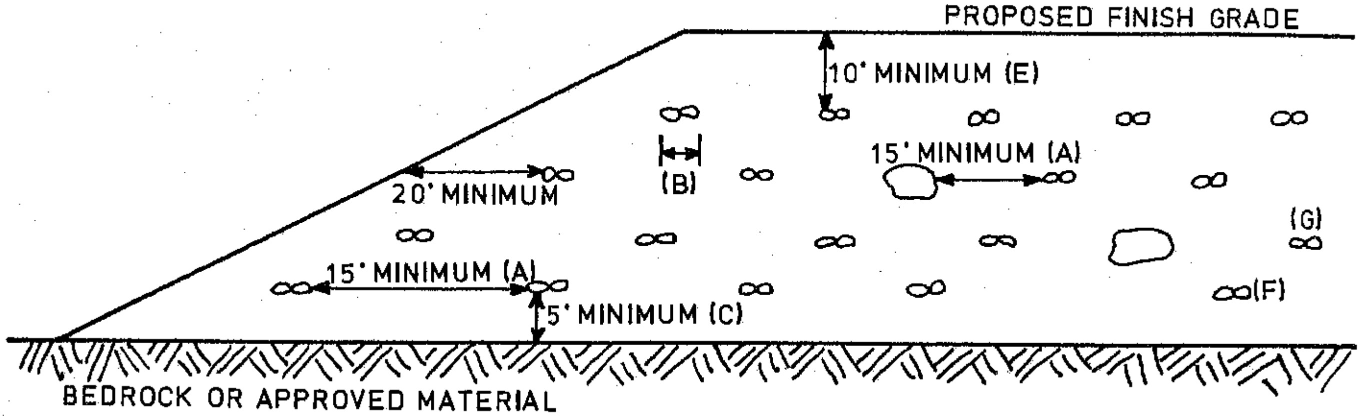
TOP VIEW



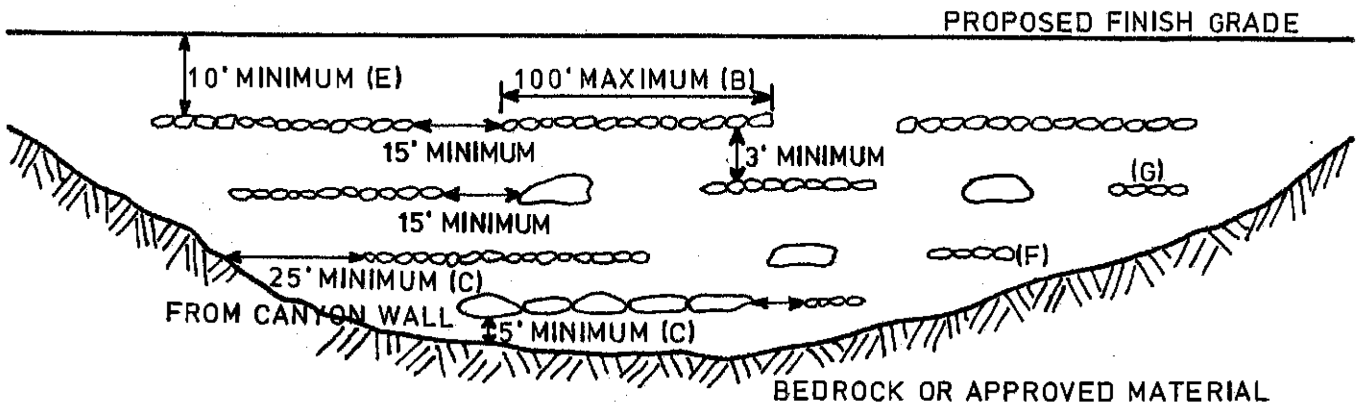
(NOT TO SCALE)

OVERSIZE ROCK DISPOSAL

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



NOTE: (A) ONE EQUIPMENT WIDTH OR A MINIMUM OF 15 FEET.

(B) HEIGHT AND WIDTH MAY VARY DEPENDING ON ROCK SIZE AND TYPE OF EQUIPMENT. LENGTH OF WINDROW SHALL BE NO GREATER THAN 100' MAXIMUM.

(C) IF APPROVED BY THE SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST, WINDROWS MAY BE PLACED DIRECTLY ON COMPETENT MATERIAL OR BEDROCK PROVIDED ADEQUATE SPACE IS AVAILABLE FOR COMPACTION.

(D) ORIENTATION OF WINDROWS MAY VARY BUT SHOULD BE AS RECOMMENDED BY THE SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST. STAGGERING OF WINDROWS IS NOT NECESSARY UNLESS RECOMMENDED.

(E) CLEAR AREA FOR UTILITY TRENCHES, FOUNDATIONS AND SWIMMING POOLS.

(F) ALL FILL OVER AND AROUND ROCK WINDROW SHALL BE COMPACTED TO 90% RELATIVE COMPACTION OR AS RECOMMENDED.

