

APPENDIX F
GEOTECHNICAL/HAZARDS REPORTS
AND PEER REVIEW COMMENTS

SIERRA GEOTECHNICAL SERVICES INC.

October 4, 2006

Project No. 3.00856.1

The Chadmar Group
PO Box 100, PMB # 605
Mammoth Lakes, CA 93546

Attention: Mr. Daniel McGregor

Subject: **PRELIMINARY GEOTECHNICAL INVESTIGATION**
Snowcreek 8 Development
Mammoth Lakes, California

Dear Mr. McGregor:

In accordance with your authorization of our proposal dated July 21st 2006, we herein submit the results of our preliminary geotechnical investigation for the proposed Snowcreek 8 area development. The purpose of this study was to assess the geotechnical constraints to development (if any) and provide geotechnical recommendations relative to the future development of the proposed project.

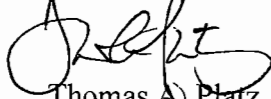
Based on the results of this study, and a review of the proposed development plan, prepared by Scheurer Architects, future site development is feasible from a geologic and geotechnical standpoint. This investigation is however considered preliminary as detailed grading and foundation plans are currently not available. Sierra Geotechnical Services Inc. should review grading and foundation plans prior to construction in order to assure that they are in conformance with this report; some of the recommendations contained herein may need to be revised after reviewing.

The conclusions and recommendations presented herein are considered site specific and should not be extrapolated to other areas or used for other projects.

We appreciate the opportunity to be of service to you. Should you have any questions regarding this report, please do not hesitate to contact us.

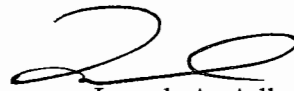
Respectfully,

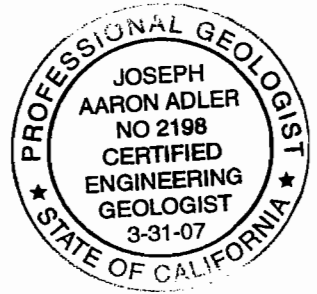
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PRELIMINARY GEOTECHNICAL INVESTIGATION
FOR
SNOWCREEK 8 DEVELOPMENT
MAMMOTH LAKES, CALIFORNIA

OCTOBER 4, 2006
PROJECT NO. 3.00856.1

Prepared By:

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1. PURPOSE AND SCOPE

This report presents the results of our preliminary geotechnical investigation for the proposed Snowcreek 8 area development to be located south of Old Mammoth Road and east of the existing Snowcreek Golf Course, in Mammoth Lakes, California. (Figures 1 and 2). The purpose of this study was to assess the geotechnical constraints to development (if any), and provide geotechnical recommendations relative to the future development of the proposed project.

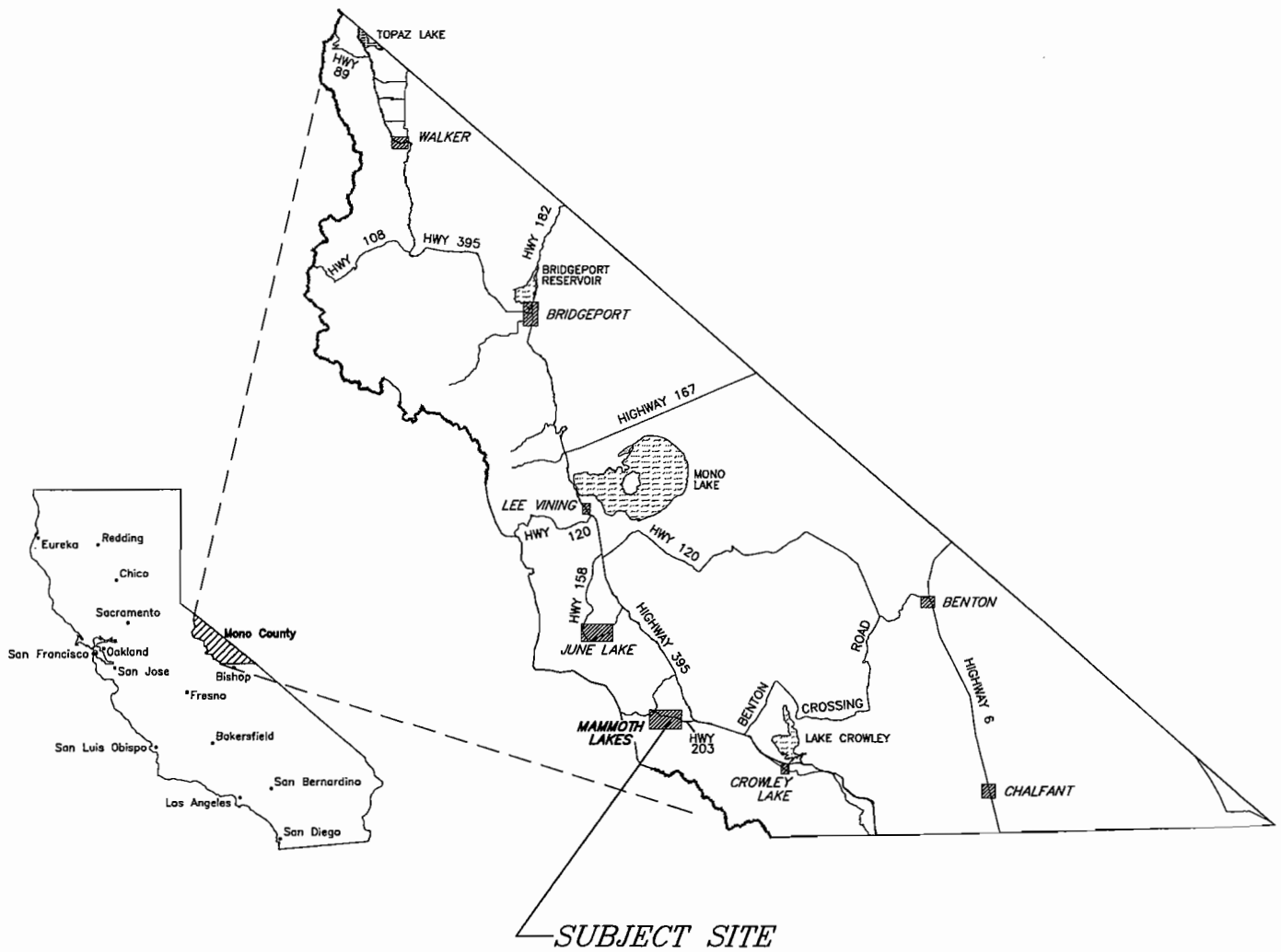
The scope of this investigation included a review of the proposed development plan, prepared by Scheurer Architects, stereoscopic aerial photographs, readily available published and unpublished geologic literature, a subsurface field investigation, seismic analysis, and preparation of this report presenting the results of our findings, conclusions, geotechnical recommendations and construction considerations for the proposed project.

The field investigation was performed on August 1st through August 3rd 2006, and included the excavation of forty-two exploratory test pits within the proposed construction areas. A geologist from our office logged the excavations as they were advanced. Soil materials were visually classified in the field according to the Unified Soil Classification System (USCS). Bulk samples of the soils encountered were obtained during the field investigation for laboratory testing. Approximate locations of the exploratory test pits are shown on the Subsurface Geotechnical Maps (Figures 3 and 4). Details of the laboratory testing are presented in Appendix B.

After the test pits were excavated and logged, they were loosely backfilled with the excavated soil and not compacted to the requirements typically specified for engineered fill. Prior to construction the test pit backfill material should be removed and compacted in accordance with the earthwork recommendations contained within this report. If the backfill materials are left "as-is" structures located over these areas may experience some degree of settlement.

2. SITE DESCRIPTION

The Snowcreek 8 property is located along the southern portion of the developable lands in Mammoth Lakes, in Mono County, California, (Figure 1). The property is bordered on the south and east by USFS land and on the north across Old Mammoth Road by undeveloped property.



NOT TO SCALE

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PROJECT:	REGIONAL MAP SNOWCREEK 8	
SCALE:	NTS	DATE: 9/2006
DRAWING:	FIG1.DWG	DRAWN BY: JAA
JOB NO.:	3.00856.1	FIGURE: FIGURE 1

The project area includes approximately 161-acres, ninety-four of which were acquired during a USFS land exchange. The ninety-four acres are designated for the additional nine hole golf course with the remainder for the residential/commercial structures.

Overall topography on the site is characterized by both relatively flat and shallow sloping hillside terrain, with elevations ranging between 7835 and 7930-feet mean sea level (MSL). Shallow drainages flow east and northeast towards Mammoth Creek. Vegetation consists of abundant sagebrush and grasses as well as a few pine trees.

3. PROPOSED DEVELOPMENT

Based upon a review of the proposed development plan(s), it is our understanding that the majority of construction will occur south of Old Mammoth Road and will include a multi-story hotel with at least 2 levels of underground parking, several low to high density residential townhouse structures, some with at least 1-level of underground parking, and others with "tuck-under" parking, an additional nine holes of golf with associated structures and basins, and an outfitters cabin, paved access drives, at-grade parking areas, and associated utilities and appurtenances. Construction to the north of Old Mammoth Road will include a general store, interpretative center, paved parking and access drive areas and associated utilities.

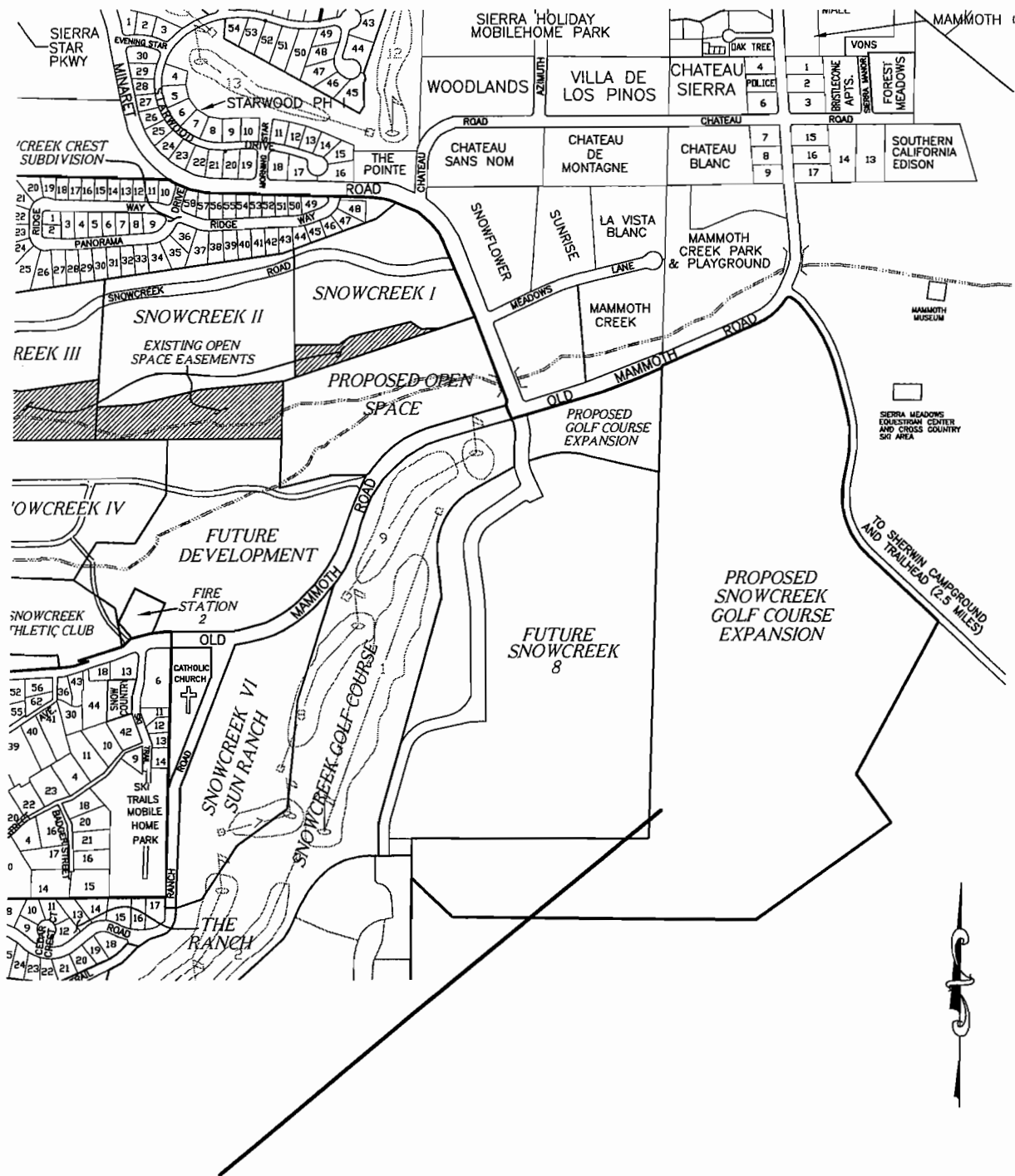
This report is deemed preliminary until foundation and grading plans can be reviewed by SGSI to ensure general conformance with our recommendations. As previously noted, detailed plans for construction are currently not available.

4. AERIAL PHOTOGRAPHIC REVIEW

Prior to our field investigation, we acquired and reviewed aerial photographs to assist in our evaluation of geomorphic features that could be indicative of geologic hazards at the property. Details from the earliest available photographs (1944) did not show any evidence of lineations, scarps, or other ground-surface fault, landslide, or recent avalanche related features.

5. GEOLOGIC AND GEOTECHNICAL SITE CONSTRAINTS

Geotechnical constraints to development include the potential for moderate ground shaking ($M_w \sim 6.6$) along the nearby Hartley Springs fault located approximately 0.7 mi (1.1 km)



PROJECT SITE

PROJECT: VICINITY MAP SNOWCREEK 8	
SCALE: <i>NTS</i>	DATE: <i>9/2006</i>
DRAWING: <i>FIG1.DWG</i>	DRAWN BY: <i>JAA</i>
JOB NO.: <i>3.00856.1</i>	FIGURE: FIGURE 2

west/northwest of the subject parcel. The above concern is addressed in the site seismicity section (see Sections 8 and 9) of this report.

6. GEOLOGY AND SUBSURFACE CONDITIONS

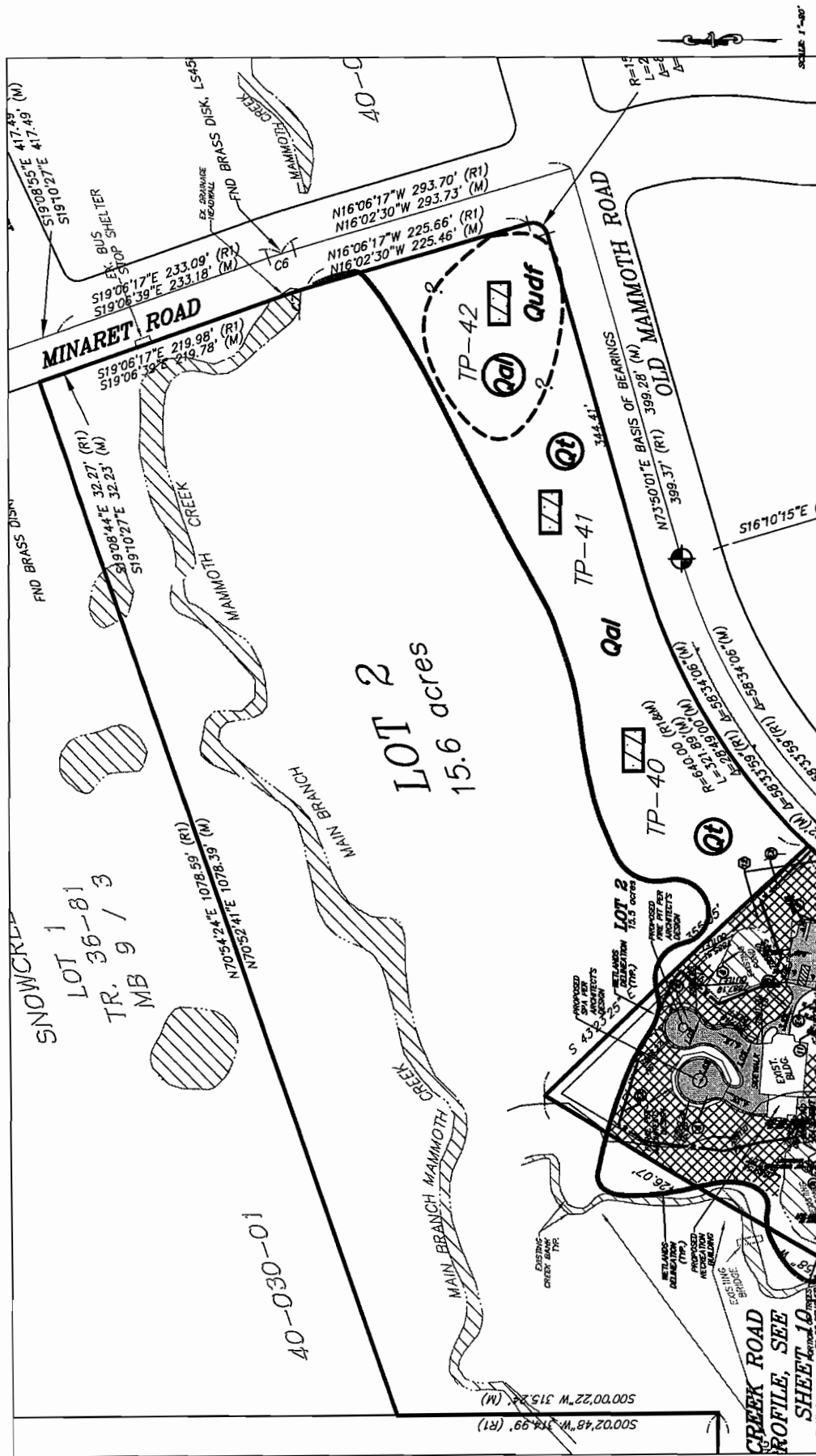
The project site is located within the Sierra Nevada province, a generally north to northwesterly trending, asymmetric, and tilted fault-block, bordered on the east by the Sierra Nevada frontal-fault system. Predominant basement rock types of the Sierra Nevada include Cretaceous granitics with associated Paleozoic roof pendants along the west margin of Mono Basin, and to a lesser degree, Paleozoic meta-sedimentary formations mantled by Pleistocene glacial tills.

More specifically, the project site is located at the southwestern edge of the Long Valley caldera near the eastern flank of the Sierra Nevada. The caldera (collapsed volcano) is an east-west elongate, oval depression formed approximately 760,000 years ago with continued volcanic activity to the present (Bailey, 1989). The pre-volcanic basement rock in the Mammoth Lakes area is predominantly Mesozoic granitic rocks of the Sierra Nevada batholith. The batholith is a series of intrusions that displaced overlying ancient sedimentary sea floor rocks (roof pendants) during the Jurassic and Cretaceous Periods. Piedmont glaciation occurred throughout the Pleistocene leaving a mantle of glacial till covering the basement and volcanic rocks throughout the area now occupied by the Town of Mammoth Lakes.

As observed during this investigation, 3 general soil types underlie the site, consisting of Undocumented fill, Topsoil/Colluvium, and Glacial Deposits. Logs of the subsurface conditions encountered in the exploratory test pits are provided in Appendix A. Generalized descriptions of the materials encountered during this investigation follow.

6.1 Undocumented Fill (Qudf)

Undocumented fill soils were encountered in test pits TP-27 through TP-29, and TP-42, to an approximate maximum depth of 5-feet. In general, the undocumented fill consisted of a dark grayish brown to mottled brown, moist, loose to dense, silty to clayey very fine to coarse SAND (Unified Soil Classification Symbols: SM, SP, and SC-SM) with few cobbles and boulders to 36-inches diameter. This material is considered unsuitable for the support of additional fill or structural loads and should be



LEGEND

Qudf	UNDOCUMENTED FILL (QUERIED WHERE UNCERTAIN)
Qal	TOPSOIL/ALLUVIUM (CIRCLED WHERE BURIED)
Qt	GLACIAL TILL (CIRCLED WHERE BURIED)
---	GEOLOGIC CONTACT
TP-3	APPROXIMATE LOCATION OF EXPLORATORY TEST PIT

PROJECT: **SUBSURFACE GEOTECHNICAL MAP**

SNOWCREEK 8

SCALE: NTS DATE: 10/2006

DRAWING: FIG1DWG DRAWN BY: BM

JOB NO.: 3.00856.1 FIGURE: **FIGURE 4**

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removed from within all structural areas. This material may however be used as fill or backfill if processed in accordance with the earthwork recommendations contained within this report.

6.2 Topsoil/Alluvium (Qal)

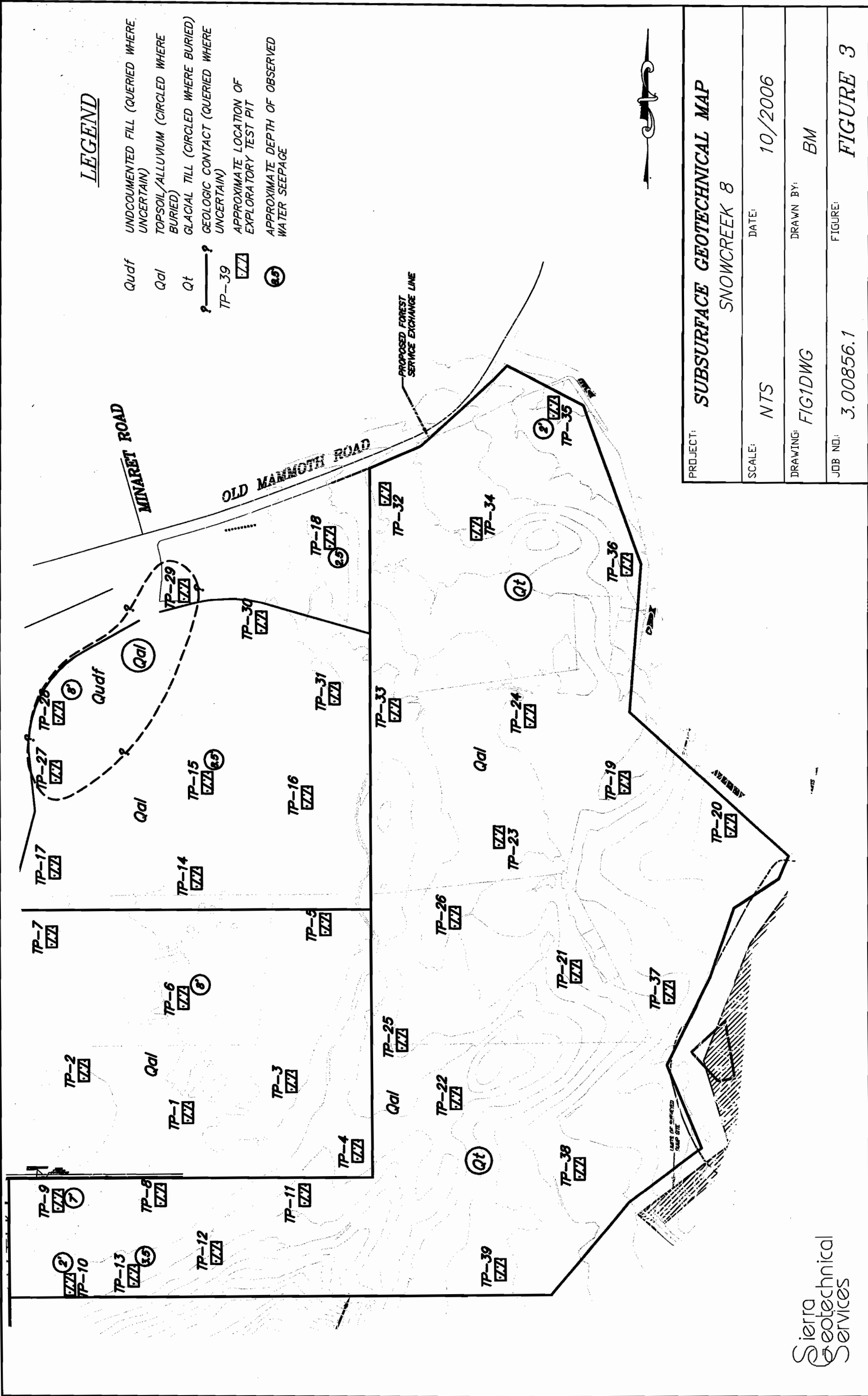
Topsoil/Alluvium was encountered within all the test pits either underlying the undocumented fill or above the glacial deposits. In general, the topsoil/alluvium consisted of a light to dark brown and light to dark gray and olive, moist to wet, loose to dense, silty to clayey, very fine to coarse-grained SAND (Unified Soil Classification Symbols: SP, SM, and SP-SM), with few to abundant organics, random carbonate stringers, and areas with few to abundant cobbles and boulders to 3-feet in diameter. In addition, mottled soils indicative of shallow groundwater conditions were also observed. In some areas the thickness of this deposit was not determined during this investigation. This deposit below approximately 2 to 3-feet is however considered suitable for additional fill and/or structural support provided the earthwork and grading recommendations included within this report are adhered to during site development.

6.3 Glacial Deposits (Qt)

Glacial deposits were encountered in all the test pits, excluding test pits TP-1 through TP-6, TP-8, TP-11, TP-19, TP-24 through TP-25, TP-29 through TP-31, TP-33 through TP-37, and TP-39 through TP-42 below the topsoil/alluvium. In general, the glacial deposits consisted of a light to dark gray and light to medium brown, and reddish-brown, moist to wet, loose to dense, silty, very fine to coarse SAND (SP, SM, SP-SM) and sandy SILT (ML) with few to abundant gravels, and few to abundant cobbles and boulders to approximately 3-feet in diameter. The thickness of the glacial deposits was not determined during this investigation. This deposit is considered suitable for additional fill and/or structural support provided the earthwork and grading recommendations included within this report are adhered to during site development.

6.4 Groundwater

Groundwater seepage was encountered in test pits TP-6, TP-9, TP-10, TP-13, TP-15, TP-18, TP-28 and TP-35 during our field investigation at approximately at depths as high as 2-feet and as low as 8½-feet below the existing grade. In addition mottled soils



indicative of high groundwater conditions were observed in test pits TP-3 through TP-5, TP-14, TP-17, and TP-41. The depths to groundwater reflect site conditions at the time of this investigation. Groundwater conditions often fluctuate seasonally, and the depths recorded may not necessarily be reflective of groundwater elevations during construction. Excavations performed in the spring and early summer in the low lying areas of the site and/or near drainage outlets should anticipate some seepage. Pad grades should be raised from at least 18-inches to 2-feet in areas where shallow groundwater conditions and mottling were observed. In addition, perimeter footing drain systems should be installed along the building perimeters to mitigate against potential moisture intrusion (see attached detail in Appendix D).

Subsurface strata which would retard the flow of water downward were not observed during the investigation. However, any drywell or infiltrator system proposed to be embedded either deeper than the depths explored or in the areas where shallow groundwater was observed should be monitored during construction to ensure that they do not interact with any groundwater. Drywell design may need to be mitigated during construction.

7. FAULTING

Our discussion of faults on the site is prefaced with a discussion of California legislation and state policies concerning the classification and land-use criteria associated with faults. By definition of the California Geological Survey, an "active fault" is a fault that has had surface displacement within Holocene time (about the last 11,000 years); hence constituting a potential hazard to structures that might be located across it. This definition is used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazards Zones Act of 1972, which is detailed in the California Geological Survey Special Publication SP-42 (Hart and Bryant, 1999). The intent of this act is to assure that unwise urban development does not occur across the traces of active faults. Based on our review, the site is not located within any "Earthquake Fault Zones" or Alquist-Priolo Hazard Zones as identified in this document.

8. SITE SEISMICITY

Site coordinates of latitude 37.6296° north and longitude 118.9677° west were estimated using the computer program **Microsoft Streets and Trips (2004)**. The computer programs

EQFAULT and **EQSEARCH** (Blake, 2000) were used to estimate peak horizontal accelerations from regional faults and tabulate data from historical earthquakes.

A deterministic seismic analysis was performed within a 62.2 mi (100 km) radius of the site using the computer program **EQFAULT** (Blake, 2000). The results of the analysis indicate that the peak ground acceleration estimated for a maximum earthquake event within the specified radius is 0.46g. This acceleration represents deterministic peak ground accelerations and could occur from a magnitude 6.6 (M_w) earthquake on the Hartley Springs fault located approximately 0.7 mi (1.1 km) northwest the site. The Hilton Creek fault, located approximately 6.1 mi (9.8 km) from the site could produce a magnitude 6.7 (M_w) earthquake resulting in a peak horizontal ground acceleration of 0.28g at the site. The tabulated results of the deterministic seismic analysis are presented in Appendix C. The Fault Location Map, which depicts active faults within a 62.2 mi (100 km) radius of the site, is also presented in Appendix C.

The computed maximum site acceleration within a 62.2 mi (100 km) radius of the site was derived from **EQSEARCH** (Blake, 2000) during the time period of 1800 to 2005. The largest estimated site acceleration based on the Boore et al. (1997) model, was 0.24g, which occurred during the Mammoth Lakes Earthquake of January 7, 1983. This earthquake was located approximately 2.8 mi (4.5 km) from the site. The Modified Mercalli Intensity and earthquake magnitude were IX and 5.6 (M_w) respectively. The largest earthquake recorded within the specified distance and time period was a magnitude 6.6 (M_w) earthquake (Modified Mercalli Intensity of VII) which occurred in The Owens Valley on April 11, 1872. A site acceleration of 0.09g was estimated from this earthquake which was located approximately 27.1 mi (43.6 km) from the site.

The tabulated results of the historical analysis are presented in Appendix C. The Earthquake Epicenter Map, which depicts the epicenters and magnitudes of historical earthquakes that have affected the site, an Earthquake Recurrence Curve, and a plot depicting Earthquake Events versus Magnitude also presented in Appendix C.

The computer program **FRISKSP** (Blake, 2000) was used to perform a probabilistic analysis of seismicity at the subject site. The probabilistic analysis was used to define the Upper-Bound and Design Basis Earthquakes at the site for use in structural design. These results as well as Probability of Exceedance versus Acceleration graphs, and Return Period versus Acceleration graphs are presented in Appendix C. Based on the results of the probabilistic analysis, the

Upper-Bound Earthquake (Non-Magnitude Weighted) for the site, defined as the ground motion that has a 10-percent chance of exceedance in 100 years, with a statistical return period of ~ 949 years, is 0.44g. The Design Basis Earthquake (Non-Magnitude Weighted) for the site, defined as the ground motion that has a 10-percent chance of exceedance in 50 years, with a statistical return period of ~ 475 years, is 0.35g.

8.1 Seismic Design Criteria

Table 1 presents the Seismic Parameters for use in preparing a Design Response Spectra for the site. The program used to obtain the seismic parameters is **UBCSEIS** which is based upon the 1997 Uniform Building Code (UBC) and 2001 California Building Code (CBC). The results of the UBC Seismic Design Parameters as well as the Design Response Spectra are presented in Appendix C.

TABLE 1

UBC-CHAPTER 16 TABLE NO.	SEISMIC PARAMETER	RECOMMENDED VALUE
16-I	Seismic Zone Factor Z	0.4
16-J	Soil Profile Type	S_c
16-Q	Seismic Coefficient C_a	0.57
16-R	Seismic Coefficient C_v	1.02
16-S	Near Source Factor N_a	1.3
16-T	Near Source Factor N_v	1.6
16-U	Seismic Source Type	B

The subject site is situated in Seismic Zone 4 ($Z=0.4$) based on the 1997 UBC, and the 2001 CBC. A geologic subgrade type S_c , "very dense soil" was assumed for the site based upon visual observations of the subsurface conditions.

