Appendix G Geotechnical Report

MEMORANDUM

To: Alison E. Rondone, Project Manager, PBS&J

From: George J. Burwasser, California Professional Geologist 7151, PBS&J

Date: Monday 04 October 2010

Project: Palm-Industrial Warehouse Project EIR - PBS&J Project Number D21331.00

Re: Update of Baseline Data for Geology, Soils, and Mineral Resources: Palm Avenue and Industrial Avenue, San Bernardino, California

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Introduction

This memorandum brings Chapter 4.5 Geology and Soils/Mineral Resources of the Screencheck Environmental Impact Report (EIR) submitted in April 2008 up to date.

Documentation

Key Sources Consulted - Web Searches

- The City of San Bernardino (http://www.ci.san-bernardino.ca.us/) webpages for the Planning Division (http://www.ci.san-bernardino.ca.us/depts/community_development/planning/default.asp), Planning Division documents (http://www.ci.san-bernardino.ca.us/depts/community_development/planning/planning_documents.asp), Community Development Agendas (http://www.ci.san-bernardino.ca.us/depts/community_development/agendas/default.asp),and Development Code (http://www.ci.san-bernardino.ca.us/depts/community_development/development_code.asp) were searched for information about updates to the City's General Plan since 2005 related to geology, soils, and mineral resources.
- The California Code of Regulations (http://government.westlaw.com/linkedslice/search/default.asp?tempinfo=find&RS=GVT1.0&VR=2.0&SP=CCR-1000), Governor's Office of Planning and Research "CEQAnet Database" (http://www.ceqanet.ca.gov/), and California Natural Resources Agency "California Environmental Resource Evaluation System" (http://ceres.ca.gov/ceqa/) were searched for information about updates to the California Environmental Quality Act (CEQA) since 2005 related to geology, soils, and mineral resources.

Research Results

Review of the San Bernardino General Plan determined no new policies regarding geology, soils, and mineral resources were adopted that were not previously been considered in the April 2008 submittal of the DEIR.

Review of the CEQA requirements related to geology, soils, and mineral resources determined no changes have occurred that were not previously been considered in the April 2008 submittal of the EIR.

Conclusion

Geology, soils, and mineral resources conditions are essentially the same as they were at the time of the April 2008 submittal of the EIR. No text alterations are necessary at this time.

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PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED 682K DISTRIBUTION BUILDING
SOUTHEAST CORNER PALM AVENUE
AND INDUSTRIAL PARKWAY
SAN BERNARDINO, CALIFORNIA

PROJECT NO. 22448.1
AUGUST 13, 2007

Prepared For:

IDS Real Estate
515 D. Figueroa Street, 16th Floor
Los Angeles, California 90071

Attention: Mr. Steve Methot

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August 13, 2007

IDS Real Estate
515 D. Figueroa Street, 16th Floor
Los Angeles, California 90071

Project No. 22448.1

Attention: Mr. Steve Methot

Subject: Preliminary Geotechnical Investigation, 682k Distribution Building, San Bernardino, California.

LOR Geotechnical Group, Inc., is pleased to present this report summarizing our geotechnical investigation for the proposed 682k distribution building to be located southwest of Interstate 215, southeast of Palm Avenue, and both east and north of Industrial Parkway in the city of San Bernardino, California. This report was based upon a scope of services generally outlined in our proposal letter dated June 14, 2007 and other written and verbal communications with you.

In summary, it is our opinion that the site can be developed from a geotechnical perspective, provided the recommendations presented in the attached report are incorporated into design and construction. The following executive summary reviews some of the important elements of the project. However, this summary should not be solely relied upon.

The subject site is underlain by materials of recent alluvial valley and wash deposits in addition to Pelona Schist metamorphic bedrock. In addition to these materials, wind blown sand (aeolian) deposits were identified during this investigation. Also noted during this investigation, the lower lying areas of the site contained a relatively thin, surficial layer of fill materials. These materials are the result of past and current weed abatement (discing) practices at the site.

It is our opinion that existing fills and the upper portion of the alluvial and wind blown sand deposits will not provide uniform and/or adequate support for the proposed distribution building. Thus, we recommend that the building be founded entirely on an engineered compacted fill mat constructed over competent native soils. Based upon

our field and laboratory data, we anticipate removals on the order of 2 to 5 feet below the existing ground surface will be required within alluvial and/or wind blown sand areas to expose competent soils with a minimum relative compaction of 85 percent (per ASTM D 1557) upon which structural fills can be placed. Removals are not anticipated within areas exposing metamorphic bedrock. However, as the currently proposed building will span across several cut to fill transitions, over-excavation of the cut bedrock portions of the building will be necessary to mitigate the potential for differential settlement. A minimum subexcavation of 24 inches below the base grade of the building footings in bedrock areas is recommended. Removals within proposed parking and drive areas are expected to be on the order of 1 to 2 feet deep. The given removal depths are preliminary. The actual depths of removals should be verified during the grading operation by observation and in-place density testing.

Very low expansive soils, good R-value quality, and a negligible sulfate content were encountered on the site. However, during grading it is anticipated that the on site soils will be mixed and blended. Therefore, at the near completion and completion of site grading, additional foundation and subgrade soils should be tested to verify their expansion potential, sulfate content, and R-value quality.

LOR Geotechnical Group, Inc.

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Table of Contents

Page No.

INTRODUCTION 1

PROJECT CONSIDERATIONS 2

EXISTING SITE CONDITIONS 2

AERIAL PHOTOGRAPH REVIEW 3

SUBSURFACE FIELD INVESTIGATION 4

LABORATORY TESTING PROGRAM 4

GEOLOGIC CONDITIONS 5
- Site Geologic Conditions 6
  - Pelona Schist Bedrock (Mzps) 6
  - Alluvium (Qal) 7
  - Wind Blown Sand (Qs) 7
  - Man Made Fill (Qf) 7
  - Landslide (Qls) 7
- Groundwater Hydrology 8
- Surface Runoff 9
- Mass Movement 9
- Faulting 9
- Historical Seismicity 10
- Secondary Seismic Hazards 11
  - Liquefaction 11
  - Seiches/Tsunamis 12
  - Flooding (Water Storage Facility Failure) 12
  - Seismically-Induced Landsliding: 12
  - Rockfalls 12
  - Seismically-Induced Settlement 12

SOILS AND SEISMIC DESIGN CRITERIA (California Building Code) 12
- CBC Division IV: Earthquake Design Criteria Selection 13
  - Seismic Zone 13
  - Near Source Factor 13
- CBC Division V: Soil Profile 14
- CBC Earthquake Design Summary 15

DRAFT

Table of Contents

Page No.

SLOPE STABILITY ANALYSIS 16

CONCLUSIONS 17
General 17
Seismicity 17
Foundation Support 19

RECOMMENDATIONS 19
General Site Grading 19
Initial Site Preparation 20
Preparation of Fill Areas 21
Preparation of Building Pad Area 21
Engineered Compacted Fill 21
Short-Term Excavations 23
Slope Construction 23
Slope Protection 24
Soil Expansiveness 24
Foundation Design 24
Settlement 25
Building Area Slab-on-Grade Design 26
Exterior Flatwork and Driveway Construction 26
Wall Pressures 26
Preliminary Pavement Design 27
Sulfate Protection 28
Construction Monitoring 29

LIMITATIONS 29

TIME LIMITATIONS 30

CLOSURE 31

REFERENCES 32

AERIAL PHOTOGRAPHS 33

INTRODUCTION

During July and August of 2007, a Preliminary Geotechnical Investigation was performed by LOR Geotechnical Group, Inc., for proposed distribution building to be located southwest of Interstate 215, southeast of Palm Avenue, and both east and north of Industrial Parkway in the city of San Bernardino, California. The property is identified as Assessor's Parcel Number 266-041-62. The purpose of this investigation was to provide a technical evaluation of the geologic setting of the site and to provide an opinion on the feasibility of the proposed development as well as preliminary geotechnical design recommendations for the planned warehouse distribution project. The scope of our services included:

- Review of available pertinent geotechnical literature, reports, maps, and agency information pertinent to the study area;
- Interpretation of stereo aerial photograph pairs of the site and immediate surrounding region dated 1938 through 2005;
- Geologic field reconnaissance mapping to verify the areal distribution of earth units and significance of surficial features as compiled from documents, literature, and reports reviewed;
- A subsurface field investigation to determine the physical soil conditions pertinent to the proposed development;
- A geophysical survey by our sub-consultant, Terra Geosciences, using non-destructive seismic methods to determine the velocity characteristics of the underlying earth materials;
- Laboratory testing of selected soil samples obtained during the field investigation;
- Development of geotechnical recommendations for site grading and foundation design; and
- Preparation of this report summarizing our findings, and providing conclusions and recommendations for site development.

The approximate location of the site is shown on the attached Index Map, Enclosure A-1, within Appendix A.

PROJECT CONSIDERATIONS

To orient our investigation at the site, a Preliminary Grading Plan and Overall Site Plan, both prepared by HPA, Inc., dated June and October 2006, respectively, were provided for our use. A copy of the Preliminary Grading Plan was utilized as a base map for this investigation and is attached to this report as Plate 1.

The proposed development indicated on the plans includes the grading of the site to create one large, planar building pad (682,408 square feet) and the associated parking, driveway, and landscape areas.

Relatively extensive grading will be required to achieve the proposed configuration. As shown on the grading plan, cuts to a maximum depth of approximately 100 feet and fills up to a maximum depth of approximately 40 feet are proposed. In addition, cut and fill slopes are currently proposed at maximum gradients of 2 horizontal to 1 vertical, up to a maximum height of approximately 36 feet.

The future structure is anticipated to be of concrete tilt-up or masonry construction. Moderate foundation loads are anticipated with such a structure.

EXISTING SITE CONDITIONS

The site consists of an irregular shaped parcel of land comprising approximately 43.5 acres of vacant land located southwest of Interstate 215, southeast of Palm Avenue, and both east and north of Industrial Parkway in the city of San Bernardino. This area lies along the margin of the Cajon Wash, which lies approximately 0.8 kilometers (0.5 miles) to the west-southwest of the site, and the alluvial fans of the San Bernardino Mountains, which lie approximately 2.0 kilometers (1.25 miles) to the northeast. Cable Creek lies just beyond Interstate 215 to the northeast of the site. The topography of the site is dominated by two, relatively small, roughly northwest to southeast trending hills, separated by in-filled valleys. The topography rises steeply from the lower lying valley areas of the site up to the hillside areas of the site on the order of 130 feet, with an overall fall to the south-southeast.

The smaller of the onsite hills lies within the northwestern portion of the site while the larger hill comprises nearly half of the site within the east-southeast portion of the site. Both hills have small, vertical cuts along the current roadways. Just northwest of the larger hill, a small valley is present. Just beyond this low lying area, the topography rises steeply atop another hill. However, this hill has been cut down and lies approximately 30 feet higher than the adjacent low lying areas. During our review of historic aerial photographs, it was noted that prior to 1962 this hill was in a natural state and extended northwestwards in the area currently occupied by the Palm Avenue / I-215 on and off ramps. This hill was similar to the large hill on the site in relief.

At the time of our field investigation, the vegetation across the site consisted of a moderately heavy growth of weeds and shrubs along the elevated portions of the site, while the low lying areas of the site were recently disced. However, within the lower lying areas of the site, several dense trees were present, primarily within the southeastern area of the site. A small gully traverse the site from an existing storm drain located north of the small onsite hill. This gully trends southeastward to the large onsite hill which redirects the flow southward. Another small gully is present along north side of the large onsite hill, trending southeast.

Man-made improvements on the site include utilities (light standards and fiber optic lines) along Industrial Parkway and several end dumped piles of trash and concrete debris. A small concrete pad was present in the eastern portion of the site.

Palm Avenue, a partially improved roadway, and a small commercial development consisting of a restaurant and two smaller, unoccupied commercial buildings lie northwest of the site. Interstate 215 lies north and northeast of the site. Industrial Parkway, an improved roadway, lies west and southwest of the site. The far southern portion of the site is bordered by an easement belonging to M.W.D., followed by vacant land.

AERIAL PHOTOGRAPH REVIEW

During the course of this study, an analysis of time sequential stereoscopic aerial photograph pairs of the site and immediate surrounding region, on file at the San Bernardino County Flood Control District, were reviewed. Stereoscopic pairs of the site and immediate surrounding region dating from 1938 through 2005 were

examined. A complete list of the photographs studied is provided in the reference section at the back of this report.

In summary, the site has consisted of vacant land since 1938 with occasional plowing most likely for weed control. The geomorphology suggests that bedrock is present under the hillside portion transitioning to relatively deep alluvial deposits within the valleys. A circular, hummocky depression along the eastern slope face of the southern portion of the largest hill suggest some slope instabilities. No evidence of on-site faulting was noted in the photographs reviewed.

SUBSURFACE FIELD INVESTIGATION

Our subsurface field exploration program was conducted on July 17, and 25, 2007 and consisted of excavating ten exploratory trenches using a tractor-mounted backhoe and drilling seven exploratory borings with a track-mounted CME 55 drill rig equipped with 8-inch diameter hollow stem augers. The trenches were excavated to depths ranging from approximately 8 to 13 feet below the existing ground surface. The borings were drilled to depths ranging from approximately 30.5 to 60 feet. The approximate locations of our exploratory trenches and borings are presented on Plate 1 of this report.

The subsurface conditions encountered in the exploratory trenches were logged by a staff geologist from this firm. In-place density tests were taken in accordance with ASTM D 2922, the Nuclear Density Method. Bulk samples of the encountered materials were obtained and returned to our geotechnical laboratory in sealed containers for further testing and evaluation.

The subsurface conditions encountered in the exploratory borings were logged by a staff geologist from this firm. Relatively undisturbed and bulk samples were obtained at a maximum depth interval of 5 feet and returned our geotechnical laboratory in sealed containers for further testing and evaluation.

A detailed description of the field exploration program and the boring and trench logs are presented in Appendix B.

LABORATORY TESTING PROGRAM

Selected soil samples obtained during the field investigation were subjected to laboratory testing to evaluate their physical and engineering properties. Laboratory testing included moisture content, dry density, laboratory compaction, direct shear, consolidation, sieve analysis, sand equivalent, R-value, and soluble sulfate content. A detailed description of the laboratory testing program and the test results are presented in Appendix C.

GEOLOGIC CONDITIONS

The subject site is located very near the boundary of two major physiographic provinces of California; the Transverse Ranges geomorphic province to the north and the Peninsular Ranges Province to the south. The site is located just south of and very near the lower foothills of the San Bernardino mountains to the north and the San Gabriel mountains to the northwest. These mountains form the eastern portion of the Transverse Ranges geomorphic province. The Transverse Ranges province is composed of a series of numerous mountain ranges that extend from the little San Bernardino and Pinto Mountains in the Joshua Tree National Monument westward across the southern end of California out into the Pacific Ocean west of Ventura. These ranges are transversely oriented to the normal north-south trend of most major prominent mountain ranges across the continent. The current doctrine is that these ranges have been uplifted in reaction to compressional forces created by the " big bend" of the San Andreas in southern California. The San Bernardino Mountains themselves are composed chiefly of igneous plutonic rocks with lesser, but significant amounts of, metamorphic rocks. These mountains are bordered by the San Andreas fault on the south and a series of thrust faults on the north. The San Andreas fault lies to the northeast of the site.

The Peninsular Ranges Geomorphic Province incorporates the vast region extending southward from the foothills of the San Gabriel and San Bernardino Mountains south into the Baja California Peninsula. This region is characterized by a series of northwest trending small mountain ranges. According to a study conducted by the United States Geological Survey (Miller, Matti, and Carson, 2001) in the region of the site, the low lying areas are underlain by units of very young wash deposits of alluvium, consisting of recently transported and deposited, unconsolidated sediments of sand, gravel and

boulder deposits with some young alluvial valley deposits having slightly dissected surfaces consisting of slightly consolidated silt, sand, and pebbles.

The hill areas of the site are composed of metamorphic rock of schist. These units are foliated in fine to thick laminations that dip to the south.

The site lies within a relatively seismically active region of southern California with several active faults in the immediate region. Perhaps the best known of these is the San Andreas fault, which lies just northeast of the site. A large amount of right lateral strike slip motion has been demonstrated along this feature, which dominates the structural grain of the region. This fault offsets and separates the San Bernardino Mountains from the San Gabriel Mountains to the northwest of the site. Another significant fault that lies relatively close to the site is the San Jacinto fault that lies just southwest of the site. This feature extends from the Lytle Creek area of the San Gabriel Mountains southeast all the way into the Imperial Valley near the Mexican border.

The general geologic conditions of the site and the immediate surrounding area as mapped by the U.S.G.S. (Miller, Matti, and Carson, 2001) are shown on the Regional Geologic Map, Enclosure A-2, within Appendix A. A description of onsite geologic units is presented on Enclosure A-3.

Site Geologic Conditions

As noted on the regional geologic map, Enclosure A-2, the lower lying valley areas of the subject site are underlain by materials of recent alluvial valley and wash deposits, while the hillside exposed bedrock units of Pelona Schist metamorphic bedrock. In addition to these materials, wind blown sand (aeolian) deposits and a landslide feature were identified during this investigation. Also noted during this investigation, the lower lying areas of the site contained a relatively thin, surficial layer of fill materials. These materials are the result of past and current weed abatement (discing) practices at the site. These units, as encountered, are described in further detail in the following sections.

Pelona Schist Bedrock (Mzps): The hillsides of the site expose dvarious units of a dark colored metamorphic rock that has been mapped by past geologic studies (Miller, Matti, and Carson, 2001 and Dibblee, 2004) as the Miocene age Pelona Schist. Where encountered, these were noted to be gray to greenish gray, foliated, generally soft to moderately hard at the surface, becoming hard to very hard with depth. These units were typically friable with some areas containing numerous fractures. While metamorphic rocks do not contain true bedding layers, they do tend to develop planar features by parallel alignment of mineral grains, called foliation. When well developed these can have similar weakness as true bedding. At the subject site, the schist was noted to be typically foliated, in many areas strongly developed. The foliation of these units were generally noted to be dipping at relatively gentle angles to the east-southeast on the order of 20 to 30 degrees.