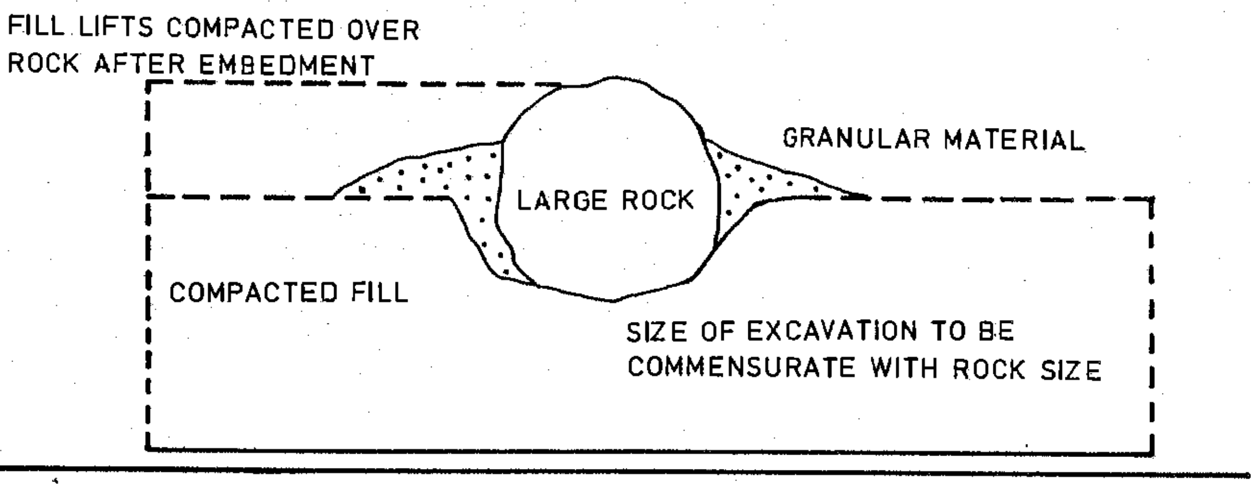
(G) AFTER FILL BETWEEN WINDROWS IS PLACED AND COMPACTED WITH THE LIFT OF FILL COVERING WINDROW, WINDROW SHOULD BE PROOF ROLLED WITH A D-9 DOZER OR EQUIVALENT.

VIEWS ARE DIAGRAMMATIC ONLY. ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED IN.

PLATE RD-1

ROCK DISPOSAL PITS

VIEWS ARE DIAGRAMMATIC ONLY. ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED IN.



ROCK DISPOSAL LAYERS

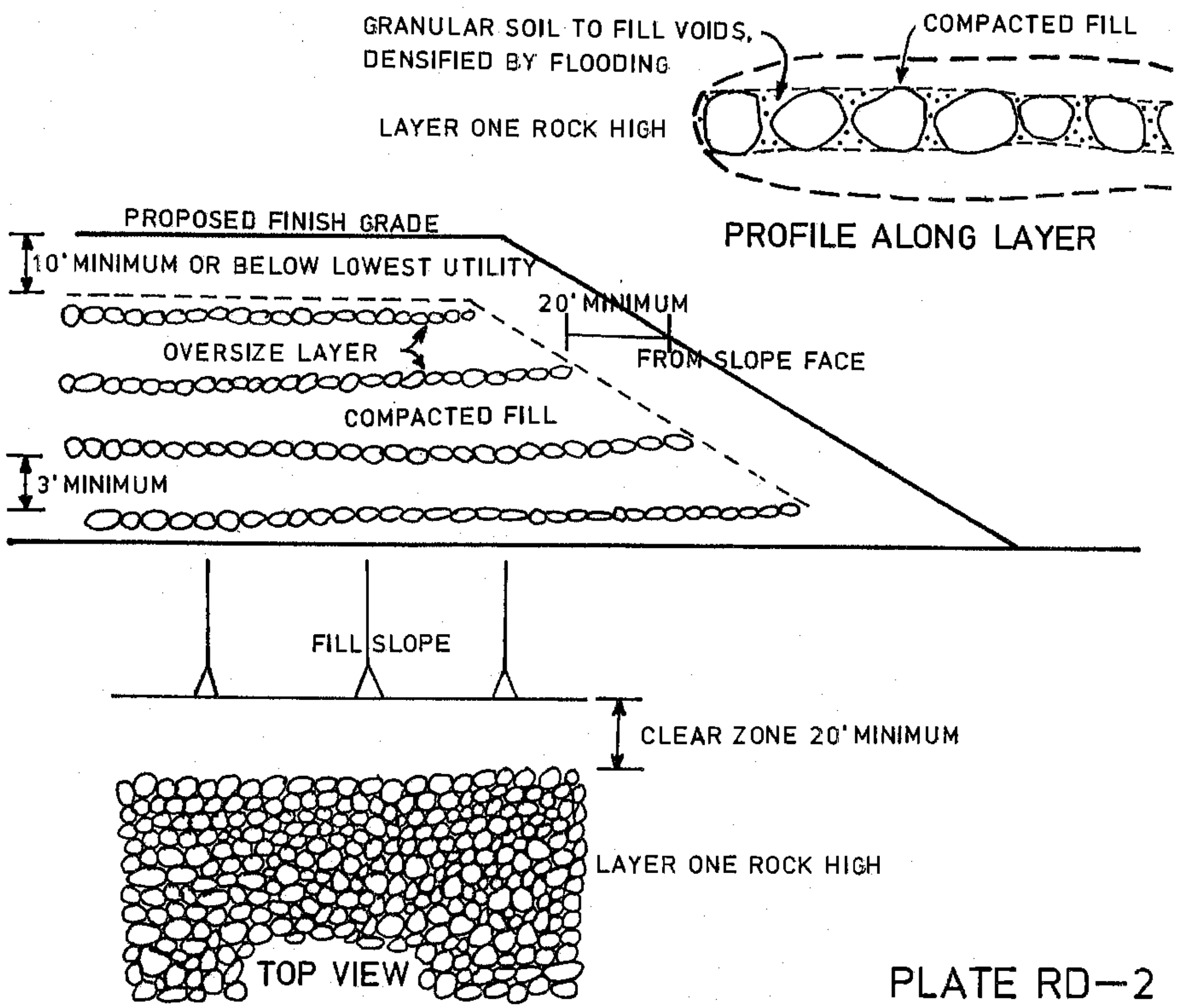


PLATE RD-2