The Boore et al (1997) NEHRP C (520) acceleration-attenuation relation was used to estimate ground accelerations at the site based upon the shear wave velocity data. The seismic coefficients of acceleration and velocity C_a and C_v , as derived from the soil profile type and seismic zone factor, are 0.57 and 1.02 respectively.

The distance between the site and the nearest active fault is less than 2 km; therefore the near-source acceleration and velocity factors N_a and N_v are 1.3 and 1.6 respectively. The nearest known active fault is the Hartley Springs fault located approximately 0.7 mi (1.1 km) northwest of the site. The Hartley Springs fault is a Type B Seismic Source.

Conformance to the above criteria for strong ground shaking does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur during a large magnitude earthquake. Design of structures should comply with the requirements of the governing jurisdictions, building codes, and standard practices of the Association of Structural Engineers of California. A Design Civil or Structural Engineer in conjunction with the State Architect should determine what level of risk is acceptable for the project considering the recommendations contained in this report, economics, and safety.

9. SECONDARY EARTHQUAKE EFFECTS

Secondary effects that can be associated with severe ground shaking following a relatively large earthquake include shallow ground rupture; soil lurching, liquefaction, dynamic settlement, seiches and tsunamis, and avalanches (rockfall and snow). These secondary effects of seismic shaking are discussed in the following sections.

9.1 Shallow Ground Rupture

Ground surface rupture results when the movement along a fault is sufficient to cause a gap or break along the upper edge of the fault zone on the surface. Our review of available geologic literature indicated that there are no known active, potentially active, or inactive faults that transect the subject site. The nearest known active regional fault is the Hartley Springs Fault. The closest projected trace for this fault zone is located approximately 0.7 mi (1.1 km) northwest of the site.

9.2 Soil Lurching

Soil lurching refers to the rolling motion on the ground surface by the passage of seismic surface waves. Effects of this nature are likely to be most severe where the thickness of soft sediments varies appreciably under structures. In its present condition,

the potential for lurching below the proposed structures is considered low to moderate due to the existence of potentially compressible soils within the upper few feet of material below existing grades. The potential for lurching may be greatly reduced if the potentially compressible soils, present on site, are removed and properly compacted during grading, as per the earthwork recommendations provided herein.

9.3 Liquefaction

Liquefaction of cohesionless soils can be caused by strong vibratory motion due to earthquakes. Research and historical data indicate that loose granular soils below a near-surface groundwater table are most susceptible to liquefaction. Liquefaction is characterized by a loss of shear strength in the affected soil layers, thereby causing the soil to behave as a viscous liquid. This effect may be manifested at the ground surface by settlement and, possibly, sand boils where insufficient confining overburden is present over layers. In order for the potential effects of liquefaction to be manifested at the ground surface, the soils generally have to be granular, loose to medium-dense and saturated relatively near the ground surface, and must be subjected to ground shaking of a sufficient magnitude and duration. The potential for liquefaction to occur is considered very low, given the lack of a water table and the dense nature of bearing soils present on site.

9.4 Dynamic Settlement

Portions of the shallow granular on-site soils may be loose and susceptible to dynamic settlement if strongly shaken by the design level earthquake. The potential for dynamic settlement will be greatly reduced if the loose and compressible soils near the surface are removed and properly compacted in accordance with the earthwork and grading recommendations contained within this report.

9.5 Seiches and Tsunamis

The potential for seiches and tsunamis as the result of the design level earthquake in a nearby fault are considered non-existent, due to the distance of the ocean or large open bodies of water from the project site.

9.6 Avalanches (Rockfall and Snow)

Avalanches can occur as a result of moderate to large earthquakes in Alpine terrain, which can cause rock and snow to move vertically and laterally downslope. These hazards typically affect structures which are located at the base of slopes or within close proximity to the area of flow. The potential for rockfall or snow avalanches to occur at the subject site is considered low, though evidence of avalanches along the mountain range to south and southwest were observed.

10. LANDSLIDES

Evidence of past landslides was not observed either during aerial photographic review or in the field.

11. VOLCANIC HAZARDS

The area of eastern California that includes the Long Valley Caldera and the Mono-Inyo Craters volcanic chain has a long history of geologic activity that includes earthquakes and volcanic eruptions. Studies within this area indicate that massive eruptions of the size that accompanied formation of Long Valley Caldera approximately 760,000 years ago are extremely rare (none have occurred during the period of written human history). Currently, there is no evidence that an eruption of such catastrophic proportions might be forming beneath the Long Valley caldera (Miller, 1985; 1989).

A small to moderate volcanic eruption could occur however; somewhere along Mono-Inyo Craters volcanic chain producing pyroclastic flows and surges, as well as volcanic ash and pumice fallout, which could significantly impact the subject site. The odds however, of such an eruption are roughly one in a thousand in a given year (Miller, 1985; 1989).

12. CONCLUSIONS

Based on the results of this investigation, it is our opinion that the construction of the proposed project is feasible from a geotechnical standpoint provided the following recommendations are incorporated into the design and construction. The following sections discuss the principal geotechnical concerns affecting site development and grading and provide preliminary grading

and foundation design recommendations which should be implemented during site development to mitigate site geologic constraints.

- There are no known active, potentially active, or inactive faults that transect the subject site. Evidence of past soil failures, landslides, or active faulting on the site was not encountered. Seismic hazards at the site may be caused by ground shaking during seismic events on regional active faults. The nearest known active regional fault is the Hartley Springs fault located approximately 0.7 mi (1.1 km) west/northwest of the site.
- Based on the results of the probabilistic analysis, the Upper-Bound and Design Basis Earthquakes for the site yielded peak ground accelerations of 0.44g and 0.35g respectively.
- The project consultants and the Client should discuss various seismic design parameters and decide upon an appropriate design value based upon their seismic performance goals. A design value of 0.35g is the lowest value that should be considered.
- A volcanic eruption could occur somewhere along Mono-Inyo Craters volcanic chain producing pyroclastic flows and surges, as well as volcanic ash and pumice fallout, which could significantly impact the subject site. The odds however, of such an eruption are roughly one in a thousand in a given year (Miller, 1985; 1989).
- The subject property is situated on relatively flat to slightly sloping terrain underlain in areas by undocumented fills, and loose topsoil/alluvium, to approximately 2 to 3-feet in depth, considered "unsuitable" for the support of new fill or structural loads. Where these soils will be subjected to increased loads from new fills, remedial grading consisting of overexcavation and compaction is recommended to improve the bearing capacity of those materials. Remedial grading recommendations are provided in this report.
- The depth of the unsuitable soils is based upon the areas observed during the field investigation. It should be anticipated that the overall depth of the unsuitable materials exposed during construction may vary from that encountered in the test pits. Reasonably continuous construction observation and review during site grading and foundation installation allows for evaluation of the actual soil conditions and the ability to provide appropriate revisions where required during construction.

- Reasonably continuous construction observation and review during site grading and foundation installation allows for evaluation of the actual soil conditions and the ability to provide appropriate revisions where required during construction.
- Groundwater seepage was encountered during our field investigation at depths between approximately 2-feet to 8½-feet below grade. The depths to groundwater reflect site conditions at the time of this investigation. Groundwater conditions fluctuate seasonally, and the depths recorded may not necessarily be reflective of groundwater elevations during construction. Excavations performed in the spring and early summer in the low lying areas of the site and/or near drainage outlets should anticipate some seepage. Pad grades should be raised from at least 18-inches to 2-feet in areas where shallow groundwater conditions and mottling were observed. The depths to groundwater (where observed) are included on Figure 3.
- Subsurface strata which would retard the flow of water downward were not observed during the investigation. However, any drywell or infiltrator system proposed to be embedded either deeper than the depths explored or in the areas where shallow groundwater was observed should be monitored during construction to ensure that they do not interact with any groundwater. Drywell design may need to be mitigated during construction.
- Site soils encountered during our field investigation generally consist of loose to dense, silty to clayey, very fine to coarse-grained sand deposits with cobbles and boulders to approximately 3-feet in diameter. Excavations at the site should be achievable using standard earthmoving equipment.

13. RECOMMENDATIONS

The following recommendations should be adhered to during site development. These recommendations are based on empirical and analytical methods typical of the standard of practice in California. If these recommendations appear not to cover any specific feature of the project, please contact our office for additions or revisions to the recommendations.

13.1 **Geotechnical Review**

Geotechnical review is of paramount importance in engineering practice. The poor performance of many foundation and earthwork projects has been attributed to inadequate construction review. Sierra Geotechnical Services, Inc. should be provided the opportunity to review the following items or we waive all liability for any and all geotechnical issues associated with grading or construction relative to the subject site.

13.1.1 Plan and Specification Review

Detailed plans for construction and grading were not available at the time of this report. SGSI should review grading and foundation plans prior to construction in order to assure that they are in conformance with this report; some of the recommendations contained herein may need to be revised after reviewing.

13.2 Earthwork

Earthwork should be performed in accordance with the General Earthwork and Grading Specifications in Appendix D and the following recommendations. The recommendations contained in Appendix D are general grading specifications provided for typical grading projects. Some of the recommendations may not be strictly applicable to this project. The specific recommendations contained in the text of this report supersede the general recommendations in Appendix D. The contract between the developer and earthwork contractor should be worded such that it is the responsibility of the contractor to place the fill properly in accordance with the recommendations of this report and the specifications in Appendix D notwithstanding the testing and observation of the geotechnical consultant.

13.2.1 Site Preparation

Prior to grading, the proposed structural improvement areas (i.e. all structural fill, pavements areas and structural building, etc.) of the site should be cleared of surface and subsurface obstructions, including vegetation. Vegetation and debris should be disposed of off site. Holes resulting from removal of buried obstructions, which extend below the recommended removal depths described herein or below finished site grades (whichever is lower) should be filled with properly compacted soil. Should existing underground utilities be encountered they should be completely removed and properly backfilled. Alternatively if the utility is not within the influence zone of the foundation it may be abandoned in place by fully grouting the pipe.

13.2.2 Removals and Compaction

The subject property is underlain by up to approximately 3-feet of loose, undocumented fills and 2 to 3-feet of loose, topsoil/alluvium considered unsuitable for the support of new fill or structural loads. If grading will consist of the overexcavation and removal of “unsuitable” soils from within all structural areas, then excavations should extend below the unsuitable material and to a minimum horizontal distance of one-half the footing width or 5-feet (whichever is greater) horizontally outside the footing footprint. If earthwork will consist of foundation excavations only, then all footings shall be embedded at least 12-inches below the estimated removal depth (3-feet). These excavations will likely extend deeper than the minimum 18-inch depth below outside adjacent grade requirement (see section 13.4.1).

For the paved roadways, parking areas and other improvements a 1 to 3-foot removal is recommended depending on site conditions (i.e. depth of root zone, and depth of disturbance which may have locally deeper removal depths). The removal should also extend a minimum horizontal distance of 2-feet beyond the back of curbs and pavement. In addition, the removal bottom should be observed (tested as needed) by the geotechnical consultant prior to placing fill soils. Removals and Compaction recommendations are provided in Appendix D.

13.3 Excavation and Grading Observation

Site grading and footing excavations should be observed by SGSI. Such observations are considered essential to identify field conditions that differ from those anticipated by the investigation, to adjust design to actual field conditions, and to determine that the grading is accomplished in general accordance with the recommendations of this report. Earthwork and grading recommendations which include guidelines for site preparation fill compaction, slopework, temporary excavations, and trench backfill are provided in Appendix D.

13.4 Preliminary Foundation Preparation and Design

The following preliminary recommendations are presented as minimum design recommendations; they are not intended to supercede design by the structural engineer. Preliminary foundations should be designed in accordance with structural considerations and the following recommendations. Upon the completion of the grading and structural plans, Sierra Geotechnical Services Inc. should review the foundation loads and embedment in order to confirm the implementation of the recommendations herein.

Continuous or pad footings may be used to support the proposed structures provided they are founded entirely upon properly compacted fill, competent alluvial deposits encountered below approximately 3-feet in depth, or competent glacial deposits. Continuous and isolated column foundations should be sized according to the allowable soil bearing pressures shown in Table II below. The pressures shown on Table II are for dead load plus long-term live load, including snow load, and for total load including wind and seismic forces.

Table II – Allowable Soil Bearing Pressures

Depth Below Existing Ground Surface	Allowable Soil Bearing Pressure (psf)
0-3'	2,500
3-10'	3,000
10'+	4,000

The allowable pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces. A friction coefficient for concrete of 0.35 may be employed to resist lateral loads. Continuous and isolated footings should be designed in accordance with the structural engineer requirements. Reinforcement of footings should be per the structural engineer's design.

13.4.1 Foundation Construction

Based upon our observations and past experience relative to the general site area, low expansive soils exist onsite. The following preliminary recommendations assume low expansive soils near finish pad grade.

- Footings should be designed in accordance with the structural engineer requirements regarding width. Exterior and interior foundations shall be founded within compacted fill or competent native soils. Exterior foundations shall have a minimum embedment depth of 18-inches below outside adjacent grade. If earthwork will consist of foundation excavations only, then all footings shall be embedded at least 12-inches below the estimated removal depth of 3-feet.
- All footings should be reinforced to at least the minimum reinforcement for temperature as required in Chapter 19 of the 1997 UBC.
- Although no specific pre-saturation is required for these soil conditions, footing trench excavations should be well moistened prior to pouring concrete.
- Footing trenches should not have any rocks or boulders protruding into the trench bottom. Soft soil pockets created by rock removal during foundation excavation shall be replaced with approved fill material, and compacted to 95-percent of the material's maximum dry density.

13.5 Foundation Setback

We recommend a minimum 5-foot horizontal setback distance from the face of slopes for all structural footings and settlement-sensitive structures (i.e. fences, walls, signs, etc.). This distance is measured from the outside edge of the footing, horizontally to the slope face (or to the face of a retaining wall). We should note that the soils within a slope setback area possess poor long term lateral stability, and improvements (such as retaining wall, sidewalks, fences, pavement, underground utilities, etc.) constructed within this setback area may be subject to lateral movement and/or differential settlement.

Utility trenches that parallel or nearly parallel structure footings should not encroach within a 1:1 plane extending downward and outward from the outside edge of the footing.

13.6 Concrete Slab-on-Grade Floors

Compacted fill materials will provide adequate support for concrete slabs provided the on-site materials are prepared per our grading recommendations prior to placement of the slab.

Structural fill and subgrade soils underlying concrete slabs shall be compacted to a minimum of 95-percent of the material's maximum dry density for the upper 12-inches. Concrete slabs should be underlain by a 1-inch layer of clean sand (SE greater than 30) to aid in concrete curing, which is underlain by a 10-mil (or heavier) moisture barrier, which is, in turn, underlain by a 1-inch layer of clean sand to act as a capillary break. All penetrations and laps in the moisture barrier should be appropriately sealed.

Minimum slab reinforcement shall consist of #3 rebar placed at 18-inches on center each way. The slab reinforcement shall be placed, vertically, in the middle of the slab. Slab thickness shall be a minimum of 4-inches. In areas where heavy equipment or loading will stress the slab, the thickness and reinforcement will meet the requirements of the Structural Engineer of record. Our experience indicates that the use of reinforcement in slabs and foundations will generally reduce the potential for drying and shrinkage cracking. However, some cracking may be expected as the concrete cures. Concrete cracking and/or spalling is often aggravated by a high cement ratio, high or low concrete temperature at the time of placement, small nominal aggregate size and rapid moisture loss. The use of low slump concrete (not exceeding 4-inches at the time of placement) and proper curing methods can reduce the potential for shrinkage cracking.

Moisture barriers can retard, but not eliminate moisture vapor movement from the underlying soils up through the slab. We recommend that the floor coverings installer test the moisture vapor flux rate prior to attempting application of the flooring. "Breathable" floor coverings should be considered if the vapor flux rates are high. A slipsheet should be used if crack sensitive floor coverings are planned.

13.7 Preliminary Pavement Recommendations

A preliminary pavement section of 3-inch asphalt concrete underlain by 4-inches of compacted Class II aggregate base may adequately support any access drives. For areas

where heavier loading (truck traffic, buses parking etc...) are anticipated, a pavement section of 4-inch asphalt concrete underlain by 6-inches of compacted Class II aggregate base should be used.

The upper 12-inches of subgrade material along with the Class II Aggregate Base and the Asphaltic concrete shall be compacted to a minimum of 95-percent of the material's maximum dry density as determined by ASTM D1557-2000. The subgrade and aggregate base shall be moisture-conditioned and compacted to 95-percent of the material's maximum dry density as determined by ASTM D-1557-2000 to a depth of 12-inches.

Based upon our experience in the Mammoth Lakes area, environmental conditions such as freeze-thaw and thermal cracking will most likely govern the life of the asphalt pavement. Therefore, a 3-inch AC section is the minimum recommended.

If pavement areas are adjacent to heavily watered landscape areas, some deterioration of the subgrade load bearing capacity may result. We recommend some measures of moisture control (such as deepened curbs or other moisture barrier materials) be provided to prevent the subgrade soils from becoming saturated.

13.8 Lateral Earth Pressures and Resistance

Embedded structural walls or cantilever retaining walls should be designed for lateral earth pressures exerted on them. The magnitude of these pressures depends on the amount of deformation that the wall can yield under load. If a wall can yield enough to mobilize the full shear strength of the soil; it can be designed for "active" pressure.

If a wall cannot yield under the applied load, the shear strength of the soil cannot be mobilized and the earth pressure will be higher. Such walls should be designed for "at rest" conditions. If a structure moves toward the soils, the resulting resistance developed by the soil is the "passive" resistance.

For design purposes, the recommended equivalent fluid pressure for each case for walls founded above the static ground water and backfilled with soils of very low to low expansion potential is provided. The equivalent fluid pressure values assume free-draining conditions. If conditions other than those assumed above are anticipated the

equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer. Surcharge loading effects from the adjacent structures should be evaluated by the structural engineer. The select backfill should have an expansion index (EI) of no greater than 50 and a sand equivalent (SE) greater than 30. The backfill soils should be tested by the soils engineer prior to backfill operations starting for the retaining wall/basement wall structures.

Slope of Backfill Behind Retaining Wall

Lateral Earth Pressure in Equivalent Fluid Weight (pcf)

	Active Case	Passive Case
Horizontal	40	355
At-Rest	55	

For sliding resistance, the friction coefficient of 0.35 may be used at the concrete and soil interface. Wall footings should be designed in accordance with structural considerations. If both the passive and frictional resistances are assumed to act together than we recommend that a minimum factor of safety of 2.0 is used for design. The passive resistance value may be increased by one-third when considering loads of short duration, including wind or seismic loads. The horizontal distance between foundations providing passive resistance should be a minimum of three times the depth of the foundations to allow full development of passive pressures. The total depth of retained earth for design of cantilever walls should be the vertical distance below the ground surface measured at the wall face for stem design or measured at the heel of the footing for overturning and sliding.

Wall backcut excavations less than 5-feet in height can be made near vertical. All retaining wall structures should be provided with appropriate drainage and waterproofing. Drainage should consist of continuous drains installed along the base of the wall outletting to a storm drain system or the surface if grade allows.

13.8.1 Earthquake Induced Lateral Earth Pressures

$$P_e = \text{Horizontal Force} = \frac{3}{8} (\alpha_{\max}/g) \gamma_t H^2 \text{ (acting @ } 0.6H \text{ above base of wall)}$$

Per Seed and Whitman 1970

$\alpha_{\max}$ = Peak acceleration at the ground surface = **0.35g** (per FRISK)

γ_t = Avg. unit wt. of native soil (below 3-feet) = **118 pcf**

H = Wall Max. Height = **10-feet**

g = Gravity (9.81m/s²)

P_e = 1,549 lbs per linear foot of wall

For free standing or cantilevered walls the seismic increment of lateral pressure should be applied at a distance of $0.6H$ above the toe of the wall. The pressure increment for cantilevered retaining walls should be taken as an inverted triangular distribution from the stem of the cantilevered retaining wall to the top of the cantilevered retaining wall. For resistive walls, i.e. basement walls, the pressure increment should be taken as a rectangular force applied from the stem of the basement wall to the top of the basement wall.

Under the combined effects of static and earthquake loads on the wall, a factor of safety between 1.1 and 1.2 is acceptable when evaluating the stability (sliding, overturning) of the wall (NAVFAC DM 7.2). Because P_e is a short term load that may never occur during the life of the wall, it is common practice to allow a $1/3$ increase in passive resistance for earthquake analysis rather than apply the more conservative value presented above.

13.9 Estimated Settlement

Post construction settlement is estimated to be one-half inch or less if the foundation recommendations provided in this report are conformed to. Post-construction differential settlements should be one-quarter inch or less. Settlements for similarly loaded footings located on varying thicknesses of fill may experience differential settlements on the order of 0.5 percent of the difference in fill thickness beneath the footings. We recommend that the foundation plans be reviewed once detailed loading conditions are known to confirm the estimated settlements mentioned above.

13.10 Drainage

We recommend that measures be taken to properly finish grade the building area, such that drainage water from the building area is directed away from building foundations (2-percent minimum grade on soil or sod for a distance of 5-feet). Ponding of water should not be permitted. Erosion is possible surrounding the structures if left unprotected during the snowmelt run-off season.

13.11 Erosion

A Storm Water Pollution Prevention Plan (SWMP) will need to be prepared for this site in compliance the Town of Mammoth Lakes and State Water Quality Control Board – Lahontan Region requirements. Graded areas shall be protected against erosion once they are brought to final grade. No graded areas shall be left unstabilized between October 15th and April 15th.

14. LIMITATIONS

This report has been prepared for the sole use and benefit of our client. The conclusions of this report pertain only to the site investigated. The intent of the report is to advise our client of the geologic and geotechnical recommendations relative to the future development of the proposed project. It should be understood that the consulting provided and the contents of this report are not perfect. Any errors or omissions noted by any party reviewing this report, and/or any other geotechnical aspects of the project, should be reported to this office in a timely fashion. The client is the only party intended by this office to directly receive this advice. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Sierra Geotechnical Services Incorporated from and against any liability, which may arise as a result of such use or reliance, regardless of any fault, negligence, or strict liability of Sierra Geotechnical Services Incorporated.

Conclusions and recommendations presented herein are based upon the evaluation of technical information gathered, experience, and professional judgment. Other consultants could arrive at different conclusions and recommendations. Final decisions on matters presented are the responsibility of the client and/or the governing agencies. No warranties in any respect are made as to the performance of the project.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings within this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

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Stereoscopic Aerial Photographs

OV, FL-1, 7-84 & 85; Dated 10/19/1944, 1:24,000

GS-QN, FL-3, 65 & 66; Dated 8/10/1951, 1:47,200

United States Department of Agriculture, Stereoscopic Aerial Photographs: EAD -19-131 to EAD-19-132; Dated 8/23/56, 1:24,000.

United States Department of Agriculture, Stereoscopic Aerial Photographs: EMG-7-147 to EMG-7-148; Dated 9/10/63, 1:12,500

United States Department of Agriculture, Stereoscopic Aerial Photographs: FL-7, 483-36&39; Dated 8/06/84, 1:12,000

United States Department of Agriculture, Stereoscopic Aerial Photographs: 615040: 501-86 to 501-87; Dated 7/14/01, 1:12,500

APPENDIX A

EXPLORATORY TEST PIT LOGS

A field investigation was performed from August 1st, through August 3rd, 2006 that included the excavation of forty-two exploratory test pits with a Case 580 backhoe equipped with a 24-inch bucket. A geologist from our office logged the excavations as they were advanced. Logs of the exploratory test pits are presented herein. The approximate locations of the exploratory test pits are shown on the Subsurface Geotechnical Maps (Figures 3 and 4).

In-place nuclear density tests and bulk samples of the soils encountered were obtained during the field investigation. Results of the in-place nuclear density tests are presented on the logs of the exploratory test pits. Details of the laboratory testing are presented in Appendix B.

SIERRA GEOTECHNICAL SERVICES INC.

P.O. BOX 5024

MAMMOTH LAKES, CA 93546

(760) 934-3992

TEST PIT LOGSJOB NO: 3.00856.1DATE: 8/1/06PROJECT: Snowcreek 8LOGGED BY: PS

TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
1	0 - 2	SM				<u>Topsoil/Alluvium</u> Light brown to light grayish-brown, moist, loose to medium dense, silty, very fine to fine SAND, moderate roots.
	2 - 4	SM	2	16.9	101.8	<u>Glacial Deposits</u> Light to medium gray, moist, medium dense, silty, very fine to fine SAND, with cobbles and large boulders up to 30-inches in diameter.
	4 - 6	SP-SM				Medium to olive-gray, moist, medium dense to dense, silty, very fine to medium grained SAND, with gravels. ----- <i>Total depth = 6-feet. No groundwater encountered. Backfilled 8/1/06.</i>
2	0 - 4	SM				<u>Topsoil/Alluvium</u> Medium brown, moist, loose, silty, very fine to medium grained SAND, with cobbles, and boulders up to 36-inches in diameter. Rock content comprising 25-35% of deposit.
	2 - 4	SP-SM				Light to medium brown, moist, loose to medium dense, silty, very fine to medium grained SAND, with cobbles and boulders to 14-inches diameter.
	3½ - 5	SM				<u>Glacial Deposits</u> Gray, moist, medium dense to dense, silty, fine to medium grained SAND, with gravels, and cobbles. ----- <i>Total depth = 5-feet. No groundwater encountered. Backfilled 8/1/06.</i>

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TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
3	0 - 1½	SM				<u>Topsoil/Alluvium</u> Light grayish-brown, moist, loose, silty, very fine to fine SAND, moderate roots.
	1½ - 4	SM				<u>Glacial Deposits</u> Light gray, moist, medium dense, silty, very fine to fine SAND, with abundant angular rock fragments comprising 25-35% of deposit.
	4 - 7	SP-SM				Mottled gray to reddish-brown, moist, medium dense, silty, fine to coarse grained SAND, with gravels, cobbles, and boulders to 14-inches diameter. ----- <i>Total depth = 7-feet. Groundwater mottling @ 4-feet. No groundwater encountered. Backfilled 8/1/06.</i>
4	0 - 4	SP-SM				<u>Topsoil/Alluvium</u> Medium brown, moist, loose, silty, fine to coarse grained SAND, with cobbles and boulders up to 42-inches in diameter, moderately rooted. Rock content comprising 25-35% of deposit.
	4 - 6	SP	4	8.3	120.0	<u>Glacial Deposits</u> Gray, moist, medium dense to dense, fine to coarse grained SAND, with gravels and cobbles. ----- <i>Total depth = 6-feet. Groundwater mottling below 5-feet. No groundwater encountered. Backfilled 8/1/06.</i>

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TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
5	0 - 3	SM	1	8.6	72.8	<u>Topsoil/Alluvium</u> Light brown, moist, loose, silty, very fine to fine SAND, with cobbles, moderately rooted.
	3 - 4½	SM	3	12.1	91.0	Light brown to light grayish-brown, moist, medium dense, silty, very fine to fine SAND, with gravels and cobbles.
	4½ - 6	SM				<u>Glacial Deposits</u> Light gray to gray, moist, medium dense, silty, very fine to coarse grained SAND, with gravels. ----- Total depth = 6-feet. Groundwater mottling @ 4-feet. No groundwater encountered. Backfilled 8/1/06.
6	0 - 4	SM	2	40.4	63.8	<u>Topsoil/Alluvium</u> Light grayish-brown to light olive gray, moist, silty, very fine to fine SAND moderately rooted. Carbonate staining below 2-feet.
	4 - 7	SP	4	17.3	102.8	Olive brown to reddish-brown, moist, medium dense, very fine to coarse grained SAND, and carbonate staining.
	7 - 9	SP				<u>Glacial Deposits</u> Dark reddish-brown, moist to saturated, medium dense, medium to coarse grained SAND.
	9 - 10	SP-SM				Gray, moist, medium dense to dense, silty, very fine to medium grained SAND. ----- Total depth = 10-feet. Groundwater seepage at approximately 8-feet. Backfilled 8/1/06.

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TEST PIT LOGSJOB NO: 3.00856.1DATE: 8/1/06PROJECT: Snowcreek 8LOGGED BY: PS

TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
7	0 - 3	SC-SM	3	17.3	93.8	<u>Topsoil/Alluvium</u> Light to medium gray, moist, loose to medium dense, silty to clayey, very fine to fine SAND, moderately rooted, with thin carbonate lenses at 2-feet.
	3 - 5	SP-SM				Gray to olive gray, moist, medium dense, fine to coarse grained SAND, with gravels, cobbles, and thin carbonate lenses. ----- Total depth = 5-feet. No groundwater encountered. Backfilled 8/1/06.
8	0 - 3	SP-SM				<u>Topsoil/Alluvium</u> Light grayish-brown, moist, loose, silty, very fine to medium grained SAND, with gravels moderately rooted.
	3 - 10	SP-SM				<u>Glacial Deposits</u> Brown to light grayish-brown, moist, medium dense to dense, fine to coarse grained SAND, with gravels and cobbles. ----- Total depth = 10-feet. No groundwater encountered. Backfilled 8/1/06.