Within our excavation, the bedrock materials became difficult to excavate at a depth of approximately 4 feet, with refusal at approximately 7 feet with the tractor-mounted backhoe. The drill rig experienced moderate difficulty at approximately 7 to 25 feet with increased difficulty at approximately 43 feet and practical refusal at approximately 60 feet. Additional information regarding the rippability of the bedrock is presented in the later sections of this report, as well as within the Seismic Refraction Survey conducted by our subconsultant, Terra Geosciences. Their report is present within Appendix E of this report.

Alluvium (Qal): Within the lower, flatter areas of the site erosion of surrounding mountains has deposited relatively thick amounts of unconsolidated materials of sand and silt. In our trenches and borings these materials were noted to consist primarily of well to poorly graded sand and silty sand. These units were typically brown to light brown in color, dry to damp, and loose near the surface becoming slightly denser with depth. Some caving was experienced within the dry, relatively cohesionless alluvial materials. Based on the results of in-situ density tests and compaction data, it was noted that the upper alluvial units were typically in a loose to medium dense state becoming dense with depth. Consolidation testing was conducted on samples of the alluvium. These tests indicate normal consolidation characteristics. Details of the consolidation testing and the test results are presented within Appendix C of this report. While the maximum depth of the alluvial materials was not determined, our boring B-1 placed in the lower portions of the center valley did not encounter any bedrock materials to a depth of 50 feet.

Wind Blown Sand (Qs): Within the northeastern portion of the site, wind blown sand (aeolian) deposits were observed and encountered within two of our exploratory trenches (T-5 and T-6). As encountered, these materials consisted of poorly graded sand which was dry, tan in color, and based on our in-place density testing, was in a medium dense to dense state.

Man Made Fill (Qf): As previously mentioned, man made fill materials are present across the surface of the low lying areas of the site. These materials are the result of past and current weed abatement (discing) practices at the site. These materials were encountered to consist of well graded sand to silty sand which were dry, light brown, and loose and on the order of 1 to 2 feet in thickness. In addition, several relatively small stockpiles of fill were noted at the site. The neighboring commercial development to the north-northwest contains an approximately 8 foot high, two horizontal to one vertical fill slope along the boundary with the subject site. According to the map used during this investigation, this slope lies within the subject site.

Landslide (Qls): First noted during our review of stereo aerial photographs, it appears that the eastern face of the south portion of the larger on-site hill has experience some movement. During our field reconnaissance, evidence for such a feature was noted as a circular, hummocky depression. It appears to be a relatively shallow landslide that occurred along the foliation plane of the metamorphic bedrock.

The approximate lateral extent of the geologic units described above are shown on Plate 1. The relative thin, surficial fill materials were omitted from Plate 1 as their inclusion would mask the native geologic units.

Geotechnical cross sections throughout various portions of the site deemed most representative are presented on Plate 2, within back pocket. These cross sections depict the approximate current geologic conditions as well as the proposed development configuration.

Groundwater Hydrology

Groundwater was not encountered within any of borings or trenches, nor was any groundwater seepage observed during our site reconnaissance.

For historical groundwater records, we consulted the database maintained by the Western Municipal Water District (2007), the State of California Department of Water Resources, and we reviewed a groundwater contour map prepared by the U.S. Geological Survey (Carson and Matti, 1985).

The nearest wells listed on the W.M.W.D. database lie approximately 600 feet to the east. Seventeen wells were listed in that area. Groundwater records for those wells ranged from December of 1991 through October of 2006. Groundwater depths for that time ranged from approximately 112 to 351 feet beneath the existing ground surface. The latest measurements were in October of 2006 and groundwater lied at an average depth of approximately 200 feet in those wells.

No wells were found nearby the site within the State of California, Department of Water Resources database.

According to the U.S. Geological Survey (Carson and Matti, 1985), groundwater lied at a depth of approximately 100 feet beneath the site from the time period of 1973 to 1979.

Therefore, groundwater is anticipated to lie at a depth of approximately 200 feet and historically may have lied at a depth of approximately 100 feet. Thus, groundwater is not anticipated to be a factor in site development.

Surface Runoff

Current surface runoff of precipitation waters across the site is as sheetflow from the from the north to the south.

Mass Movement

As the overall local trend of the bedrock foliation dips to the east-southeast, only the east-southeast facing slopes of the site appeared to be susceptible to landslide failures. As previously noted, one such feature appears to be present at the site. This area is indicated on Plate 1.

Faulting

No active or potentially active faults are known to exist at the subject site. In addition, the subject site does not lie within a current State of California Earthquake Fault Zone (Hart and Bryant, 1997).

The closest known active fault in relation to the site is the San Jacinto fault located approximately 0.8 kilometers (0.5 miles) to the west. In addition, other relatively close active faults include the San Andreas fault zone located approximately 2.3 kilometers (1.5 miles) to the northeast, and the Cucamonga fault zone located approximately 7.2 kilometers (4.5 miles) to the northwest.

The San Jacinto fault zone is a sub-parallel branch of the San Andreas fault zone, extending from the northwestern San Bernardino area, southward into the El Centro region. This fault has been active in recent times with several large magnitude events. It is believed that the San Jacinto fault is capable of producing an earthquake magnitude on the order of 6.5 or greater.

The San Andreas fault is considered to be the major tectonic feature of California, separating the Pacific Plate and the North American Plate. While estimates vary, the San Andreas fault is generally thought to have an average slip rate on the order of 24mm/yr and capable of generating large magnitude events on the order of 7.5 or greater.

The Cucamonga fault is part of the Sierra Madre fault system which marks the southern boundary of the San Gabriel Mountains. This system is comprised of steeply, north dipping, thrust, range-front faults along which most of the uplift of the San Gabriel Mountains has occurred. The Cucamonga fault marks the eastern portion of the Sierra Madre fault system while the San Fernando fault marks the western portion. It is believed that the Cucamonga fault is capable of producing an earthquake on the order of 7.0 or greater.

Current standards of practice included a discussion of all potential earthquake sources within a 100 kilometer (62 mile) radius. However, while there are other large earthquake faults within a 100 kilometer (62 mile) radius of the site, none of these are

considered as relevant to the site as the faults described above, due to their closer distance and larger anticipated magnitudes.

Historical Seismicity

In order to obtain a general perspective of the historical seismicity of the site and surrounding region a search was conducted for seismic events at and around the area within various radii. This search was conducted utilizing the historical seismic search program by EPI Software, Inc. (Reeder, 2000) This program conducts a search of a user selected cataloged seismic events database, within a specified radius and selected magnitudes, and then plots the events onto an overlay map of known faults. For this investigation the database of seismic events utilized by the EPI program was obtained from the Southern California Seismic Network (SCSN) available from the Southern California Earthquake Center. At the time of our search the data base contained data from January 1, 1932 through July 30, 2007.

In our first search, the general seismicity of the region was analyzed by selecting an epicenter map listing all events of magnitude 4.0 and greater, recorded since 1932, within a 100 kilometer (62 mile)radius of the site, in accordance with guidelines of the California Division of Mines and Geology. This map illustrates the regional seismic history of moderate to large events. As noted on Enclosure A-4, within Appendix A, the site lies within a relatively active region associated with the San Andreas and San Jacinto fault zones trending northwest-southeast. Of these events, the closest was a magnitude 4.3 located approximately 3 kilometers (1.9 miles) to the west of the site.

In the second search, the micro seismicity of the area lying within a 10 kilometer (6.2 mile) radius of the site was examined by selecting an epicenter map listing events on the order of 0.0 and greater since 1977. In addition, only the " A" events, or most accurate events were selected. Caltech indicates the accuracy of the " A" events to be approximately 1 kilometer. The results of this search is a map that presents the seismic history around the area of the site with much greater detail, not permitted on the larger map. The reason for limiting the events to the last 30 ± years on the detail map is to enhance the accuracy of the map. Events recorded prior the mid 1970's are generally considered to be less accurate due to advancements in technology. As noted on this map, Enclosure A-5, the San Andreas fault appears to be relatively quiet, while the San Jacinto fault zones appear to be the source of numerous events.

In summary, the historical seismicity of the site entails numerous small to medium magnitude earthquake events occurring around the subject site, predominately associated with the presence of the San Jacinto fault, and to a lesser degree the San Andreas fault. Any future developments at the subject site should anticipate that a moderate to very large seismic events could occur very near the site along either one of these features.

Secondary Seismic Hazards

Other secondary seismic hazards generally associated with severe ground shaking during an earthquake include liquefaction, seiches and tsunamis, earthquake induced flooding, landsliding and rockfalls, and seismic-induced settlement.

Liquefaction: The potential for liquefaction generally occurs during strong ground shaking within loose, granular sediments where the groundwater is usually less than 50-feet. Based on our field investigation, the lower lying areas of the site are underlain at depth by relatively dense alluvial materials and the depth to current groundwater levels is thought to be in excess of 50 feet, therefore, the possibility of liquefaction at the site is considered nil. Liquefaction is also considered nil within the elevated portions of the site underlain by metamorphic bedrock.

Seiches/Tsunamis: The potential for the site to be affected by a seiche or Tsunami (earthquake generated wave) is considered nil due to absence of any large bodies of water near the site.

Flooding (Water Storage Facility Failure): There are no large water storage facility located near and above the site which could possibly rupture during in earthquake and effect the site by flooding.

Seismically-Induced Landsliding: The eastern face of the southern portion of the larger hill at the site appears to be a landslide feature. Additional downslope activity along this feature during an earthquake, especially if it occurs after a period of precipitation, can not be completely ruled out in its current configuration.

Rockfalls: No large, exposed, loose or unrooted boulders are present above the site that would affect the integrity of the site.

Seismically-Induced Settlement: Settlement generally occurs within areas of loose, granular soils with relatively low density. Since the low lying areas of the site are underlain by relatively dense alluvial materials at depth and hard metamorphic bedrock at the surface of the elevated portions of the site, the potential for settlement is considered low. In addition, the recommended earthwork operations during the development of the site should mitigate any surficial loose soil conditions.

SOILS AND SEISMIC DESIGN CRITERIA (California Building Code)

Design requirements for structures can be found within Chapter 16 of the 2001 California Building Code (CBC) based on building type, use and/or occupancy. The classification of use and occupancy of all proposed structures at the site, and thus design requirements, shall be the responsibility of the structural engineer and the building official. For structures at the site to be designed in accordance with the provisions of Chapter 16, the subject site specific soils and seismic criteria are provided in the following sections.

CBC Division IV: Earthquake Design Criteria Selection

Procedure and limitations for the earthquake design of applicable structures can be obtained from Division IV of Chapter 16 of the 2001 California Building Code (CBC). However, it should be noted that the building code requires the minimum design to allow a structure to remain standing after a seismic event, in order to allow for safe evacuation. As stated in section 1626.1, " The purpose of the earthquake provisions herein is primarily to safeguard against major structural failures and loss of life, not to limit damage or maintain function." Therefore a structure built to CBC code may still sustain damage which might ultimately result in the demolishing of the structure.

The CBC Division IV requires that all sites, unless exempted, be assigned a soil profile type and a regional seismic zone. The criteria for the selection of a site soil profile can be found in the 2001 CBC Division V, discussed in later sections.

Seismic Zone: As shown on Figure 16-2 within Chapter 16 of the 2001 CBC, the site is located in Seismic Zone 4. Section 1629.4.2 of the 2001 CBC directs that all sites in Seismic Zone 4, unless exempted, shall have a near source factor determined.

Near Source Factor: Near source factors are determined based on the distance to the nearest Type A or B seismic source (earthquake fault). Once these are determined, near source values can be obtained, dependent on structure type, from Tables 16-S or 16-T within the 2001 CBC. Seismic source types are classified as A, B, or C, based on description, maximum anticipated magnitude, and slip rate. Type C sources are not considered as they do not increase the standard near source factor value of 1.0. The following table lists the seismic source type requirements.

TABLE 16-U SEISMIC SOURCE TYPE[1]

Seismic Source Type	Seismic Source Description	Seismic Source Definitions	
		Maximum Magnitude	Slip Rate (mm/yr)
A	Faults capable of large magnitude events, and have a high rate of seismic activity.	M ≥ 7.0	SR ≥ 5
B	All faults other than A and C.	M ≥ 7.0 M < 7.0 M ≥ 6.5	SR < 5 SR > 2 SR < 2
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity.	M < 6.5	SR ≤ 2

[1]Source 2001 CBC

Specific parameters for earthquake faults within the State of California can be obtained from The Revised 2002 California Probabilistic Seismic Hazards Maps, June 2003 (Cao et al.). As noted in the Faulting section of this report, the nearest known active fault to the site is the San Jacinto fault, located approximately 0.8 kilometers (0.5 miles) to the west is the closest fault to the site. According to Cao et al. (2003), the Glen Helen segment of the San Jacinto fault has a slip rate of 12 mm/year, ± 6, and an estimated magnitude event of 6.7. According to the CBC table above, the Glen Helen segment of the San Jacinto fault is, therefore, classified as a Type B fault. The nearest known active Type A fault, according to the table above and the Uniform Building Code Maps of Known Active Fault Near-Source Zones (UBC, 1998), is the San Andreas fault located approximately 2.3 kilometers (1.5 miles) to the northeast.

According to Cao et al. (2003), the San Bernardino segment of the San Andreas fault has a slip rate of 24 mm/year, ± 6, and an estimated magnitude event of 7.3.

CBC Division V: Soil Profile

As noted in our excavations at the site and previously published literature (Miller, Matti, and Carson, 2001 and Dibblee, 2004), the subject site is thought to be underlain by relatively medium dense to dense alluvial materials based on our Standard Penetration Test (SPT) and in-place density data, and metamorphic bedrock. Bedrock is assigned one of three soil types by the CBC: S_A, S_B, or S_C. Soil types S_A and S_B shall be measured on-site or estimated by a geotechnical engineer, engineering geologist or seismologist for competent rock with moderate fracturing and weathering. Soil types S_A and S_B shall not be used if more than 10 feet of soil is between the rock surface and the bottom of the spread footing or mat foundation. The soil type S_C can be used for very dense soil and soft rock. Therefore, the bedrock at the site falls into the soil type S_C and the alluvial materials are soil type S_D. Since final development will consist of a structure which will span these soil types, the conservative soil profile type of S_D should be used.

CBC Earthquake Design Summary

As determined in the previous sections, the following earthquake design criteria have been formulated for the site. However, these values should be reviewed and the final design should be performed by a qualified structural engineer familiar with the region.

SEISMIC AND SOIL CRITERIA **Seismic Zone = 4, Seismic Zone Factor (Z) = 0.40, Soil Type = S_D**						
Nearest Source Type	**Fault Name**	**Distance* (km)**	**N_a**	**N_v**	**C_a (0.44 N_a)**	**C_v (0.64 N_V)**
A	San Andreas, San Bernardino segment	2.5	1.5	1.9	0.66	1.22
B	San Jacinto, Glen Helen segment	1.0	1.3	1.6	0.57	1.02
$T_s = C_v/2.5C_a = 0.74$ $T_o = 0.2T_s = 0.15$						
* Distances rounded to the nearest 0.5 km						

SLOPE STABILITY ANALYSIS

As shown on the referenced preliminary grading for the site, the proposed development will include the grading of fill slopes up to 36 feet in height with minor cut slopes up to 8 feet in height. All slopes are planned to be graded to a maximum inclination of 2 horizontal to 1 vertical.

Per current standards of practice, any slope higher than 30 feet and/or any cut slope steeper than 2 horizontal to 1 vertical will require a detailed slope stability analysis. Therefore, we have conducted a slope stability analysis for the proposed fill slopes at the site higher than 30 feet. Such slopes are planned to be built along the southeastern portion of the site. A representative cross section of the subject slope, Section A-A', showing the geologic units comprising the slope is presented on Plate 2. As displayed on this illustration, the subject slope is 36 feet tall and will comprise of fill materials placed over alluvial materials. The location of this cross section is shown on Plate 1.

The stability analysis were performed using the method of limit equilibrium with the aid of the computer program PCSTABLE 5 (Purdue, 1999). This program calculates the factors for safety along numerous failure planes and establishes the surface with the lowest factor of safety. The failure surface with the lowest factor of safety is termed the "critical slip surface". By convention, slopes with static values greater than 1.50 and pseudostatic values greater than 1.10 along the critical slip surface are considered stable.

The parameters utilized during this assessment were a moist unit weight of 120 pounds per cubic foot (pcf), a cohesion value of 100 pounds per square foot (psf), and a friction angle of 30 degrees for the engineered compacted fills. For the native alluvium, a moist unit weight of 120 pcf, a cohesion of 50 psf, and a friction angle of 32 degrees were applied. These parameters were obtained from direct shear tests conducted in our geotechnical laboratory on representative samples of the slope materials. The results of our direct shear testing is presented within Appendix C.

Both static and pseudostatic conditions were analyzed for the design. In the pseudo static analysis a horizontal earthquake coefficient of 0.15g was utilized. Our calculations show a static factor of safety of 1.55 and a pseudostatic factor of safety

of 1.15 for the slope. The complete results of the slope stability analysis are presented within Appendix D.

In summary, our preliminary calculations show satisfactory factors of safety for the intended fill slopes. Therefore, we believe that the slopes should be stable to the proposed maximum inclination and height of 2 horizontal to 1 vertical and 36 feet, respectively. However, since much of the onsite materials are composed of clean, cohesionless sands and silty sands and bedrock materials with relatively low cohesion, the slopes could experience surficial stability problems. Therefore, some mitigation measures will be required to prevent shallow slope failures.