SIERRA GEOTECHNICAL SERVICES INC.**P.O. BOX 5024****MAMMOTH LAKES, CA 93546****(760) 934-3992****TEST PIT LOGS****JOB NO: 3.00856.1****DATE: 8/1/06****PROJECT: Snowcreek 8****LOGGED BY: PS**

TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
9	0 - 3	SC-SM				<u>Topsoil/Alluvium</u> Medium gray, moist, medium dense to stiff, silty to clayey, very fine to fine SAND Moderately rooted.
	3 - 7	SP-SM				Medium gray, moist to saturated, medium dense to dense, fine to coarse grained SAND, with gravels, cobbles, and few boulders. ----- <i>Total depth = 7-feet. Groundwater seepage at approximately 7-feet. Backfilled 8/1/06.</i>
10	0 - 3	SM				<u>Topsoil/Alluvium</u> Dark brown, moist to saturated, loose, silty, very fine to medium grained SAND, with gravels and cobbles. Organic rich, highly rooted.
	3 - 4	SP				Brown to reddish-brown, saturated, medium dense, fine to coarse grained SAND, with gravels and cobbles. ----- <i>Total depth = 4-feet. Groundwater encountered at approximately 2-feet. Backfilled 8/1/06.</i>

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TEST PIT LOGSJOB NO: 3.00856.1DATE: 8/1/06PROJECT: Snowcreek 8LOGGED BY: PS/SH

TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
11	0 - 6	SP-SM				<u>Topsoil/Alluvium</u> Brown, moist, loose to medium dense, silty, very fine to medium grained SAND, with cobbles and boulders to 36-inches in diameter. Rock content comprising 30-40% of deposit. Moderately rooted.
	6 - 8	SP				<u>Glacial Deposits</u> Gray, moist, medium dense to dense, fine to coarse grained SAND, with gravels, cobbles and boulders. ----- <i>Total depth = 8-feet. No groundwater encountered. Backfilled 8/1/06.</i>
12	0 - 9	SM				<u>Topsoil/Alluvium</u> Medium to dark brown, moist, medium dense, silty, fine to medium grained SAND, with gravels and cobbles. ----- <i>Total depth = 9-feet. No groundwater encountered. Backfilled 8/1/06.</i>
13	0 - 3½	SM				<u>Topsoil/Alluvium</u> Medium to dark brown, moist, loose to medium dense, silty, fine to medium grained SAND, with few gravels.
	3½ - 7	SM				Medium brown, saturated, medium dense, silty, medium to coarse grained SAND, with gravels and cobbles. ----- <i>Total depth = 7-feet. Backhoe refused at 7-feet. Groundwater seepage encountered at approximately 3½ -feet. Backfilled 8/1/06.</i>

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TEST PIT LOGSJOB NO: 3.00856.1DATE: 8/2/06PROJECT: Snowcreek 8LOGGED BY: PS

TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
14	0 - 5	SM	2	22.5	67.8	<u>Topsoil/Alluvium</u> Light brown to gray, moist, loose to medium dense, silty to clayey, very fine to fine SAND. Moderately rooted.
			4	38.8	58	
	5 - 7	SP				Gray to grayish-brown, moist, medium dense to dense, very fine to medium grained SAND, with gravels, cobbles, and lenses of medium to coarse grained SAND. ----- <i>Total depth = 7-feet. Groundwater mottling at approximately 5½-feet. No groundwater encountered. Backfilled 8/2/06.</i>
15	0 - 4	SC-ML				<u>Topsoil/Alluvium</u> Light gray to gray, moist, firm, silty to clayey, very fine to fine sandy SILT. Moderately rooted.
	4 - 10	SP-SM				Gray to dark gray, moist, medium dense, very fine to coarse grained SAND, with gravels, and cobbles. ----- <i>Total depth = 10-feet. Groundwater seepage at approximately 8½-feet. Backfilled 8/2/06.</i>
16	0 - 1	SM	1½	8.3	93.8	<u>Topsoil/Alluvium</u> Light gray, moist, loose, silty, very fine to fine SAND.
	1 - 3	SP-SM				Light gray, moist, medium dense, silty, fine to coarse grained SAND, with cobbles.
	3 - 5	SP	4	24.8	94.5	Dark gray, moist, medium dense to dense, fine to fine SAND. ----- <i>Total depth = 5-feet. No groundwater encountered. Backfilled 8/2/06.</i>

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TEST PIT LOGSJOB NO: 3.00856.1DATE: 8/2/06PROJECT: Snowcreek 8LOGGED BY: PS

TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
17	0 - 3	SC-SM	2	38.6	59	<u>Topsoil/Alluvium</u> Light gray, moist, loose to medium dense, silty to clayey, very fine to fine SAND. Moderately rooted.
	3 - 5	SM	3½	13.7	103.8	Gray to grayish-brown, moist, medium dense to dense, very fine to medium grained SAND, with gravels, cobbles, and thin lenses of medium to coarse grained SAND. ----- <i>Total depth = 5-feet. Groundwater mottling at approximately 3½-feet. No groundwater encountered. Backfilled 8/2/06.</i>
18	0 - 7	SP				<u>Undocumented Fill</u> Brown, moist to saturated, loose, fine to coarse grained SAND, with abundant cobbles and boulders comprising 80-90% of deposit.
	7 - 8	SP				<u>Toposil/Alluvium</u> Gray, saturated, medium dense to dense, very fine to coarse grained SAND, with gravels. ----- <i>Total depth = 8-feet. Groundwater seepage at approximately 2½-feet. Backfilled 8/2/06.</i>
19	0 - 2½	SP-SM				<u>Topsoil/Alluvium</u> Medium brown, loose, silty, very fine to coarse grained SAND, with abundant cobbles and boulders to 24-inches in diameter. Rock content comprising 60-70% of deposit. Moderately rooted.
	2 ½ - 3	SM				<u>Glacial Deposits</u> Gray, moist, medium dense to dense, silty, very fine to coarse grained SAND, with cobbles, and boulders. ----- <i>Total depth = 3-feet. No groundwater encountered. Backfilled 8/2/06.</i>

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TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
20	0 - 2½	SP-SM				<u>Topsoil/Alluvium</u> Medium brown, moist, loose, silty, very fine to medium grained SAND, with cobbles and boulders to 12-inches in diameter. Rock content comprising 5-10% of deposit. Highly rooted.
	2½ - 3½	SP-SM				Medium brown, moist, loose to medium dense, silty, very fine to coarse grained SAND, with abundant cobbles and boulders to 18-inches in diameter. Rock content comprising 30-40% of deposit. ----- <i>Total depth = 3½-feet. No groundwater encountered. Backfilled 8/2/06.</i>
21	0 - 6	SM				<u>Topsoil/Alluvium</u> Medium brown, moist, loose, silty, very fine to coarse grained SAND, with cobbles.
	6 - 6½	SP-SM				Medium brown, moist, loose to medium dense, silty, fine to coarse grained SAND, with cobbles and boulders. ----- <i>Total depth = 6½-feet. No groundwater encountered. Backfilled 8/2/06.</i>
22	0 - 5	SM				<u>Topsoil/Alluvium</u> Light brown, moist, loose, silty, very fine to medium grained SAND, with cobbles, and large boulders to 3-feet in diameter. Rock content comprising 50-60% of deposit. ----- <i>Total depth = 5-feet. No groundwater encountered. Belling and caving due to loose soil matrix. Backfilled 8/2/06.</i>

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TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
23	0 - 3	SP				<u>Topsoil/Alluvium</u> Brown, moist, loose, silty, very fine to medium grained SAND, with gravels and cobbles. Moderately rooted.
	3 - 5½	SM				Brown, moist, loose to medium dense, silty, very fine to coarse grained SAND, with cobbles. ----- <i>Total depth = 5½-feet. No groundwater encountered. Backfilled 8/2/06.</i>
24	0 - 5	SP-SM				<u>Topsoil/Alluvium</u> Brown, moist, loose, silty, very fine to medium grained SAND, with gravels and cobbles. Moderately rooted,
	5 - 7	ML				<u>Glacial Deposits</u> Light gray, moist, stiff, clayey, very fine to fine sandy SILT. ----- <i>Total depth = 7-feet. No groundwater encountered. Backfilled 8/2/06.</i>
25	0 - 2	SP-SM				<u>Topsoil/Alluvium</u> Brown, moist, loose, silty, very fine to medium grained SAND, with gravels and cobbles.
	2 - 5	SP-SM				Reddish-brown, moist, loose to medium dense, silty, fine to coarse SAND, with gravels, cobbles, and boulders to 3-feet in diameter. Rock content comprising 50% of deposit.
	5 - 6	SP				<u>Glacial Deposits</u> Gray, moist, medium dense to dense, fine to coarse grained SAND, with gravels, cobbles, and boulders to 3-feet in diameter. ----- <i>Total depth = 6-feet. No groundwater encountered. Backhoe refused at approximately 6-feet. Backfilled 8/2/06.</i>

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TEST PIT LOGSJOB NO: 3.00856.1DATE: 8/3/06PROJECT: Snowcreek 8LOGGED BY: PS

TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
26	0 - 3	SP-SM				<u>Topsoil/Alluvium</u> Brown, moist, loose, silty, very fine to medium grained SAND, with gravels. Moderately rooted.
	3 - 4	SP				Light gray to brown, moist, loose to medium dense, fine to coarse grained SAND, with few cobbles. ----- Total depth = 4-feet. No groundwater encountered. Backfilled 8/3/06.
27	0 - 2	SC-SM				<u>Undocumented Fill</u> Mottled grayish-brown, moist, dense, silty to clayey, very fine to fine SAND, with few cobbles.
	2 - 3	SM				<u>Topsoil/Alluvium</u> Dark brown, moist, loose, silty, very fine to fine SAND. Highly rooted. ----- Total depth = 3-feet. No groundwater encountered. Backfilled 8/3/06.
28	0 - 5	SP	2	9.3	113.5	<u>Undocumented Fill</u> Mottled brown to grayish-brown, moist, medium dense, silty, very fine to coarse, with few cobbles and boulders to 36-inches in diameter.
	5 - 10	SP-SM				<u>Topsoil/Alluvium</u> Dark gray, moist, dense, silty, fine to coarse grained SAND, with gravels and cobbles. ----- Total depth = 10-feet. Groundwater encountered at approximately 8-feet. Piezometer installed to 10-feet. Backfilled 8/3/06.

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TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
29	0 - 3	SM				<u>Undocumented Fill</u> Dark grayish-brown, moist, loose, silty, very fine to medium grained SAND, with cobbles, and boulders to 18-inches in diameter. Moderately rooted. Rock content comprising 25-35% of deposit.
	3 - 4	SM	3	11	117.5	<u>Topsoil/Alluvium</u> Reddish-brown, moist, loose to medium dense, silty, very fine to coarse grained SAND, with cobbles and boulders to 14-inches diameter.
	4 - 5	SP				<u>Glacial Deposits</u> Gray, moist, medium dense to dense, fine to coarse grained SAND, with gravels, cobbles, and boulders to 14-inches diameter. ----- <i>Total depth = 5-feet. No groundwater encountered. Backfilled 8/3/06.</i>
30	0 - 2	SC-SM	2	18.2	83.8	<u>Topsoil/Alluvium</u> Light grayish-brown, moist, loose, silty to clayey, highly rooted, very fine to fine SAND, with few cobbles, and boulders.
	2 - 4½	SP-SM				<u>Glacial Deposits</u> Gray, moist, medium dense to dense, silty, very fine to coarse grained SAND, with few gravels, and cobbles. ----- <i>Total depth = 4½-feet. No groundwater encountered. Backfilled 8/3/06.</i>
31	0 - 2	SP-SM				<u>Topsoil/Alluvium</u> Gray, moist, loose, silty, very fine to coarse grained SAND, with abundant cobbles, and boulders. Highly rooted.
	2 - 4½	SP				<u>Glacial Deposits</u> Gray to light grayish-brown, moist, medium dense, fine to coarse grained SAND, with abundant gravels, cobbles, and boulders. ----- <i>Total depth = 4½-feet. No groundwater encountered. Backfilled 8/3/06.</i>

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TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
32	0 - 3	SP				<u>Topsoil/Alluvium</u> Brown, moist, loose, silty, very fine to medium grained SAND, with few gravels, cobbles, and boulders to 3-feet in diameter.
	3 - 4½	SP				Brown to reddish-brown, moist, medium dense to dense, fine to coarse grained SAND, with gravels, cobbles, and boulders to 3-feet in diameter. ----- <i>Total depth = 4½-feet. No groundwater encountered. Backfilled 8/3/06.</i>
33	0 - 3	SP-SM				<u>Topsoil/Alluvium</u> Reddish-brown, moist, loose to medium dense, silty, fine to coarse grained SAND, with gravels, cobbles, and few boulders to 30-inches in diameter.
	3 - 4½	SP-SM				<u>Glacial Deposits</u> Gray, moist, medium dense to dense, silty, very fine to medium grained SAND, with abundant cobbles, and few boulders to 30-inches in diameter. ----- <i>Total depth = 4½-feet. No groundwater encountered. Backfilled 8/3/06.</i>
34	0 - 2	SM				<u>Topsoil/Alluvium</u> Brown, moist, loose, silty, very fine to coarse grained SAND, with few cobbles and few boulders to 30-inches in diameter.
	2 - 5	SP-SM				<u>Glacial Deposits</u> Gray, moist, dense, very fine to coarse grained SAND, with gravels, cobbles, and few boulders. ----- <i>Total depth = 5-feet. No groundwater encountered. Backfilled 8/3/06.</i>

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TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
35	0 - 1	SM				<u>Topsoil/Alluvium</u> Dark brown, moist, loose, silty, very fine to medium grained SAND, with few gravels. Organic rich, highly rooted,
	1 - 3	SP				<u>Glacial Deposits</u> Gray, moist to saturated, medium dense, fine to coarse grained SAND, with gravels and cobbles. ----- <i>Total depth = 3-feet. Groundwater seepage at approximately 2-feet. Backfilled 8/3/06.</i>
36	0 - 2½	SM				<u>Topsoil/Alluvium</u> Brown to reddish-brown, moist, loose to medium dense, silty, very fine to medium grained SAND, with abundant cobbles, and boulders to 16-inches diameter.
	2½ - 3	SP-SM				<u>Glacial Deposits</u> Light gray, moist, medium dense, silty, very fine to fine SAND, with abundant angular road fragments comprising 25-35% of deposit. ----- <i>Total depth = 3-feet. No groundwater encountered. Backfilled 8/3/06.</i>
37	0 - 4	SM				<u>Topsoil/Alluvium</u> Brown, moist, loose, silty, very fine to medium grained SAND, with gravels, cobbles, and few boulders to 14-inches diameter.
	4 - 8	SP-SM				<u>Glacial Deposits</u> Gray, moist, medium dense to dense, silty, fine to coarse grained SAND, with gravels, cobbles, and few boulders to 14-inches diameter. ----- <i>Total depth = 8-feet. No groundwater encountered. Backhoe refused at 8-feet. Backfilled 8/3/06.</i>

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TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
38	0 - 2	SP-SM				<u>Topsoil/Alluvium</u> Brown, moist, loose, silty, highly rooted, very fine to medium grained SAND, with gravels, cobbles, and few boulders.
	2 - 4	SP				Brown, moist, loose to medium dense, fine to coarse grained SAND, with abundant cobbles, and boulders to 18-inches in diameter. Rock content comprising 25-35% of deposit. ----- <i>Total depth = 4-feet. No groundwater encountered. Backfilled 8/3/06.</i>
39	0 - 2	SP				<u>Topsoil/Alluvium</u> Brown, moist, loose, silty, very fine to medium grained SAND, with gravels, cobbles, and few boulders to 14-inches diameter. Highly rooted.
	2 - 4	SP-SM				<u>Glacial Deposits</u> Gray, moist, medium dense to dense, fine to coarse grained SAND, with gravels, cobbles, and few boulders to 14-inches diameter. ----- <i>Total depth = 4-feet. No groundwater encountered. Backfilled 8/3/06.</i>

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TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
40	0 - 2½	SM	1½	5.6	116.0	<u>Topsoil/Alluvium</u> Grayish-brown, moist, loose to medium dense, silty, very fine to coarse grained SAND, with cobbles and boulders to 14-inches diameter. Moderately rooted.
	2½ - 4	SP-SM				<u>Glacial Deposits</u> Gray, moist, medium dense to dense, fine to coarse grained SAND, with gravels, cobbles and boulders to 14-inches diameter. ----- <i>Total depth = 4-feet. No groundwater encountered. Backfilled 8/3/06.</i>
41	0 - 2	SM	1	6.5	95.8	<u>Topsoil/Alluvium</u> Brown, moist, loose, silty, highly rooted, very fine to medium grained SAND, with gravels, cobbles, and few large boulders.
	2 - 3½	SP-SM	3½	9.4	106.5	Reddish-brown, moist, medium dense, silty, fine to coarse grained SAND, with cobbles and boulders to 18-inches diameter. Rock content comprising 60-75% of deposit.
	3½ - 5	SP-SM				<u>Glacial Deposits</u> Grayish-brown to reddish-brown, moist, medium dense to dense, fine to coarse grained SAND, with gravels, cobbles and boulders to 18-inches diameter. ----- <i>Total depth = 5-feet. Backhoe refused at 5-feet. Groundwater mottling at 3½-feet. No groundwater encountered. Backfilled 8/3/06.</i>

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(760) 934-3992**

JOB NO: 3.00856.1
DATE: 8/3/06

TEST PIT	DEPTH (FT)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
42	0 - 2	SM	2	13.6	97.0	<u>Undocumented Fill</u> Brown to mottled brown, moist, medium dense, silty, very fine to medium grained SAND, with gravels, cobbles, and chunks of asphalt.
	2 - 6	SM				<u>Topsoil/Alluvium</u> Brown, moist, medium dense to dense, silty, very fine to coarse grained SAND, with gravels, cobbles, and few boulders to 13-inches diameter.
	6 - 8	SP-SM				<u>Glacial Deposits</u> Grayish-brown, moist, dense, fine to coarse grained SAND, with gravels, cobbles and boulders to 15-inches diameter.
----- <i>Total depth = 8-feet. No groundwater encountered.</i> <i>Backfilled 8/3/06.</i>						

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed on the representative test samples acquired during this investigation to provide a basis for the development of design parameters. Soil materials were visually classified in the field according to the Unified Soil Classification System (USCS). Selected samples were tested for the following parameters: Classification and grain size determination, direct shear, and maximum dry density (Proctor). Laboratory tests were performed in general accordance with the American Society of Testing and Materials (ASTM) procedures. The results of our laboratory testing are presented herein. USCS classifications are presented on the test pit logs (Appendix A).

LABORATORY TESTING

Classification or Grain Size Tests: Typical materials were subjected to mechanical grain-size analysis by sieving from U.S. Standard brass screens (ASTM Test Method C136). The data was evaluated in determining the classification of the materials. Unified Soil Classification (USCS) is presented in both the test data and the test pit logs.

Direct Shear Tests: Seven direct shear tests were performed on selected disturbed samples remolded from 85 to 90-percent maximum dry density, and saturated under a surcharge load. After a period of 24-hours, the sample was then tested under various normal/axial loads, using a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of 0.04 inches per minute. The test results are presented herein.

Sample Location	Sample Description	Friction Angle (degrees) (relaxed)	Apparent Cohesion (psf)
TP-3 @ 4 - 5'	Mottled gray to reddish-brown, silty, fine to coarse SAND	32	64
TP-5 @ 3 - 4'	Light brown to light grayish-brown, silty, very fine to fine SAND	30	304
TP-16 @ 3 - 5'	Dark gray, fine SAND	32	0
TP-17 @ 3 - 5'	Gray to grayish-brown, silty, very fine to fine SAND	27	492
TP-29 @ 3 - 4'	Reddish-brown, silty, very fine to coarse SAND	28	272
TP-40 @ 1½ - 2½'	Grayish-brown, silty, fine to coarse SAND	34	64
TP-42 @ 2½ - 3½'	Brown, silty, very fine to coarse SAND	29	328

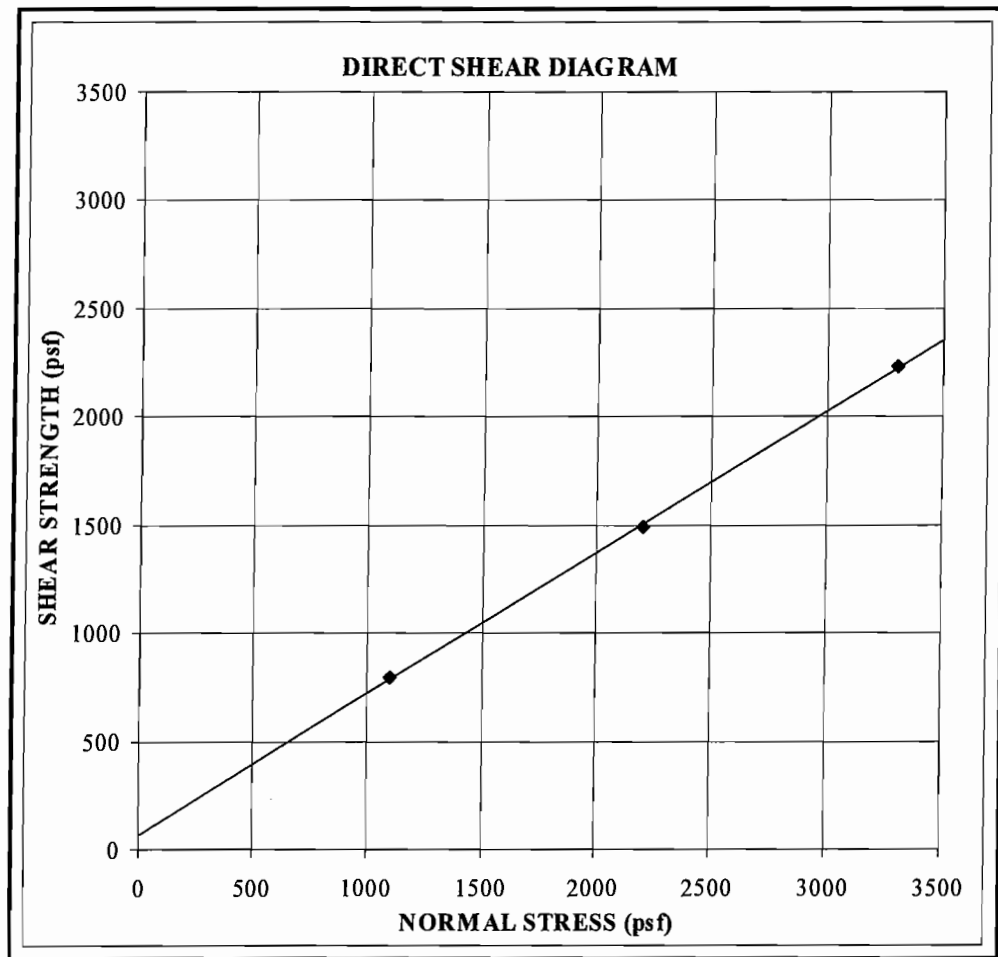
LABORATORY TESTING (Continued)

Maximum Density Tests: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM Test Method D1557-2000. The results of these tests are presented in the table below:

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
TP-3 @ 4 - 5'	Mottled gray to reddish-brown, silty, fine to coarse SAND	130.0	8.5
TP-5 @ 3 - 4'	Light brown to light grayish-brown, silty, very fine to fine SAND	107.3	16.0
TP-16 @ 3 - 5'	Dark gray, fine SAND	112.8	12.2
TP-17 @ 3 - 5'	Gray to grayish-brown, silty, very fine to fine SAND	120.5	12.0
TP-29 @ 3 - 4'	Reddish-brown, silty, very fine to coarse SAND	118.0	11.0
TP-40 @ 1½ - 2½'	Grayish-brown, silty, fine to coarse SAND	111.5	13.5
TP-42 @ 2½ - 3½'	Brown, silty, very fine to coarse SAND	112.0	14.0