RIPPABILITY

Our subconsultant, Terra Geosciences, conducted a detailed study regarding the rippability of the bedrock. The excavation characteristics of the metamorphic bedrock is anticipated to be highly variable at the site. This variability is due to the variable lithologic nature of the Palona Schist, and localized potential for quartzite and granitic dikes. Excavation difficulties are anticipated at depths as shallow as approximately 35 feet and as great as approximately 80 feet. However, it should be noted that localized hard rock floaters and dikes may also be present within the upper, weathered materials considered to be rippable. Such materials may require special handling. Special handling will most likely result in over-sized materials. Recommendations for over-sized materials are presented in the Engineered Compacted Fill section of this report. Details of the excavation characteristics of the bedrock materials are provided within Appendix E. The locations of Terra Geoscience's seismic survey is shown on Plate 1.

The most important consideration for the proposed grading should include selecting an experienced, well-qualified contractor. The success to excavating the bedrock materials at the site will require the contractor to have knowledge of the appropriate ripper-equipment selection (i.e. down pressure available at the tip, tractor fly wheel horsepower, tractor grass weight, etc.) and ripping techniques (i.e. single or multi teeth, pair spacing, tandem pushing, etc.). Selecting the most qualified contractor cannot be overemphasized.

CONCLUSIONS

General

This investigation provides a broad overview of the geotechnical and geologic factors which are expected to influence future site planning and development. On the basis of our field investigation and testing program, it is the opinion of LOR Geotechnical Group, Inc. that the proposed development is feasible from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into design and implemented during grading and construction.

The subsurface conditions encountered in our exploratory borings and trenches are indicative of the locations explored. The subsurface conditions presented here are not to be construed as being present the same everywhere on the site. If conditions are encountered during the construction of the project which differ significantly from those presented in this report, this firm should be notified immediately so we may assess the impact to the recommendations provided.

Seismicity

Seismic ground rupture is generally considered most likely to occur along pre-existing active faults. Based on our review of historical aerial photographs, site mapping, and the referenced regional geologic maps prepared by the State of California and USGS, no faults are known to exist at, or project into the site. Therefore, it appears that the probability of ground surface rupture occurring at the site is considered nil.

Due to the site's close proximity to the San Andreas and San Jacinto fault zones, described above, it is reasonable to expect a strong to very strong ground motion seismic event to occur during the lifetime of the proposed development on the site. Large earthquakes could occur on other faults in the general area, but because of their lesser anticipated magnitude and/or greater distance, they are considered less significant than the San Andreas and San Jacinto fault zones from a ground motion standpoint.

The effects of ground shaking anticipated at the subject site, not including the secondary effects of slope stability, can be mitigated by the seismic design requirements and procedures outlined in Chapter 16 of the California Building Code. However, it should be noted that the current building code requires the minimum design to allow a structure to remain standing after a seismic event, in order to allow for safe evacuation. A structure built to code may still sustain damage which might ultimately result in the demolishing of the structure (Larson and Slosson, 1992).

Slope Stability

Based on our slope stability analysis and data collected, we believe that the proposed slopes at the project should be grossly stable. However, as shown by our data, the slopes are likely to be composed of the on-site remolded bedrock units and alluvial materials which are mostly granular materials, with very low cohesion, which could lead to surficial failures of the slopes. To remediate this problem, we recommend that slopes at the project be planted with a deep rooted ground cover as soon as possible after completion. The use of succulent ground covers such as iceplant or sedum is not recommended because it just sits on top of the ground without providing any relatively deeper protection/reinforcement to the soil. If watering is necessary to sustain plant growth on slopes, then the watering operation should be monitored to assure proper operation of the irrigation system and to prevent over watering.

The currently proposed grading at the site will most likely remove the majority of the existing landslide feature at the site. In addition, the area beyond to toe of this feature is panned for approximately 34 feet of fill, extending at the approximate depth for a distance of approximately 200 feet, essentially prohibiting movement of any left in-place slide materials.

Foundation Support

Based upon the field investigation and test data, it is our opinion that the existing fill materials, landslide feature, and the upper portions of the native alluvial and wind blown sand materials will not, in their present condition, provide uniform and/or adequate support for the proposed structures. Our observations and in-place density data indicated variable in-situ conditions of the upper native soils, ranging from loose

to medium dense states. This condition may cause unacceptable differential and/or overall settlements upon application of the anticipated foundation loads at the site.

To provide adequate support for the proposed building, all foundations should rest entirely on entirely on an engineered compacted fill mat placed on competent native materials. Conventional foundation systems will provide adequate support for the anticipated downward and lateral loads when utilized in conjunction with the recommended fill mat or native materials.

RECOMMENDATIONS

Geologic Recommendations

During site rough grading, all cut slopes and excavations into the existing hillsides should be observed in the field by the project engineering geologist. This includes excavations in the area of the landslide feature. Should this feature be larger than currently anticipated, complete removal of this feature may be warranted.

General Site Grading

It is imperative that no clearing and/or grading operations be performed without the presence of the project geotechnical consultant. An on-site, pre-job meeting with the owner, the developer, the contractor, the jurisdictional agency, and the project geotechnical consultant should occur prior to all grading related operations. Operations undertaken at the site without the project geotechnical consultant present may result in exclusions of affected areas from the final compaction report for the project.

Grading of the subject site should be performed in accordance with the following recommendations as well as applicable portions of Appendix Chapter 33 of the California Building Code, and/or applicable local ordinances.

All areas to be graded should be stripped of significant vegetation and other deleterious materials.

Cavities created by removal of subsurface obstructions should be thoroughly cleaned of loose soil, organic matter, and other deleterious materials, shaped to provide access for construction equipment, and backfilled as recommended in the following Engineered Compacted Fill section of this report.

Initial Site Preparation

All fill materials, any noted landslide units, and the upper loose portions of the alluvial and wind blown sand materials should be removed from structural areas and areas to receive structural fills. Based upon the data collected during this study, removals on the order of 3 to 5 feet below the existing ground surface will be required within alluvial and/or wind blown sand areas to expose competent soils with a minimum relative compaction of 85 percent (per ASTM D 1557) upon which structural fills can be placed. Removals are not anticipated within areas exposing metamorphic bedrock. The currently proposed grading is anticipated to remove the noted landslide feature, thus, additional earthwork removals in this area are not anticipated. Removals should be extended at a 1:1 (horizontal to vertical) projection beyond the footing lines. The actual depths of removals should be verified during the grading operation by observation and in-place density testing.

Over-excavation within proposed parking and drive areas underlain by bedrock is not required. However, consideration should be given to over-excavate any utility corridors which may be present in such areas in order to facilitate their installation.

Preparation of Fill Areas

After conducting the removals discussed above and prior to placing fill, the surfaces of alluvial and wind blown sand areas to receive fill should be scarified to a depth of at least 12 inches. The scarified soil should be brought to near optimum moisture content and recompacted to a relative compaction of at least 90 percent (ASTM D 1557).

Rippability

As previously mentioned, the weathered, fractured, well foliated Pelona Schist noted within the upper portions are anticipated to be rippable. However, areas of hard rock floaters and/or quartzite and granitic dikes may be present, which are anticipated to be difficult to excavate. In addition, difficult excavation is anticipated at depths as shallow as approximately 35 feet to greater than approximately 80 feet. Areas of difficult excavation are anticipated to require special handling which is anticipated to result in over-sized materials. Such materials will require special disposal, as outlined in the Engineered Compacted Fill section of this report.

Consideration should be given to over-excavate areas underlain by bedrock and to receive utilities and/or other subterranean structures. By doing so, the installation of such improvements will be facilitated.

The most important factor for the proposed grading should include selecting an experienced, well-qualified contractor. Selecting the most qualified contractor cannot be over-emphasized.

The velocity characteristics and the expected rippability characteristics of the bedrock materials were investigated by our sub-consultant Terra Geosciences. The locations of their geophysical survey lines are shown on Plate 1. These locations were selected based on currently proposed grading. A copy of their report is present within Appendix E.

Preparation of Building Pad Area

As currently proposed, the structure will span across several cut to fill and bedrock to alluvium transitions. Due to the potential for differential settlement across such transitions, foundations should rest entirely on a minimum of 24 inches of properly compacted fill material placed over competent native materials. In areas where the required fill thickness is not accomplished by site rough grading and remedial removals, the footing areas should be further subexcavated to a depth of at least 24 inches below the proposed footing base grade, with the subexcavation extending at least 5 feet beyond. The bottom of this excavation should then be scarified, for alluvial and/or wind blown sand materials, to a depth of at least 12 inches, brought to near optimum

moisture content, and recompacted to at least 90 percent relative compaction (ASTM D 1557) prior to refilling the excavation to grade as properly compacted fill.

As stated above, a minimum compacted fill mat of 24 inches below the base grade of footings is recommended in areas of shallow fills or cut areas. However, bedrock areas to be cut, deeper over-excavations should be considered to facilitate the placement of utilities and other subterranean improvements.

To provide adequate support, concrete slabs-on-grade should bear on a minimum of 12 inches of compacted soil placed on competent native materials. The compacted soil should have a density of at least 90 percent relative compaction (ASTM D 1557).

Engineered Compacted Fill

The on-site soils should provide adequate quality fill material, provided they are free from organic matter and other deleterious materials. Unless approved by the geotechnical engineer, rock or similar irreducible material with a maximum dimension greater than 12 inches should not be buried or placed in fills within the upper 10 feet of fill areas. Oversized material (more than 12 inches) is anticipated and may be stockpiled for landscaping purposes or placed in a rock disposal area as approved by the owner, geotechnical engineer, and local agency having jurisdiction.

An acceptable rock disposal method may consist of spreading the rock out over an area of competent materials and filling and compacting the voids between the oversized material with granular materials. Such granular materials are readily available at the site. Once the voids have been properly filled and compacted, a 2-foot thick layer of engineered compacted fill should be placed over the top of this rock ‘ blanket’ prior to construction of another rock disposal blanket.

Import fill should be inorganic, non-expansive granular soils free from rocks or lumps greater than 12 inches in maximum dimension. Sources for import fill should be approved by the geotechnical engineer prior to their use.

Fill should be spread in maximum 8-inch uniform, loose lifts, each lift brought to near optimum moisture content, and compacted to a relative compaction of at least 90 percent in accordance with ASTM D 1557.

Based upon the relative compaction of the near surface alluvial/wind blown sand materials determined during this investigation and the relative compaction anticipated for compacted fill soil, we estimate a compaction shrinkage of approximately 15 to 20 percent. Therefore, 1.15 cubic yards to 1.20 cubic yards of in-place alluvial materials would be necessary to yield one cubic yard of properly compacted fill material. For bedrock materials to be used as fill, a compaction bulking factor of approximately 0 to 5 percent is anticipated. Thus one cubic yard of in-place bedrock material would yield 1.00 to 1.05 cubic yards of engineered compacted fill. These values are for estimating purposes only, and are exclusive of losses due to stripping or the removal of subsurface obstructions. These values may vary due to differing conditions within the project boundaries and the limitations of this investigation. Shrinkage/bulking should be monitored during construction. If percentages vary, provisions should be made to revise final grades or adjust quantities of borrow or export.

Short-Term Excavations

Following the California Occupational and Safety Health Act (CAL-OSHA) requirements, excavations deeper than 5 feet should be sloped or shored. All excavations and shoring should conform to CAL-OSHA requirements.

Short-term excavations greater than 5 feet deep shall conform to Title 8 of the California Code of Regulations, Construction Safety Orders, Section 1504 and 1539 through 1547. Based on our exploratory trenches and borings, it appears that Type C soil is the predominant type of soil across the lower, flatter areas of the site, while Type A soil is the prevailing type of material across the hillsides. We recommend that short-term excavations be designed based on the preceding soil types. Deviation from the standard short-term slopes are permitted using option 4, Design by a Registered Professional Engineer (Section 1541.1).

Short-term excavation construction and maintenance are the responsibility of the contractor and should be a consideration of his methods of operation and the actual soil conditions encountered.

Slope Construction

Preliminary data indicates that cut and fill slopes should be constructed no steeper than 2 horizontal to 1 vertical. Fill slopes should be overfilled during construction and then cut back to expose fully compacted soil. A suitable alternative would be to compact the slopes during construction, then roll the final slopes to provide dense, erosion-resistant surfaces.

Where fills are to be placed against existing slopes steeper than 5:1 (horizontal to vertical), the hillside should be properly keyed and benched to expose competent native materials. The key, constructed across the toe of the proposed slope, should be a minimum of 12 to 15 feet wide, a minimum of 2 feet deep at the toe, and sloped back at 2 percent. Benches should be constructed at approximately 2 to 4 feet vertical intervals. Typical keying and benching operations are presented on Enclosure F-1 within Appendix F.

Slope Protection

Since the native materials are susceptible to erosion by running water, measures should be provided to prevent surface water from flowing over slope faces. Slopes at the project should be planted with a deep rooted ground cover as soon as possible after completion. The use of succulent ground covers such as iceplant or sedum is not recommended. If watering is necessary to sustain plant growth on slopes, then the watering operation should be monitored to assure proper operation of the irrigation system and to prevent over watering.

Soil Expansiveness

The materials encountered during this investigation were observed to be granular and are considered to have a very low expansion potential. Therefore, specialized construction procedures to specifically resist expansive soil activity are not anticipated at this time. In order to verify this, additional evaluation of on-site and imported soils for their expansion potential should be conducted during the grading operation.

Foundation Design

If the site is prepared as recommended, the proposed distribution building may be safely founded on conventional shallow foundations bearing entirely on a minimum of 24 inches of engineered compacted fill placed over competent native materials. Footings should have a minimum width of 12 inches and should be established a minimum of 12 inches below lowest adjacent grade.

For design of footings about 12-inch wide and 12-inch below the lowest adjacent final grade, a maximum soil bearing pressure of 2,000 pounds per square foot (psf) for dead plus live loads should be used. This bearing pressure may be increased by 400 psf for each additional foot of width, and by 400 psf for each additional foot of depth, up to a maximum of 5,000 psf. For example, a footing 4 feet wide and embedded 1.5 feet will have an allowable bearing pressure of 3,400 psf.

The above values are net pressures; therefore, the weight of the foundations and the backfill over the foundations may be neglected when computing dead loads. The values apply to the maximum edge pressure for foundations subjected to eccentric loads or overturning. The recommended pressures apply for the total of dead plus frequently applied live loads, and incorporate a factor of safety of at least 3.0. The allowable bearing pressures may be increased by one-third for temporary wind or seismic loading. The resultant of the combined vertical and lateral seismic loads should act within the middle one-third of the footing width. The maximum calculated edge pressure under the toe of foundations subjected to eccentric loads or overturning should not exceed the increased allowable pressure. Foundations should be setback from slopes as detailed on Enclosure F-2, within Appendix F.

Resistance to lateral loads will be provided by passive earth pressure and base friction. For footings bearing against compacted fill, passive earth pressure may be considered to be developed at a rate of 300 pounds per square foot per foot of depth. Base friction may be computed at 0.30 times the normal load. Base friction and passive earth pressure may be combined without reduction. These values are for dead load plus live load and may be increased by one-third for wind or seismic loading.

Settlement

As stated before, the planned development of the site will require placement of fills up to 40 feet in depth in areas underlying by alluvial/wind blown sand deposits and cuts up to 100 feet in areas underlying by bedrock. Our calculations suggest that total settlements up to 2.5 inches could take place within fill areas. These settlements are believed to occur rapidly, primarily as a result of elastic compression of supporting soils as the fills are applied, and should be essentially completed shortly after the placement of fills and before the construction of the foundations for the distribution building. Areas to be cut into bedrock will have a settlement of approximately zero.

Static total settlement of individual foundations of the building will vary depending on the width of the foundation and the actual load supported. Maximum total static settlement of foundations designed and constructed in accordance with the preceding recommendations are estimated to be on the order of 0.5 inches. Differential settlements between adjacent footings should be about one-half of the total settlement.

Building Area Slab-on-Grade Design

To provide adequate support, concrete slabs-on-grade should bear on a minimum of 12 inches of compacted soil placed on competent native materials. The compacted soil should have a density of at least 90 percent relative compaction (ASTM D 1557). The final pad surfaces should be rolled to provide smooth, dense surfaces upon which to place the concrete.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor barrier. This barrier may consist of an impermeable membrane. Two inches of sand over the membrane will reduce punctures and aid in obtaining a satisfactory concrete cure. The sand should be moistened just prior to placing of concrete.

For design of slabs and estimating slab deflection, a modulus of subgrade reaction (k) of 200 pounds per square inch per inch of deflection may be used. Settlements of lightly loaded floor slabs should be negligible. Where feasible, we recommend that the pouring of the floor slabs be deferred until most of the column dead loads have been applied.

The slabs should be protected from rapid and excessive moisture loss which could result in slab curling. Careful attention should be given to slab curing procedures, as the site area is subject to large temperature extremes, humidity, and strong winds.

Exterior Flatwork and Driveway Construction

To provide adequate support, exterior flatwork and driveway improvements should rest on a minimum of 12 inches of compacted soil with a relative compaction of at least 90 percent (ASTM D 1557) or upon bedrock.

Wall Pressures

The design of footings for retaining walls should be performed in accordance with the recommendations described earlier under Preparation of Building Pad Area and Foundation Design. For design of retaining wall footings, the resultant of the applied loads should act in the middle one-third of the footing, and the maximum edge pressure should not exceed the basic allowable value without increase.

For design of retaining walls unrestrained against movement at the top, we recommend an equivalent fluid density of 40 pounds per cubic foot (pcf) be used. This assumes level backfill consisting of recompacted, non-expansive, native soils placed against the structures and within the back cut slope extending upward from the base of the stem at 35 degrees from the vertical or flatter.

Retaining structures subject to uniform surcharge loads within a horizontal distance behind the structures equal to the structural height should be designed to resist additional lateral loads equal to 0.32 times the surcharge load. Any isolated or line loads from adjacent foundations or vehicular loading will impose additional wall loads and should be considered individually.

To avoid over stressing or excessive tilting during placement of backfill behind walls, heavy compaction equipment should not be allowed within the zone delineated by a 45 degree line extending from the base of the wall to the fill surface. The backfill directly behind the walls should be compacted using light equipment such as hand operated vibrating plates and rollers. No material larger than 3 inches in diameter should be placed in direct contact with the wall.