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Boring No: TP-3

Sample Depth: 4 to 5-feet

Friction Angle: 32 degrees

Cohesion: 64 psf

Dry Density: 117 pcf

Remolded to: 90%

Date Tested: 9/7/06

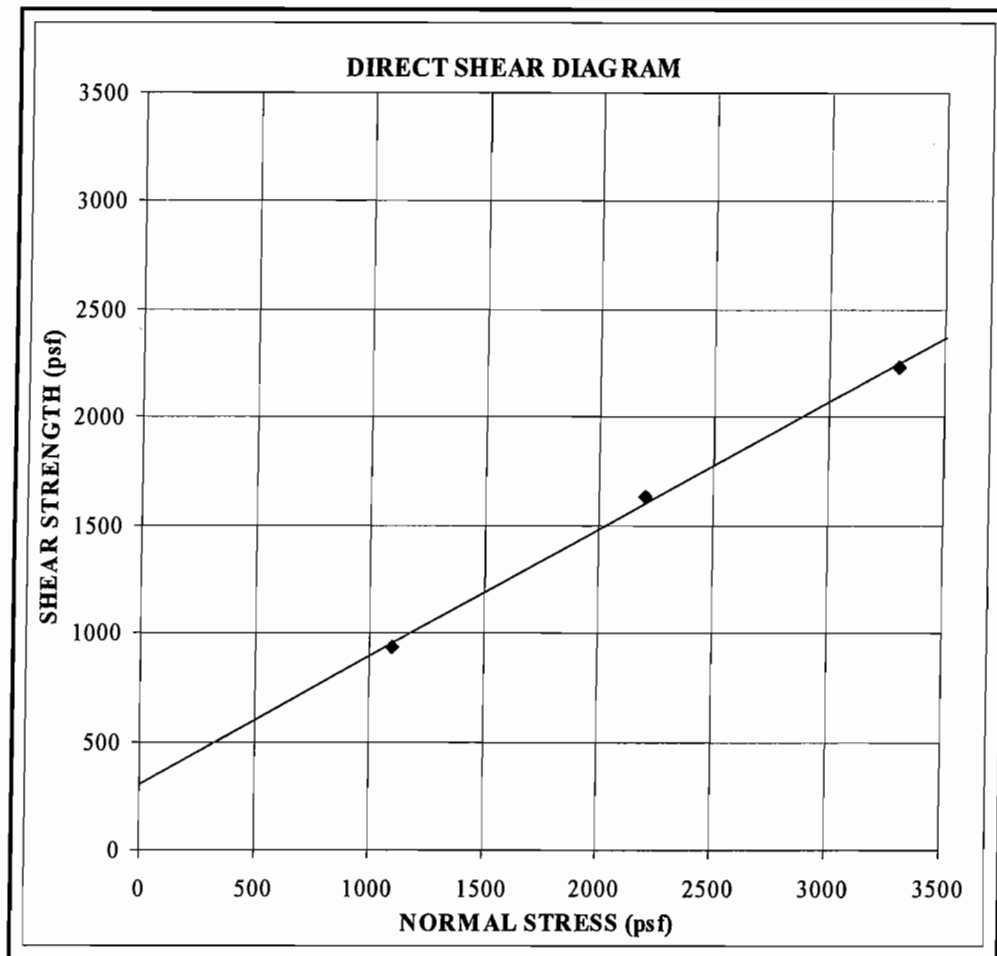
PROJECT: Snowcreek 8

3.00856.1

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Boring No: TP-5

Sample Depth: 3 to 4-feet

Friction Angle: 30 degrees

Cohesion: 304 psf

Dry Density: 91.4 pcf

Remolded to: 85%

Date Tested: 9/1/06

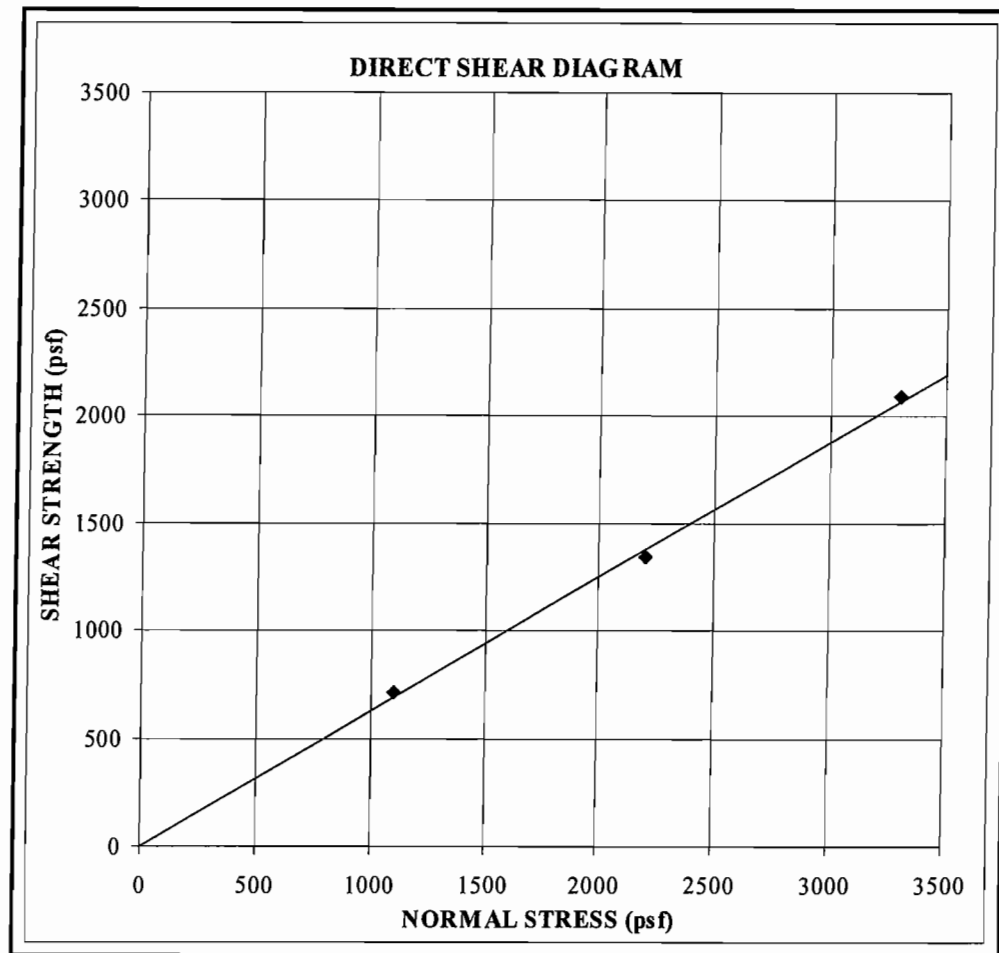
PROJECT: Snowcreek 8

3.00856.1

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Boring No: TP-16

Sample Depth: 3 to 5-feet

Friction Angle: 32 degrees

Cohesion: 0 psf

Dry Density: 95.9 pcf

Remolded to: 85%

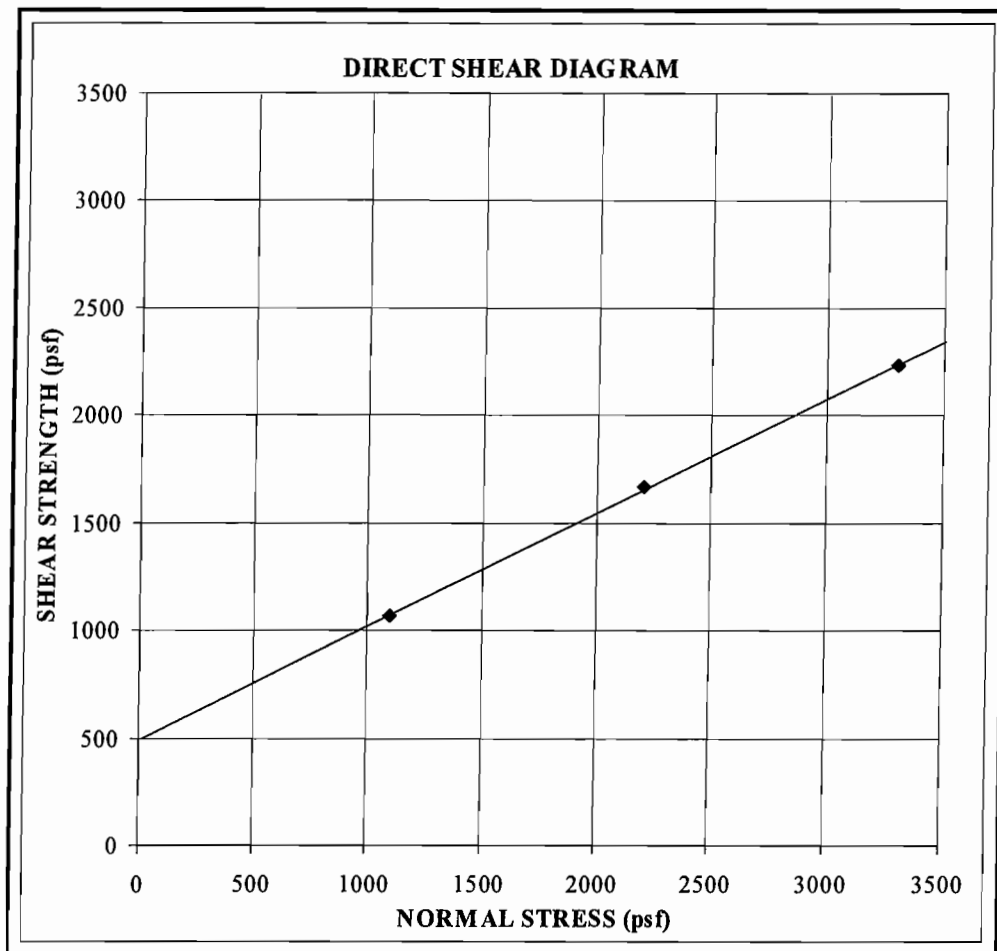
Date Tested: 9/13/06

PROJECT: Snowcreek 8

3.00856.1

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Boring No: TP-17

Sample Depth: 3.5-feet

Friction Angle: 27 degrees

Cohesion: 492 psf

Dry Density: 102.4 pcf

Remolded to: 85%

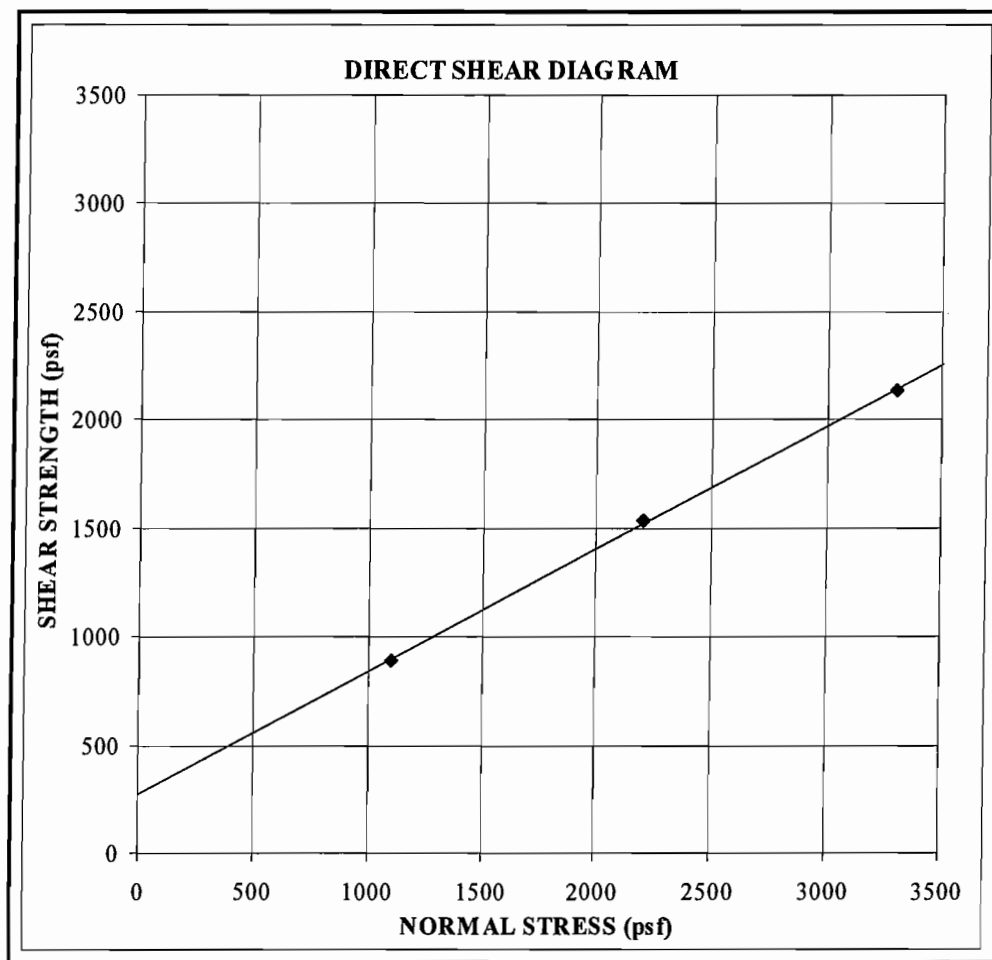
Date Tested: 9/11/06

PROJECT: Snowcreek 8

3.00856.1

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Boring No: TP-29

Sample Depth: 3-feet

Friction Angle: 28 degrees

Cohesion: 272 psf

Dry Density: 106.2 pcf

Remolded to: 90%

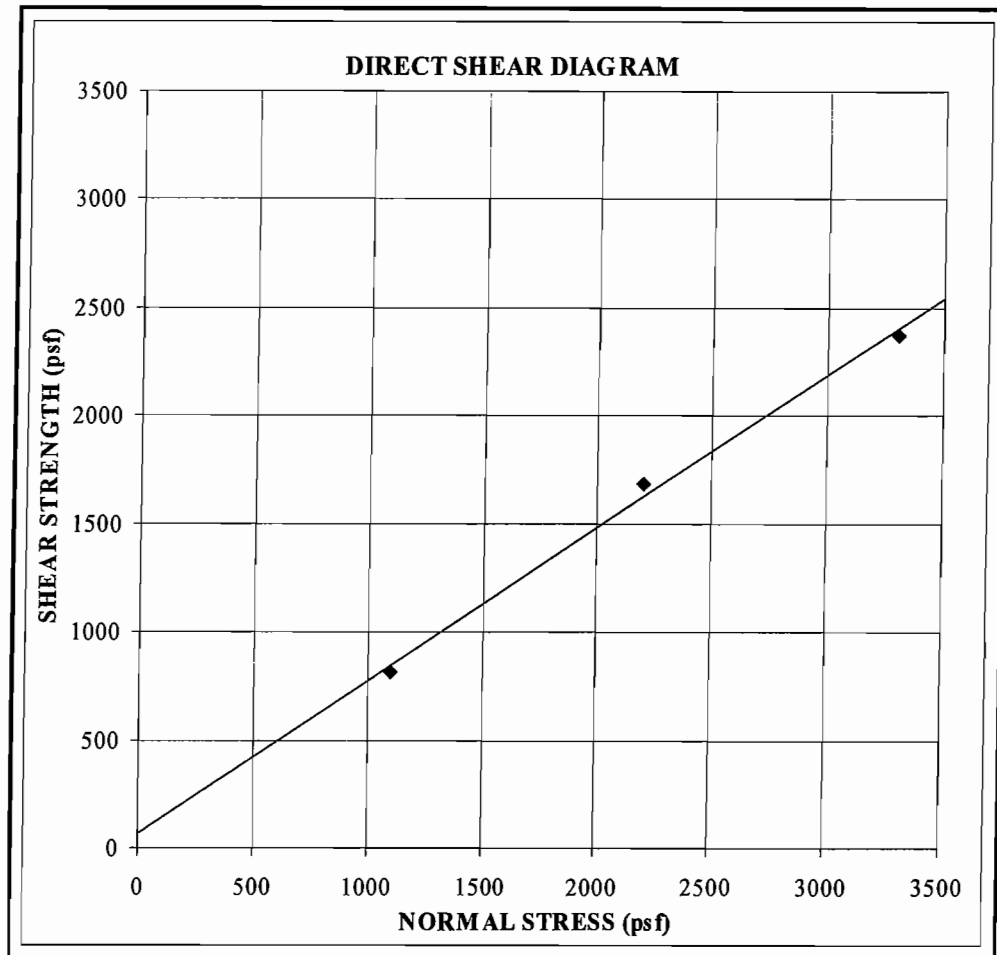
Date Tested: 9/17/06

PROJECT: Snowcreek 8

3.00856.1

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Boring No: TP-40

Sample Depth: 1½ to 2½-feet

Friction Angle: 34 degrees

Cohesion: 64 psf

Dry Density: 100.4 pcf

Remolded to: 90%

Date Tested: 8/28/06

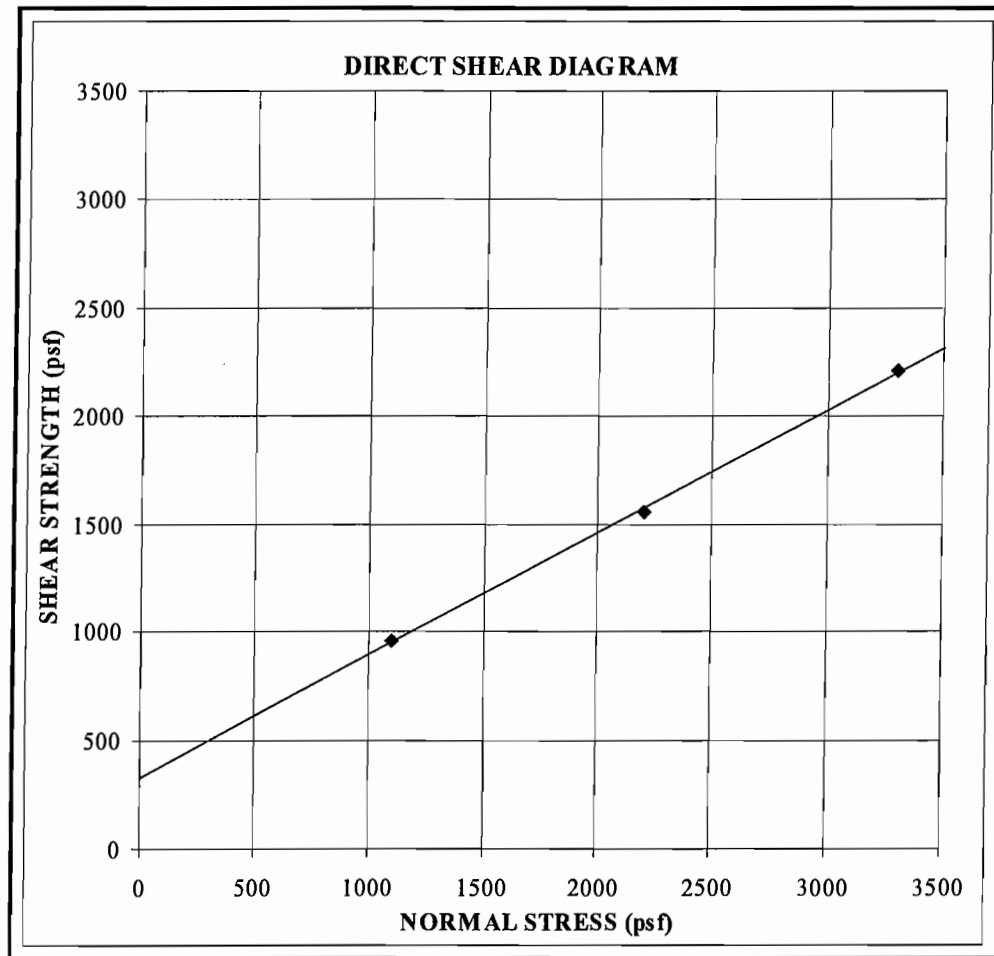
PROJECT: Snowcreek 7

3.00856.1

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Boring No: TP-42

Sample Depth: 2½ to 3½-feet

Friction Angle: 29 degrees

Cohesion: 328 psf

Dry Density: 95.2 pcf

Remolded to: 85%

Date Tested: 8/29/06

PROJECT: Snowcreek 7

3.00856.1

MAXIMUM DENSITY CURVE

PER ASTM TEST METHOD 1557-00 AND CTM 231-01

JOB NUMBER: 3.00856.1

DATE: 9/5/06

PROJECT: SNOWCREEK 8

SAMPLED BY: PS

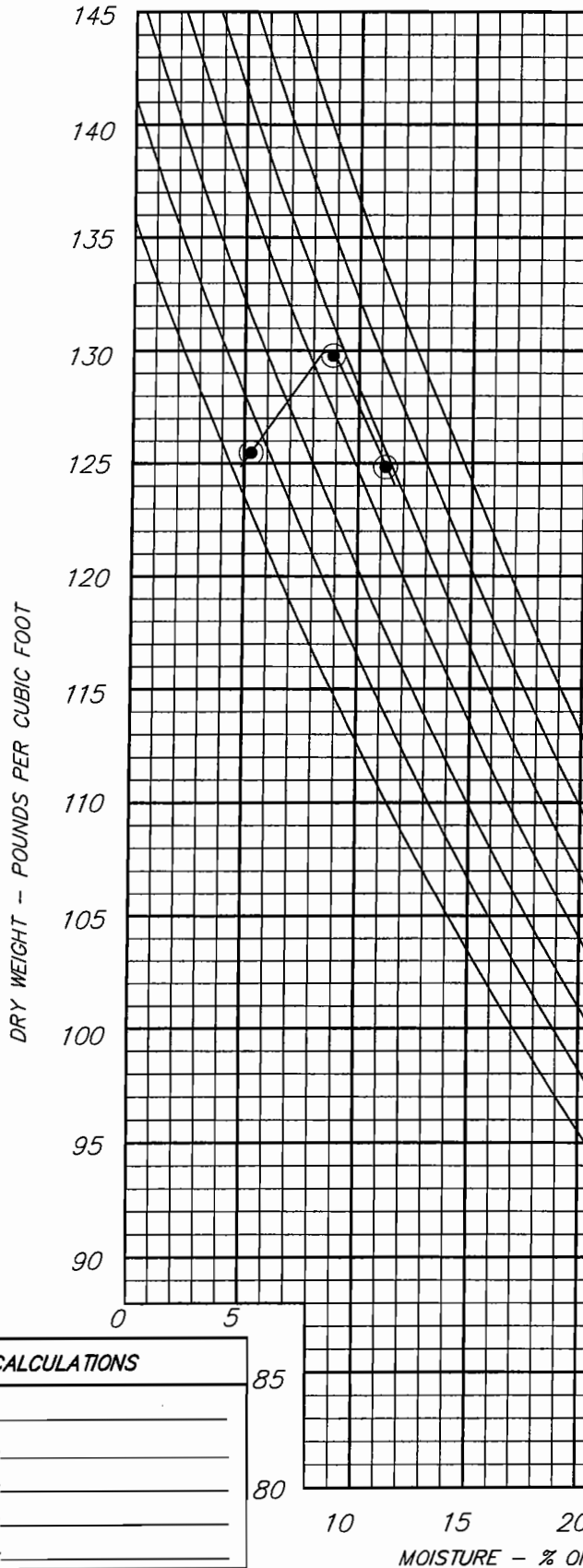
TESTED BY: PS

EXCAVATION: TP-3

DEPTH (FT.): 4-5

SOIL CLASSIFICATION: GLACIAL DEPOSITS

DESIGNATION: SP-SM



TEST #	1	2	3	4	5
WEIGHT OF SOIL AND MOLD	6379	6519	6479		
WEIGHT OF MOLD	4381	4381	4381		
NET WET WEIGHT OF SOIL	1998	2138	2098		
WET DENSITY LBS./CU.FT.	132.1	141.4	138.8		
DRY DENSITY LBS./CU.FT.	125.5	129.8	124.9		

MOISTURE DETERMINATION

WET WEIGHT SOIL	459	487	490		
TARE (WT. OF PAN)					
DRY WEIGHT SOIL	436	447	441		
WEIGHT OF MOISTURE	23	40	49		
PERCENT MOISTURE	5.3	8.9	11.1		

ROCK CORRECTION

A	TOTAL SAMPLE WEIGHT	
B	+ 3/4" WT. IN AIR (gm)	
C	+ 3/4" WT. IN WATER (gm)	
D	+ 3/4" VOLUME (cc) (B-C)	
E	% + 3/4" 100(B/A)	
F	% - 3/4" (100-E)	
G	DENSITY OF + 3/4" (B/D)	
H	% + 3/4" / DENSITY OF + 3/4"	
I	% - 3/4" / DENSITY OF - 3/4"	
J	SUM OF H AND I	
K	ADJUSTED DENSITY gm/cc (100/J)	

RESULTS

MOLD VOLUME: _____
 SIEVE USED: _____
 MAX. DENSITY (LBS./FT³): 130.0
 OPT. MOISTURE (% DRY WT.): 8.5

CALCULATIONS

1: _____
 2: _____
 3: _____
 4: _____
 5: _____

MAXIMUM DENSITY CURVE

PER ASTM TEST METHOD 1557-00 AND CTM 231-01

JOB NUMBER: 3.00856.1

DATE: 8/30/06

PROJECT: SNOWCREEK 8

SAMPLED BY: PS

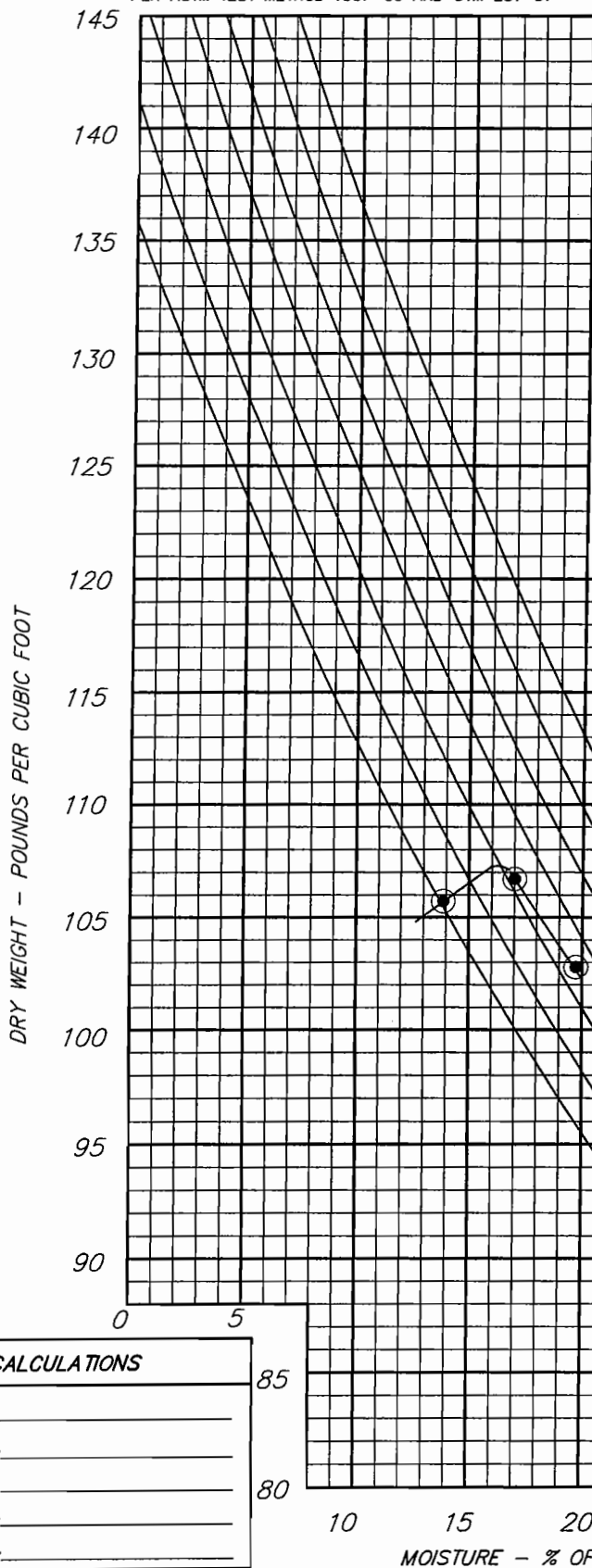
TESTED BY: PS

EXCAVATION: TP-5

DEPTH (FT.): 3-4

SOIL CLASSIFICATION: TOPSOIL/ALLUVIUM

DESIGNATION: SM



TEST #	1	2	3	4	5
WEIGHT OF SOIL AND MOLD	6270	6243	6202		
WEIGHT OF MOLD	4381	4381	4381		
NET WET WEIGHT OF SOIL	1890	1862	1821		
WET DENSITY LBS./CU.FT.	125.0	123.2	105.7		
DRY DENSITY LBS./CU.FT.	106.8	102.8	105.7		

MOISTURE DETERMINATION

WET WEIGHT SOIL	488	485	491		
TARE (WT. OF PAN)					
DRY WEIGHT SOIL	417	405	431		
WEIGHT OF MOISTURE	71	80	60		
PERCENT MOISTURE	17.0	19.8	13.9		

ROCK CORRECTION

A	TOTAL SAMPLE WEIGHT	
B	+ 3/4" WT. IN AIR (gm)	
C	+ 3/4" WT. IN WATER (gm)	
D	+ 3/4" VOLUME (cc) (B-C)	
E	% + 3/4" 100(B/A)	
F	% - 3/4" (100-E)	
G	DENSITY OF + 3/4" (B/D)	
H	% + 3/4" / DENSITY OF + 3/4"	
I	% - 3/4" / DENSITY OF - 3/4"	
J	SUM OF H AND I	
K	ADJUSTED DENSITY gm/cc (100/J)	

RESULTS

MOLD VOLUME: _____

SIEVE USED: _____

MAX. DENSITY (LBS./FT³): 107.3

OPT. MOISTURE (% DRY WT.): 16

CALCULATIONS

1: _____

2: _____

3: _____

4: _____

5: _____

MAXIMUM DENSITY CURVE

PER ASTM TEST METHOD 1557-00 AND CTM 231-01

JOB NUMBER: 3.00856.1

DATE: 8/30/06

PROJECT: SNOWCREEK 8

SAMPLED BY: PS

TESTED BY: PS

EXCAVATION: TP-16

DEPTH (FT.): 3-5

SOIL CLASSIFICATION: TOPSOIL/ALLUVIUM

DESIGNATION: SP

DRY WEIGHT - POUNDS PER CUBIC FOOT

145

140

135

130

125

120

115

110

105

100

95

90

0 5

85

80

10

15

20

25

30

35

40

45

MOISTURE - % OF DRY WEIGHT

TEST #

1

2

3

4

5

WEIGHT OF SOIL AND MOLD

6174

6265

6283

WEIGHT OF MOLD

4381

4381

4381

NET WET WEIGHT OF SOIL

1793

1884

1902

WET DENSITY LBS./CU.FT.

118.6

124.6

125.8

DRY DENSITY LBS./CU.FT.

110.9

112.4

110.2

MOISTURE DETERMINATION

WET WEIGHT SOIL

436

448

490

TARE (WT. OF PAN)

DRY WEIGHT SOIL

408

404

429

WEIGHT OF MOISTURE

28

44

61

PERCENT MOISTURE

6.9

10.9

14.2

ROCK CORRECTION

A

TOTAL SAMPLE WEIGHT

B

+ 3/4" WT. IN AIR (gm)

C

+ 3/4" WT. IN WATER (gm)

D

+ 3/4" VOLUME (cc) (B-C)

E

% + 3/4" 100(B/A)

F

% - 3/4" (100-E)

G

DENSITY OF + 3/4" (B/D)

H

% + 3/4" / DENSITY OF + 3/4"

I

% - 3/4" / DENSITY OF - 3/4"

J

SUM OF H AND I

K

ADJUSTED DENSITY gm/cc (100/J)

RESULTS

MOLD VOLUME:

SIEVE USED:

MAX. DENSITY (LBS./FT³): 112.8

OPT. MOISTURE (% DRY WT.): 12.2

CALCULATIONS

- 1: _____
- 2: _____
- 3: _____
- 4: _____
- 5: _____

MAXIMUM DENSITY CURVE PER ASTM TEST METHOD 1557-00 AND CTM 231-01

JOB NUMBER: 3.00856.1

DATE: 9/5/06

PROJECT: SNOWCREEK 8

SAMPLED BY: PS

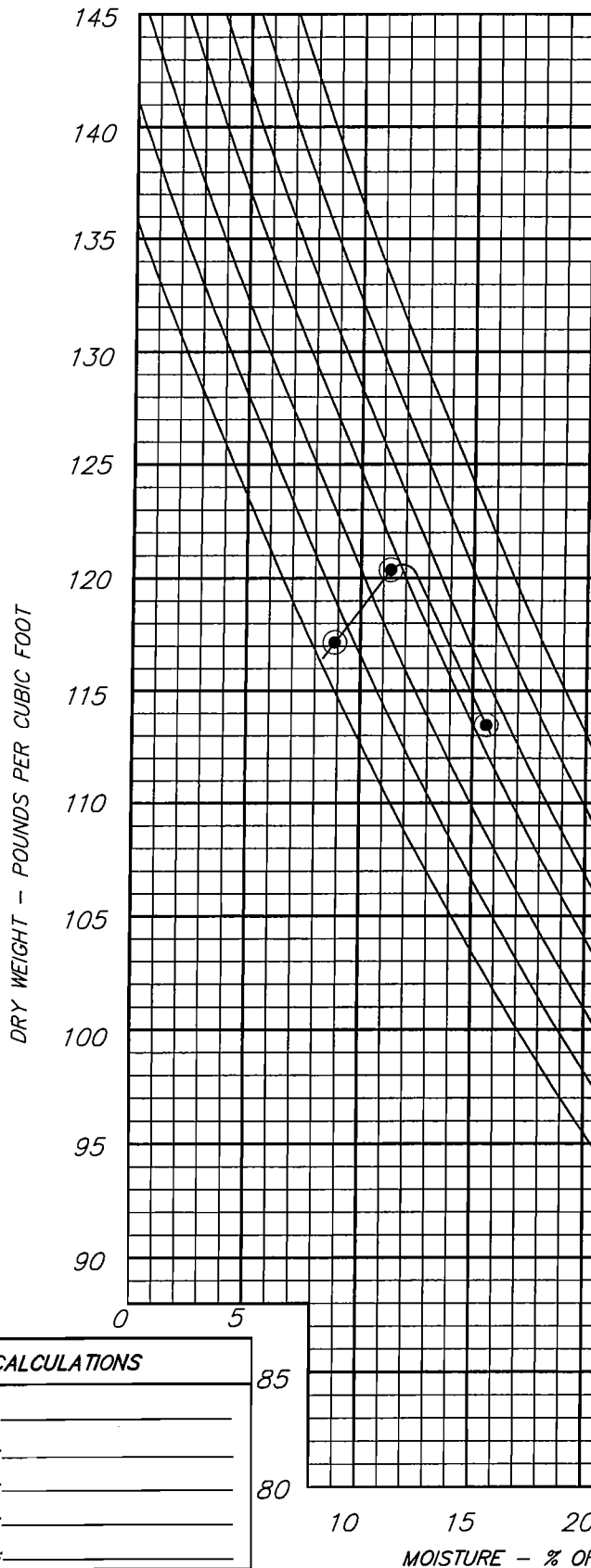
TESTED BY: PS

EXCAVATION: TP-17

DEPTH (FT.): 3 1/2 - 4 1/2

SOIL CLASSIFICATION: TOPSOIL/ALLUVIUM

DESIGNATION: SM



TEST #	1	2	3	4	5
WEIGHT OF SOIL AND MOLD	6311	6406	6365		
WEIGHT OF MOLD	4381	4381	4381		
NET WET WEIGHT OF SOIL	1930	2025	1984		
WET DENSITY LBS./CU.FT.	127.7	134.0	131.2		
DRY DENSITY LBS./CU.FT.	117.2	120.4	113.5		

MOISTURE DETERMINATION

WET WEIGHT SOIL	475	483	489		
TARE (WT. OF PAN)					
DRY WEIGHT SOIL	436	434	423		
WEIGHT OF MOISTURE	39	49	66		
PERCENT MOISTURE	8.9	11.3	15.6		

ROCK CORRECTION

A	TOTAL SAMPLE WEIGHT	
B	+ 3/4" WT. IN AIR (gm)	
C	+ 3/4" WT. IN WATER (gm)	
D	+ 3/4" VOLUME (cc) (B-C)	
E	% + 3/4" 100(B/A)	
F	% - 3/4" (100-E)	
G	DENSITY OF + 3/4" (B/D)	
H	% + 3/4" / DENSITY OF + 3/4"	
I	% - 3/4" / DENSITY OF - 3/4"	
J	SUM OF H AND I	
K	ADJUSTED DENSITY gm/cc (100/I)	

RESULTS

MOLD VOLUME: _____

SIEVE USED: _____

MAX. DENSITY (LBS./FT³): 120.5

OPT. MOISTURE (% DRY WT.): 12.0

CALCULATIONS

- 1: _____
- 2: _____
- 3: _____
- 4: _____
- 5: _____

MAXIMUM DENSITY CURVE PER ASTM TEST METHOD 1557-00 AND CTM 231-01

JOB NUMBER: 3.00856.1

DATE: 9/13/06

PROJECT: SNOWCREEK 8

SAMPLED BY: PS

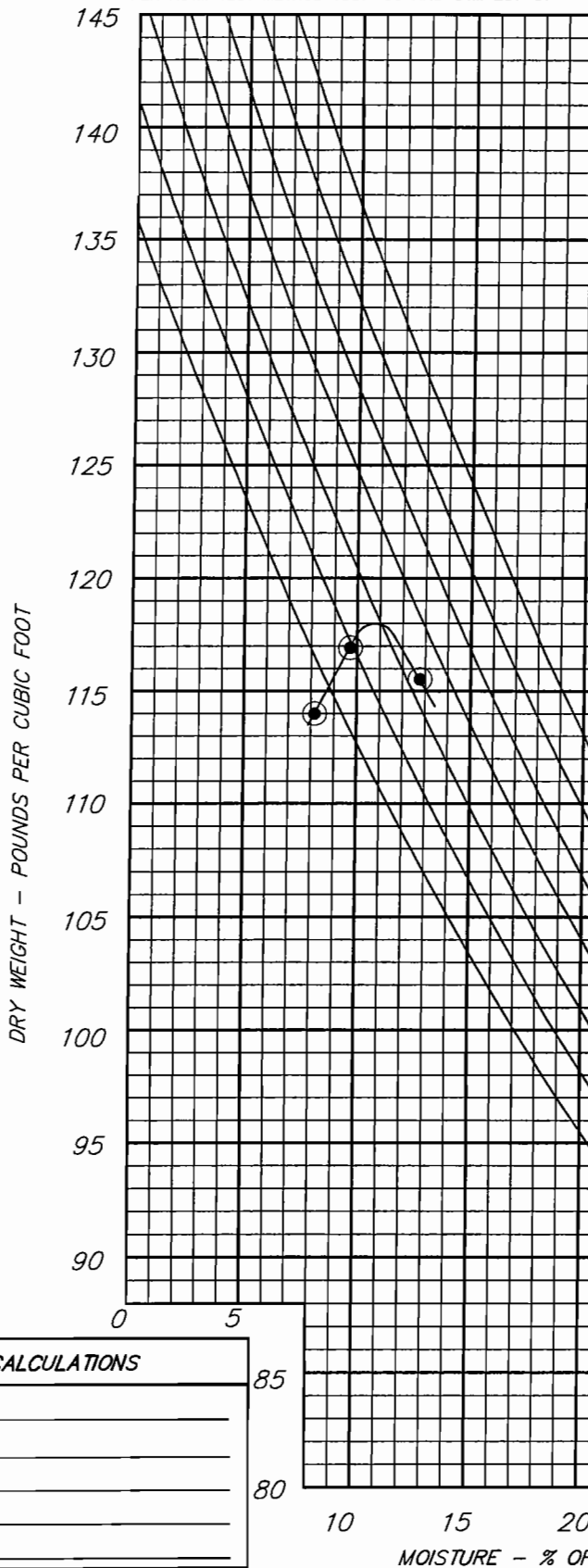
TESTED BY: PS

EXCAVATION: TP-29

DEPTH (FT.): 3-4

SOIL CLASSIFICATION: TOPSOIL/ALLUVIUM

DESIGNATION: SP



TEST #	1	2	3	4	5
WEIGHT OF SOIL AND MOLD	6320	6351	6240		
WEIGHT OF MOLD	4381	4381	4381		
NET WET WEIGHT OF SOIL	1939	1970	1862		
WET DENSITY LBS./CU.FT.	128.2	130.3	123.2		
DRY DENSITY LBS./CU.FT.	116.9	115.5	114		

MOISTURE DETERMINATION

WET WEIGHT SOIL	487	485	492		
TARE (WT. OF PAN)					
DRY WEIGHT SOIL	444	430	455		
WEIGHT OF MOISTURE	43	55	37		
PERCENT MOISTURE	9.7	12.8	8.1		