Wall pressures should be verified prior to construction, when the actual backfill materials and conditions have been determined. Recommended pressures are applicable only to level, non-expansive, properly drained backfill with no additional surcharge loadings. If inclined backfills are proposed, this firm should be contacted to develop appropriate active earth pressure parameters. Toe bearing pressure for non-structural walls on soils, not prepared as described earlier under Preparation of Building Pad Area, should not exceed California Building Code values, (CBC Table 18-1-A).

Preliminary Pavement Design

Testing and design for preliminary on-site pavement was conducted in accordance with the California Highway Design Manual and Portland Cement Association Guidelines. Based upon our preliminary sampling and testing, and upon Traffic Indices generally associated with these types of projects, it appears that the structural sections tabulated below should provide satisfactory pavements for the subject improvements:

AREA	T.I.	DESIGN R-VALUE	PRELIMINARY SECTION
Light Vehicular Traffic with Occasional Truck Traffic	5.0	50	0.25' AC/0.35' AB or 6" PCC
Heavy Truck Traffic	7.0	50	0.30' AC/0.45' AB or 7" PCC
AC - Asphalt Concrete AB - Class 2 Aggregate Base PCC - Portland Cement Concrete with a minimum modulus of rupture of 550 psi			

The above structural sections are predicated upon 90 percent relative compaction (ASTM D 1557) of all utility trench backfills and 95 percent relative compaction (ASTM D 1557) of the upper 12 inches of pavement subgrade soils and of any aggregate base utilized. In addition, the aggregate base should meet Caltrans specifications for Class 2 Aggregate Base.

The concrete pavement section may be placed directly over the native subgrade prepared as described above. The concrete to be utilized for the concrete pavement should have a minimum modulus of rupture of 550 psi. Transverse joints should be

sawcut in the pavement at approximately 15 foot intervals within 4 to 6 hours of concrete placement, or preferable sooner. Sawcut depth should be equal to approximately one quarter of slab thickness. Construction joints should be constructed such that adjacent sections butt directly against each other and are keyed into each other. Parallel pavement sections should also be keyed into each other. It should be noted that distributed steel reinforcement (welded wire fabric) is not necessary, nor will any decrease in section thickness result from its inclusion.

The above pavement designs were based upon the results of preliminary sampling and testing, and should be verified by additional sampling and testing when the actual subgrade soils are exposed.

Sulfate Protection

The results of the soluble sulfate tests conducted on selected subgrade soils expected to be encountered at foundation levels are presented in Appendix C.

Based on the test results the sulfate exposures of on site soils is considered negligible by the California Building Code. Therefore, no specific recommendations are given for concrete elements to be in contact with on site soils.

Construction Monitoring

Post investigative services are an important and necessary continuation of this investigation. Project plans and specifications should be reviewed by this firm prior to construction to confirm that the intent of the recommendations presented in this report have been incorporated into the design. Verification testing including soluble sulfate content and expansion index should be performed during the site rough grading.

During construction, sufficient and timely geotechnical observation and testing should be provided to correlate the findings of this investigation with the actual subsurface conditions exposed during construction. Items requiring observation and testing include, but are not necessarily limited to, the following:

1. Site preparation-stripping and removals.
2. Excavations, including approval of the bottom of excavations prior to backfilling.
3. Scarifying and recompacting prior to fill placement.
4. Subgrade preparation for pavements and slabs-on-grade.
5. Placement of engineered compacted fill and backfill, including approval of fill materials and the performance of sufficient density tests to evaluate the degree of compaction being achieved.
6. Foundation excavations, including footings.

LIMITATIONS

This report contains geotechnical conclusions and recommendations developed solely for use by IDS Real Estate and their design consultants, for the purposes described earlier. It may not contain sufficient information for other uses or the purposes of other parties. The contents should not be extrapolated to other areas or used for other facilities without consulting LOR Geotechnical Group, Inc.

The recommendations are based on interpretations of the subsurface conditions concluded from information gained from subsurface explorations, and a surficial site reconnaissance. The interpretations may differ from actual subsurface conditions, which can vary horizontally and vertically across the site. Due to possible subsurface variations, all aspects of field construction addressed in this report should be observed and tested by the project geotechnical consultant.

If parties other than LOR Geotechnical Group, Inc. provide construction monitoring services, they must be notified that they will be required to assume responsibility for the geotechnical phase of the project being completed by concurring with the recommendations provided in this report or by providing alternative recommendations.

The report was prepared using generally accepted geotechnical engineering practices under the direction of a state licensed geotechnical engineer. No warranty, expressed or implied, is made as to conclusions and professional advice included in this report. Any persons using this report for bidding or construction purposes should perform such independent investigations as deemed necessary to satisfy themselves as to the surface and subsurface conditions to be encountered and the procedures to be used in the performance of work on this project.

TIME LIMITATIONS

The findings of this report are valid as of this date. Changes in the condition of a property can, however, occur with the passage of time, whether they be due to natural processes or the work of man on this or adjacent properties. In addition, changes in the Standards-of-Practice and/or Governmental Codes may occur. Due to such changes, the findings of this report may be invalidated wholly or in part by changes beyond our control. Therefore, this report should not be relied upon after a significant amount of time without a review by LOR Geotechnical Group, Inc., verifying the suitability of the conclusions and recommendations.

CLOSURE

It has been a pleasure to assist you with this project. We look forward to being of further assistance to you as construction begins. Should conditions be encountered during construction that appear to be different than indicated by this report, please contact this office immediately in order that we might evaluate their effect.

Should you have any questions regarding this report, please do not hesitate to contact our office at your convenience.

Respectfully submitted,
LOR Geotechnical Group, Inc.

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Miller, R.K., Matti, J.C., and Carson, S.E., 2001, Geologic Map of the San Bernardino North 7.5' Quadrangle, San Bernardino County, California, Version 1.0, U.S. Geological Survey Open File Report 01-131.

Purdue University, 1999, Stedwin 2.3 Slope Stability Computer Program.

Reeder, W., 2000, Earthquake Plotting Program, EPI Software.

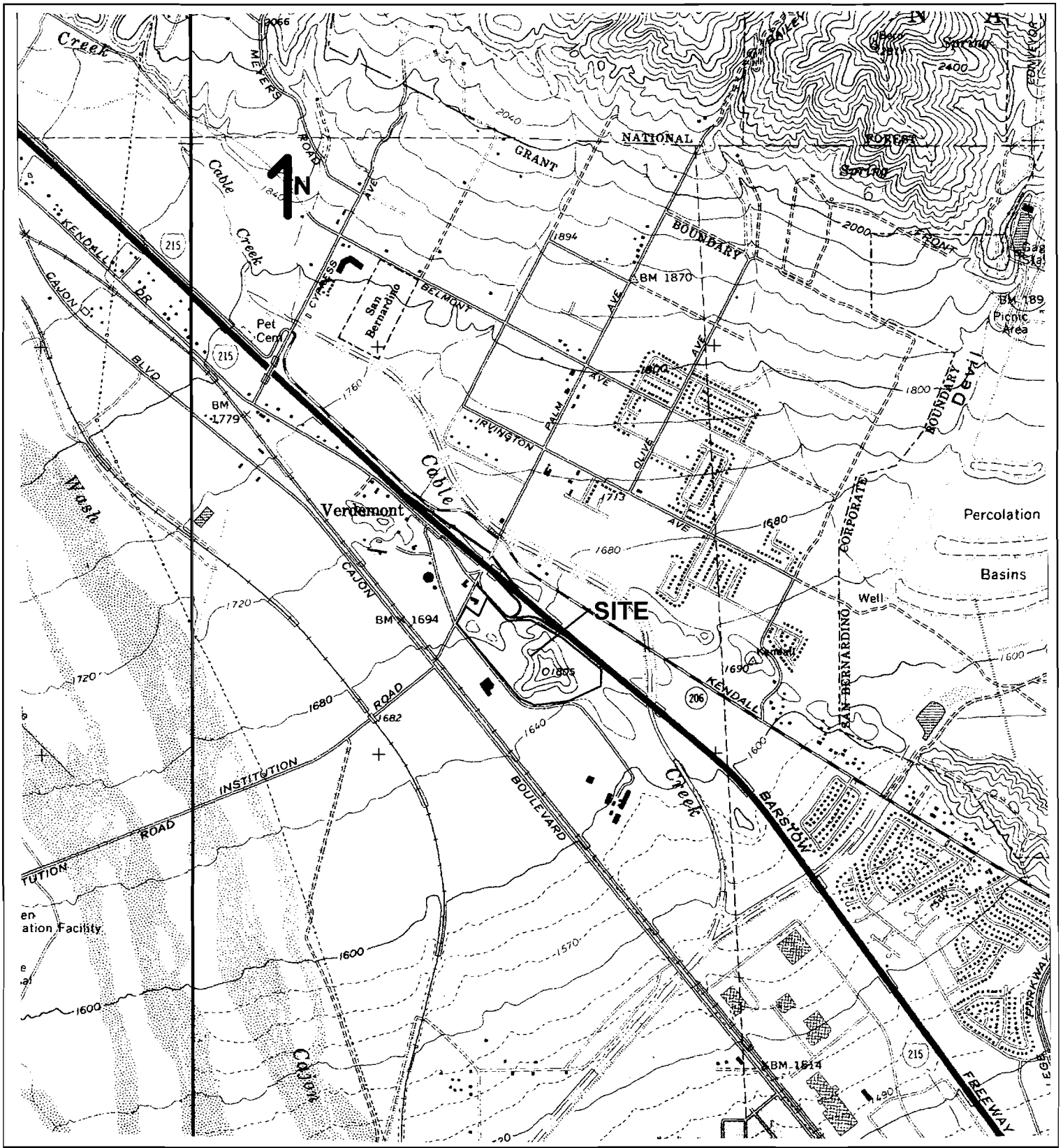
AERIAL PHOTOGRAPHS

(Source: San Bernardino County Flood Control)

DATE	FLIGHT NO.	PHOTO NO(S).	SCALE
July 9, 1938	USDA	AXL-63-80, & 81	1"= 2,000'
November 10, 1955	F-34	7-30 and 7-31	1"= 1,200'
May 22, 1962	C-16	22 and 23	1"= 2,000'
October 8, 1971	C-186	14 and 15	1"= 2,000'
February 25, 1986	C-450	169 and 170	1"= 2,000'
July 1, 1991	C-487	196 and 197	1"= 2,000'
January 19, 2005	C-553	17-32 and 17-33	1"= 1,000'

APPENDIX A

Index Map, Regional Geologic Map, Description of Geologic Units, and Historical Seismicity Maps



INDEX MAP

PROJECT:	682K DISTRIBUTION BUILDING, SAN BERNARDINO, CALIFORNIA	PROJECT NO.:	22448.1
CLIENT:	IDS REAL ESTATE	ENCLOSURE:	A-1
LOR Geotechnical Group, Inc.		DATE:	AUGUST 2007
		SCALE:	1" = 2,000'

VERY YOUNG SURFICIAL DEPOSITS—Sediment recently transported and deposited in channels and washes, on surfaces of alluvial fans and alluvial plains, and on hillslopes. Soil-profile development is non-existant to minimal. Includes:

Qw — **Very young wash deposits (late Holocene)**—Unconsolidated to locally cemented sand, gravel, and boulder deposits in active washes of streams and on active surfaces of alluvial fans. Typically shows fresh flood scours and channel-and-bar morphology. Locally includes small areas of older surficial deposits. Most deposits are centimeters to a few meters thick. Includes:

Qw_2 — **Very young wash deposits, Unit 2 (late Holocene)**—Chiefly sand and gravel. Distinguished as lower level terraces in Qw terrace sequence; pebble to boulder clasts are sparse to abundant

Qw_1 — **Very young wash deposits, Unit 1 (late Holocene)**—Chiefly sand and gravel. Distinguished as upper level terraces in Qw terrace sequence; pebble to boulder clasts are sparse to abundant

Young alluvial-valley deposits (Holocene and late Pleistocene)—Slightly to moderately dissected, consolidated to cemented deposits of angular to subangular silt, sand and pebbles. Generally finer grained than Qyf units Includes:

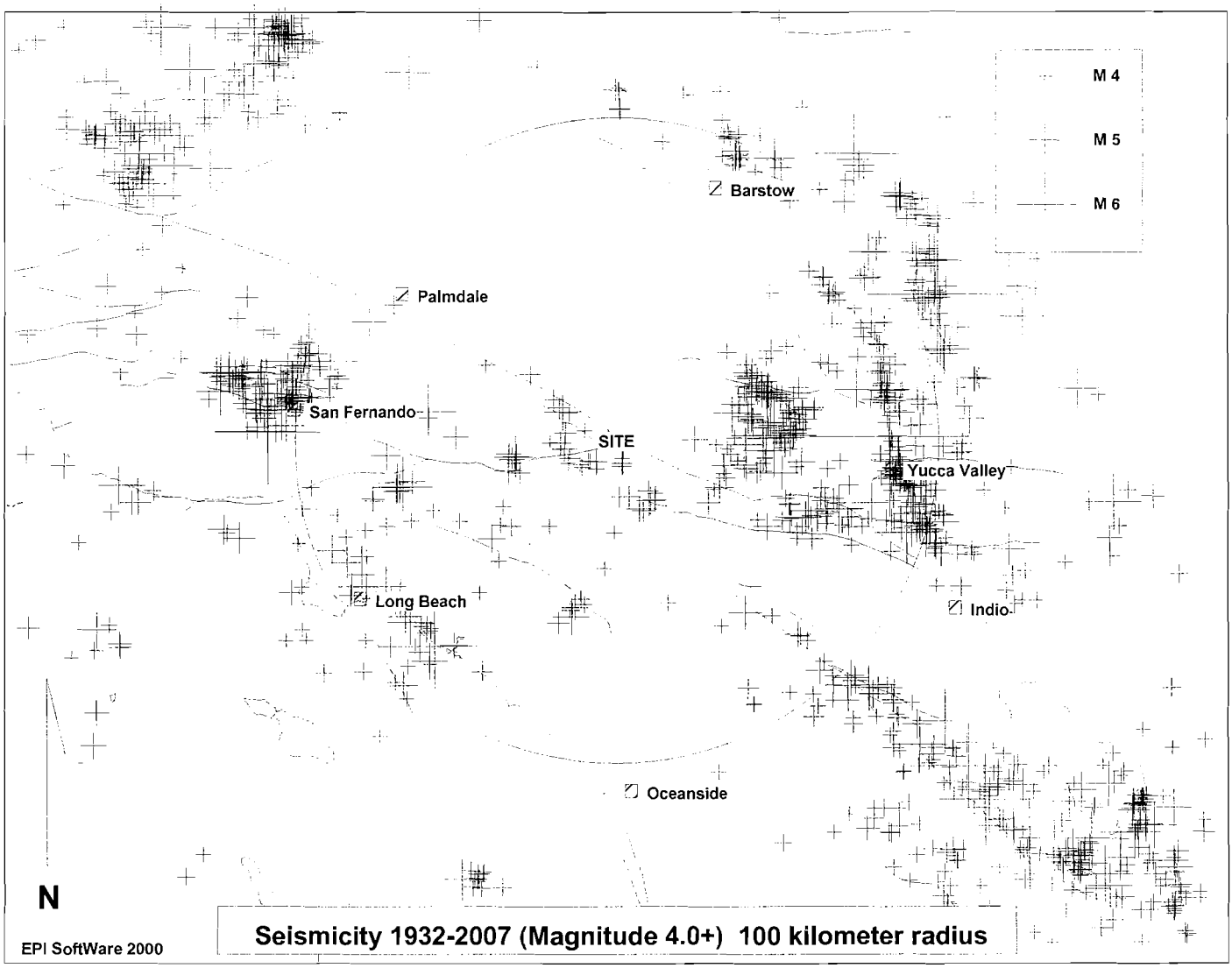
Qya_5 — **Young alluvial-valley deposits, Unit 5 (Holocene and late Pleistocene)**—Slightly dissected, slightly consolidated to cemented deposits of angular to subangular silt, sand and pebbles. Differs from Qya_4 by relative position in terrace sequence, consolidation, and surface dissection

Hypabyssal granitic dikes (Miocene)—Hornblende-biotite granodiorite in Shandin Hill area and leucocratic biotite-quartz-plagioclase porphyry in Perris Hill area. Granodiorite is medium and fine grained, porphyry is fine grained. Presumed to be early Miocene to late Oligocene because of similarity to dated rocks intruding Pelona Schist in other areas

Mzps — **Pelona Schist (Mesozoic)**—Muscovite-chlorite-albite-quartz schist; contains numerous, but volumetrically minor, layers of fine-grained quartzite and greenstone. Typically fine-grained, green, gray, and brown. Layering ranges from fine laminations to massive zones several meters thick. Layering is probably all transposed bedding, but locally some could be primary bedding; no primary sedimentary structures identified. Unit is highly susceptible to landsliding. Age is very poorly constrained; based primarily upon plutonic intrusions into Pelona (Mukasa and others, 1984; Silver and Nourse, 1986) which have ambiguous contact relations

DESCRIPTION OF GEOLOGIC UNITS (Miller, Matti, and Carson, 2001)

PROJECT:	682K DISTRIBUTION BUILDING, SAN BERNARDINO, CALIFORNIA	PROJECT NO.:	22448.1
CLIENT:	IDS REAL ESTATE	ENCLOSURE:	A-3
LOR Geotechnical Group, Inc.		DATE:	AUGUST 2007
		SCALE:	NO SCALE



SITE LOCATION: 34.187 LAT. -117.358 LONG.

MINIMUM LOCATION QUALITY: C

TOTAL # OF EVENTS ON PLOT: 1539

TOTAL # OF EVENTS WITHIN SEARCH RADIUS 653

MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

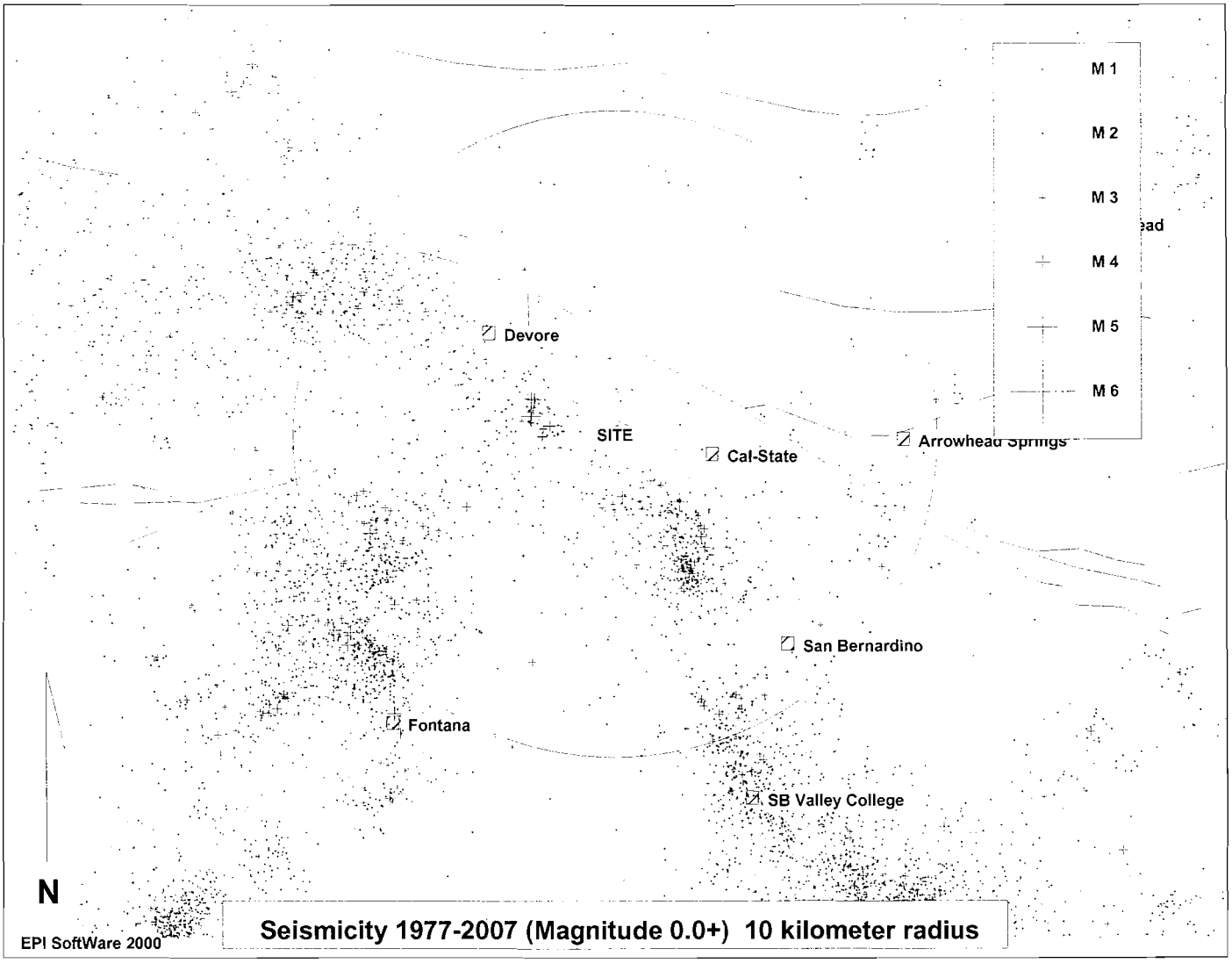
4.0- 4.9 : 592
5.0- 5.9 : 55
6.0- 6.9 : 5
7.0- 7.9 : 1
8.0- 8.9 : 0

CLOSEST EVENT: 4.3 ON THURSDAY, DECEMBER 28, 1989 LOCATED APPROX 3 KILOMETERS WEST OF THE SITE

LARGEST 5 EVENTS:

7.3 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 84 KILOMETERS EAST OF THE SITE
6.6 ON TUESDAY, FEBRUARY 09, 1971 LOCATED APPROX 98 KILOMETERS WEST OF THE SITE
6.4 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 48 KILOMETERS EAST OF THE SITE
6.4 ON SATURDAY, MARCH 11, 1933 LOCATED APPROX 84 KILOMETERS SOUTHWEST OF THE SITE
6.1 ON THURSDAY, APRIL 23, 1992 LOCATED APPROX 99 KILOMETERS EAST OF THE SITE

0 50 100
KILOMETERS



SITE LOCATION: 34.187 LAT. -117.358 LONG.

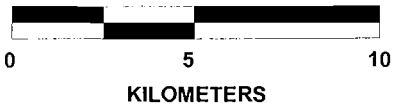
MINIMUM LOCATION QUALITY: A

TOTAL # OF EVENTS ON PLOT: 5481

TOTAL # OF EVENTS WITHIN SEARCH RADIUS 1610

MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

0.0- .9 : 108
1.0- 1.9 : 1229
2.0- 2.9 : 236
3.0- 3.9 : 34
4.0- 4.9 : 3
5.0- 5.9 : 0
6.0- 6.9 : 0
7.0- 7.9 : 0
8.0- 8.9 : 0



CLOSEST EVENT: 1.1 ON SUNDAY, OCTOBER 12, 1997 LOCATED APPROX .2 KILOMETER OF THE SITE

LARGEST 5 EVENTS:

4.3 ON THURSDAY, DECEMBER 28, 1989 LOCATED APPROX 2 KILOMETERS WEST OF THE SITE
4.3 ON THURSDAY, DECEMBER 28, 1989 LOCATED APPROX 2 KILOMETERS WEST OF THE SITE
4.2 ON SATURDAY, JUNE 28, 1997 LOCATED APPROX 2 KILOMETERS SOUTHEAST OF THE SITE
3.9 ON THURSDAY, MAY 24, 2007 LOCATED APPROX 2 KILOMETERS NORTHWEST OF THE SITE
3.9 ON THURSDAY, MAY 24, 2007 LOCATED APPROX 2 KILOMETERS NORTHWEST OF THE SITE

APPENDIX B

Field Investigation Program and Trench and Boring Logs

APPENDIX B
FIELD INVESTIGATION

Subsurface Exploration

Our subsurface exploration of the site consisted of drilling seven exploratory borings to depths ranging from approximately 30.5 to 60 feet using a CME drill rig on July 17, 2007and then excavating ten backhoe trenches on July 25, 2007 to depths of approximately 8 to 13 feet below the existing ground surface. The approximate locations of the trenches and borings are shown on Plate 1 within the back pocket of this report.

The drilling exploration was conducted using a CME-55 drill rig equipped with 8-inch diameter hollow stem augers. The soils were continuously logged by a staff geologist from this firm who inspected the site, maintained detailed logs of the borings, obtained undisturbed, as well as disturbed, soil samples for evaluation and testing, and classified the soils by visual examination in accordance with the Unified Soil Classification System.

Relatively undisturbed samples of the subsoils were typically obtained at a maximum interval of 5 feet. The samples were recovered by using a California split barrel sampler of 2.40-inch inside diameter and 3.25-inch outside diameter from the ground surface to the maximum depths attained. The samplers were driven by a 140-pound automatic trip hammer dropped from a height of 30 inches. The number of hammer blows required to drive the sampler into the ground the final 12 inches were recorded and further converted to an equivalent SPT N-values, which are included in the boring logs, Enclosures B-3 through B-9.

The undisturbed soil samples were retained in brass sample rings of 2.42 inches in diameter and 1.00 inch in height, and placed in sealed plastic containers. Disturbed soil samples were obtained at selected levels within the borings and placed in sealed containers for transport to our geotechnical laboratory.

The trenching exploration was conducted using a CAT 436 backhoe with a 24-inch bucket. The soils encountered were continuously logged by a staff geologist from this firm who visually observed the site, maintained detailed logs of the trenches, obtained disturbed soil samples for laboratory evaluation and testing, and classified the soils encountered by visual examination in accordance with the Unified Soil Classification System.

In-place density determinations were conducted at selected levels within the trenches utilizing the Nuclear Gage Method (ASTM D 2922). Disturbed soil samples were obtained at soil changes and other selected levels within the trenches. The samples were placed in sealed containers for transport to our geotechnical laboratory.

All samples obtained were taken to our geotechnical laboratory for storage and testing. Detailed logs of the trenches and borings are presented on the enclosed Trench and Boring Logs, Enclosures B-3 through B-19. A Boring/Trench log legend is presented on Enclosure B-1. A Unified Soil Classification chart is present as Enclosure B-2

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

PARTICLE SIZE LIMITS

BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

12" 3" 3/4" No. 4 No. 10 No. 40 200

(U.S. STANDARD SIEVE SIZE)

SOIL CLASSIFICATION CHART

PROJECT:	682K DISTRIBUTION BUILDING, SAN BERNARDINO, CALIFORNIA	PROJECT NO.:	22448.1
CLIENT:	IDS REAL ESTATE	ENCLOSURE:	B-2
LOR Geotechnical Group, Inc.		DATE:	AUGUST 2007

CONSISTENCY OF SOIL

SANDS

SPT BLOWS	CONSISTENCY
0-4	Very Loose
4-10	Loose
10-30	Medium Dense
30-50	Dense
Over 50	Very Dense

COHESIVE SOILS

SPT BLOWS	CONSISTENCY
0-2	Very Soft
2-4	Soft
4-8	Medium
8-15	Stiff
15-30	Very Stiff
30-60	Hard
Over 60	Very Hard

SAMPLE KEY

Symbol	Description
	INDICATES CALIFORNIA SPLIT SPOON SOIL SAMPLE
	INDICATES BULK SAMPLE
	INDICATES SAND CONE OR NUCLEAR DENSITY TEST
	INDICATES STANDARD PENETRATION TEST (SPT) SOIL SAMPLE

TYPES OF LABORATORY TESTS

1. Atterberg Limits
2. Consolidation
3. Direct Shear (undisturbed or remolded)
4. Expansion Index
5. Hydrometer
6. Organic Content
7. Proctor (4", 6", or Cal216)
8. R-value
9. Sand Equivalent
10. Sieve Analysis
11. Soluble Sulfate Content
12. Swell
13. Wash 200 Sieve

TRENCH AND BORING LOG LEGEND

PROJECT:	682K DISTRIBUTION BUILDING, SAN BERNARDINO, CALIFORNIA	PROJECT NO.:	22448.1
CLIENT:	IDS REAL ESTATE	ENCLOSURE:	B-1
LOR Geotechnical Group, Inc.		DATE:	AUGUST 2007

LOG OF BORING B-1

DEPTH IN FEET	ESTIMATED SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@0 feet FILL: SILTY SAND, approximately 5% gravel to 1/2", 20% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 15% silty fines, light brown, dry, loose
	18		1.7		120.1	■		SW SM	@2 feet ALLUVIUM: WELL GRADED SAND with Silt, approximately 25% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 10% silty fines, damp, brown
5	8		4.3		108.4	■		SM	
	19	3	1.4		112.4	■		SP	@5 feet, SILTY SAND, approximately 10% coarse grained sand, 30% medium grained sand, 45% fine grained sand, 15% silty fines, damp, brown
10	23		3.2		112.6	■		SW	@7 feet, POORLY GRADED SAND, Trace Gravel to 1/2", about 15% coarse grained sand, 25% medium grained sand, 55% fine grained sand, 5% silty fines, damp, brown.
15	19		3.1		120.8	■			@10 feet, WELL GRADED SAND, approximately 10% gravel to 3/4", 25% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 5% silty fines, damp, brown
20	19		4.4		111.4	■		SM	@20 feet, SILTY SAND, approximately 5% gravel to 1/2", 20% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 15% silty fines, damp, brown
25	16		6.5		117.8	■			
30	28		4.9		117.7	■			
35	31		3.9		108.0	■		SW	@35 feet WELL GRADED SAND, approximately 30% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 5% silty fines, tan, damp
40	59		1.7		111.2	■			
45	38		2.0		104.1	■			
50	33		9.7		121.7	■		SM	@50 feet, SILTY SAND, approximately 15% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 30% silty fines, damp
55									END OF BORING

Fill from 0-2 feet
No Groundwater
No Bedrock

PROJECT:	682k Distribution Building	PROJECT NUMBER:	22448.1
CLIENT:	IDS Real Estate Group	ELEVATION:	1670
		DATE DRILLED:	July 17, 2007
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-3

LOR GEOTECHNICAL GROUP INC.

LOG OF BORING B-2

DEPTH IN FEET	ESTIMATED SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0									@0 feet BEDROCK: PELONA SCHIST, fine grained, weathered, friable, red-brown
									@3 feet, becomes less weathered, light green
5	29-5"		1.9		114.6				
10									
15									@15 feet, becomes slightly harder, relatively easy to drill
20									
25									@25 feet, becomes much harder, moderate drilling
30									
35									END OF BORING No fill No groundwater Bedrock 0-35 feet

PROJECT:	682k Distribution Building	PROJECT NUMBER:	22448.1
CLIENT:	IDS Real Estate Group	ELEVATION:	1722
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	July 17, 2007
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-4

LOG OF BORING B-3

DEPTH IN FEET	ESTIMATED SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SW	@0 feet FILL: WELL GRADED SAND, approximately 5% cobble and gravel, 25% coarse grained sand, 30% medium grained sand, 35% find grained sand, 5% silty fines, dry, tan, loose
	20		2.0		107.9			SW	@2 feet ALLUVIUM: WELL GRADED SAND, approximately 10% cobble with gravel, 25% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 5% silty fines.
5	25		2.2		114.0				
	49		1.8		123.9				
10	33		3.6		117.3				
15	21		5.4		106.3				@15 feet, becoming slightly finer grained, damp, brown
20	31		8.2		115.6			SP SM	@20 feet, POORLY GRADED SAND with Silt, approximately 5% coarse grained sand, 15% medium grained sand, 70% fine grained sand, 10% silty fines, red-brown, damp, some iron oxide staining
25	48		12.9		117.0			SC	@25 feet, CLAYEY SAND/SANDY CLAY approximately 15% medium grained sand, 35% fine grained sand, 50% clayey fines, red-brown-, moist
30	34		14.1		118.3				
35									END OF BORING Fill from 0-2 feet No Groundwater No Bedrock

PROJECT:	682k Distribution Building	PROJECT NUMBER:	22448.1
CLIENT:	IDS Real Estate Group	ELEVATION:	1679
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	July 17, 2007
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-5

LOG OF BORING B-4

DEPTH IN FEET	ESTIMATED SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0	35		3.1		114.6			SM	@0 feet FILL SILTY SAND, approximately 10% gravel to 1/2 ", 20% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 15% silty fines, tan, dry, loose
5	13	3	4.0		106.6			SP SM	@5 feet, ALLUVIUM: POORLY GRADED SAND with Silt, approximately 15% coarse grained sand, 25% medium grained sand, 50% fine grained sand, 10% silty fines, brown, damp
	14	2	4.1		105.3				
10	19		2.1		108.0			SW	@10 feet, WELL GRADED SAND, approximately 25% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 5% silty fines, dry, tan
	22		3.1 4.7		100.4 102.5			SP	@12 feet, POORLY GRADED SAND, approximately 15% coarse grained sand, 20% medium grained sand, 60% fine grained sand, 5% silty fines, brown, damp
15	19								
20	50		3.7		103.3				
25	24		6.3		109.6			SM	@25 feet, SILTY SAND, approximately 5% coarse grained sand, 10% medium grained sand, 70% fined grained sand, 15% silty fines
30	32		6.0		108.0				
35									END OF BORING Fill from 0-5 feet No Groundwater No Bedrock

PROJECT:	682k Distribution Building	PROJECT NUMBER:	22448.1
CLIENT:	IDS Real Estate Group	ELEVATION:	1671
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	July 17, 2007
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-6

LOG OF BORING B-5

DEPTH IN FEET	ESTIMATED SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SW	@0 feet, FILL: WELL GRADED SAND, approximately 25% coarse graded sand, 35% medium graded sand, 35% fine graded sand, 5% silty fines, tan, dry, loose
	11		1.3		108.6			SM	@2 feet, ALLUVIUM: SILTY SAND, approximately 5% gravel to 1/2", 20% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 15% silty fines, dry, tan
5	18		2.4		107.2				
	22		2.0		108.1			SW	@7 feet, WELL GRADED SAND, approximately 5% gravel to 1/2", 30% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 5% silty fines, tan, dry
10	19		2.1						
	19		8.2		119.0			SM	@12 feet, SILTY SAND, approximately 10% coarse grained sand 30% medium grained sand, 40% fine grained sand, 20% silty fines, brown, damp
15	44		2.7		125.1			SW	@15 feet, WELL GRADED SAND, approximately 25% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 5% silty fines, brown, damp
20	70		1.9		111.6			SW	@20 feet, WELL GRADED SAND with gravel, approximately 20% gravel to 2 1/2", 20% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 5% silty fines, light brown, dry
25	44-5"		1.9		116.8				
30	44-4"								@30 feet no recovery.
									END OF BORING
35									

Fill from 0-2 feet
No groundwater
No bedrock

PROJECT:	682k Distribution Building	PROJECT NUMBER:	22448.1
CLIENT:	IDS Real Estate Group	ELEVATION:	1648
		DATE DRILLED:	July 17, 2007
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-7

LOG OF BORING B-6

DEPTH IN FEET	ESTIMATED SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SW SM	@0 feet, FILL: WELL GRADED SAND WITH SILT, approximately 10% gravel to 1 1/2", 25% coarse grained sand 25% medium grained sand, 30% fine grained sand, 10% silty fines, tan, dry, loose
	19		3.7		108.6			SP SM	@2 feet, ALLUVIUM: POORLY GRADED SAND WITH SILT, approximately 10% coarse grained sand, 15% medium grained sand, 65% fine grained sand, 10% silty fines, red-brown, damp
5	21		4.5		110.0				
	21		6.1		114.3				
10	19	2	8.1		112.2			SM	@10 feet, SILTY SAND, approximately 5% coarse grained sand, 10% medium grained sand, 65% fine grained sand, 20% silty fines, red-brown, damp
	22		8.4		113.0				
15	24		9.0		115.3				
20	20		10.3		121.5				@20 feet, SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 50% fine grained sand, 30% silty fines, red-brown, damp, trace pinhole porosity
			10.1		121.8				
25	17								@25 feet, porosity no longer visible
30	28		5.0		104.6			SP	@30 feet, POORLY GRADED SAND, approximately 5% coarse grained sand, 10% medium grained sand, 80% fine grained sand, 5% silty fines, tan, damp
35	30		8.8		110.4				

END OF BORING

Fill from 0-2 feet
No Groundwater
No Bedrock

PROJECT:	682k Distribution Building	PROJECT NUMBER:	22448.1
CLIENT:	IDS Real Estate Group	ELEVATION:	1641
		DATE DRILLED:	July 17, 2007
		EQUIPMENT:	CME 55
		HOLE DIA.: 8"	ENCLOSURE: B-8

LOR GEOTECHNICAL GROUP INC.