ROCK CORRECTION

A	TOTAL SAMPLE WEIGHT	
B	+ 3/4" WT. IN AIR (gm)	
C	+ 3/4" WT. IN WATER (gm)	
D	+ 3/4" VOLUME (cc) (B-C)	
E	% + 3/4" 100(B/A)	
F	% - 3/4" (100-E)	
G	DENSITY OF + 3/4" (B/D)	
H	% + 3/4" / DENSITY OF + 3/4"	
I	% - 3/4" / DENSITY OF - 3/4"	
J	SUM OF H AND I	
K	ADJUSTED DENSITY gm/cc (100/J)	

RESULTS

MOLD VOLUME: _____

SIEVE USED: _____

MAX. DENSITY (LBS./FT³): 118.0

OPT. MOISTURE (% DRY WT.): 11.0

CALCULATIONS

1: _____

2: _____

3: _____

4: _____

5: _____

MAXIMUM DENSITY CURVE

PER ASTM TEST METHOD 1557-00 AND CTM 231-01

JOB NUMBER: 3.00856.1

DATE: 8/25/06

PROJECT: SNOWCREEK 8

SAMPLED BY: PS

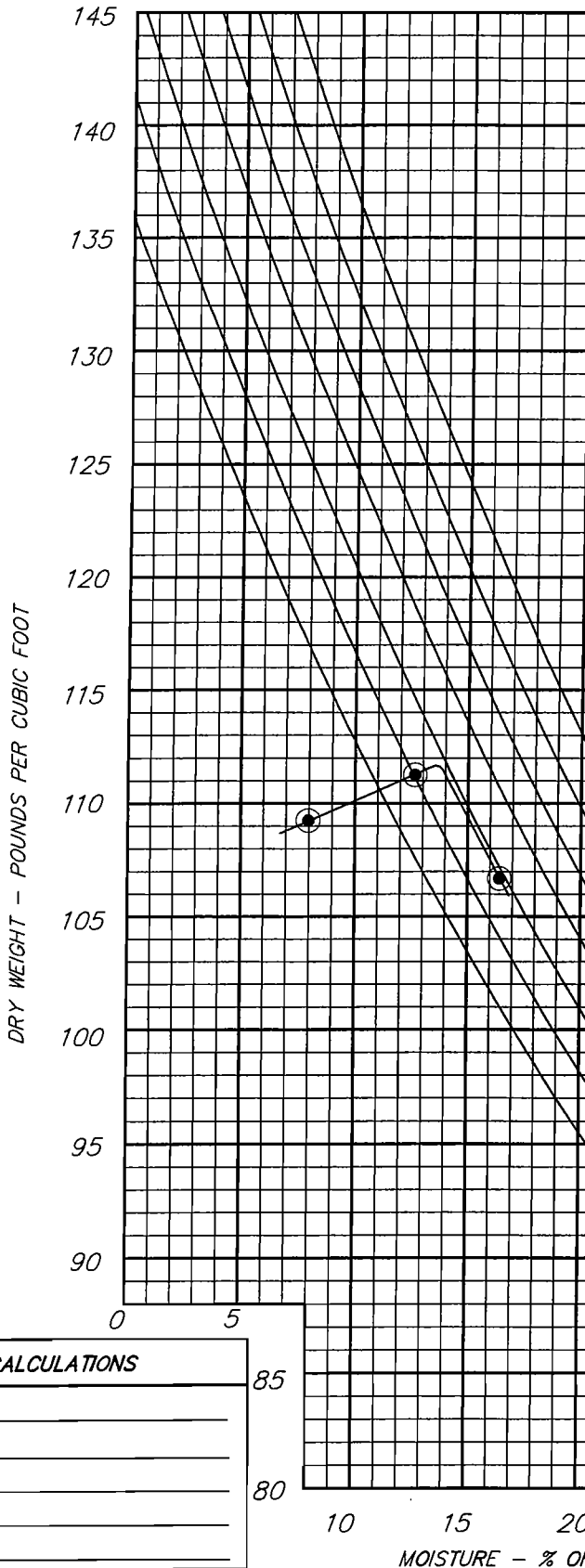
TESTED BY: PS

EXCAVATION: TP-40

DEPTH (FT.): 1 1/2 - 2 1/2

SOIL CLASSIFICATION: TOPSOIL/ALLUVIUM

DESIGNATION: SM



TEST #	1	2	3	4	5
WEIGHT OF SOIL AND MOLD	6165	6224	6261		
WEIGHT OF MOLD	4381	4381	4381		
NET WET WEIGHT OF SOIL	1784	1893	1880		
WET DENSITY LBS./CU.FT.	118	125.2	124.3		
DRY DENSITY LBS./CU.FT.	109.3	111.2	106.8		

MOISTURE DETERMINATION

WET WEIGHT SOIL	472	483	482		
TARE (WT. OF PAN)					
DRY WEIGHT SOIL	437	429	414		
WEIGHT OF MOISTURE	35	54	68		
PERCENT MOISTURE	8.0	12.6	16.4		

ROCK CORRECTION

A	TOTAL SAMPLE WEIGHT	
B	+ 3/4" WT. IN AIR (gm)	
C	+ 3/4" WT. IN WATER (gm)	
D	+ 3/4" VOLUME (cc) (B-C)	
E	% + 3/4" 100(B/A)	
F	% - 3/4" (100-E)	
G	DENSITY OF + 3/4" (B/D)	
H	% + 3/4" / DENSITY OF + 3/4"	
I	% - 3/4" / DENSITY OF - 3/4"	
J	SUM OF H AND I	
K	ADJUSTED DENSITY gm/cc (100/J)	

RESULTS

MOLD VOLUME: _____

SIEVE USED: _____

MAX. DENSITY (LBS./FT³): 111.5

OPT. MOISTURE (% DRY WT.): 13.5

CALCULATIONS

- 1: _____
- 2: _____
- 3: _____
- 4: _____
- 5: _____

MAXIMUM DENSITY CURVE

PER ASTM TEST METHOD 1557-00 AND CTM 231-01

JOB NUMBER: 3.00856.1

DATE: 8/25/06

PROJECT: SNOWCREEK 8

SAMPLED BY: PS

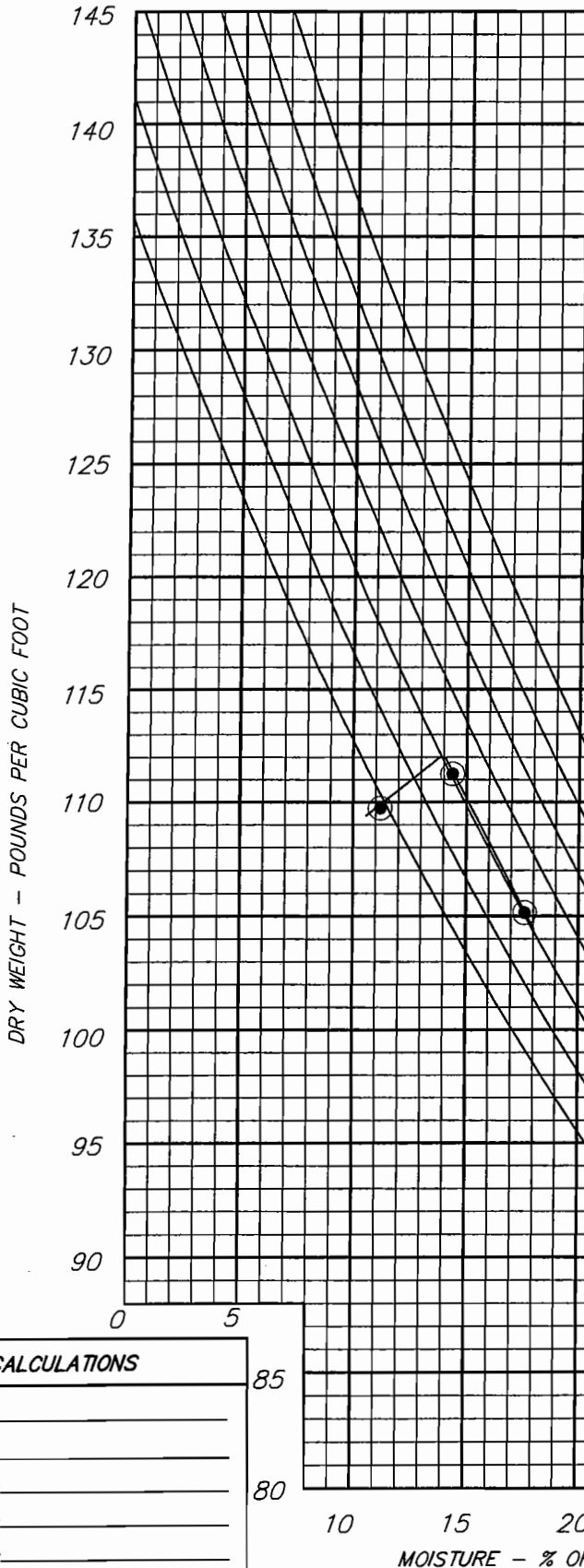
TESTED BY: PS

EXCAVATION: TP-42

DEPTH (FT.): 2-3

SOIL CLASSIFICATION: TOPSOIL/ALLUVIUM

DESIGNATION: SM



TEST #	1	2	3	4	5
WEIGHT OF SOIL AND MOLD	6224	6306	6252		
WEIGHT OF MOLD	4381	4381	4381		
NET WET WEIGHT OF SOIL	1843	1925	1871		
WET DENSITY LBS./CU.FT.	121.9	127.3	123.7		
DRY DENSITY LBS./CU.FT.	109.8	111.3	105.1		

MOISTURE DETERMINATION

WET WEIGHT SOIL	462	493	493		
TARE (WT. OF PAN)					
DRY WEIGHT SOIL	416	431	419		
WEIGHT OF MOISTURE	46	62	74		
PERCENT MOISTURE	11.1	14.4	17.7		

ROCK CORRECTION

A	TOTAL SAMPLE WEIGHT	
B	+ 3/4" WT. IN AIR (gm)	
C	+ 3/4" WT. IN WATER (gm)	
D	+ 3/4" VOLUME (cc) (B-C)	
E	% + 3/4" 100(B/A)	
F	% - 3/4" (100-E)	
G	DENSITY OF + 3/4" (B/D)	
H	% + 3/4" / DENSITY OF + 3/4"	
I	% - 3/4" / DENSITY OF - 3/4"	
J	SUM OF H AND I	
K	ADJUSTED DENSITY gm/cc (100/J)	

RESULTS

MOLD VOLUME: _____

SIEVE USED: _____

MAX. DENSITY (LBS./FT³): 112

OPT. MOISTURE (% DRY WT.): 14

CALCULATIONS

1: _____

2: _____

3: _____

4: _____

5: _____

SIERRA GEOTECHNICAL SERVICES INC.

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(760) 934-3992; (760) 934-8832 Fax

SIEVE ANALYSIS OF FINE AND COARSE AGGREGATES

Per CTM 202 / ASTM C136 (underline one)

Project:	Snowcreek 8	Job No.:	3.00856.1
Client:	Chadmar	Tested by:	JK
Sampled by:	PS	Delivered by:	PS
Sample Date/time:	8/1/06	Delivered Date/time:	
Sample Location:	TP-5 @ 3-4'	Test Date:	9/11/06
Description:	Topsoil/Alluvium - SM		

Dry Sample Total Weight (g): 218			#4 Minus Dry Wt. (g):						% Passing by Dry Weight:	
Sieve Size			Fine Wt. Ret.	Fine % Ret.	Fine % Pass.	Coarse Wt. Ret.	Coarse % Ret.	Coarse % Pass.	Coarse + Fine	Specified
Inches	mm	Mesh								
2.0	50.0	2"								
1.5	37.5	1 1/2"				0	0	100	100	
1.0	25.0	1"				60	3	97	97	
0.750	19.0	3/4"				85	4	96	96	
0.500	12.7	1/2"				130	6	94	94	
0.375	9.5	3/8"				200	10	90	90	
0.250	6.3	1/4"								
0.187	4.75	#4				402	19	81	81	
0.0937	2.36	#8	28	13	87				71	
0.0469	1.18	#16	53	24	76				62	
		#20								
0.0234	0.60	#30	75	34	66				54	
		#40								
0.0117	0.30	#50	88	40	60				49	
		#80								
0.0059	0.15	#100	128	59	41				33	
0.0029	0.075	#200	147	67.4	32.6				26.4	
PAN			153							
TOTAL										

Underline "WASH" if #200 wash analysis was done.

Remarks:

SIERRA GEOTECHNICAL SERVICES INC.

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(760) 934-3992; (760) 934-8832 Fax

SIEVE ANALYSIS OF FINE AND COARSE AGGREGATES

Per CTM 202 / ASTM C136 (underline one)

Project:	Snowcreek 8	Job No.:	3.00856.1
Client:	Chadmar	Tested by:	JK
Sampled by:	PS	Delivered by:	PS
Sample Date/time:	8/1/06	Delivered Date/time:	
Sample Location:	TP-6 @ 2-3'	Test Date:	8/30/06
Description:	Topsoil/Alluvium -SM		

Dry Sample Total Weight (g): 253			#4 Minus Dry Wt. (g):			% Passing by Dry Weight:				
Sieve Size			Fine Wt. Ret.	Fine % Ret.	Fine % Pass.	Coarse Wt. Ret.	Coarse % Ret.	Coarse % Pass.	Coarse + Fine	Specified
Inches	mm	Mesh								
2.0	50.0	2"								
1.5	37.5	1 1/2"								
1.0	25.0	1"								
0.750	19.0	3/4"								
0.500	12.7	1/2"								
0.375	9.5	3/8"								
0.250	6.3	1/4"								
0.187	4.75	#4	8	3	97					
0.0937	2.36	#8	48	19	81					
0.0469	1.18	#16	92	36	64					
		#20								
0.0234	0.60	#30	125	49	51					
		#40								
0.0117	0.30	#50	138	55	45					
		#80	171	68	32					
0.0059	0.15	#100	188	74.3	25.7					
0.0029	0.075	#200	194	77.1	22.9					
PAN										
TOTAL										

Underline "WASH" if #200 wash analysis was done.

Remarks:

SIERRA GEOTECHNICAL SERVICES INC.

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SIEVE ANALYSIS OF FINE AND COARSE AGGREGATES

Per CTM 202 / ASTM C136 (underline one)

Project:	Snowcreek 8	Job No.:	3.00856.1
Client:	Chadmar	Tested by:	JK
Sampled by:	PS	Delivered by:	PS
Sample Date/time:	8/1/06	Delivered Date/time:	
Sample Location:	TP-7 @ 3-4'	Test Date:	8/30/06
Description:	Topsoil/Alluvium – SC-SM		

Dry Sample Total Weight (g): 350			#4 Minus Dry Wt. (g):						% Passing by Dry Weight:	
Sieve Size			Fine Wt. Ret.	Fine % Ret.	Fine % Pass.	Coarse Wt. Ret.	Coarse % Ret.	Coarse % Pass.	Coarse + Fine	Specified
Inches	mm	Mesh								
2.0	50.0	2"								
1.5	37.5	1 1/2"								
1.0	25.0	1"								
0.750	19.0	3/4"								
0.500	12.7	1/2"								
0.375	9.5	3/8"	3	1	99					
0.250	6.3	1/4"								
0.187	4.75	#4	40	11	89					
0.0937	2.36	#8	67	19	81					
0.0469	1.18	#16	92	26	74					
		#20								
0.0234	0.60	#30	120	34	66					
		#40								
0.0117	0.30	#50	149	43	57					
		#80								
0.0059	0.15	#100	214	61	39					
0.0029	0.075	#200	240	68.6	31.4					
PAN			245							
TOTAL										

Underline "WASH" if #200 wash analysis was done.

Remarks:

SIERRA GEOTECHNICAL SERVICES INC.

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SIEVE ANALYSIS OF FINE AND COARSE AGGREGATES

Per CTM 202 / ASTM C136 (underline one)

Project:	Snowcreek 8	Job No.:	3.00856.1
Client:	Chadmar	Tested by:	JK
Sampled by:	PS	Delivered by:	PS
Sample Date/time:	8/2/06	Delivered Date/time:	
Sample Location:	TP-17 @ 3-4'	Test Date:	9/11/06
Description:	Topsoil/Alluvium - SM		

Dry Sample Total Weight (g): 250			#4 Minus Dry Wt. (g):						% Passing by Dry Weight:	
Sieve Size			Fine Wt. Ret.	Fine % Ret.	Fine % Pass.	Coarse Wt. Ret.	Coarse % Ret.	Coarse % Pass.	Coarse + Fine	Specified
Inches	mm	Mesh								
2.0	50.0	2"								
1.5	37.5	1 1/2"				0	0	100	100	
1.0	25.0	1"				0	0	100	100	
0.750	19.0	3/4"				5	0	100	100	
0.500	12.7	1/2"				83	5	95	95	
0.375	9.5	3/8"				167	9	91	91	
0.250	6.3	1/4"								
0.187	4.75	#4				443	25	75	75	
0.0937	2.36	#8	50	20	80				60	
0.0469	1.18	#16	85	34	66				50	
		#20								
0.0234	0.60	#30	122	49	51				38	
		#40								
0.0117	0.30	#50	150	60	90				30	
		#80								
0.0059	0.15	#100	200	80	20				15	
0.0029	0.075	#200	219	87.6	12.4				9.3	
PAN			222							
TOTAL										

Underline "WASH" if #200 wash analysis was done.

Remarks:

SIERRA GEOTECHNICAL SERVICES INC.

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SIEVE ANALYSIS OF FINE AND COARSE AGGREGATES

Per CTM 202 / ASTM C136 (underline one)

Project:	Snowcreek 8	Job No.:	3.00856.1
Client:	Chadmar	Tested by:	JK
Sampled by:	PS	Delivered by:	PS
Sample Date/time:	8/3/06	Delivered Date/time:	8/3/06
Sample Location:	TP-29 @ 3-4'	Test Date:	9/22/06
Description:	Topsoil/Alluvium - SM		

Dry Sample Total Weight (g): 684			#4 Minus Dry Wt. (g):						% Passing by Dry Weight:	
Sieve Size			Fine Wt. Ret.	Fine % Ret.	Fine % Pass.	Coarse Wt. Ret.	Coarse % Ret.	Coarse % Pass.	Coarse + Fine	Specified
Inches	mm	Mesh								
2.0	50.0	2"								
1.5	37.5	1 1/2"								
1.0	25.0	1"								
0.750	19.0	3/4"	24	4	96					
0.500	12.7	1/2"	41	6	94					
0.375	9.5	3/8"	57	8	92					
0.250	6.3	1/4"								
0.187	4.75	#4	128	19	81					
0.0937	2.36	#8	190	28	72					
0.0469	1.18	#16	252	37	63					
		#20								
0.0234	0.60	#30	312	46	54					
		#40								
0.0117	0.30	#50	381	56	44					
		#80								
0.0059	0.15	#100	446	65	35					
0.0029	0.075	#200	494	72	28					
PAN			509							
TOTAL										

Underline "WASH" if #200 wash analysis was done.

Remarks:

SIERRA GEOTECHNICAL SERVICES INC.

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SIEVE ANALYSIS OF FINE AND COARSE AGGREGATES

Per CTM 202 / ASTM C136 (underline one)

Project:	Snowcreek 8	Job No.:	3.00856.1
Client:	Chadmar	Tested by:	JK
Sampled by:	PS	Delivered by:	PS
Sample Date/time:	8/3/06	Delivered Date/time:	
Sample Location:	TP-30 @ 2-3'	Test Date:	8/30/06
Description:	Topsoil/Alluvium – SC-SM		

Dry Sample Total Weight (g): 281			#4 Minus Dry Wt. (g):						% Passing by Dry Weight:	
Sieve Size			Fine Wt. Ret.	Fine % Ret.	Fine % Pass.	Coarse Wt. Ret.	Coarse % Ret.	Coarse % Pass.	Coarse + Fine	Specified
Inches	mm	Mesh								
2.0	50.0	2"								
1.5	37.5	1 1/2"								
1.0	25.0	1"								
0.750	19.0	3/4"								
0.500	12.7	1/2"								
0.375	9.5	3/8"	5	2	98					
0.250	6.3	1/4"								
0.187	4.75	#4	32	11	89					
0.0937	2.36	#8	43	15	85					
0.0469	1.18	#16	70	25	75					
		#20								
0.0234	0.60	#30	99	35	65					
		#40								
0.0117	0.30	#50	118	42	58					
		#80								
0.0059	0.15	#100	160	57	43					
0.0029	0.075	#200	183	65.1	34.9					
PAN			188							
TOTAL										

Underline "WASH" if #200 wash analysis was done.

Remarks: _____

APPENDIX C

SEISMIC ANALYSIS

Seismic analysis was conducted for the subject site in order to develop parameters for structural design. This appendix presents the raw data from our analysis from three commercially available computer programs, **EQFAULT**, **EQSEARCH**, and **FRISKSP** (Blake, 2000). All three analyses used the same published attenuation relationship for “Very Dense Soil and Soft Rock” sites (Boore et. al., 1997).

EQFAULT: The program **EQFAULT** was used to develop the deterministic peak ground acceleration parameters summarized in herein. The Fault Location Map, which depicts active faults within a 62 mile (100 km) radius of the site, is also presented herein.

EQSEARCH: The program **EQSEARCH** was used to generate a table of estimated characteristics of nearby seismic events which were recorded between 1800 and 2005. This table is presented herein, and shows the epicenters, magnitudes, and dates of these nearby earthquakes, along with the estimated peak ground acceleration for the site. The Earthquake Epicenter Map, which depicts the epicenters and magnitudes of historical earthquakes that have affected the site, an Earthquake Recurrence Curve, and a plot depicting Earthquake Events versus Magnitude are also presented herein.

FRISKSP: The program **FRISKSP** was used to perform a probabilistic analysis of seismicity at the subject site. The probabilistic analysis was used to define the Upper-Bound and Design Basis Earthquakes at the site for use in structural design. The results of the analysis are presented in herein. Graphs including Non-magnitude Weighting Factors are also included herein.

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*
*   E Q F A U L T   *
*
*   Version 3.00     *
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 3.00856.1

DATE: 08-24-2006

JOB NAME: Snowcreek 8 Development

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: CDMGFLTE.DAT

SITE COORDINATES:

SITE LATITUDE: 37.6296

SITE LONGITUDE: 118.9677

SEARCH RADIUS: 62.2 mi

ATTENUATION RELATION: 2) Boore et al. (1997) Horiz. - NEHRP C (520)

UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0

DISTANCE MEASURE: cd_2drp

SCOND: 0

Basement Depth: 5.00 km Campbell SSR: Campbell SHR:

COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: CDMGFLTE.DAT

MINIMUM DEPTH VALUE (km): 0.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT			
		MAXIMUM	PEAK	EST. SITE	
		EARTHQUAKE MAG. (Mw)	SITE ACCEL. g	INTENSITY MOD.MERC.	
=====	=====	=====	=====	=====	=====
HARTLEY SPRINGS	0.7 (1.1)	6.6	0.456	X	
HILTON CREEK	6.1 (9.8)	6.7	0.282	IX	
ROUND VALLEY	13.1 (21.1)	6.8	0.178	VIII	
MONO LAKE	22.4 (36.0)	6.6	0.107	VII	
FISH SLOUGH	25.4 (40.9)	6.6	0.097	VII	
MOHWAK - HONEY LAKE ZONE	25.6 (41.2)	7.3	0.115	VII	
WHITE MOUNTAINS	32.2 (51.9)	7.1	0.087	VII	
ROBINSON CREEK	41.1 (66.1)	6.4	0.061	VI	
OWENS VALLEY	44.3 (71.3)	7.6	0.088	VII	
DEATH VALLEY (N. of Cucamongo)	44.7 (71.9)	7.0	0.064	VI	
BIRCH CREEK	47.9 (77.1)	6.4	0.054	VI	
FOOTHILLS FAULT SYSTEM	52.1 (83.8)	6.5	0.053	VI	
DEEP SPRINGS	52.3 (84.2)	6.6	0.056	VI	

-END OF SEARCH- 13 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

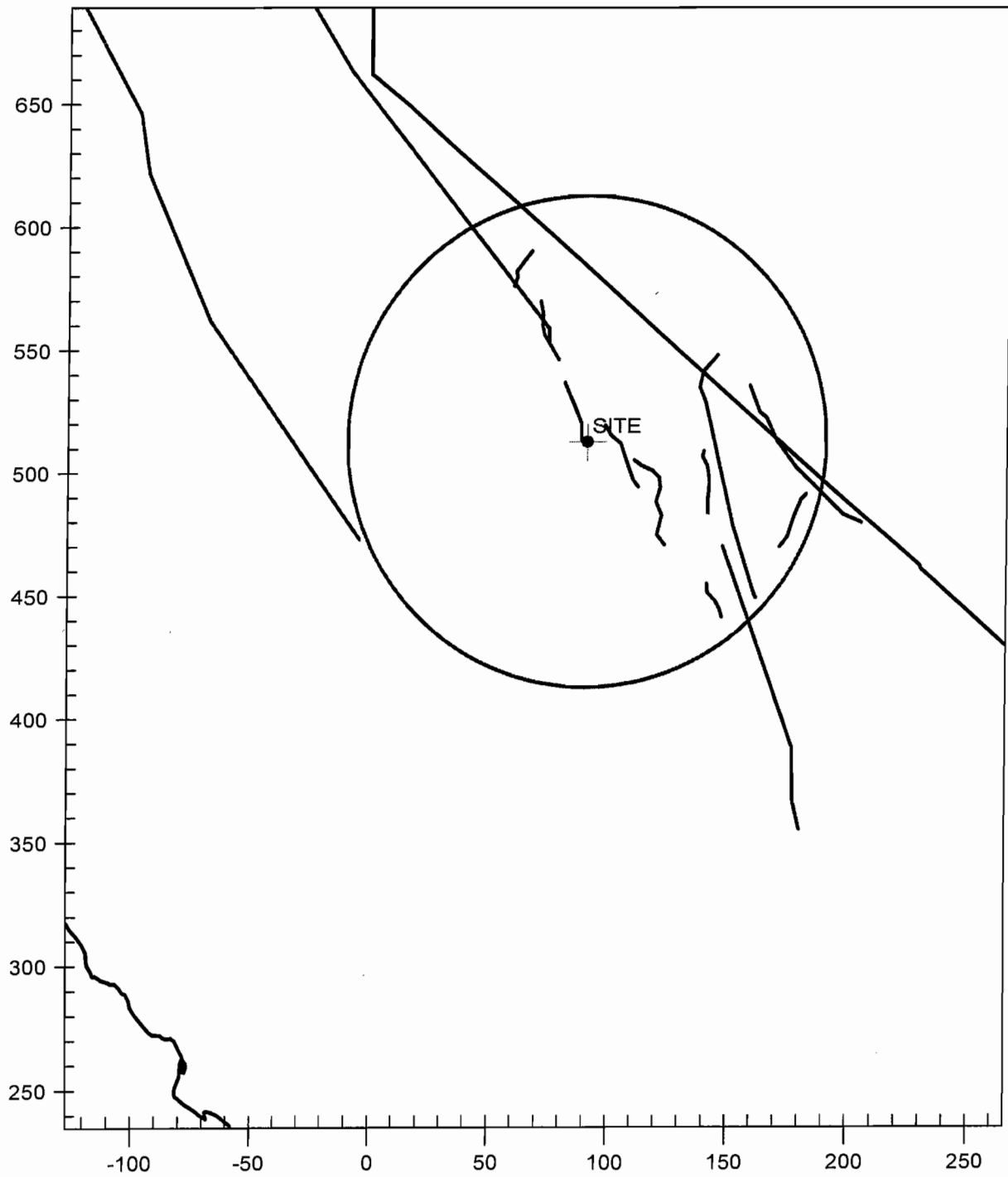
THE HARTLEY SPRINGS FAULT IS CLOSEST TO THE SITE.