LOG OF BORING B-7

DEPTH IN FEET	TEST DATA: ESTIMATED SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0									@0 feet, BEDROCK: **PELONA SCHIST, moderately weathered, fine grained, red-brown, friable**
5									
									@ 7 feet, **becomes less weathered, hard, moderate drilling, green**
10									
15									
20									
25									
30									
35									
40									
									@43 feet, **becomes somewhat difficult to drill**
45									
50									
55									@55 feet, **becomes difficult to drill**
60									**END OF BORING DUE TO SLOW PROGRESS**
65									**No Fill** **No Groundwater** **Bedrock from 0-60 feet**

PROJECT:	**682k Distribution Building**	PROJECT NUMBER:	**22448.1**
CLIENT:	**IDS Real Estate Group**	ELEVATION:	**1785**
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	**July 17, 2007**
		EQUIPMENT:	**CME 55**
		HOLE DIA.: **8"**	ENCLOSURE: **B-9**

LOG OF TRENCH T-1

DEPTH IN FEET	TEST DATA: LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0				0.5				SW SM	@ 0 feet FILL: WELL GRADED GRAVEL with silt, approximately 10% gravel to 1/2", 25% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 10% silty fines, light brown, dry, loose.
			89	1.1	107.5			SP SM	@ 1 foot ALLUVIUM: POORLY GRADED SAND with silt, approximately 15% coarse grained sand, 35% medium grained sand, 40% fine grained sand, 10% silty fines, light brown, dry.
	7								
			84	1.1	101.5			SW	@ 3.5 feet WELL GRADED SAND, approximately 25% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 5% silty fines, light brown, dry, caving.
5	3, 7, 11								
								SM	@ 8 feet SILTY SAND, approximately 15% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 15% silty fines, brown, damp, no caving.
10									
									END OF TRENCH Fill from 0 to 1 foot Caving from 3.5 to 8 feet No groundwater No bedrock
15									

PROJECT:	682k Distribution Building	PROJECT NUMBER:	22448.1
CLIENT:	IDS Real Estate Group	ELEVATION:	1675
		DATE EXCAVATED:	July 25, 2007
		EQUIPMENT:	CAT 436
		BUCKET W.: 24"	ENCLOSURE: B-10

LOR GEOTECHNICAL GROUP INC.

LOG OF TRENCH T-2

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0	8, 9, 10			0.5				SW SM SM	@ 0 feet FILL: WELL GRADED SAND with silt, approximately 5% gravel to 1/2", 25% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 10% silty fines, light brown, dry, loose. @ 1 foot ALLUVIUM: SILTY SAND, approximately 5% gravel to 1/2", 10% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 25% silty fines, tan, dry. @ 2 feet occasional cobbles to 10", caving to 4.5 feet, faint, horizontal bedding 1/2" thick.
			85	1.1	107.7				
5			93	1.1	112.4			SP	@ 4.5 feet POORLY GRADED SAND, approximately 5% gravel to 1/2", 10% coarse grained sand, 30% medium grained sand, 50% fine grained sand, 5% silty fines, light brown, dry.
10									@ 10 feet becomes slightly coarser grained.
									END OF TRENCH Fill from 0 to 1 foot Caving from 2 to 4.5 feet No groundwater No bedrock
15									

PROJECT:	682k Distribution Building	PROJECT NUMBER:	22448.1
CLIENT:	IDS Real Estate Group	ELEVATION:	1665
LOR GEOTECHNICAL GROUP INC.		DATE EXCAVATED:	July 25, 2007
		EQUIPMENT:	CAT 436
		BUCKET W.: 24"	ENCLOSURE: B-11

LOG OF TRENCH T-3

DEPTH IN FEET	TEST DATA: LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0	9, 10			0.3				SW SM SW	@ 0 feet FILL: WELL GRADED SAND with gravel and silt, approximately 15% gravel to 3", 25% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 10% silty fines, light brown, dry, loose.
			93	1.8	112.2				@ 1 foot ALLUVIUM: WELL GRADED SAND, approximately 5% gravel to 1/2", 30% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 5% silty fines, tan, dry. @ 2 feet minor caving to 12 feet.
			91	0.5	109.8				@ 4 feet trace cobbles to 4".
5									@ 6 feet becomes damp.
10									
									END OF TRENCH Fill from 0 to 1 foot Caving from 2 to 12 feet (minor) No groundwater No bedrock
15									

PROJECT:	682k Distribution Building	PROJECT NUMBER:	22448.1
CLIENT:	IDS Real Estate Group	ELEVATION:	1684
LOR GEOTECHNICAL GROUP INC.		DATE EXCAVATED:	July 25, 2007
		EQUIPMENT:	CAT 436
		BUCKET W.: 24"	ENCLOSURE: B-12

LOG OF TRENCH T-4

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0				0.3				SW SM SW	@ 0 feet FILL: WELL GRADED SAND with silt, approximately 10% gravel to 1", 25% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 10% silty fines, light brown, dry, loose. @ 1 foot ALLUVIUM: WELL GRADED SAND with gravel, approximately 15% gravel to 1", 25% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 5% silty fines, tan, damp. @ 2 feet thin 1 to 1/2" thick horizontal bedding.
	7		89	1.4	110.1				
5			87	1.8	107.8				@ 6 feet occasional cobbles to 10". @ 9 feet becomes slightly finer grained.
10									
15									END OF TRENCH Fill from 0 to 1 foot No caving No groundwater No bedrock

PROJECT:	682k Distribution Building	PROJECT NUMBER:	22448.1
CLIENT:	IDS Real Estate Group	ELEVATION:	1675
		DATE EXCAVATED:	July 25, 2007
		EQUIPMENT:	CAT 436
		BUCKET W.: 24"	ENCLOSURE: B-13

LOR GEOTECHNICAL GROUP INC.

LOG OF TRENCH T-5

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0				0.2				SP SM	@ 0 feet AEOLIAN SAND: **POORLY GRADED SAND with silt, approximately 15% medium grained sand, 75% fine grained sand, 10% silty fines, tan, dry, loose.**
	3, 7, 11		85	2.0	100.8			SP	@ 2 feet **POORLY GRADED SAND, trace coarse grained sand, 15% medium grained sand, 80% fine grained sand, 5% silty fines, tan, dry.**
			87	1.3	103.3				
5									
10									
									END OF TRENCH **No fill** **No caving** **No groundwater** **No bedrock**
15									

PROJECT:	**682k Distribution Building**	PROJECT NUMBER:	**22448.1**
CLIENT:	**IDS Real Estate Group**	ELEVATION:	**1687**
LOR GEOTECHNICAL GROUP INC.		DATE EXCAVATED:	**July 25, 2007**
		EQUIPMENT:	**CAT 436**
		BUCKET W.: **24"**	ENCLOSURE: **B-14**

LOG OF TRENCH T-6

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0				0.0				SW SM	@ 0 feet FILL: **WELL GRADED SAND with silt, approximately 30% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 10% silty fines, light brown, dry, loose.**
								SP	@ 1 foot AEOLIAN SAND: **POORLY GRADED SAND, approximately 15% medium grained sand, 80% fine grained sand, 5% silty fines, tan, dry.**
			80	1.9	95.0				
5				20.7	83.4				
								SW	@ 7 feet ALLUVIUM: **WELL GRADED SAND, approximately 5% gravel to 1/2", 20% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 5% silty fines, tan, damp.**
10									
								SP	@ 12 feet **POORLY GRADED SAND, approximately 25% medium grained sand, 70% fine grained sand, 5% silty fines, brown, damp, 1/2" horizontal bedding.**
									END OF TRENCH **Fill from 0 to 1 foot** **No caving** **No groundwater** **No bedrock**
15									

PROJECT:	**682k Distribution Building**	PROJECT NUMBER:	**22448.1**
CLIENT:	**IDS Real Estate Group**	ELEVATION:	**1657**
LOR GEOTECHNICAL GROUP INC.		DATE EXCAVATED:	**July 25, 2007**
		EQUIPMENT:	**CAT 436**
		BUCKET W.: **24"**	ENCLOSURE: **B-15**

LOG OF TRENCH T-7

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0				0.3				SM	@ 0 feet FILL: **SILTY SAND, approximately 5% gravel to 1/2", 20% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 15% silty fines, light brown, dry.**
								SM	@ 1 foot <<B>**ALLUVIUM**: SILTY SAND, approximately 20% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 15% silty fines, light brown, dry.
			81	2.2	102.4				**@ 2.5 feet minor caving to 12 feet.**
	3, 7, 11								
			94	1.6	113.8			SW	**@ 4 feet WELL GRADED SAND, approximately 5% gravel to 1/2", 30% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 5% silty fines, tan, dry, 1 to 2" thick horizontal bedding.**
5									**@ 6 feet some gravel to 1".**
				1.8					**@ 9 feet occasional cobbles to 8".**
10									
									END OF TRENCH **Fill fro 0 to 1 foot** **Caving from 2.5 to 12 feet (minor)** **No groundwater** **No bedrock**
15									

PROJECT:	**682k Distribution Building**	PROJECT NUMBER:	**22448.1**
CLIENT:	**IDS Real Estate Group**	ELEVATION:	**1644**
LOR GEOTECHNICAL GROUP INC.		DATE EXCAVATED:	**July 25, 2007**
		EQUIPMENT:	**CAT 436**
		BUCKET W.: **24"**	ENCLOSURE: **B-16**

LOG OF TRENCH T-8

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0				0.2				SM	@ 0 feet FILL: SILTY SAND, approximately 25% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 15% silty fines, light brown, dry, loose.
								SP SM	@ 0.5 feet ALLUVIUM: POORLY GRADED SAND with silt, approximately 13% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 12% silty fines, light brown, dry.
			87	1.1	105.4				
			95	1.6	114.4				
5								SM	@ 6 feet SILTY SAND, approximately 15% coarse grained sand, 35% medium grained sand, 30% fine grained sand, 20% silty fines, brown, damp. @ 7 feet occasional gravel to 3".
10									
									END OF TRENCH Fill from 0 to 1 foot No caving No groundwater No bedrock
15									

PROJECT:	682k Distribution Building	PROJECT NUMBER:	22448.1
CLIENT:	IDS Real Estate Group	ELEVATION:	1650
LOR GEOTECHNICAL GROUP INC.		DATE EXCAVATED:	July 25, 2007
		EQUIPMENT:	CAT 436
		BUCKET W.: 24"	ENCLOSURE: B-17

LOG OF TRENCH T-9

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0				0.2				SM SP	@ 0 feet TOPSOIL: **SILTY SAND, approximately 15% coarse grained sand, 25% medium grained sand, 45% fine grained sand, 15% silty fines, brown, dry, loose.** @ 0.5 feet ALLUVIUM: **POORLY GRADED SAND, approximately 10% coarse grained sand, 25% medium graine sand, 60% fine grained sand, 5% silty fines, tan, dry, massive.**
			83	0.7	98.5				
5			85	0.8	100.9				
10									@ 10 feet becomes slightly coarser grained, damp.
									END OF TRENCH **No fill** **No caving** **No groundwater** **No bedrock**
15									

PROJECT:	**682k Distribution Building**	PROJECT NUMBER:	**22448.1**
CLIENT:	**IDS Real Estate Group**	ELEVATION:	**1672**
LOR GEOTECHNICAL GROUP INC.		DATE EXCAVATED:	**July 25, 2007**
		EQUIPMENT:	**CAT 436**
		BUCKET W.: **24"**	ENCLOSURE: **B-18**

LOG OF TRENCH T-10

DEPTH IN FEET	TEST DATA: LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0	3, 7, 11								@ 0 feet BEDROCK: Pelona Schist, highly weathered, fine grained, recovers as gravel to cobble size, angular fragments, dry, friable, fractured, foliated.
									@ 3 feet becomes less weathered, somewhat difficult to excavate, somewhat friable, some caving.
									@ 4 feet becomes much less weathered, difficult to excavate.
5									@ 6 feet becomes very difficult to excavate.
									END OF TRENCH DUE TO PRACTICAL REFUSAL No fill Caving at 3 feet No groundwater Bedrock at 0 feet
10									
15									

PROJECT:	682k Distribution Building	PROJECT NUMBER:	22448.1
CLIENT:	IDS Real Estate Group	ELEVATION:	1719
LOR GEOTECHNICAL GROUP INC.		DATE EXCAVATED:	July 25, 2007
		EQUIPMENT:	CAT 436
		BUCKET W.: 24"	ENCLOSURE: B-19

APPENDIX C

Laboratory Testing Program and Test Results

APPENDIX C
LABORATORY TESTING

General

Selected soil samples obtained from the trenches and borings were tested in our laboratory to evaluate the physical properties of the soils affecting foundation design and construction procedures. The laboratory testing program performed in conjunction with our investigation included moisture content, dry density, laboratory compaction, direct shear, consolidation, sieve analysis, sand equivalent, R-value, and soluble sulfate content. Descriptions of the laboratory tests are presented in the following paragraphs.

Moisture-Density Tests

The moisture content and dry density information provides an indirect measure of soil consistency for each stratum, and can also provide a correlation between soils on this site. The dry unit weight and field moisture content were determined for selected undisturbed samples, and the results are shown on the trench and boring logs, Enclosures B-3 through B-19, for convenient correlation with the soil profile.

Laboratory Compaction

Selected soil samples were tested in the laboratory to determine compaction characteristics using the ASTM D 1557-00 compaction test method. The results are presented in the following table:

LABORATORY COMPACTION				
Trench Number	**Sample Depth (feet)**	**Material Description (U.S.C.S.)**	**Maximum Dry Density (pcf)**	**Optimum Moisture Content (percent)**
T-1	2-3	(SP-SM) Poorly Graded Sand with silt	121.0	11.0
T-1	4-5	(SW) Well Graded Sand	120.5	5.5
T-4	3-4	(SW) Well Graded Sand with gravel	123.5	9.5
T-5	2-3	(SP) Poorly Graded Sand	118.5	9.5
T-7	2-3	(SM) Silty Sand	127.0	9.5
T-10	1-2	Remolded Bedrock	132.5	7.5

Direct Shear Tests

Shear tests are performed with a direct shear machine at a constant rate-of-strain (usually 0.04 inches/minute). The machine is designed to test a sample partially extruded from a sample ring in single shear. Samples are tested at varying normal loads in order to evaluate the shear strength parameters, angle of internal friction and cohesion. Samples are tested in a remolded (r) or undisturbed (u) state and soaked, to represent the worst-case conditions expected in the field. The results of the shear tests are presented in the following table.

The results of the shear tests are presented in the following table:

DIRECT SHEAR TESTS				
Trench/ Boring Number	**Sample Depth (feet)**	**Material Description (U.S.C.S.)**	**Angle of Internal Friction (degrees)**	**Apparent Cohesion (psf)**
T-1	4-5 (r)	(SW) Well Graded Sand	32	0
T-5	2-3 (r)	(SP) Poorly Graded Sand	31	20
T-7	2-3 (r)	(SM) Silty Sand	31	100
T-10	1-2 (r)	Remolded Bedrock	30	150
B-1	7 (u)	(SP) Poorly Graded Sand	34	180
B-4	5 (u)	(SM) Silty Sand	32	0

Consolidation Tests

The apparatus used for the consolidation tests (odometer) is designed to test a one-inch high portion of the undisturbed soil sample as contained in a sample ring. Porous stones and filler paper are placed in contact with the top and bottom of the specimen to permit the addition or release of water. Loads are applied to the test specimen in specified increments and the resulting axial deformations are recorded. The results are plotted as log of axial pressure versus consolidation or compression, expressed as strain or sample height. The rest of our tests are provided on Enclosure C-1 and C-2.

Sieve Analysis

A quantitative determination of the grain size distribution was performed for selected samples in accordance with the ASTM D 422 laboratory test procedure. The determination is performed by passing the soil through a series of sieves, and recording the weights of retained particles on each screen. The results of the sieve analyses are presented graphically on Enclosure C-3.

Sand Equivalent

The sand equivalent of selected subgrade soils were evaluated using the California Sand Equivalent Test Method, Caltrans Number 217. The results of the sand equivalent tests are presented with the grain size distribution analyses on Enclosure C-3.