IT IS ABOUT 0.7 MILES (1.1 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.4559 g

CALIFORNIA FAULT MAP

Snowcreek 8 Development



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

ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 3.00856.1

DATE: 08-24-2006

JOB NAME: Snowcreek 8 Development

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

MAGNITUDE RANGE:

MINIMUM MAGNITUDE: 4.50
MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:

SITE LATITUDE: 37.6296
SITE LONGITUDE: 118.9677

SEARCH DATES:

START DATE: 1800
END DATE: 2005

SEARCH RADIUS:

62.2 mi
100.1 km

ATTENUATION RELATION: 2) Boore et al. (1997) Horiz. - NEHRP C (520)
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
ASSUMED SOURCE TYPE: DS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
SCOND: 0 Depth Source: A
Basement Depth: 5.00 km Campbell SSR: Campbell SHR:
COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 0.0

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

Page 1

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]
PAS	37.6250	118.9530	01/07/1983	33023.8	5.0	4.50	0.149	VIII	0.9(1.4)
GSB	37.6360	118.9490	11/22/1997	181059.4	8.0	4.80	0.173	VIII	1.1(1.8)
GSB	37.6340	118.9460	11/30/1997	211705.4	7.0	4.90	0.181	VIII	1.2(2.0)
GSB	37.6380	118.9340	11/22/1997	172035.2	7.0	4.90	0.170	VIII	1.9(3.1)
DMG	37.6250	118.9260	10/03/1969	131010.3	-2.0	4.90	0.164	VIII	2.3(3.7)
PAS	37.6560	118.9290	01/07/1983	13810.6	5.7	5.70	0.237	IX	2.8(4.5)
GSB	37.6350	118.9170	11/22/1997	120656.0	8.0	4.60	0.133	VIII	2.8(4.5)
PAS	37.6640	119.0080	01/07/1983	32419.1	5.0	5.60	0.214	VIII	3.2(5.2)
PAS	37.6300	118.8920	04/28/1984	224821.3	6.0	4.80	0.127	VIII	4.1(6.7)
UNR	37.5820	118.9090	05/28/1980	6 743.4	0.2	4.50	0.103	VII	4.6(7.4)
PAS	37.6220	118.8810	09/30/1981	115327.0	6.0	5.80	0.201	VIII	4.8(7.7)
PAS	37.5940	118.8920	12/24/1980	154833.8	6.0	4.70	0.112	VII	4.8(7.7)
GSB	37.6310	118.8700	12/31/1997	203647.3	6.0	4.80	0.112	VII	5.3(8.6)
PAS	37.5800	118.8860	05/28/1980	51623.4	3.3	4.80	0.108	VII	5.6(9.1)
UNR	37.5580	118.9110	06/19/1980	71931.5	0.4	5.00	0.118	VII	5.8(9.4)
UNR	37.5600	118.9060	06/19/1980	72226.1	0.4	4.80	0.106	VII	5.9(9.4)
UNR	37.5670	118.8930	05/30/1980	154158.1	3.3	4.50	0.090	VII	5.9(9.6)
PAS	37.5550	118.9090	01/25/1983	101041.5	8.0	4.70	0.098	VII	6.1(9.8)
PAS	37.6720	118.8680	01/07/1983	13126.1	5.0	4.50	0.088	VII	6.2(9.9)
PAS	37.6620	118.8590	09/30/1981	13 548.2	3.0	4.60	0.091	VII	6.3(10.2)
PAS	37.5540	118.8970	08/01/1980	163856.3	4.7	5.40	0.137	VIII	6.5(10.4)
UNR	37.6360	118.8470	06/19/1980	14430.2	8.3	5.20	0.122	VII	6.6(10.6)
USG	37.5560	118.8820	08/01/1980	172821.4	1.2	4.79	0.095	VII	6.9(11.1)
USG	37.5610	118.8740	08/01/1980	164745.9	1.9	5.15	0.115	VII	7.0(11.2)
UNR	37.5710	118.8630	06/02/1980	203413.8	6.5	4.60	0.086	VII	7.0(11.3)
GSB	37.5650	118.8610	02/10/1997	232628.9	9.0	4.60	0.083	VII	7.3(11.8)
PAS	37.5920	118.8420	07/05/1980	115859.7	4.6	4.50	0.079	VII	7.3(11.8)
USG	37.5600	118.8660	08/01/1980	173153.5	8.7	4.99	0.102	VII	7.3(11.8)
GSB	37.5970	118.8360	03/05/1995	000703.1	11.0	4.50	0.077	VII	7.5(12.1)
USG	37.5420	118.8830	06/05/1980	20 452.3	1.6	5.05	0.103	VII	7.6(12.3)
PAS	37.6080	118.8210	05/25/1980	163344.8	3.7	6.40	0.200	VIII	8.2(13.1)
PAS	37.5440	118.8610	05/26/1980	185756.3	2.8	4.90	0.090	VII	8.3(13.4)
GSB	37.5270	118.8850	08/11/1993	054821.0	5.0	4.70	0.080	VII	8.4(13.5)
PAS	37.6010	118.8170	05/25/1980	202327.1	2.7	4.70	0.080	VII	8.5(13.6)
UNR	37.5290	118.8760	06/19/1980	44532.1	2.0	4.90	0.088	VII	8.6(13.8)
UNR	37.5360	118.8610	05/25/1980	183415.0	0.2	4.60	0.074	VII	8.7(14.0)
PAS	37.5580	118.8360	09/30/1983	1614 0.9	6.0	4.50	0.070	VI	8.7(14.1)
UNR	37.5180	118.8880	06/17/1980	122639.5	1.6	4.60	0.074	VII	8.8(14.2)
UNR	37.5360	118.8510	06/20/1980	152459.6	8.7	5.30	0.104	VII	9.1(14.6)
DMG	37.6170	118.8000	07/08/1940	105736.5	0.0	4.80	0.080	VII	9.2(14.8)
PAS	37.6760	118.8090	05/26/1980	162021.7	10.4	4.50	0.068	VI	9.2(14.9)
PAS	37.5090	119.0430	06/11/1980	441 1.1	14.1	5.00	0.088	VII	9.3(14.9)
PAS	37.5430	118.8370	05/26/1980	55628.1	11.0	4.60	0.071	VI	9.3(15.0)
PAS	37.6360	118.7950	05/31/1980	151611.7	5.6	4.90	0.082	VII	9.4(15.2)
DMG	37.5830	118.8000	06/22/1933	1241 2.0	0.0	4.90	0.081	VII	9.7(15.6)
DMG	37.5830	118.8000	06/22/1933	123628.0	0.0	4.90	0.081	VII	9.7(15.6)
GSB	37.5890	118.7950	06/09/1998	052440.2	6.0	5.20	0.094	VII	9.8(15.8)
GSG	37.5640	118.8050	07/15/1998	045319.2	6.0	5.10	0.088	VII	10.0(16.1)
UNR	37.5160	118.8370	06/18/1980	185537.7	6.3	5.30	0.094	VII	10.6(17.1)
GSG	37.5290	118.8170	05/15/1999	132210.7	5.0	5.60	0.109	VII	10.8(17.3)
PAS	37.5220	118.8230	05/25/1980	194125.1	4.2	4.50	0.061	VI	10.9(17.5)
PAS	37.5560	118.7910	05/25/1980	194452.2	6.4	6.50	0.173	VIII	10.9(17.6)
USG	37.5180	118.8260	07/10/1980	143053.9	4.2	4.93	0.076	VII	10.9(17.6)

EARTHQUAKE SEARCH RESULTS

Page 2

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]
PAS	37.6040	118.7700	05/27/1980	19 1 8.3	3.8	5.00	0.078	VII	11.0 (17.6)
PAS	37.7040	119.1450	05/28/1980	54827.9	7.7	4.80	0.070	VI	11.0 (17.6)
DMG	37.6330	118.7670	07/22/1940	23 032.9	0.0	4.60	0.063	VI	11.0 (17.7)
GSB	37.5070	118.8330	05/15/1999	175408.8	8.0	4.60	0.062	VI	11.2 (18.1)
USG	37.4980	118.8380	06/06/1980	141817.2	2.0	5.27	0.087	VII	11.5 (18.5)
PAS	37.4960	118.8200	05/27/1980	213454.3	4.7	4.50	0.056	VI	12.3 (19.7)
UNR	37.4730	118.8600	06/18/1980	115820.8	0.9	4.90	0.068	VI	12.3 (19.8)
PAS	37.4740	118.8490	05/25/1980	17 627.2	5.0	4.90	0.067	VI	12.6 (20.2)
PAS	37.8090	118.8830	07/03/1980	23956.2	3.9	4.50	0.053	VI	13.2 (21.3)
PAS	37.6210	118.7260	05/25/1980	165229.2	4.3	4.60	0.055	VI	13.2 (21.3)
PAS	37.4960	118.7920	05/25/1980	174833.8	4.5	4.60	0.055	VI	13.3 (21.4)
USG	37.5270	118.7590	06/08/1980	61139.6	1.3	5.12	0.072	VI	13.4 (21.6)
PAS	37.6340	118.7210	05/31/1980	05817.4	12.6	4.60	0.055	VI	13.5 (21.7)
DMG	37.5670	118.7330	09/14/1941	182118.7	0.0	5.50	0.087	VII	13.5 (21.8)
DMG	37.5670	118.7330	09/14/1941	164331.8	0.0	5.80	0.102	VII	13.5 (21.8)
DMG	37.5670	118.7330	09/14/1941	165458.0	0.0	4.50	0.052	VI	13.5 (21.8)
DMG	37.5670	118.7330	10/23/1941	204431.0	0.0	4.50	0.052	VI	13.5 (21.8)
DMG	37.5670	118.7330	09/14/1941	2116 1.0	0.0	5.00	0.067	VI	13.5 (21.8)
DMG	37.5670	118.7330	09/14/1941	183911.9	0.0	6.00	0.114	VII	13.5 (21.8)
DMG	37.5670	118.7330	01/01/1942	341 1.0	0.0	4.50	0.052	VI	13.5 (21.8)
DMG	37.5670	118.7330	12/05/1942	1852 7.0	0.0	4.50	0.052	VI	13.5 (21.8)
DMG	37.5670	118.7330	07/06/1942	211140.0	0.0	4.50	0.052	VI	13.5 (21.8)
DMG	37.5670	118.7330	12/31/1941	18 544.0	0.0	4.50	0.052	VI	13.5 (21.8)
DMG	37.5670	118.7330	02/04/1942	332 3.0	0.0	4.50	0.052	VI	13.5 (21.8)
DMG	37.5670	118.7330	12/31/1941	64844.0	0.0	5.40	0.083	VII	13.5 (21.8)
PAS	37.5530	118.7330	05/26/1980	119 4.8	4.5	4.70	0.056	VI	13.9 (22.3)
PAS	37.4640	118.8230	05/27/1980	145057.1	2.4	6.30	0.131	VIII	13.9 (22.4)
PAS	37.4810	118.7940	05/25/1980	205924.9	4.7	4.90	0.062	VI	14.0 (22.5)
PAS	37.4860	118.7830	05/25/1980	164930.3	4.7	5.80	0.099	VII	14.2 (22.8)
PAS	37.5170	118.7430	05/26/1980	122427.3	2.0	5.20	0.071	VI	14.5 (23.4)
DMG	37.5000	118.7500	09/18/1927	2 7 7.0	0.0	6.00	0.106	VII	14.9 (24.0)
DMG	37.5170	118.7330	06/05/1960	747 7.0	0.0	5.20	0.069	VI	15.0 (24.2)
PAS	37.5130	118.7320	05/25/1980	17 830.6	5.0	4.60	0.050	VI	15.2 (24.5)
PAS	37.5370	118.7130	05/25/1980	203551.0	5.0	5.50	0.080	VII	15.3 (24.7)
PAS	37.6070	118.6660	07/21/1986	145750.2	6.0	4.70	0.049	VI	16.6 (26.7)
PAS	37.5380	118.6750	10/04/1978	1739 3.3	6.3	5.30	0.066	VI	17.2 (27.7)
DMG	37.3830	118.9170	05/16/1935	325 0.0	0.0	4.50	0.043	VI	17.2 (27.8)
PAS	37.5140	118.6830	10/04/1978	164248.7	5.6	5.80	0.085	VII	17.5 (28.2)
DMG	37.7000	119.2800	06/21/1957	04125.0	0.0	4.60	0.045	VI	17.7 (28.6)
GSB	37.4870	118.6350	11/28/1984	165738.1	18.0	4.60	0.040	V	20.7 (33.3)
PAS	37.4490	118.6530	11/26/1984	162141.4	6.0	5.50	0.062	VI	21.3 (34.2)
PAS	37.5830	118.5830	07/21/1986	145358.1	6.0	4.90	0.046	VI	21.3 (34.2)
DMG	37.5670	118.5830	12/28/1951	24927.0	0.0	5.20	0.053	VI	21.5 (34.6)
PAS	37.6130	118.5690	07/21/1986	22 922.1	6.0	4.70	0.040	V	21.8 (35.1)
DMG	37.4170	118.6670	02/02/1961	0 742.0	0.0	5.10	0.049	VI	22.1 (35.5)
PAS	37.5930	118.5670	10/25/1980	52646.4	5.0	4.50	0.036	V	22.1 (35.5)
DMG	37.4500	118.6330	02/02/1961	0 416.0	0.0	5.30	0.055	VI	22.1 (35.6)
DMG	37.5500	118.5670	06/18/1959	02940.0	0.0	4.70	0.039	V	22.6 (36.4)
PAS	37.4510	118.6200	11/25/1984	2310 9.6	6.0	4.70	0.039	V	22.7 (36.5)
PAS	37.4690	118.5980	05/04/1985	32246.2	6.0	4.70	0.039	V	23.1 (37.1)
PAS	37.4700	118.5970	11/23/1984	18 825.6	6.0	6.20	0.085	VII	23.1 (37.2)
DMG	37.4530	118.6040	12/03/1938	174252.6	10.0	5.70	0.065	VI	23.3 (37.6)
PAS	37.4230	118.6150	08/27/1985	3 4 6.8	6.0	4.50	0.034	V	24.0 (38.6)

EARTHQUAKE SEARCH RESULTS

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FILE	LAT.	LONG.	DATE	TIME	DEPTH	QUAKE	SITE	SITE	APPROX.
CODE	NORTH	WEST		(UTC)	(km)	MAG.	ACC.	MM	DISTANCE
				H M Sec			g	INT.	mi [km]
PAS	37.4230	118.6080	11/23/1984	191235.3	6.0	5.40	0.054	VI	24.3(39.1)
DMG	37.5000	118.5330	02/24/1940	938 0.0	0.0	4.50	0.032	V	25.4(40.9)
PAS	37.6470	118.5020	07/21/1986	22 718.7	6.0	5.40	0.052	VI	25.5(41.0)
GSB	37.5470	118.5030	07/21/1986	152649.3	10.0	4.60	0.033	V	26.1(41.9)
DMG	37.3330	118.6670	10/25/1932	328 0.0	0.0	4.50	0.031	V	26.3(42.3)
PAS	37.4480	118.5450	03/25/1985	16 513.6	6.0	5.00	0.041	V	26.3(42.3)
DMG	37.3830	118.6000	05/31/1943	201653.0	0.0	4.50	0.031	V	26.4(42.4)
GSB	37.6650	118.4870	07/21/1986	154624.8	10.0	4.50	0.031	V	26.4(42.5)
PAS	37.5970	118.4860	07/21/1986	151130.8	6.0	4.70	0.035	V	26.4(42.5)
UNR	37.4330	118.5500	07/07/1976	182738.0	8.1	4.60	0.033	V	26.6(42.8)
PAS	37.6120	118.4740	07/29/1986	95757.4	6.0	4.60	0.032	V	27.0(43.5)
DMG	37.5000	118.5000	04/11/1872	19 0 0.0	0.0	6.60	0.093	VII	27.1(43.6)
PAS	37.5340	118.4810	07/22/1986	122450.3	6.0	4.60	0.032	V	27.4(44.1)
PAS	37.5800	118.4680	07/21/1986	111522.0	6.0	4.60	0.032	V	27.5(44.3)
DMG	37.3170	118.6500	01/13/1946	163115.0	0.0	4.70	0.033	V	27.7(44.6)
PAS	37.3500	118.6030	11/28/1984	162326.7	6.0	4.70	0.033	V	27.8(44.7)
PAS	37.5890	118.4620	07/30/1986	64153.0	6.0	4.80	0.035	V	27.8(44.7)
PAS	37.5860	118.4570	07/21/1986	151935.7	6.0	4.50	0.030	V	28.1(45.2)
PAS	37.5830	118.4500	07/20/1986	142946.3	6.0	5.90	0.062	VI	28.5(45.9)
PAS	37.5420	118.4440	07/21/1986	144226.7	6.0	5.90	0.060	VI	29.3(47.1)
USG	38.0020	118.6900	06/30/1980	172917.6	8.2	5.29	0.043	VI	29.8(48.0)
DMG	37.3500	118.5500	08/04/1959	73659.0	0.0	5.20	0.041	V	29.9(48.2)
PAS	37.5280	118.4300	05/17/1980	0 117.0	4.9	4.50	0.028	V	30.2(48.7)
PAS	37.5830	118.4170	07/21/1986	144521.0	6.0	4.60	0.030	V	30.3(48.7)
PAS	37.5830	118.4170	07/21/1986	145439.2	6.0	4.50	0.028	V	30.3(48.7)
USG	38.0680	119.0020	07/05/1983	142725.9	19.3	4.60	0.030	V	30.3(48.8)
DMG	37.4160	118.4790	09/22/1965	214925.9	7.7	4.50	0.028	V	30.6(49.2)
GSP	37.6060	118.4090	07/22/1986	000953.6	5.0	4.60	0.029	V	30.6(49.2)
GSB	38.0470	119.1570	10/24/1990	061520.7	12.0	5.70	0.053	VI	30.6(49.3)
PAS	37.6200	118.3980	07/22/1986	134859.0	6.0	5.20	0.040	V	31.2(50.1)
PAS	37.6550	118.3980	07/21/1986	17 532.3	6.0	4.60	0.029	V	31.2(50.2)
DMG	37.6480	118.3960	12/06/1963	83421.5	1.7	4.70	0.031	V	31.3(50.3)
UNR	37.4990	118.4180	12/10/1975	191924.4	0.9	4.50	0.027	V	31.4(50.5)
PAS	37.6270	118.3860	07/22/1986	133358.9	6.0	5.00	0.035	V	31.8(51.2)
DMG	37.6670	118.3830	04/13/1949	75826.0	0.0	4.50	0.027	V	32.1(51.6)
PAS	37.6420	118.3730	07/20/1986	183851.9	6.0	4.80	0.031	V	32.5(52.3)
PAS	37.9500	118.5280	05/07/1981	1 238.0	4.0	4.60	0.028	V	32.6(52.5)
DMG	37.6100	118.3680	05/10/1936	174013.2	10.0	5.00	0.034	V	32.8(52.8)
DMG	37.2000	118.7000	09/30/1889	520 0.0	0.0	5.60	0.047	VI	33.1(53.3)
PAS	38.0440	118.6600	06/29/1980	74613.8	7.6	4.70	0.029	V	33.2(53.4)
GSB	37.3750	118.4420	08/01/1986	142818.0	5.0	4.70	0.029	V	33.7(54.3)
PAS	37.4730	118.3720	07/31/1986	72240.5	6.0	5.90	0.053	VI	34.3(55.3)
DMG	37.4670	118.3670	12/09/1949	1239 2.0	0.0	4.60	0.027	V	34.7(55.9)
PAS	37.5990	118.3200	07/21/1986	1451 8.8	6.0	5.40	0.040	V	35.5(57.1)
USG	38.0450	118.5730	04/15/1982	2152 8.6	18.7	5.10	0.034	V	35.9(57.7)
GSB	38.1400	118.8380	01/24/1985	112721.7	7.0	5.30	0.038	V	35.9(57.8)
DMG	37.3300	118.4200	05/06/1910	1640 0.0	0.0	5.50	0.041	V	36.4(58.6)
DMG	37.3300	118.4200	01/05/1912	354 0.0	0.0	5.50	0.041	V	36.4(58.6)
DMG	37.2500	118.5000	06/19/1935	955 0.0	0.0	4.50	0.024	V	36.7(59.0)
PAS	38.0690	118.5760	11/11/1980	1019 2.7	5.0	4.60	0.025	V	37.1(59.7)
DMG	37.8090	118.3230	12/03/1938	184116.4	10.0	4.50	0.024	V	37.3(60.1)
DMG	37.6840	118.2630	01/22/1972	25718.5	8.0	4.60	0.025	V	38.7(62.3)
PAS	37.9940	118.4020	09/07/1980	43741.1	5.2	5.70	0.043	VI	39.8(64.1)

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

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FILE	LAT.	LONG.	DATE	TIME	DEPTH	QUAKE	SITE	SITE	APPROX.
CODE	NORTH	WEST		(UTC)	(km)	MAG.	ACC.	MM	DISTANCE
				H M Sec			g	INT.	mi [km]
PAS	38.0030	118.4010	09/07/1980	64313.4	4.9	4.50	0.023	IV	40.2 (64.8)
PAS	38.0050	118.3930	09/04/1980	21 336.7	4.5	4.60	0.024	IV	40.7 (65.4)
PAS	38.1430	118.5740	04/28/1981	225449.0	5.0	4.80	0.026	V	41.4 (66.7)
DMG	38.0000	118.3700	02/06/1968	34810.8	0.0	4.50	0.022	IV	41.4 (66.7)
GSB	38.2270	118.7580	08/12/1991	211150.8	8.0	4.80	0.025	V	42.8 (68.9)
DMG	38.2500	118.9300	12/31/1956	173745.0	0.0	5.00	0.028	V	42.9 (69.0)
DMG	38.0200	118.3500	02/06/1968	04138.0	0.0	4.90	0.026	V	43.1 (69.4)
PAS	38.2250	118.6910	09/04/1980	1339 6.7	4.9	4.50	0.021	IV	43.8 (70.5)
DMG	38.0000	118.3000	08/21/1925	1114 0.0	12.0	4.80	0.025	V	44.5 (71.6)
DMG	38.2800	118.9700	12/31/1956	173924.0	0.0	5.10	0.029	V	44.9 (72.3)
PAS	38.0500	118.3330	09/07/1980	64833.9	5.0	5.40	0.033	V	45.2 (72.7)
DMG	38.2500	119.2500	01/05/1933	912 0.0	0.0	4.50	0.021	IV	45.5 (73.2)
DMG	38.2500	119.2500	01/05/1933	743 0.0	0.0	4.50	0.021	IV	45.5 (73.2)
GSB	37.8010	118.1430	11/02/1997	085154.2	5.0	5.50	0.034	V	46.6 (74.9)
UNR	38.2330	119.3520	10/07/1979	205441.2	8.4	4.90	0.025	V	46.6 (75.0)
GSB	37.8540	118.1610	11/02/1997	150304.3	5.0	4.60	0.021	IV	46.7 (75.1)
PAS	38.0390	118.2670	09/07/1980	13045.8	4.8	5.00	0.026	V	47.5 (76.5)
DMG	38.3000	119.2000	06/03/1965	162627.4	15.0	4.80	0.023	IV	48.0 (77.2)
GSB	37.9880	118.2100	01/15/1990	052903.5	5.0	5.00	0.026	V	48.2 (77.5)
PAS	38.1210	118.3390	01/28/1981	20 851.9	5.0	4.60	0.021	IV	48.2 (77.6)
PAS	38.2350	118.5200	12/28/1980	2258 7.2	6.0	4.70	0.022	IV	48.4 (77.9)
DMG	38.2200	119.4500	04/13/1962	153851.9	6.0	5.10	0.027	V	48.5 (78.0)
PAS	37.8810	118.1350	09/24/1982	74024.6	11.6	5.20	0.028	V	48.7 (78.3)
DMG	38.3000	119.3000	06/03/1965	1631 2.2	15.0	4.90	0.024	IV	49.7 (80.0)
DMG	38.3300	118.6700	08/08/1955	103535.0	0.0	5.20	0.027	V	51.0 (82.1)
PAS	38.1460	118.2590	12/28/1982	19 622.9	6.0	5.20	0.027	V	52.6 (84.6)
DMG	38.3000	119.5000	12/19/1919	1359 0.0	0.0	5.20	0.026	V	54.6 (87.9)
MGI	38.3000	118.4000	01/30/1934	1924 0.0	0.0	5.70	0.033	V	55.7 (89.6)
DMG	38.2800	118.3600	01/30/1934	201631.0	0.0	6.30	0.045	VI	55.8 (89.8)
DMG	38.2000	118.2000	08/09/1943	530 4.0	0.0	5.50	0.029	V	57.4 (92.4)
GSB	38.4170	119.3200	10/31/1986	035729.0	3.0	4.60	0.018	IV	57.6 (92.8)
DMG	37.0800	118.1700	09/23/1931	825 0.0	0.0	4.50	0.017	IV	57.9 (93.2)
DMG	38.0000	118.0000	03/23/1934	2249 0.0	0.0	4.50	0.017	IV	58.6 (94.4)
MGI	38.0000	118.0000	11/22/1910	6 5 0.0	0.0	5.70	0.032	V	58.6 (94.4)
DMG	38.0000	118.0000	08/19/1937	7 3 0.0	0.0	4.50	0.017	IV	58.6 (94.4)
MGI	38.0000	118.0000	11/22/1910	030 0.0	0.0	5.00	0.022	IV	58.6 (94.4)
MGI	38.0000	118.0000	11/19/1910	225 0.0	0.0	5.70	0.032	V	58.6 (94.4)
DMG	38.0000	118.0000	05/11/1939	242 0.0	0.0	4.50	0.017	IV	58.6 (94.4)
DMG	38.0000	118.0000	03/19/1934	1041 0.0	0.0	4.50	0.017	IV	58.6 (94.4)
DMG	38.0000	118.0000	01/02/1933	134 0.0	0.0	4.50	0.017	IV	58.6 (94.4)
DMG	38.0000	118.0000	04/02/1934	8 5 0.0	0.0	4.50	0.017	IV	58.6 (94.4)
MGI	38.0000	118.0000	11/21/1910	2323 0.0	0.0	6.30	0.044	VI	58.6 (94.4)
DMG	38.0000	118.0000	09/23/1938	820 0.0	0.0	4.50	0.017	IV	58.6 (94.4)
DMG	38.0000	118.0000	01/30/1936	1832 0.0	0.0	4.50	0.017	IV	58.6 (94.4)
DMG	38.0000	118.0000	03/13/1934	1620 0.0	0.0	5.00	0.022	IV	58.6 (94.4)
DMG	38.0000	118.0000	02/19/1937	23 6 0.0	0.0	4.50	0.017	IV	58.6 (94.4)
DMG	38.0000	118.0000	03/13/1934	1611 0.0	0.0	4.50	0.017	IV	58.6 (94.4)
DMG	38.3000	118.3000	02/19/1937	9 9 0.0	0.0	5.00	0.022	IV	58.8 (94.7)
DMG	38.4500	118.6200	07/06/1956	33135.0	0.0	4.90	0.021	IV	59.7 (96.1)
DMG	37.0000	118.2000	04/03/1872	1215 0.0	0.0	6.10	0.038	V	60.6 (97.4)
MGI	37.0000	118.1700	12/02/1929	7 0 0.0	0.0	5.30	0.025	V	61.7 (99.3)
MGI	37.0000	118.1700	12/08/1929	1245 0.0	0.0	5.30	0.025	V	61.7 (99.3)
UNR	38.4880	119.2850	02/22/1977	624 6.4	6.7	4.90	0.020	IV	61.7 (99.3)

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

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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]
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-END OF SEARCH- 212 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2005

LENGTH OF SEARCH TIME: 206 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 0.9 MILES (1.4 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 6.6

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.237 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:

a-value= 2.952

b-value= 0.690

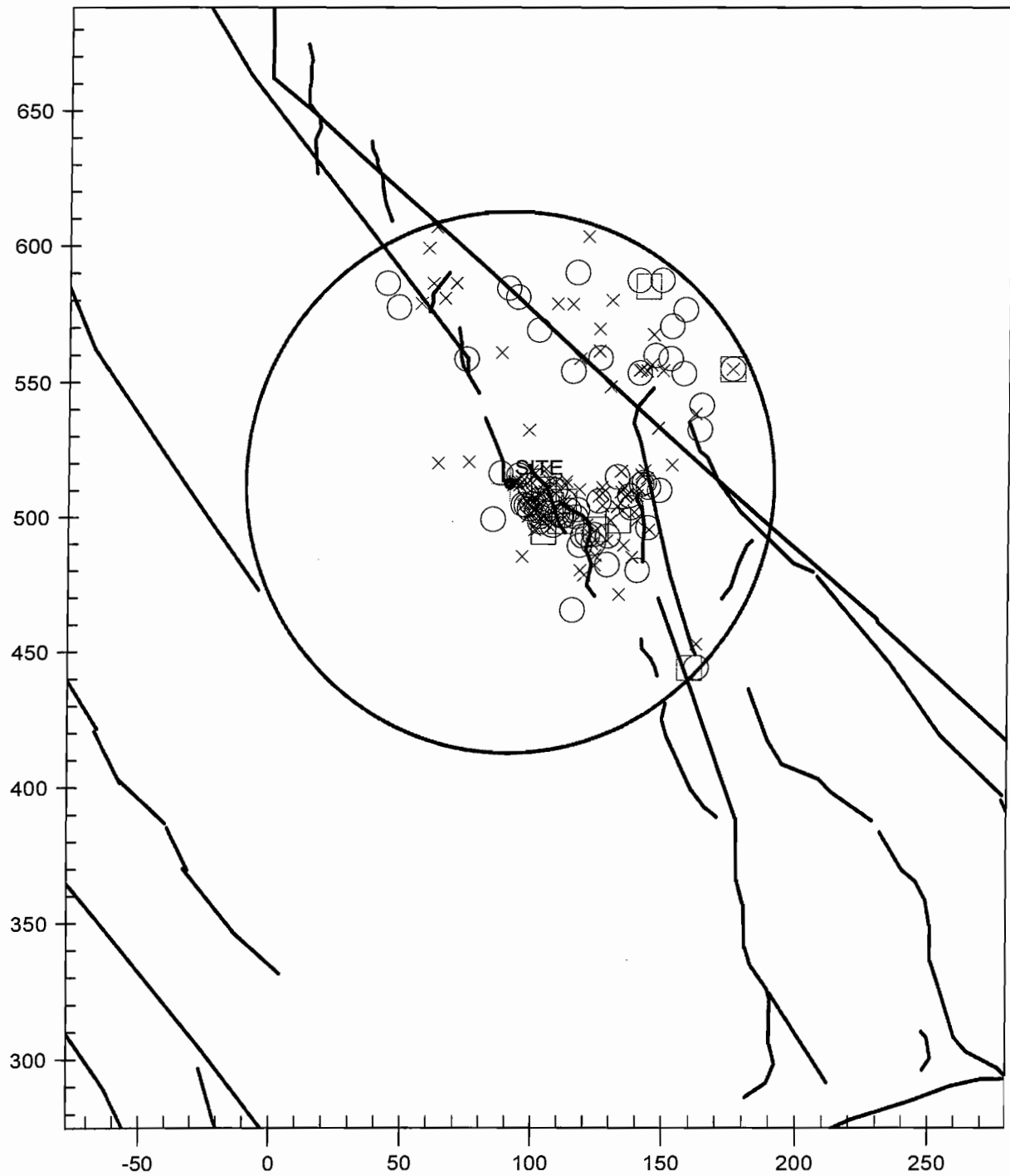
beta-value= 1.588

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

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	212	1.02913
4.5	212	1.02913
5.0	81	0.39320
5.5	34	0.16505
6.0	10	0.04854
6.5	2	0.00971