R-Value Test

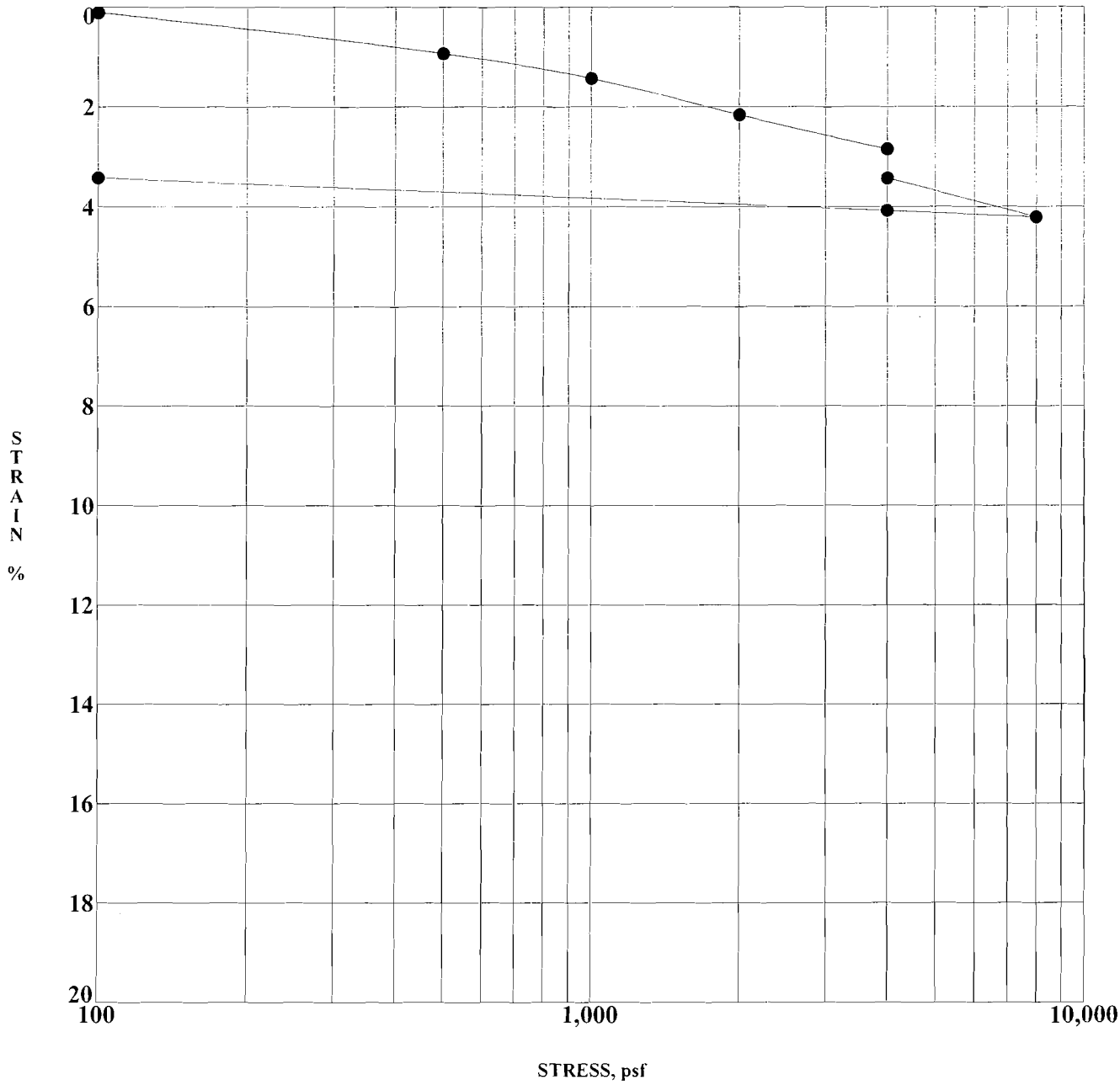
Soil samples were obtained at probable pavement subgrade level and sieve analysis and sand equivalent tests were conducted. Based on these indicator tests, a selected soil sample was tested to determine its R-value using the California R-Value Test Method, Caltrans Number 301. The results of the sieve analysis, sand equivalent, and R-value tests are presented on Enclosure C-3.

Soluble Sulfate Content Tests

The soluble sulfate content of selected subgrade soils were evaluated. The concentration of soluble sulfates in the soils was determined by measuring the optical density of a barium sulfate precipitate. The precipitate results from a reaction of barium chloride with water extractions from the soil samples. The measured optical density is correlated with readings on precipitates of known sulfate concentrations. The test results are presented on the following table:

SOLUBLE SULFATE CONTENT TESTS

Trench Number	Sample Depth (feet)	Material Description (U.S.C.S.)	Sulfate Content (% by weight)
T-1	4-5	(SW) Well Graded Sand	<0.005
T-5	2-3	(SP) Poorly Graded Sand	<0.005
T-7	2-3	(SM) Silty Sand	<0.005
T-10	1-2	Remolded Bedrock	<0.005



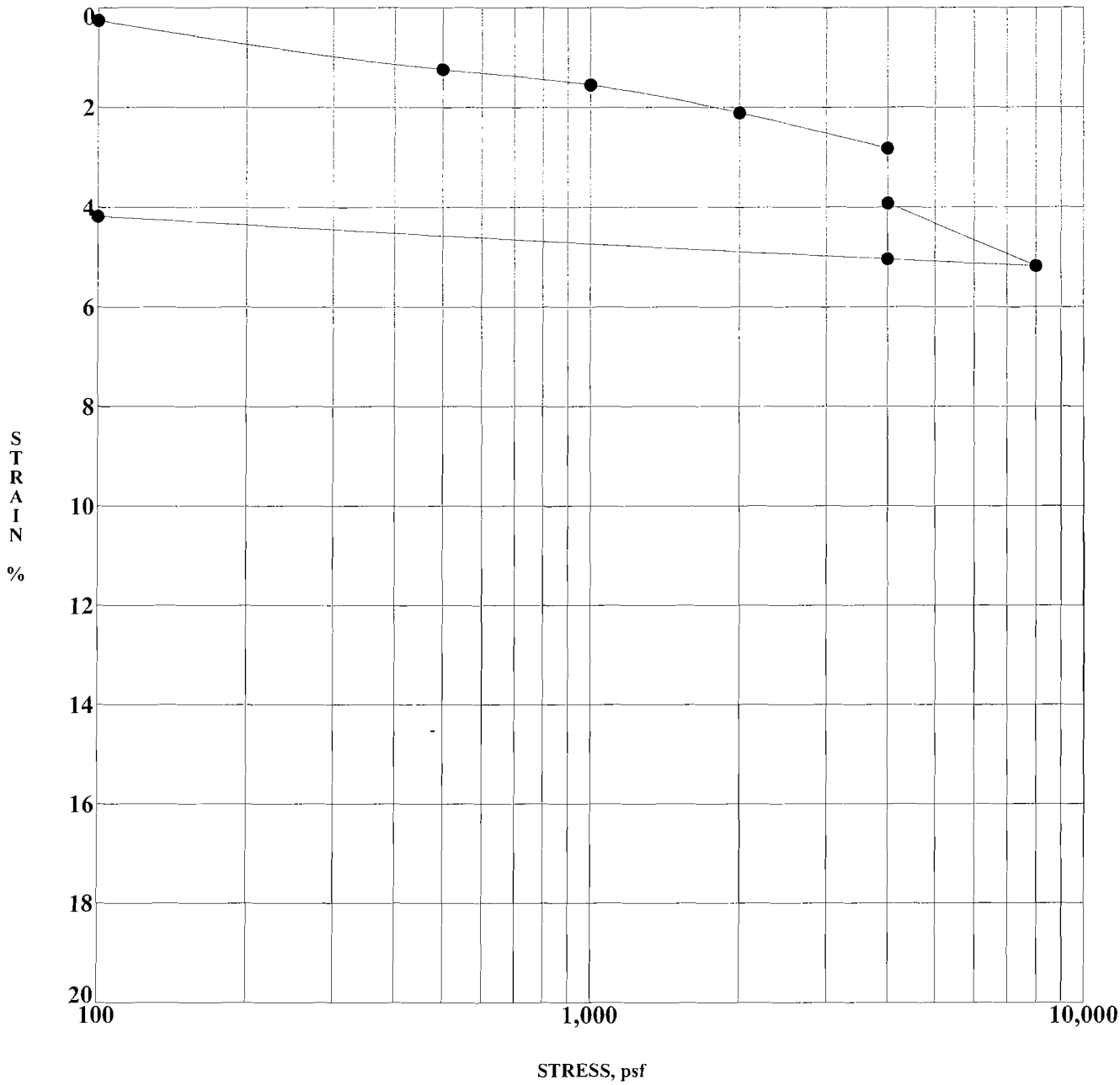
	Specimen Identification		Classification	DD	MC%
●	B-4	7.0	(SP-SM) Poorly Graded Sand with Silt	99	5
⊠					
▲					
★					
✕					
✪					

PROJECT 682k Distribution Building - San Bernardino, California

PROJECT NO. 22448.1
DATE 8/1/07

CONSOLIDATION TEST
LOR GEOTECHNICAL GROUP, INC.
RIVERSIDE, CALIFORNIA 92507

Enclosure C-1



Specimen Identification		Classification	DD	MC%
● B-6	10.0	(SM) Silty Sand	108	8

PROJECT *682k Distribution Building - San Bernardino, California*

PROJECT NO. *22448.1*

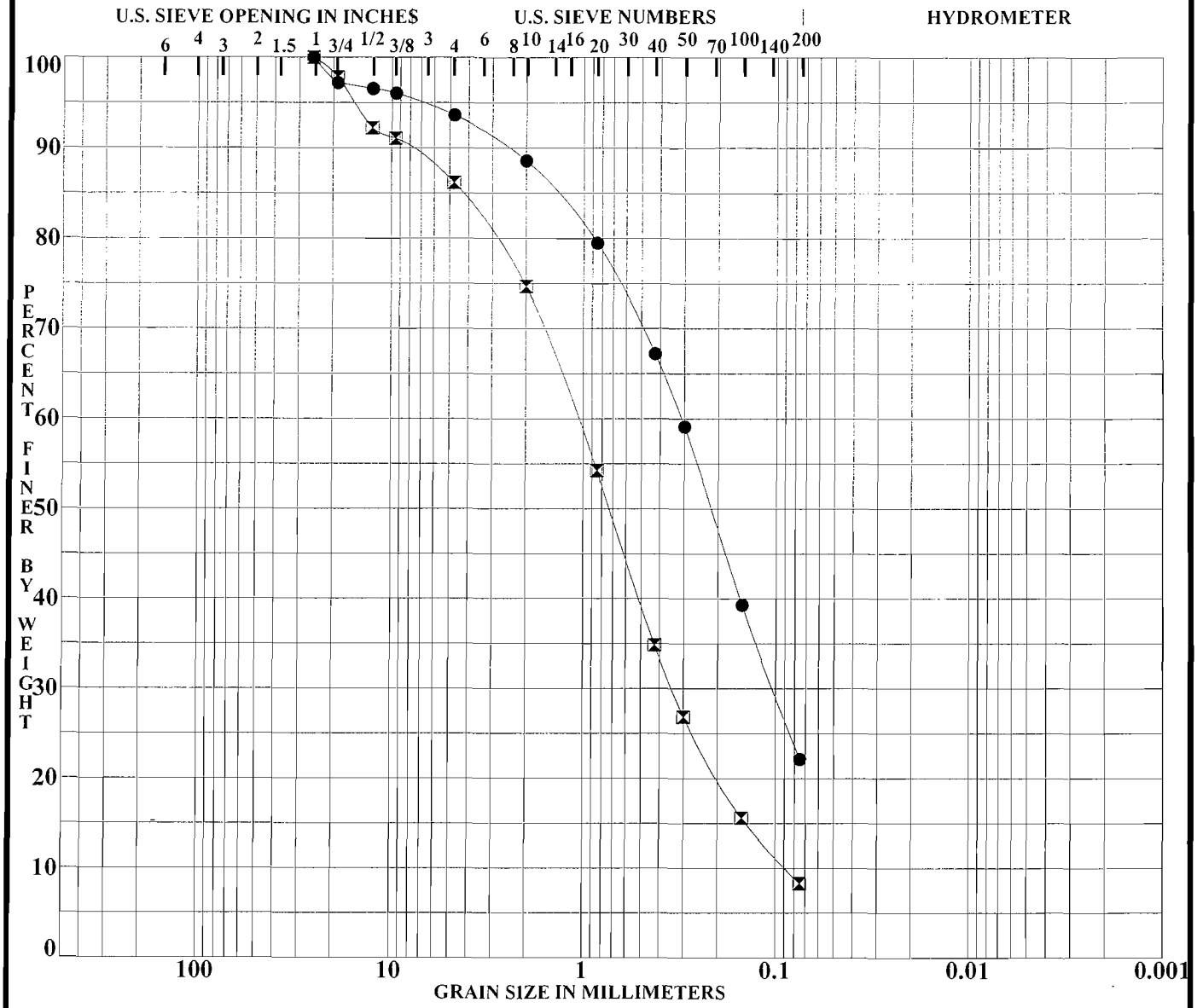
DATE *8/1/07*

CONSOLIDATION TEST

LOR GEOTECHNICAL GROUP, INC.

RIVERSIDE, CALIFORNIA 92507

Enclosure C-2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	SE	RV			Cc	Cu
●	T-02	@ 0-3 ft.	(SM) Silty Sand	29	73				
⧗	T-03	@ 0-3 ft.	(SW-SM) Well Graded Sand with silt	51	--			1.24	12.3

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	T-02	@ 0-3 ft.	25.40	0.31	0.103		6.4	71.5	22.1	
⧗	T-03	@ 0-3 ft.	25.40	1.08	0.344	0.0883	13.8	77.9	8.3	

PROJECT *682k Distribution Building - San Bernardino, California*

PROJECT NO. *22448.1*
DATE *8/1/07*

GRADATION CURVES

Enclosure C-3

LOR GEOTECHNICAL GROUP, INC.

APPENDIX D

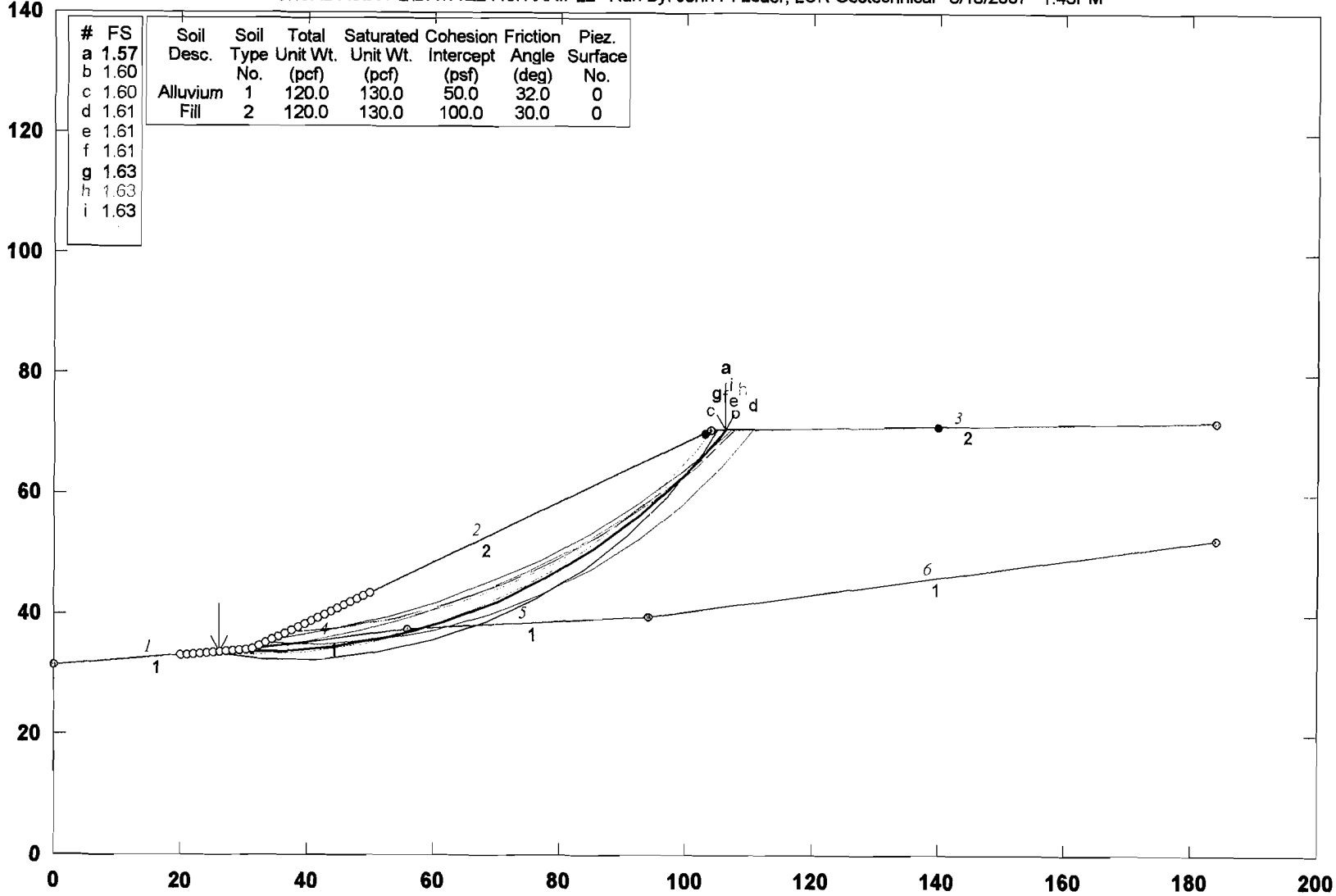
Slope Stability Calculations

Section XX' - static Analysis

C:\GABY\SLOPE\DATA\22448.1\XX.PL2 Run By: John P. Leuer, LOR Geotechnical 8/13/2007 1:43PM

#	FS
a	1.57
b	1.60
c	1.60
d	1.61
e	1.61
f	1.61
g	1.63
h	1.63
i	1.63

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Alluvium	1	120.0	130.0	50.0	32.0	0
Fill	2	120.0	130.0	100.0	30.0	0