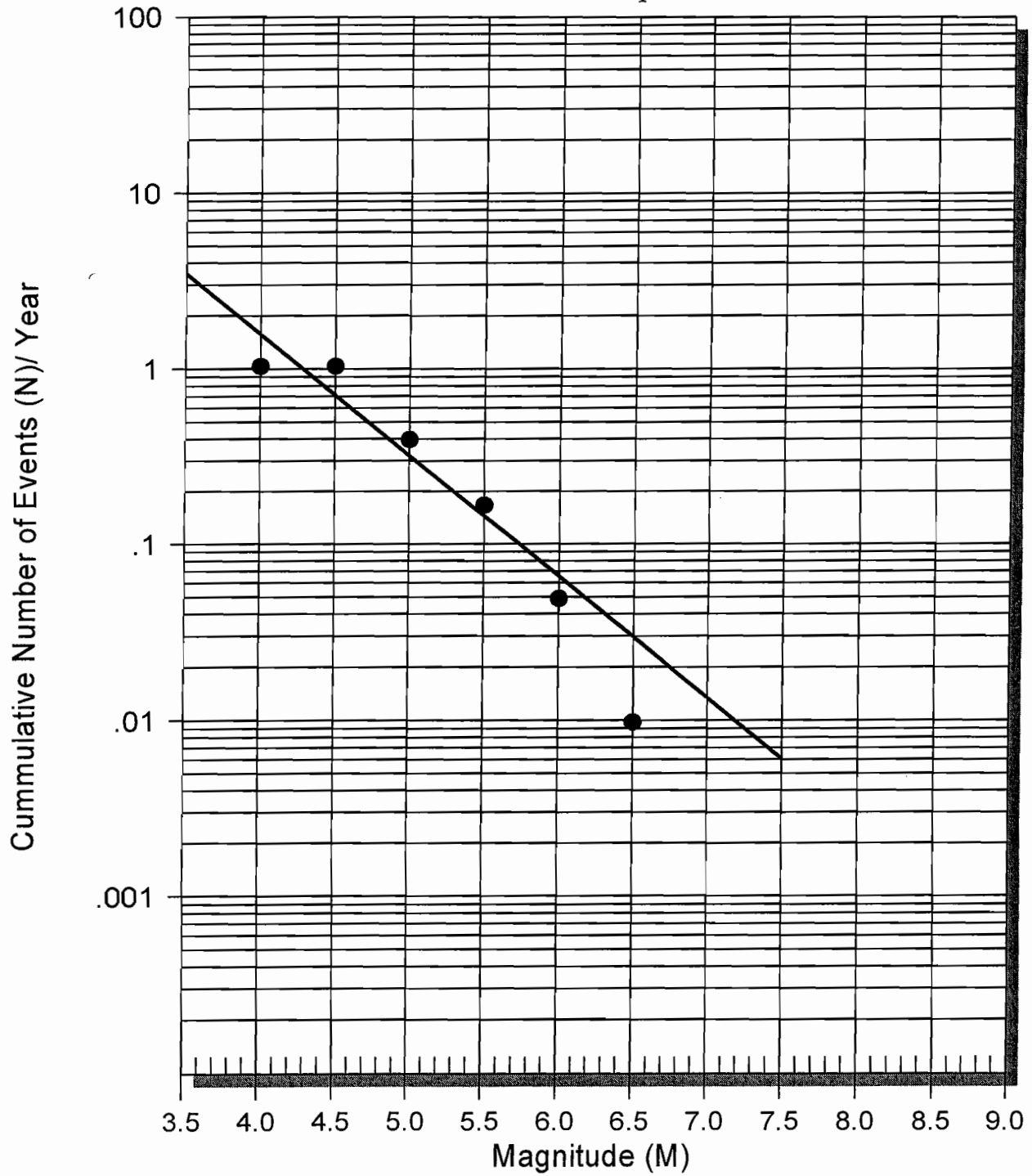
EARTHQUAKE EPICENTER MAP

Snowcreek 8 Development



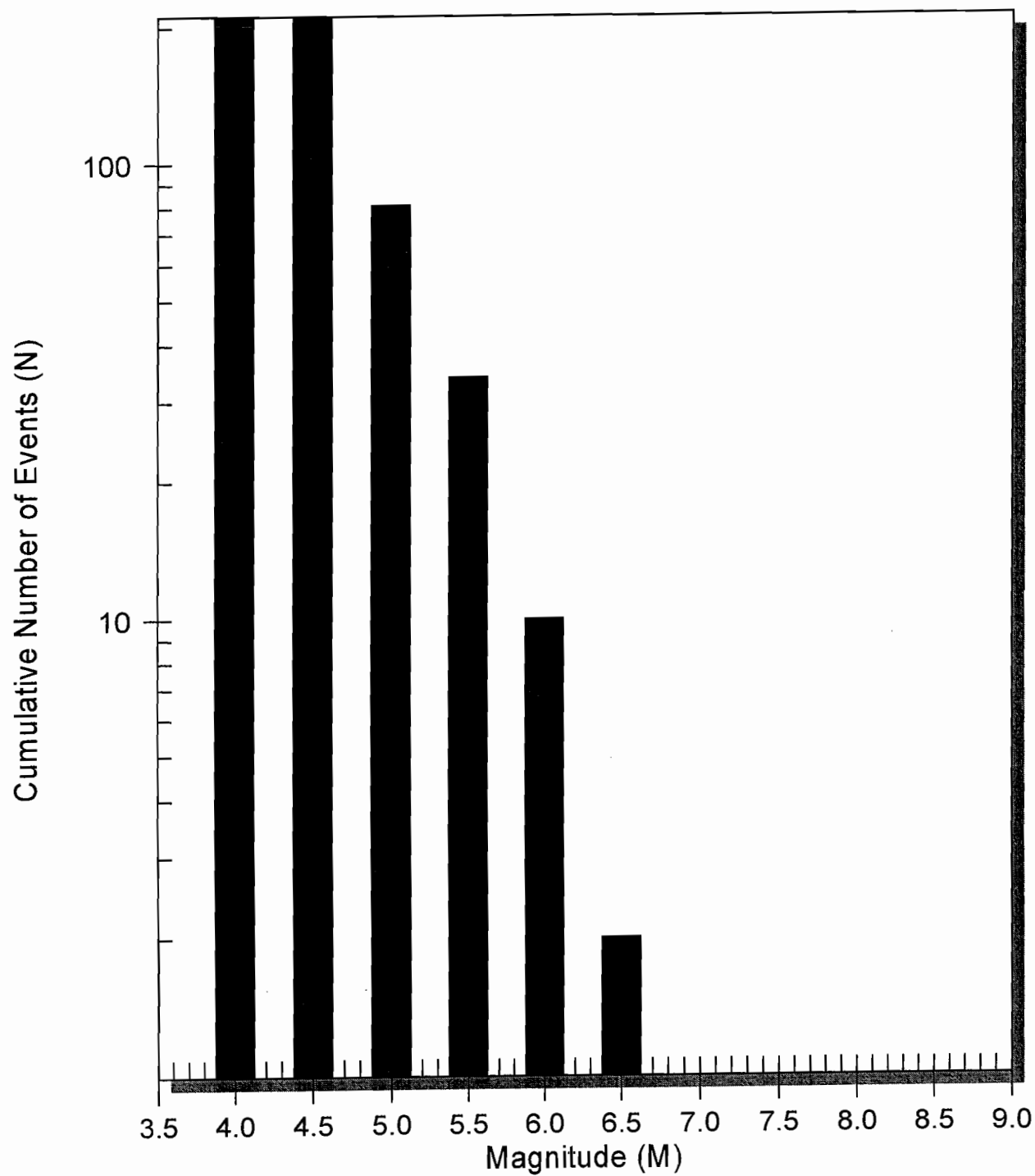
EARTHQUAKE RECURRENT CURVE

Snowcreek 8 Development



Number of Earthquakes (N) Above Magnitude (M)

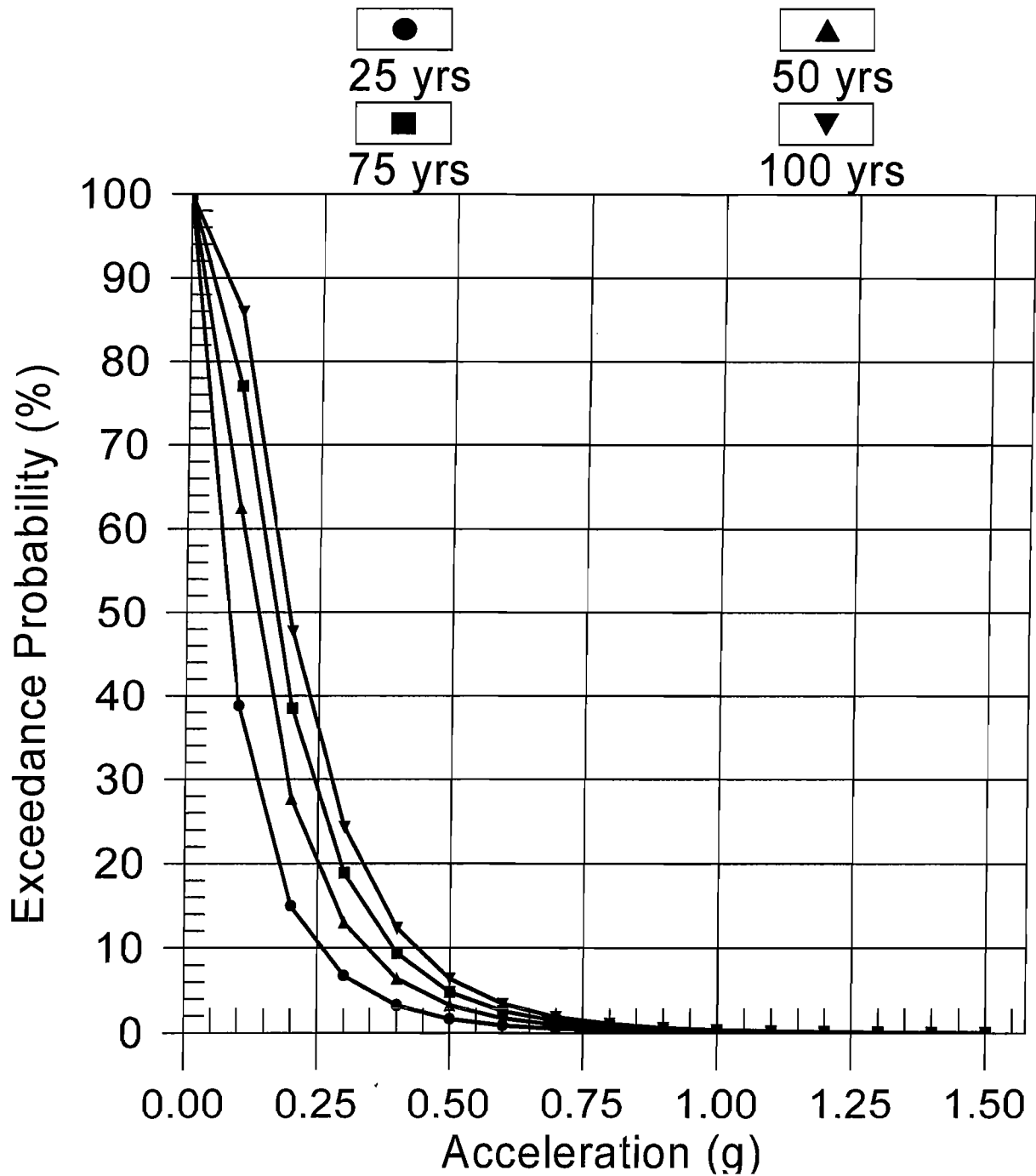
Snowcreek 8 Development



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*****
*
*      FRISKSP - IBM-PC VERSION      *
*
*   Modified from *FRISK* (McGuire 1978) *
*   To Perform Probabilistic Earthquake *
*   Hazard Analyses Using Multiple Forms *
*   of Ground-Motion-Attenuation Relations *
*
*   Modifications by:  Thomas F. Blake *
*                   - 1988-2000 -      *
*
*                   VERSION 4.00      *
*                   (Visual Fortran)  *
*****
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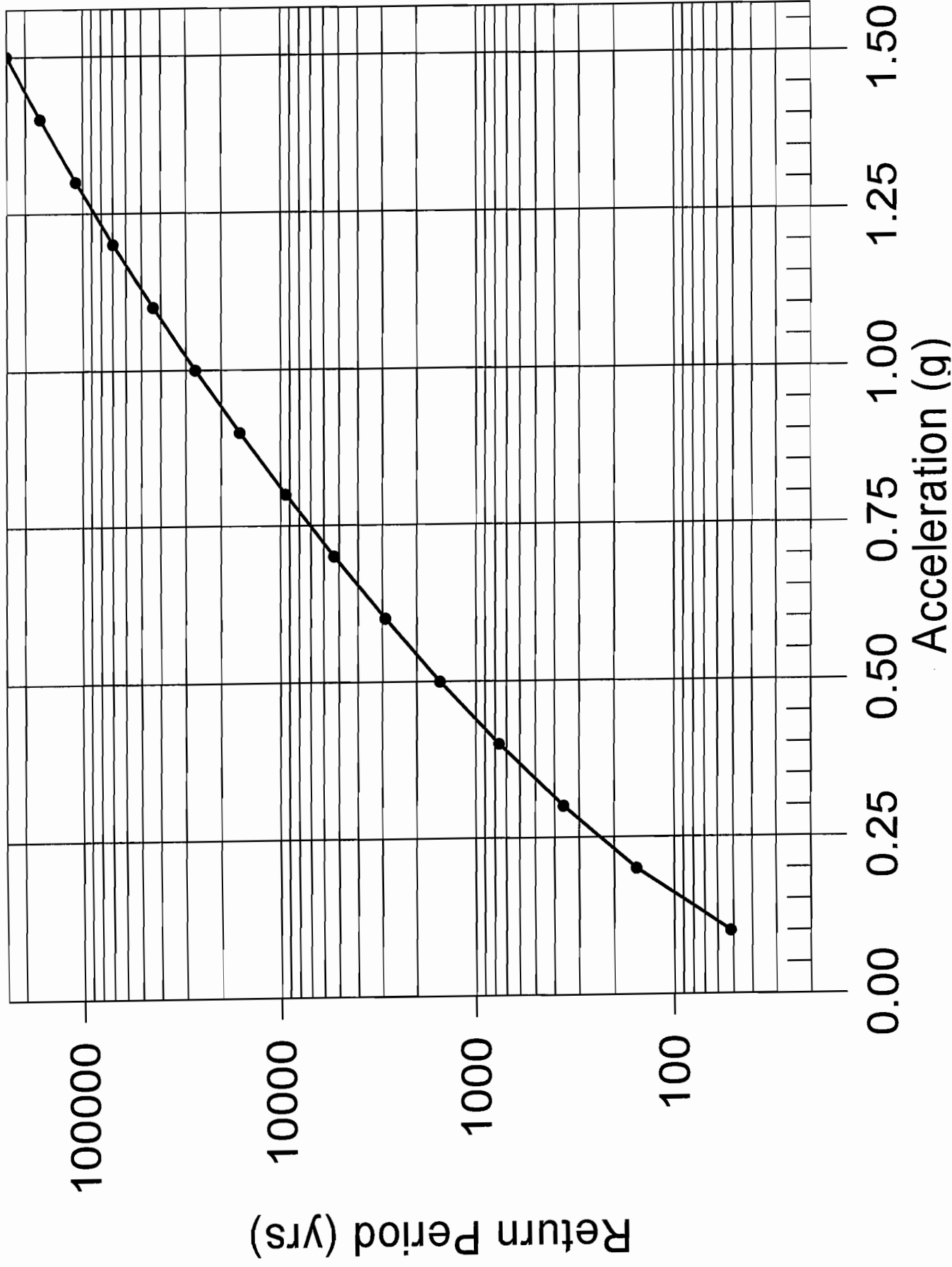

PROBABILITY OF EXCEEDANCE

BOORE ET AL(1997) NEHRP C (520)1



RETURN PERIOD vs. ACCELERATION

BOORE ET AL(1997) NEHRP C (520)1



APPENDIX C

UNIFORM BUILDING CODE DESIGN PARAMETERS

UBCSEIS: The program **UBCSEIS** was used to compute the distances between the site and faults in a data file to select corresponding Uniform Building Code seismic coefficients, and aide in the construction of a site specific design response spectrum. The results of the analysis are presented herein. A graph including the design response spectrum is also included in herein.

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*****
*
*   U B C S E I S   *
*
*   Version  1.03   *
*
*****

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COMPUTATION OF 1997
UNIFORM BUILDING CODE
SEISMIC DESIGN PARAMETERS

JOB NUMBER: 3.00856.1

DATE: 08-24-2006

JOB NAME: Snowcreek 8 Development

FAULT-DATA-FILE NAME: CDMGUBCR.DAT

SITE COORDINATES:

SITE LATITUDE: 37.6296

SITE LONGITUDE: 118.9677

UBC SEISMIC ZONE: 0.4

UBC SOIL PROFILE TYPE: SD

NEAREST TYPE A FAULT:

NAME: DEATH VALLEY (N. of Cucamongo)

DISTANCE: 71.9 km

NEAREST TYPE B FAULT:

NAME: HARTLEY SPRINGS

DISTANCE: 1.1 km

NEAREST TYPE C FAULT:

NAME:

DISTANCE: 99999.0 km

SELECTED UBC SEISMIC COEFFICIENTS:

Na: 1.3

Nv: 1.6

Ca: 0.57

Cv: 1.02

Ts: 0.716

To: 0.143

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*****
* CAUTION: The digitized data points used to model faults are *
* limited in number and have been digitized from small- *
* scale maps (e.g., 1:750,000 scale). Consequently, *
* the estimated fault-site-distances may be in error by *
* several kilometers. Therefore, it is important that *
* the distances be carefully checked for accuracy and *
* adjusted as needed, before they are used in design. *
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SUMMARY OF FAULT PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROX. DISTANCE (km)	SOURCE TYPE (A,B,C)	MAX. MAG. (Mw)	SLIP RATE (mm/yr)	FAULT TYPE (SS,DS,BT)
HARTLEY SPRINGS	1.1	B	6.6	0.50	DS
HILTON CREEK	9.8	B	6.7	2.50	DS
ROUND VALLEY (E. of S.N.Mtns.)	21.1	B	6.8	1.00	DS
MONO LAKE	36.0	B	6.6	2.50	DS
FISH SLOUGH	42.7	B	6.6	0.20	DS
WHITE MOUNTAINS	51.9	B	7.1	1.00	SS
ROBINSON CREEK	67.2	B	6.5	0.50	DS
OWENS VALLEY	71.3	B	7.6	1.50	SS
DEATH VALLEY (N. of Cucamongo)	71.9	A	7.0	5.00	SS
BIRCH CREEK	77.1	B	6.5	0.70	DS
DEEP SPRINGS	85.9	B	6.6	0.80	DS
INDEPENDENCE	101.6	B	6.9	0.20	DS
ANTELOPE VALLEY	107.5	B	6.7	0.80	DS
HUNTER MTN. - SALINE VALLEY	119.0	B	7.0	2.50	SS
DEATH VALLEY (Northern)	121.6	A	7.2	5.00	SS
GENOA	134.6	B	6.9	1.00	DS
So. SIERRA NEVADA	182.0	B	7.1	0.10	DS
PANAMINT VALLEY	191.0	B	7.2	2.50	SS
ORTIGALITA	196.5	B	6.9	1.00	SS
LITTLE LAKE	213.8	B	6.7	0.70	SS
DEATH VALLEY (Graben)	217.4	B	6.9	4.00	DS
QUIEN SABE	220.2	B	6.5	1.00	SS
SAN ANDREAS (Creeping)	224.0	B	5.0	34.00	SS
CALAVERAS (So.of Calaveras Res)	224.6	B	6.2	15.00	SS
GREENVILLE	224.6	B	6.9	2.00	SS
SAN ANDREAS - 1857 Rupture	229.0	A	7.8	34.00	SS
SARGENT	234.2	B	6.8	3.00	SS
ZAYANTE-VERGELES	238.7	B	6.8	0.10	SS
SAN ANDREAS (1906)	240.8	A	7.9	24.00	SS
HAYWARD (SE Extension)	245.3	B	6.5	3.00	SS
SAN JUAN	246.3	B	7.0	1.00	SS
CALAVERAS (No.of Calaveras Res)	250.9	B	6.8	6.00	SS
HAYWARD (Total Length)	250.9	A	7.1	9.00	SS
WHITE WOLF	252.5	B	7.2	2.00	DS
RINCONADA	254.9	B	7.3	1.00	SS
TANK CANYON	256.3	B	6.5	1.00	DS
MONTE VISTA - SHANNON	257.0	B	6.5	0.40	DS
DEATH VALLEY (South)	263.0	B	6.9	4.00	SS
MONTEREY BAY - TULARCITOS	265.3	B	7.1	0.50	DS
CONCORD - GREEN VALLEY	266.4	B	6.9	6.00	SS
GARLOCK (East)	268.1	A	7.3	7.00	SS
GARLOCK (West)	275.4	A	7.1	6.00	SS
BLACKWATER	278.4	B	6.9	0.60	SS
LENWOOD-LOCKHART-OLD WOMAN SPRGS	288.2	B	7.3	0.60	SS
OWL LAKE	290.7	B	6.5	2.00	SS
PLEITO THRUST	291.5	B	6.8	2.00	DS

SUMMARY OF FAULT PARAMETERS

Page 2

ABBREVIATED FAULT NAME	APPROX. DISTANCE (km)	SOURCE TYPE (A,B,C)	MAX. MAG. (Mw)	SLIP RATE (mm/yr)	FAULT TYPE (SS,DS,BT)
HOSGRI	292.6	B	7.3	2.50	SS
PALO COLORADO - SUR	292.7	B	7.0	3.00	SS
SAN GREGORIO	293.3	A	7.3	5.00	SS
WEST NAPA	294.6	B	6.5	1.00	SS
GRAVEL HILLS - HARPER LAKE	295.7	B	6.9	0.60	SS
HUNTING CREEK - BERRYESSA	298.8	B	6.9	6.00	SS
RODGERS CREEK	301.2	A	7.0	9.00	SS
LOS OSOS	303.6	B	6.8	0.50	DS
SAN LUIS RANGE (S. Margin)	303.7	B	7.0	0.20	DS
BIG PINE	312.3	B	6.7	0.80	SS
HELENDALE - S. LOCKHARDT	313.4	B	7.1	0.60	SS
SAN GABRIEL	324.3	B	7.0	1.00	SS
CASMALIA (Orcutt Frontal Fault)	333.4	B	6.5	0.25	DS
LIONS HEAD	336.8	B	6.6	0.02	DS
SANTA YNEZ (East)	338.4	B	7.0	2.00	SS
LOS ALAMOS-W. BASELINE	340.4	B	6.8	0.70	DS
POINT REYES	341.2	B	6.8	0.30	DS
BARTLETT SPRINGS	343.5	A	7.1	6.00	SS
CALICO - HIDALGO	344.0	B	7.1	0.60	SS
MAACAMA (South)	344.5	B	6.9	9.00	SS
SAN CAYETANO	345.9	B	6.8	6.00	DS
M. RIDGE-ARROYO PARIDA-SANTA ANA	346.6	B	6.7	0.40	DS
SANTA YNEZ (West)	350.0	B	6.9	2.00	SS
COLLAYOMI	351.1	B	6.5	0.60	SS
HOLSER	354.5	B	6.5	0.40	DS
RED MOUNTAIN	357.0	B	6.8	2.00	DS
SANTA SUSANA	357.2	B	6.6	5.00	DS
OAK RIDGE (Onshore)	359.4	B	6.9	4.00	DS
SIERRA MADRE (San Fernando)	362.0	B	6.7	2.00	DS
VENTURA - PITAS POINT	363.8	B	6.8	1.00	DS
SIMI-SANTA ROSA	366.7	B	6.7	1.00	DS
LANDERS	367.4	B	7.3	0.60	SS
SIERRA MADRE (Central)	367.8	B	7.0	3.00	DS
VERDUGO	370.4	B	6.7	0.50	DS
MAACAMA (Central)	380.0	A	7.1	9.00	SS
CLAMSHELL-SAWPIT	380.0	B	6.5	0.50	DS
PISGAH-BULLION MTN.-MESQUITE LK	381.5	B	7.1	0.60	SS
NORTH FRONTAL FAULT ZONE (West)	389.3	B	7.0	1.00	DS
SAN ANDREAS - Southern	390.8	A	7.4	24.00	SS
RAYMOND	391.7	B	6.5	0.50	DS
HOLLYWOOD	391.8	B	6.5	1.00	DS
CLEGHORN	392.0	B	6.5	3.00	SS
CUCAMONGA	392.8	A	7.0	5.00	DS
SANTA MONICA	395.1	B	6.6	1.00	DS
MALIBU COAST	395.2	B	6.7	0.30	DS
JOHNSON VALLEY (Northern)	395.2	B	6.7	0.60	SS

SUMMARY OF FAULT PARAMETERS

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ABBREVIATED FAULT NAME	APPROX. DISTANCE (km)	SOURCE TYPE (A, B, C)	MAX. MAG. (Mw)	SLIP RATE (mm/yr)	FAULT TYPE (SS, DS, BT)
ANACAPA-DUME	395.6	B	7.3	3.00	DS
SAN JACINTO-SAN BERNARDINO	397.7	B	6.7	12.00	SS
BATTLE CREEK	402.9	B	6.5	0.50	DS
NEWPORT-INGLEWOOD (L.A.Basin)	403.9	B	6.9	1.00	SS
SANTA CRUZ ISLAND	404.0	B	6.8	1.00	DS
SAN JOSE	404.4	B	6.5	0.50	DS
EMERSON So. - COPPER MTN.	405.4	B	6.9	0.60	SS
PALOS VERDES	408.4	B	7.1	3.00	SS
SANTA ROSA ISLAND	409.4	B	6.9	1.00	DS
NORTH FRONTAL FAULT ZONE (East)	412.9	B	6.7	0.50	DS
ELSINORE-WHITTIER	413.7	B	6.8	2.50	SS
CHINO-CENTRAL AVE. (Elsinore)	414.1	B	6.7	1.00	DS
ROUND VALLEY (N. S.F.Bay)	419.2	B	6.8	6.00	SS
MAACAMA (North)	425.8	A	7.1	9.00	SS
SAN JACINTO-SAN JACINTO VALLEY	429.6	B	6.9	12.00	SS
ELSINORE-GLEN IVY	435.9	B	6.8	5.00	SS
PINTO MOUNTAIN	443.6	B	7.0	2.50	SS
BURNT MTN.	450.7	B	6.5	0.60	SS
EUREKA PEAK	450.9	B	6.5	0.60	SS
NEWPORT-INGLEWOOD (Offshore)	458.5	B	6.9	1.50	SS
ELSINORE-TEMECULA	465.7	B	6.8	5.00	SS
SAN JACINTO-ANZA	468.7	A	7.2	12.00	SS
LAKE MOUNTAIN	472.7	B	6.7	6.00	SS
CORONADO BANK	493.5	B	7.4	3.00	SS
GARBERVILLE-BRICELAND	494.2	B	6.9	9.00	SS
ELSINORE-JULIAN	503.2	A	7.1	5.00	SS
SAN JACINTO-COYOTE CREEK	512.0	B	6.8	4.00	SS
ROSE CANYON	518.5	B	6.9	1.50	SS
EARTHQUAKE VALLEY	537.4	B	6.5	2.00	SS
MAD RIVER	542.7	B	7.1	0.70	DS
LITTLE SALMON (Onshore)	547.6	A	7.0	5.00	DS
SAN JACINTO - BORREGO	549.8	B	6.6	4.00	SS
TRINIDAD	552.4	B	7.3	2.50	DS
McKINLEYVILLE	552.4	B	7.0	0.60	DS
MENDOCINO FAULT ZONE	554.1	A	7.4	35.00	DS
FICKLE HILL	554.4	B	6.9	0.60	DS
BRAWLEY SEISMIC ZONE	556.1	B	6.5	25.00	SS
ELSINORE-COYOTE MOUNTAIN	567.2	B	6.8	4.00	SS
TABLE BLUFF	568.4	B	7.0	0.60	DS
ELMORE RANCH	569.5	B	6.6	1.00	SS
CASCADIA SUBDUCTION ZONE	572.0	A	8.3	35.00	DS
LITTLE SALMON (Offshore)	579.9	B	7.1	1.00	DS
SUPERSTITION MTN. (San Jacinto)	581.2	B	6.6	5.00	SS
SUPERSTITION HILLS (San Jacinto)	582.7	B	6.6	4.00	SS
BIG LAGOON - BALD MTN.FLT.ZONE	587.6	B	7.3	0.50	DS
IMPERIAL	603.4	A	7.0	20.00	SS

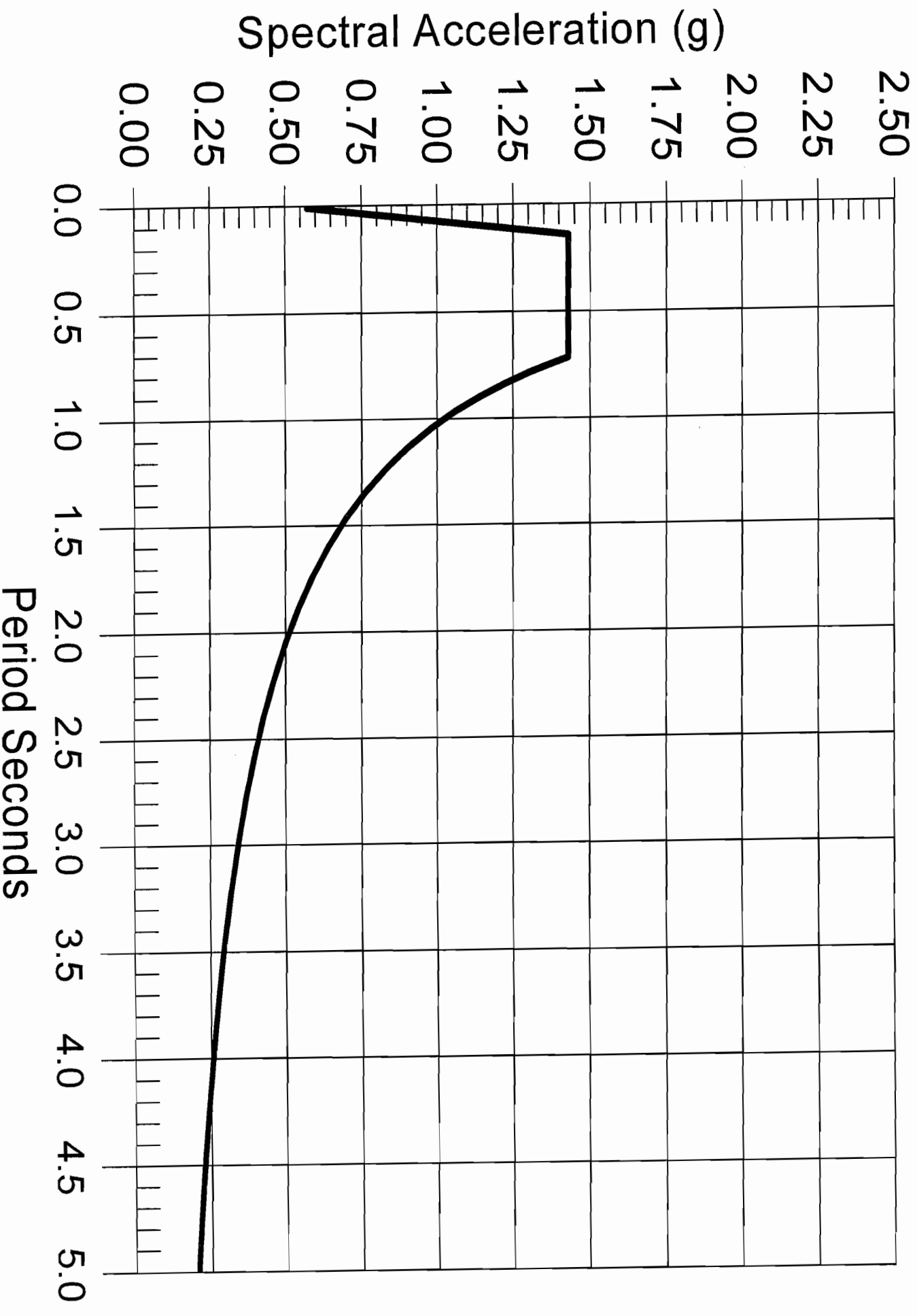
SUMMARY OF FAULT PARAMETERS

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ABBREVIATED FAULT NAME	APPROX. DISTANCE (km)	SOURCE TYPE (A,B,C)	MAX. MAG. (Mw)	SLIP RATE (mm/yr)	FAULT TYPE (SS,DS,BT)
=====	=====	=====	=====	=====	=====
ELSINORE-LAGUNA SALADA	609.3	B	7.0	3.50	SS

DESIGN RESPONSE SPECTRUM

Seismic Zone: 0.4 Soil Profile: SD



APPENDIX D
EARTHWORK
AND
GRADING RECOMMENDATIONS

EARTHWORK AND GRADING

These earthwork and grading specifications are for the grading and earthwork shown on the approved grading or construction plan(s) and/or indicated in the geotechnical report(s). Earthwork and grading should be conducted in accordance with applicable grading ordinances, the current California Building Code, and the recommendations of this report. The following recommendations are provided regarding specific aspects of the proposed earthwork construction. These recommendations should be considered subject to revision based on field conditions observed by the geotechnical consultant during grading.

Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record. The Geotechnical Consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of grading or construction.

During grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground, after it has been cleared for receiving fill but before it has been placed, bottoms of all "remedial removal areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the contractor on a routine and frequent basis.

The Earthwork Contractor

The Earthwork Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications. The Earthwork Contractor shall review and accept the plans, geotechnical report(s) and these Specifications prior to the commencement of grading. The Earthwork Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant unsatisfactory conditions, such as unstable soil, improper moisture condition, inadequate compaction, adverse weather, etc... are resulting in a quality of work less than required in these Specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

Site Preparation

General: Site preparation includes removal of deleterious materials, unsuitable materials, and existing improvements from areas where new improvements or new fills are planned. Deleterious materials, which include vegetation, trash, and debris, should be removed from the site and legally disposed of off-site. Unsuitable materials include loose or disturbed soils, undocumented fills, contaminated soils, or other unsuitable materials. The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1-percent of organic materials (by volume). Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant etc...) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fine and/or imprisonment and shall not be allowed.

Any existing subsurface utilities that are to be abandoned should be removed and the trenches backfilled and compacted. If necessary, abandoned pipelines may be filled with grout or slurry cement as recommended by, and under the observation of, the Geotechnical Consultant.

Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured, or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

Fill Compaction and Compaction

The onsite soils are suitable for placement as compacted fill provided the organics, oversized rock (greater than 6-inches in diameter) and deleterious materials are removed. Rocks greater than 6-inches and less than 2-feet in diameter can be placed in the bottom of deeper fills or approved areas provided they are selectively placed in such a manner that no large voids are created. All rocks shall be placed a minimum of 4-feet below finish grade elevation unless used for landscaping purposes. Any import soils shall be tested for suitability in advance by the project Geotechnical Engineer.

After making the recommended removals prior to fill placement, the exposed ground surface should be scarified to a depth of approximately 12-inches, moisture conditioned as necessary, and compacted to at least 90-percent of the maximum dry density obtained using ASTM D1557-2000 as a guideline. Surfaces on which fill is to be placed which are steeper than 5:1 (Horizontal to vertical) should be benched so that the fill placement occurs on relatively level ground.

For the parking areas and other improvements a one-foot removal is recommended depending on site conditions (i.e. depth of root zone, and depth of disturbance which may have locally deeper removal depths). The removal bottom should be observed (tested as needed) by the geotechnical consultant prior to placing fill soils. The upper 12-inches of subgrade material along with the Class II Aggregate Base and the Asphaltic concrete shall be compacted to a minimum of 95-percent of the materials maximum dry density as determined by ASTM D1557-2000. The subgrade and aggregate base shall be moisture-conditioned and compacted to 95-percent of the material's maximum dry density as determined by ASTM D-1557-2000 to a depth of 12-inches.

All fill and backfill to be placed in association with the proposed construction should be accomplished slightly over optimum moisture content using equipment that is capable of producing a uniformly compacted product throughout the entire fill lift. Fill materials at less than optimum moisture should have water added and the fill mixed to result in material that is uniformly above optimum moisture content. Fill materials that are too wet can be aerated by blading or other satisfactory methods until the moisture content is as required. The wet soils may be mixed with drier materials in order to achieve acceptable moisture content.

The fill and backfill should be placed in horizontal lifts at a thickness appropriate for equipment spreading, mixing, and compacting the material, but generally should not exceed eight inches in thickness.

No fill soils shall be placed during unfavorable weather conditions. When work is interrupted by rains or snow, fill operations shall not be resumed until the field tests by the geotechnical engineer indicate that the moisture content and density of the fill are as previously specified.

Slopes

All slopes shall be compacted in a single continuous operation upon completion of grading by means of sheepsfoot or other suitable equipment, or all loose soils remaining on the slopes shall be trimmed back until a firm compacted surface is exposed. Slope compaction tests shall be made within one foot of slope surface.

Cut and fill slopes shall be a maximum of 2:1 (horizontal to vertical) unless approved by the Geotechnical Consultant.

Planting and irrigation of cut and fill slopes and/or installation of erosion control and drainage devices should be completed due to the erosion potential of the soil.

Temporary Excavations

Temporary excavation shall be made no steeper than 1:1 (horizontal to vertical). The recommended slope for temporary excavations does not preclude local raveling and sloughing. Where wet soils are exposed, flatter excavation of slopes and dewatering may be necessary. In areas of insufficient space for slope cuts, or where soils with little or no binder are encountered, shoring shall be used.

All large rocks exposed above temporary cuts shall be removed prior to foundation excavation. In addition any rocks exposed during development from raveling and sloughing should be removed immediately.

All excavations should comply with the requirements of the California Construction and General Industry Safety Orders and the Occupational Safety and Health Act and other public agencies having jurisdiction.

Trench Backfill

Exterior trenches, paralleling a footing and extending below a 1:1 plane projected from the outside bottom edge of the footing, shall be compacted to a minimum of 95-percent per ASTM D1557-2000. All trenches in structural areas and under concrete flatwork shall be compacted to a minimum of 95-percent per ASTM D1557. All trenches in non-structural areas shall be compacted to a minimum of 85-percent per ASTM D1557-2000.

All material used for trench backfill shall be approved by the Geotechnical Engineer prior to placement. All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 ($SE > 30$). The bedding shall be placed to 1-foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 95-percent of maximum from 1-foot above the top of the conduit to the surface.

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

Regulations of the governing agency may supersede the above, and all trench excavations should conform to all applicable safety codes. The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations.

19 January 2007

Ms. Terry McCracken
Christopher A. Joseph & Associates
179 H Street
Petaluma, California 94952

Subject: Geotechnical/Geological Consultation
Environmental Impact Report
Snowcreek 8 Project
Mammoth Lakes, California

Dear Ms. McCracken:

This letter presents Treadwell & Rollo's (T&R's) third party geotechnical and geological review comments for the proposed Snowcreek 8 project in Mammoth Lakes, California. The proposed project site is northwest and south of Old Mammoth Road and east of the existing Snowcreek Golf Course.

PROJECT DESCRIPTION

The proposed project will consist of a planned community that integrates residential, resort, recreation, retail, and public amenities components. The project will also enhance some existing components, such as the expansion of the adjacent Snowcreek Golf Course.

The residential component will include single family dwellings, stacked flats and townhomes that will range from 650 square feet to 3,500 square feet with densities ranging from 8 to 35 units per acre. In addition, a residence club/snack bar with a pool, spa, and grill will be incorporated into the project.

The resort component will included 400 guest suites. The resort may take the form of a conventional hotel, condominium hotel, fractional ownership, or a combination thereof. The resort will include retail space, lounge, fitness area, pool, spa/wellness center, and ice skating pond.

The proposed Snowcreek plan will include a new golf clubhouse, expanding the existing golf course, attendant facilities, driving range, Outfitters Cabin, and the Snowcreek Athletic Club. An additional nine holes will be constructed on the east and south edges of the proposed Snowcreek site. The course will be designed to conserve water and better utilize natural vegetation. The existing nine holes may be modified where appropriate. The existing temporary clubhouse will be removed. Re-grading and contouring of the new portion of the golf course, and possibly portions of the existing course, will create topographic undulations in character with the surrounding landforms. Water will be routed throughout the course and fed into new ponds which will be used to store irrigation water and provide drainage retention.

The project will also include an additional retail area with a general store. Public amenities will include an interpretive center, an Outfitters Cabin, cooperation with MCWD's expansion,

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Christopher A. Joseph & Associates
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improvement and distribution of their tertiary water treatment, and the provision of hotel rooms, restaurants, retail, and conference facilities. Irrigation needs for the Snowcreek and Starwood Golf Courses can be provided by the tertiary water system resulting in a surplus of 400 acre-feet of potable water that can be diverted from use in irrigation to instead be used to meet other future needs for the Town of Mammoth Lakes.

SCOPE OF SERVICES

Our scope of services consisted of:

- reviewing available geotechnical and geologic information submitted by the project applicant;
- compiling and reviewing readily available published and unpublished geologic and seismicity data for the site vicinity; and
- preparing a written letter describing the results of our study, including a discussion of potential geotechnical and geological concerns, such as local seismicity, strong shaking from nearby earthquakes, soil liquefaction, and ground subsidence.

During the preparation of this letter, T&R reviewed the following documents:

- *Preliminary Geotechnical Investigation, Snowcreek 8 Development, Mammoth Lakes, California*, prepared by Sierra Geotechnical Services Inc. (SGSI), dated 4 October 2006; and
- *Phase I Environmental Site Assessment – Update with Limited Soil Sampling and Analysis, The Snowcreek Parcels: Site 1. Snowcreek Golf Course, Parcel 2 of Lot Line Adjustment 03-06, Tax Assessor's Parcel No. 40-070-23; Site 2. Snowcreek Maintenance Building, Hilltop and Storage Yard Areas, Lot 2 of Tentative Tract 36-205, Portion of Parcel Map 36-133, Tax Assessor's Parcel No. 40-040-20; Site 3. Proposed Snowcreek Golf Course Development Expansion Areas, Lots 3 thru 6 of Tract 36-166, Tax Assessor's Parcel Nos 40-070-10 thru 13; and Site 4. USDA Forest Service – Snowcreek Land Exchange Area, Tax Assessor's Parcel Nos. 40-070-19 & 21*, prepared by Sierra Geotechnical Services Inc. (SGSI), dated February 2005.

Our scope of services did not include a site visit, geologic reconnaissance or mapping, review of historical aerial photographs, drilling borings, laboratory testing, or performing any type of subsurface exploration for this study.

REGIONAL AND SITE GEOLOGY

The site is located at the southwestern edge of the Long Valley caldera, near the eastern flank of the Sierra Nevada. The caldera is an east-west elongated, oval depression formed approximately 760,000 years ago. The caldera experiences continued volcanic activity. The pre-volcanic basement rock in the Mammoth Lakes area is predominately Mesozoic granite rocks of the Sierra Nevada batholith. The batholith is a series of intrusions that displaced overlying ancient sedimentary sea floor rocks during the Jurassic and Cretaceous Periods. Episodic glaciation, as well as more recent volcanic eruptions occurred throughout the Pleistocene leaving a mantle of glacial till and pyroclastic deposits covering the older basement and volcanic rocks throughout the area now occupied by the Town of Mammoth Lakes.

SUBSURFACE CONDITIONS

SGSI performed 42 exploratory test pits, designated as TP-1 through TP-42, at the site and encountered the following:

Undocumented Fill

Up to five feet of undocumented fill was encountered in test pits TP-27, TP-28, TP-29, and TP-42. The undocumented fill generally consisted of loose to dense, moist, silty to clayey sand with a few cobbles and boulders with maximum dimensions of 36 inches.

Topsoil/Alluvium

A 1-1/2- to greater than 10-foot-thick layer of topsoil/alluvium was encountered within all test pits. In general, the topsoil/alluvium layer consists of loose to medium dense, moist sand, silty sand, and clayey sand with cobble, boulders, and a moderate amount of roots. It is overlain by undocumented fill at the test pit locations specified above and is underlain by glacial deposits, except at TP-7, TP-9, TP-10, TP-12 through TP-18, TP-20 through TP-23, TP-26 through TP-28, TP-32, and TP-38, where the full thickness of the topsoil/alluvium layer was not measured during the SGSI preliminary investigation.

Glacial Till Deposits

Glacial till deposits were encountered below the alluvium. The glacial till generally consists of medium dense to dense, moist to saturated, sand and silty sand, with gravels, cobbles, and boulders.

Groundwater

Groundwater was encountered during the SGSI investigation performed in August 2006 at the following locations: TP-6 at a depth of approximately 8 feet, TP-9 at a depth of approximately 7 feet, TP-10 at a depth of approximately 2 feet, TP-13 at a depth of approximately 3-1/2 feet,

TP-15 at a depth of approximately 8-1/2 feet, TP-18 at a depth of approximately 2-1/2 feet, TP-28 at a depth of approximately 8 feet, and TP-35 at a depth of approximately 2 feet. Mottling of on-site soil was identified by SGSI, which is indicative of perched groundwater in the vicinity.

REGIONAL SEISMICITY

A list of major active faults in the region, including the distance from the site and estimated maximum Moment magnitude are summarized on Table 1.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Maximum Magnitude
Hartley Springs	1.1	West	6.6
Hilton Creek	10	East	6.7
Round Valley	21	East	7.0
Mono Lake	36	North	6.6
Fish Slough	50	East	6.6
White Mountains	52	East	7.1
Robinson Creek	71	Northwest	6.4
Owens Valley	71	Southeast	7.6
Death Valley (N. of Cucamonga)	72	East	7.0
Birch Creek	77	Southeast	6.4
Deep Springs	92	East	6.6

SGSI performed deterministic and probabilistic seismic hazard analyses. SGSI's analyses indicate a magnitude 6.6 (Mw) earthquake occurring on the Hartley Springs faults located approximately 1.1 kilometers from the site could produce a deterministic peak ground acceleration (PGA) of approximately 0.46 times gravitation acceleration (0.46g).

Ms. Terry McCracken
Christopher A. Joseph & Associates
19 January 2007
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Probabilistic analyses were performed for two levels of shaking. An Upper-Bound Earthquake with a 10 percent chance of exceedance in 100 years could produce a PGA of about 0.44g. The Design Basis Earthquake with a 10 percent chance of exceedance in 50 years could produce a PGA of about 0.35g.

GEOLOGIC AND SEISMIC HAZARDS

Potential geologic and seismic hazards at the project site include strong ground shaking, fault rupture, soil liquefaction, and avalanches. These hazards are discussed in the following sections.

Strong Ground Shaking

SGSI indicates that due to the proximity of the site to the Hartley Springs fault (Type B fault), the site could be subjected to very strong ground shaking. Therefore, the proposed structures should be designed to withstand the effect of the anticipated strong ground shaking. For seismic design in accordance with the 2001 California Building Code (CBC), SGSI recommends the following parameters:

- Seismic Zone Factor 4
- Soil Profile Type S_c
- Near Source Factors N_a and N_v of 1.3 and 1.6, respectively
- Seismic Coefficients C_a and C_v of 0.57 and 1.02, respectively.

T&R concurs with SGSI's assessment of the potential for very strong shaking at the subject site. Also, T&R concurs with SGSI's preliminary recommendations for 2001 CBC seismic design parameters.

Fault Rupture

SGSI indicates there are no known active, potentially active, or inactive faults that transect the subject site. Therefore, the potential for fault rupture is very low.

T&R concurs with SGSI's assessment that the risk of ground rupture is low.

Seismic Hazards

During a major earthquake on one of the active or potentially active nearby faults, strong to very strong ground shaking is expected to occur at the project site. Strong shaking can result in ground failures, such as those associated with soil liquefaction¹, lateral spreading², post-liquefaction reconsolidation³, and cyclic soil densification⁴.

Soil Liquefaction

The SGSI report generally indicates that: 1) up to five feet of loose to medium dense undocumented fill is present at a few locations, 2) topsoil/alluvial deposits consisting of loose sand and silty sand blanket the site between the depths of approximately 1-1/2 to 10 feet, and 3) perched water may develop at the site. Based on these site conditions, the SGSI reports indicate the potential for liquefaction to occur is considered very low due to the lack of groundwater and the presence of medium dense to dense nature bearing soil at the site.

In general, where the loose to medium dense fill, topsoil and/or alluvium are less than approximately 3 to 4 feet thick and these “unsuitable” bearing materials will be excavated and replaced with well-compacted engineered fill, T&R concurs with SGSI’s conclusion that the potential for soil liquefaction is low. However, in areas where loose to medium dense fill, topsoil and/or alluvium are greater than approximately 3 to 4 feet thick, and the loose to medium dense soil is left in place, T&R concludes water may become perched beneath the proposed building sites and the potential for soil liquefaction may exist. Ground failures associated with soil liquefaction include post-liquefaction reconsolidation, lateral spreading, and loss of bearing support.

T&R believes that further subsurface investigation should be performed by SGSI to evaluate the thickness, in place density, and fines content of the underlying loose to medium soil at the proposed Snowcreek 8 site. This additional information can be used to evaluate the liquefaction

¹ Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

² Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

³ Post-liquefaction reconsolidation is a phenomenon in which a previously liquefied sand layer settles into a denser soil arrangement after dissipation of pore water pressures.

⁴ Cyclic soil densification is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, resulting in ground surface settlement.

potential of loose to medium dense granular soil beneath the planned structures and new site improvements.

Cyclic Densification

Cyclic densification should be considered a potential minor hazard at the project site. During a major earthquake on a nearby portion of one of the active faults, strong ground shaking may occur and cause the loose, unsaturated portion of the topsoil/alluvial deposit to densify and settle. T&R preliminarily estimates that upper to 1/2 inch of cyclic densification may occur at the site. This phenomenon may result in minor cracking foundations and surface improvements.

Seiches and Tsunamis

The potential for seiches and tsunamis are considered nil because there are no large bodies of water in close proximity to the site. T&R concurs with SGSI's conclusion.

Avalanches (Rockfall and Snow) and Landslides

SGSI concludes the potential for rockfall or snow avalanches to occur is low because the site is not adjacent to the base of a steep slope or within close proximity to an area of avalanche flow.

T&R concurs with SGSI's conclusion.

Volcanic Hazards

The SGSI report indicates that eastern California, including Long Valley Caldera and the Mono-Inyo Craters volcanic chain, has a long history of geologic activity that includes earthquakes and volcanic eruptions. SGSI's research indicates that massive eruptions are extremely rare and currently there is no evidence leading to the conclusion that a massive eruption is eminent. SGSI concludes that small to moderate volcanic eruptions could occur resulting in pyroclastic flows and surges, as well as volcanic ash and pumice fallout, which could impact the site.

T&R concurs with SGSI's conclusions regarding the potential risk of volcanic hazards. T&R adds that U.S. Geological Survey (USGS) estimates that eruptions at the Mono-Inyo Craters volcanic field occurred at about 500-year intervals over the past 2,000 to 3,000 years. The most recent eruption in the region was at Mono Lake between 1720 and 1850. A dome grew on the lake floor and emerged to make Paoha Island. Also in 1980, four magnitude 6 events were recorded in a 2-day period. Volcanologists interpreted the earthquakes, accompanying ground deformations, and an increase in activity at fumaroles, as an indication of magma movement beneath the caldera. In 1994, geologists investigated an area of 75 acres that contained dying forest. They studied the gas in the soil and found carbon dioxide concentrations of 30 to 96

percent. The areas of *tree kills* are generally on or adjacent to Mammoth Mountain. The USGS indicates that when carbon dioxide gas can accumulate in snowbanks, depressions, and poorly ventilated enclosures, including structures, posing a potential danger to people. The USGS scientists closely monitor the volcanic activity in the region in order to provide the public with reliable and timely warning of volcanic unrest in the Long Valley area.

RECOMMENDATIONS BY SIERRA GEOTECHNICAL SERVICES INC.

Geotechnical Review

SGSI recommends that they should be provided the opportunity to review the grading and foundation plans prior to construction in order to assure that the work is being performed in conformance with their report or SGSI will waive all liability for any and all geotechnical issues associated with grading or construction

Earthwork

SGSI recommends the earthwork be performed in accordance with the General Earthwork and Grading Specifications in Appendix D and the recommendations presented in the text of the 4 October 2006 SGSI report.

Site Preparation and Excavation

In the SGSI report, SGSI recommends removing deleterious materials, unsuitable materials, and existing improvements from areas where new improvements or new fills are planned. Unsuitable materials include loose or disturbed soils, undocumented fills, and contaminated soils. The Geotechnical Consultant should evaluate the extent of these removals depending on specific site conditions. Earth fill material should not contain more than one percent of organic material by volume. Nesting of organic materials is not allowed.

Any existing subsurface utilities that are to be abandoned should be removed and the trenches should be backfilled with compacted fill. If necessary, abandoned pipelines may be filled with grout or slurry cement as recommended by and under the observation of the Geotechnical Consultant.

Site excavations including over-excavation and removal of unsuitable undocumented fill and alluvial soil should be evaluated and approved by Geotechnical Consultant. The SGSI report indicates the upper three feet of loose undocumented fill and upper 2 to 3 feet of loose, topsoil/alluvium are considered unsuitable. Planned excavations should extend below the unsuitable material and to a minimum horizontal distance of one-half the footing width or five

feet beyond the foundation outline, whichever is greater. For paved roadways, parking areas and other improvements, SGSI recommends the upper 1 to 3 feet of the existing subgrade should be excavated and removed from the site depending on site conditions. The excavation for paved areas should extend a minimum horizontal distance of two feet beyond the back of curbs and pavements. All areas to receive fill should be observed, mapped, elevations recorded, and tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill.

Fill Placement and Compaction

SGSI recommends that after completing the excavation and removal of unsuitable soil, the exposed surface should be scarified to a depth of 12 inches, moisture-conditioned, as necessary, and compacted to at least 90 percent of the maximum dry density obtained using ASTM D1557-2000 procedure. Fill placed on surfaces steeper than 5:1 (horizontal to vertical) should be benched so that the fill placement occurs on relatively level ground.

The onsite soils are suitable for reuse as compacted fill provided the organics, oversized rock (greater than 6-inches in diameter) and deleterious materials are removed. SGSI indicates that rocks greater than 6-inches and less than 2-feet in diameter can be placed in the bottom of deeper fills or approved areas provided they are selectively placed in such a manner that no large voids are created. All rocks should be placed a minimum of four feet below finished grade selection unless used for landscaping purposes. Any imported soils should be tested for suitability in advance by the project Geotechnical Engineer.

For paved roadways, parking areas, and other improvements, SGSI recommends the upper 12 inches of subgrade material along with the Class 2 aggregate base and asphaltic concrete should be compacted to a minimum of 95 percent of the material's maximum dry density as determined by ASTM D1557-2000. SGSI indicates new fill and backfill should be moisture-conditioned slightly above optimum moisture content, placed in horizontal lifts of appropriate thickness for the equipment used, but generally should not exceed eight inches in thickness, and compacted.

SGSI indicates that new fill should not be placed during unfavorable weather conditions. When work is interrupted by rains or snow, fill operations should not resumed until the field tests by the geotechnical engineer indicate that the moisture content and density of the fill are as previously specified.

T&R generally concurs with SGSI's recommendations, with the exception that fill consisting of clean sand (less than five percent fines by weight) should be compacted to at least 95 percent of the maximum dry density of the soil, and rock fill with dimensions greater than six inches should not be placed beneath site improvements and structures that are sensitive to ground settlement primarily due to difficulties associated with properly compacting fill containing large aggregate.

Foundations

SGSI preliminarily recommends the proposed buildings be supported on spread footings that gain support in either: 1) properly compacted fill, 2) competent alluvial deposits encountered below a depth of approximately three feet, or 3) competent glacial deposits. SGSI recommends the following allowable soil bearing pressures:

- For footings bottomed 0 to 3 feet below the existing ground surface, use an allowable bearing pressure of 2,500 pounds per square foot (psf);
- For footings bottomed 3 to 10 feet below the existing ground surface, use an allowable bearing pressure of 3,000 psf; and
- For footings bottomed greater than 10 feet below the existing ground surface, use an allowable bearing pressure of 4,000 psf.

The SGSI report indicates the allowable bearing pressures may be increased by one-third when considering loads of short duration, such as wind or seismic forces. The factors of safety associated with the allowable values and the values that were increased by one-third were not stated by SGSI.

The SGSI report indicates that post construction settlement of the proposed building foundations is estimated to be less than or equal to 1/2 inch.

A friction coefficient for concrete of 0.35 may be used to resist lateral loads.

Continuous and isolated footings should be designed in accordance with the structural engineer's requirements. However, exterior foundations should have a minimum embedment depth of 18 inches below the outside adjacent grade. Also, SGSI recommends that the footings should be reinforced to at least the minimum reinforcement for temperature, as required by Chapter 19 of the 1997 UBC. If earthwork will consist of foundation excavations only, then all footings should be embedded at least 12-inches below the estimated removal depth of three feet.

SGSI recommends that footing trench excavations should be well-moistened prior to pouring concrete. Also, the footing trenches should not have any rocks or boulders protruding into the trench bottom. Soft soil pockets created by rock removal during foundation excavation shall be replaced with approved fill material, and compacted to 95 percent of the material's maximum dry density.

SGSI recommends a minimum 5-foot horizontal setback distance from the face of slopes for all structural footings and settlement-sensitive structures. The distance should be measured from the outside edge of the footing, horizontally to the slope face or to the face of a retaining wall.

SGSI recommends that utility trenches that are parallel to new footings should not encroach within a 1:1 plane extending downwards and outward from the outside edge of the footing.

T&R understands that the proposed foundation design criteria are preliminary in nature; however, T&R takes exception to some of SGSI's recommendations and has the following comments.

1. Within the hotel and low to high density housing areas, the SGSI investigation encountered up to seven feet of fill and topsoil/alluvium to depths ranging from 1-1/2 feet to greater than 10 feet below the ground surface. In August 2006, the groundwater was encountered at depths ranging between 2-1/2 to 8-1/2 feet below the ground surface. The sandy fill and topsoil/alluvium materials are generally characterized as loose to medium dense. In T&R's opinion, during the late spring or early summer, the local groundwater level is likely to rise and the lower portions of the loose to medium dense sandy fill and topsoil/alluvium layers may become saturated. Strong ground shaking associated with a large earthquake on a nearby fault could trigger soil liquefaction and associated ground failures. For this reason, T&R recommends supplemental investigation and analyses be performed by SGSI to evaluate the liquefaction potential of the soil beneath the planned residences and hotel structure.

2. SGSI's preliminary foundation recommendations consist of either: 1) over-excavating and removing "unsuitable" material from beneath the planned footing locations (if necessary), placing and compacting engineered fill, and supporting the proposed building foundations on either a layer of properly compacted fill or competent alluvium and glacial till, or 2) supporting the new structures on deepened footings that extend at least 12 inches below the estimated depth of "unsuitable" material, which corresponds to a bottom of footing depth of about four feet below the existing ground surface. In T&R's opinion, the depth to the bottom of "unsuitable" material is not clearly defined and the amount of settlement associated with supporting conventional shallow spread footings on medium dense granular soil, especially if the soil is found to be potentially liquefiable, is not fully assessed. Therefore, T&R suggests that SGSI should better define the depth and/or the properties of the foundation bearing material that will be used for support of the proposed building foundations.

Concrete Slabs-on-Grade

SGSI preliminarily recommends that interior slabs-on-grade be underlain by a water vapor retarder system consisting of a one-inch-thick layer of sand overlain by water vapor retarder

membrane that is at least 10 mils thick, which in turn is covered by a one-inch-thick layer of sand. SGSI recommends using minimum 4-inch-thick slab reinforced with #3 rebar placed at 18-inches on center each way. SGSI recommends using a low slump concrete (not exceeding 4-inches at the time of placement) and proper curing methods to reduce the potential for shrinkage cracking.

T&R suggests that SGSI consider using a capillary break consisting of at least a 4-inch-thick layer of drain rock or crushed rock (1/2- to 3/4-inch gradation) instead of a one-inch-thick layer of sand and using a vapor retarder membrane that meets the requirements for Class C vapor retarders as stated in ASTM E1745-97. Also, the vapor retarders should be placed in accordance with the requirements of ASTM E1643-98.

Pavement Design

SGSI preliminarily recommends using a pavement section consisting of three inches of asphalt concrete over four inches of Class 2 aggregate base. Where heavier loading is anticipated, SGSI recommends using a pavement section consisting of four inches of asphalt concrete over six inches of Class 2 aggregate base. The upper 12 inches of the pavement subgrade, and the asphalt concrete and aggregate base material should be compacted to at least 95 percent of the material's maximum dry density.

SGSI indicates that environmental conditions such as freeze-thaw and thermal cracking will most likely govern the life of the asphalt pavement. Therefore, SGSI recommends at least a 3-inch-thick asphalt concrete pavement section. Also, SGSI recommends using moisture control measures, such as deepened curbs or other moisture barrier materials, to reduce the potential for pavement subgrade from becoming saturated.

T&R recommends that project applicant team should define the traffic indices that will be used for the design of new pavements at the development site. Subsequently, supplemental sampling and resistance value (R-value) testing should be performed on the pavement subgrade for use in checking SGSI's flexible pavement design.

Lateral Earth Pressures and Resistance

SGSI recommends that retaining walls that are free to rotate be designed to resist lateral pressures resulting from an active earth pressure of 40 pounds per cubic foot (pcf) for level backfill conditions. Retaining walls that are restrained from rotation should be designed for an at-rest earth pressure of 55 pcf. Also, SGSI recommends using a seismic increment of 1,549 pounds per linear foot of wall applied at a distance of 0.6H above the toe of the wall for free standing or cantilevered walls or as a uniform pressure (rectangular distribution) for restrained walls such as basement walls.

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The SGSI report indicates lateral resistance can be developed using a passive earth pressure of 355 pcf and a friction coefficient of 0.35.

In T&R's opinion, the lateral pressure resulting from an active and at-rest earth pressure of 40 and 60 pcf for level backfill conditions are typical and reasonable. This design value assumes that the retaining walls are properly backdrained. Lateral resistance values in granular soils are contingent upon overburden pressures. Therefore, T&R suggests that either support from the upper one foot of soil should be ignored when computing lateral soil resistance or the soil adjacent to the footing should be confined by concrete slabs or pavements.

Also, T&R recommends that the seismic increment be applied as a function of the wall height. For example, a seismic increment of 15 times H (15H) can be used, where H represents the height of the wall in feet.

Surface Drainage Control

SGSI recommends that the areas adjacent to buildings be sloped to provide positive surface drainage away from the buildings. Slope gradients should be a minimum of two percent and extend at least five feet beyond the outline of the buildings. Ponding of water should not be permitted. Erosion of the soil adjacent to structures is possible if the soil subgrade is left unprotected during the snowmelt run-off season.

T&R concurs with SGSI's recommendations for surface drainage control.

CONCLUSIONS AND COMMENTS

Based on the review of the project documents, T&R concludes that the proposed project is feasible, but potentially constrained by: 1) strong ground shaking, 2) potentially liquefiable soil, 3) potential volcanic hazards, and 4) the presence of unsuitable near surface soil that is loose and contains organic matter. T&R reviewed SGSI's geotechnical recommendations and concur with a majority of the recommendations. Remaining issues that should be addressed or commented upon by the project applicant or applicant's consultants are summarized as follows:

Comment No. 1

T&R suggests the applicant develop an emergency evaluation plan in case the potential for volcanic hazards increases and residents need to vacate the property.

Comment No. 2

In T&R's opinion, the potential for soil liquefaction has not been fully evaluated for the soil beneath the proposed building sites. T&R recommends that supplemental subsurface investigation and analyses be performed to better define the thickness, gradation, in-place density, and liquefaction potential of the underlying granular soil.

Also, T&R generally concurs with SGSI's fill placement and compaction recommendations. However, in T&R's opinion, fill consisting of clean sand (less than five percent fines by weight) should be compacted wet of the optimum moisture content and to at least 95 percent of the maximum dry density of the fill. T&R suggests that rock and cobble fill with dimensions greater than six inches should not be placed beneath site improvements and structures that are sensitive to ground settlement primarily due to difficulties associated with properly compacting fill containing large aggregate.

Comment No. 3

T&R suggests that SGSI indicate the factors of safety, if any, that are included in their allowable foundation bearing capacity recommendations.

Comment No. 4

T&R suggests using a 4-inch-thick layer of "drain rock" or "poorly graded crushed rock" as a capillary moisture break beneath the concrete slab-on-grade floors. Also, T&R suggests SGSI consider using the requirements and specifications provided in ASTM E1745-97 and ASTM E1643-98 for vapor retarders.

Comment No. 5

T&R suggests that SGSI should modify the retaining wall design criteria. Specifically, T&R suggests that either the upper one foot of soil adjacent to the embedded footings be ignored for purposes of computing passive soil resistance or it be confined by a concrete slab or pavement. In addition, T&R suggests that SGSI should consider including a seismic increment that is variable based on the retaining wall height, such as using a uniform pressure distribution of "15H" where "H" is the height of wall in feet.

In conclusion, T&R recommends the project applicant and/or applicant's consultant consider the comments presented above and provide a response or acknowledgement that the comments presented above will be addressed during the final design of the project. T&R appreciates the

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opportunity to assist you with the evaluation of geotechnical and geological issues for this project. If you have any questions or require additional information, please call.

Sincerely yours,
TREADWELL & ROLLO, INC.

Dean H. Iwasa
Geotechnical Engineer

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