PCSTABL5M/si FSmin=1.57

Safety Factors Are Calculated By The Modified Bishop Method

STED

```
                    ** PCSTABL5M **
                          by
                   Purdue University
             --Slope Stability Analysis--
          Simplified Janbu, Simplified Bishop
             or Spencer`s Method of Slices
Run Date:                  8/13/2007
Time of Run:               1:43PM
Run By:                    John P. Leuer, LOR Geotechnical
Input Data Filename:       C:XX.IN
Output Filename:           C:XX.OUT
Unit:                      ENGLISH
Plotted Output Filename:   C:XX.PLT
PROBLEM DESCRIPTION   Section XX' - static Analysis
BOUNDARY COORDINATES
    3 Top   Boundaries
    6 Total Boundaries
Boundary     X-Left     Y-Left    X-Right    Y-Right    Soil Type
   No.        (ft)       (ft)       (ft)       (ft)     Below Bnd
    1          .00      31.50      31.20      34.10        1
    2        31.20      34.10     104.00      70.50        2
    3       104.00      70.50     184.00      72.00        2
    4        31.20      34.10      55.80      37.50        1
    5        55.80      37.50      94.00      39.50        1
    6        94.00      39.50     184.00      52.50        1
ISOTROPIC SOIL PARAMETERS
 2 Type(s) of Soil
Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
 No.  (pcf)    (pcf)     (psf)     (deg)   Param.   (psf)    No.
  1   120.0    130.0      50.0     32.0     .00       .0      0
  2   120.0    130.0     100.0     30.0     .00       .0      0
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
900 Trial Surfaces Have Been Generated.
 30 Surfaces Initiate From Each Of 30 Points Equally Spaced
Along The Ground Surface Between  X =  20.00 ft.
                              and  X =  50.00 ft.
Each Surface Terminates Between   X = 103.00 ft.
                             and   X = 140.00 ft.
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is  Y =   .00 ft.
 9.00 ft. Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Examined.  They Are Ordered - Most Critical
First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Failure Surface Specified By 12 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1        26.21       33.68
    2        35.21       33.63
    3        44.17       34.44
    4        53.02       36.09
    5        61.67       38.58
    6        70.04       41.88
    7        78.06       45.96
    8        85.66       50.78
    9        92.76       56.31
   10        99.31       62.49
   11       105.24       69.26
   12       106.16       70.54
Circle Center At X =   31.3 ; Y =  127.8  and Radius,   94.2
      ***     1.574   ***
```

Individual data on the 15 slices

Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
5.0	133.2	.0	.0	.0	.0	.0	.0	.0
4.0	701.1	.0	.0	.0	.0	.0	.0	.0
9.0	4636.2	.0	.0	.0	.0	.0	.0	.0
8.8	7997.7	.0	.0	.0	.0	.0	.0	.0
2.8	3076.0	.0	.0	.0	.0	.0	.0	.0
2.6	3040.1	.0	.0	.0	.0	.0	.0	.0
3.3	4093.8	.0	.0	.0	.0	.0	.0	.0
8.4	11252.8	.0	.0	.0	.0	.0	.0	.0
8.0	11173.4	.0	.0	.0	.0	.0	.0	.0
7.6	10082.0	.0	.0	.0	.0	.0	.0	.0
7.1	8147.8	.0	.0	.0	.0	.0	.0	.0
6.5	5591.9	.0	.0	.0	.0	.0	.0	.0
4.7	2342.3	.0	.0	.0	.0	.0	.0	.0
1.2	290.8	.0	.0	.0	.0	.0	.0	.0
.9	70.0	.0	.0	.0	.0	.0	.0	.0

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	31.38	34.19
2	40.29	35.45
3	49.09	37.33
4	57.74	39.83
5	66.19	42.93
6	74.40	46.62
7	82.32	50.88
8	89.93	55.69
9	97.18	61.02
10	104.04	66.85
11	107.83	70.57

Circle Center At X = 18.0 ; Y = 161.0 and Radius, 127.5
*** 1.599 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	22.07	33.34
2	31.06	33.01
3	40.05	33.57
4	48.93	34.99
5	57.64	37.28
6	66.08	40.41
7	74.17	44.35
8	81.84	49.06
9	89.01	54.49
10	95.62	60.60
11	101.60	67.33
12	103.86	70.43

Circle Center At X = 30.0 ; Y = 124.6 and Radius, 91.6
*** 1.599 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	33.45	35.22
2	42.45	35.00
3	51.42	35.71
4	60.27	37.32
5	68.91	39.84
6	77.25	43.22
7	85.20	47.44
8	92.67	52.46

```
    9          99.60        58.21
   10         105.89        64.64
   11         110.65        70.62
Circle Center At X =   40.1 ; Y =  122.8  and Radius,   87.8
      ***     1.606   ***
Failure Surface Specified By 10 Coordinate Points
  Point       X-Surf       Y-Surf
   No.         (ft)         (ft)
    1          36.55        36.78
    2          45.50        37.73
    3          54.34        39.43
    4          63.00        41.88
    5          71.42        45.06
    6          79.54        48.93
    7          87.31        53.48
    8          94.66        58.67
    9         101.55        64.46
   10         107.67        70.57
Circle Center At X =   29.8 ; Y =  142.6  and Radius,  106.1
      ***     1.613   ***
Failure Surface Specified By 10 Coordinate Points
  Point       X-Surf       Y-Surf
   No.         (ft)         (ft)
    1          36.55        36.78
    2          45.52        37.49
    3          54.39        39.05
    4          63.07        41.43
    5          71.48        44.63
    6          79.56        48.60
    7          87.22        53.32
    8          94.41        58.74
    9         101.05        64.81
   10         106.23        70.54
Circle Center At X =   33.6 ; Y =  131.7  and Radius,   95.0
      ***     1.613   ***
Failure Surface Specified By 12 Coordinate Points
  Point       X-Surf       Y-Surf
   No.         (ft)         (ft)
    1          24.14        33.51
    2          33.08        32.47
    3          42.08        32.47
    4          51.02        33.49
    5          59.78        35.54
    6          68.26        38.57
    7          76.33        42.55
    8          83.89        47.44
    9          90.84        53.15
   10          97.09        59.63
   11         102.56        66.78
   12         104.78        70.51
Circle Center At X =   37.7 ; Y =  110.4  and Radius,   78.1
      ***     1.625   ***
Failure Surface Specified By 13 Coordinate Points
  Point       X-Surf       Y-Surf
   No.         (ft)         (ft)
    1          22.07        33.34
    2          30.99        32.17
    3          39.99        31.95
    4          48.96        32.67
    5          57.81        34.33
    6          66.43        36.91
    7          74.73        40.38
    8          82.62        44.71
    9          90.02        49.84
```

```
   10          96.83        55.73
   11         102.98        62.29
   12         108.42        69.47
   13         109.10        70.60
Circle Center At X =   37.6 ; Y =  117.4  and Radius,   85.5
      ***     1.629   ***
Failure Surface Specified By 11 Coordinate Points
  Point       X-Surf       Y-Surf
   No.         (ft)         (ft)
    1          34.48        35.74
    2          43.37        37.20
    3          52.12        39.27
    4          60.72        41.94
    5          69.10        45.22
    6          77.24        49.07
    7          85.08        53.47
    8          92.60        58.42
    9          99.75        63.88
   10         106.51        69.83
   11         107.23        70.56
Circle Center At X =   18.2 ; Y =  163.4  and Radius,  128.7
      ***     1.629   ***
Failure Surface Specified By 11 Coordinate Points
  Point       X-Surf       Y-Surf
   No.         (ft)         (ft)
    1          33.45        35.22
    2          42.33        36.66
    3          51.12        38.60
    4          59.79        41.04
    5          68.29        43.97
    6          76.62        47.38
    7          84.74        51.27
    8          92.62        55.61
    9         100.24        60.40
   10         107.58        65.62
   11         113.90        70.69
Circle Center At X =   12.9 ; Y =  191.0  and Radius,  157.1
      ***     1.630   ***
```

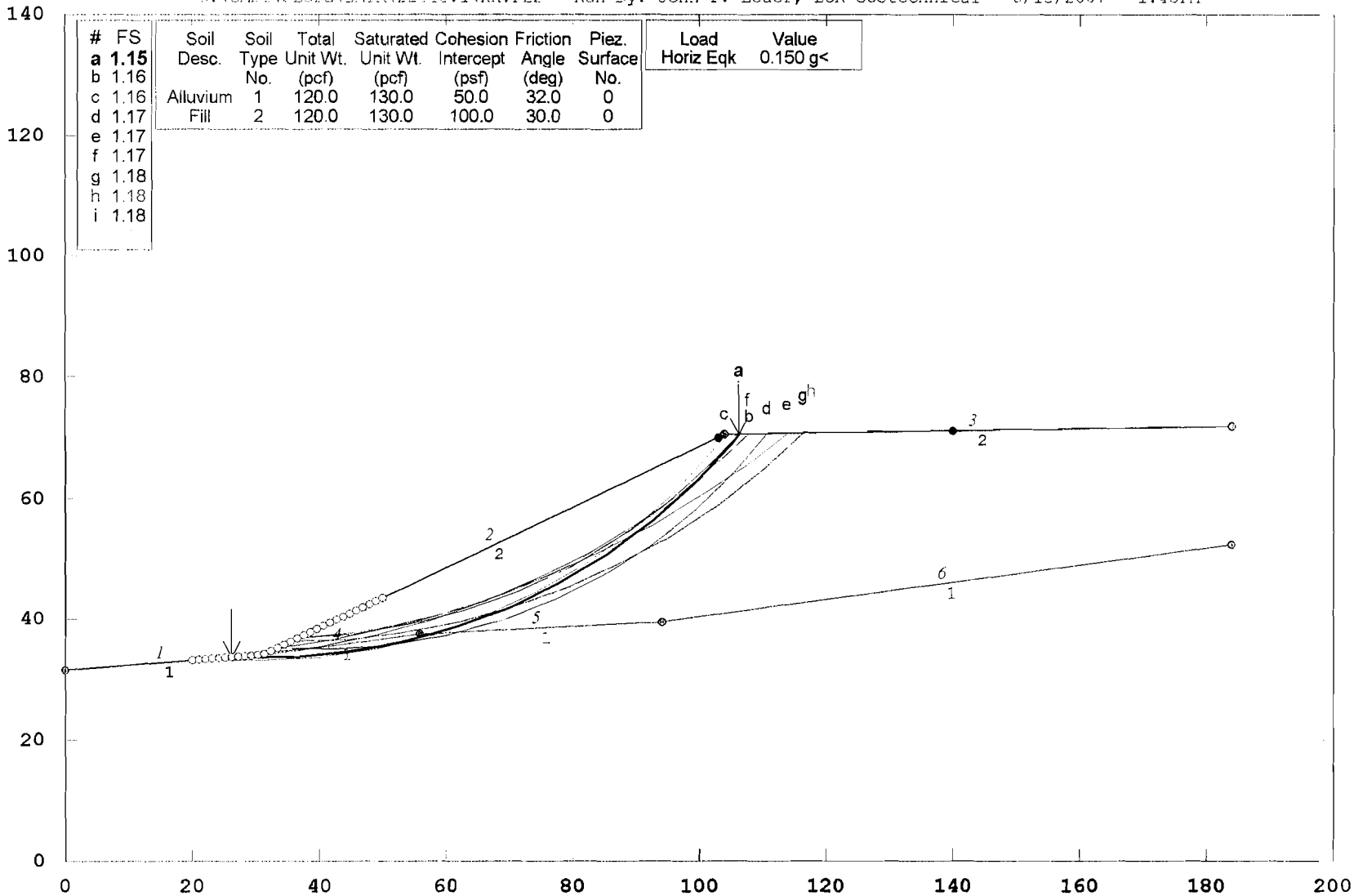
Section XX' - Pseudostatic Analysis

C:\GARY\SLOPE\DATA\22448.1\XX.PL2 Run By: John P. Leuer, LOR Geotechnical 8/13/2007 1:45PM

#	FS
a	**1.15**
b	1.16
c	1.16
d	1.17
e	1.17
f	1.17
g	1.18
h	1.18
i	1.18

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Alluvium	1	120.0	130.0	50.0	32.0	0
Fill	2	120.0	130.0	100.0	30.0	0

Load	Value
Horiz Eqk	0.150 g<



PCSTABL5M/si FSmin=1.15

Safety Factors Are Calculated By The Modified Bishop Method

STED

```
                    ** PCSTABL5M **
                          by
                   Purdue University
             --Slope Stability Analysis--
          Simplified Janbu, Simplified Bishop
             or Spencer`s Method of Slices
Run Date:                  8/13/2007
Time of Run:               1:45PM
Run By:                    John P. Leuer, LOR Geotechnical
Input Data Filename:       C:XX.IN
Output Filename:           C:XX.OUT
Unit:                      ENGLISH
Plotted Output Filename:   C:XX.PLT
PROBLEM DESCRIPTION   Section XX' - Pseudostatic Analysis
BOUNDARY COORDINATES
    3 Top   Boundaries
    6 Total Boundaries
Boundary     X-Left     Y-Left    X-Right    Y-Right    Soil Type
   No.        (ft)       (ft)       (ft)       (ft)     Below Bnd
    1          .00      31.50      31.20      34.10        1
    2        31.20      34.10     104.00      70.50        2
    3       104.00      70.50     184.00      72.00        2
    4        31.20      34.10      55.80      37.50        1
    5        55.80      37.50      94.00      39.50        1
    6        94.00      39.50     184.00      52.50        1
ISOTROPIC SOIL PARAMETERS
 2 Type(s) of Soil
Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
 No.  (pcf)    (pcf)     (psf)     (deg)   Param.    (psf)    No.
  1   120.0    130.0      50.0     32.0     .00        .0      0
  2   120.0    130.0     100.0     30.0     .00        .0      0
A Horizontal Earthquake Loading Coefficient
Of .150 Has Been Assigned
A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned
Cavitation Pressure =     .0 (psf)
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
900 Trial Surfaces Have Been Generated.
 30 Surfaces Initiate From Each Of 30 Points Equally Spaced
Along The Ground Surface Between  X =  20.00 ft.
                              and  X =  50.00 ft.
Each Surface Terminates Between   X = 103.00 ft.
                              and   X = 140.00 ft.
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is  Y =   .00 ft.
 9.00 ft. Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Examined.  They Are Ordered - Most Critical
First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Failure Surface Specified By 12 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         26.21       33.68
    2         35.21       33.63
    3         44.17       34.44
    4         53.02       36.09
    5         61.67       38.58
    6         70.04       41.88
    7         78.06       45.96
    8         85.66       50.78
    9         92.76       56.31
```

10	99.31	62.49
11	105.24	69.26
12	106.16	70.54

Circle Center At X = 31.3 ; Y = 127.8 and Radius, 94.2

*** 1.146 ***

Individual data on the 15 slices

Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
5.0	133.2	.0	.0	.0	.0	20.0	.0	.0
4.0	701.1	.0	.0	.0	.0	105.2	.0	.0
9.0	4636.2	.0	.0	.0	.0	695.4	.0	.0
8.8	7997.7	.0	.0	.0	.0	1199.7	.0	.0
2.8	3076.0	.0	.0	.0	.0	461.4	.0	.0
2.6	3040.1	.0	.0	.0	.0	456.0	.0	.0
3.3	4093.8	.0	.0	.0	.0	614.1	.0	.0
8.4	11252.8	.0	.0	.0	.0	1687.9	.0	.0
8.0	11173.4	.0	.0	.0	.0	1676.0	.0	.0
7.6	10082.0	.0	.0	.0	.0	1512.3	.0	.0
7.1	8147.8	.0	.0	.0	.0	1222.2	.0	.0
6.5	5591.9	.0	.0	.0	.0	838.8	.0	.0
4.7	2342.3	.0	.0	.0	.0	351.3	.0	.0
1.2	290.8	.0	.0	.0	.0	43.6	.0	.0
.9	70.0	.0	.0	.0	.0	10.5	.0	.0

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	31.38	34.19
2	40.29	35.45
3	49.09	37.33
4	57.74	39.83
5	66.19	42.93
6	74.40	46.62
7	82.32	50.88
8	89.93	55.69
9	97.18	61.02
10	104.04	66.85
11	107.83	70.57

Circle Center At X = 18.0 ; Y = 161.0 and Radius, 127.5

*** 1.163 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	22.07	33.34
2	31.06	33.01
3	40.05	33.57
4	48.93	34.99
5	57.64	37.28
6	66.08	40.41
7	74.17	44.35
8	81.84	49.06
9	89.01	54.49
10	95.62	60.60
11	101.60	67.33
12	103.86	70.43

Circle Center At X = 30.0 ; Y = 124.6 and Radius, 91.6

*** 1.164 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	33.45	35.22
2	42.45	35.00
3	51.42	35.71

```
    4          60.27        37.32
    5          68.91        39.84
    6          77.25        43.22
    7          85.20        47.44
    8          92.67        52.46
    9          99.60        58.21
   10         105.89        64.64
   11         110.65        70.62
Circle Center At X =   40.1 ; Y =  122.8  and Radius,   87.8
      ***     1.166   ***
Failure Surface Specified By 11 Coordinate Points
  Point       X-Surf       Y-Surf
   No.         (ft)         (ft)
    1          33.45        35.22
    2          42.33        36.66
    3          51.12        38.60
    4          59.79        41.04
    5          68.29        43.97
    6          76.62        47.38
    7          84.74        51.27
    8          92.62        55.61
    9         100.24        60.40
   10         107.58        65.62
   11         113.90        70.69
Circle Center At X =   12.9 ; Y =  191.0  and Radius,  157.1
      ***     1.173   ***
Failure Surface Specified By 10 Coordinate Points
  Point       X-Surf       Y-Surf
   No.         (ft)         (ft)
    1          36.55        36.78
    2          45.50        37.73
    3          54.34        39.43
    4          63.00        41.88
    5          71.42        45.06
    6          79.54        48.93
    7          87.31        53.48
    8          94.66        58.67
    9         101.55        64.46
   10         107.67        70.57
Circle Center At X =   29.8 ; Y =  142.6  and Radius,  106.1
      ***     1.174   ***
Failure Surface Specified By 12 Coordinate Points
  Point       X-Surf       Y-Surf
   No.         (ft)         (ft)
    1          35.52        36.26
    2          44.51        36.67
    3          53.44        37.80
    4          62.25        39.62
    5          70.89        42.14
    6          79.31        45.33
    7          87.44        49.18
    8          95.25        53.66
    9         102.68        58.73
   10         109.69        64.39
   11         116.22        70.57
   12         116.37        70.73
Circle Center At X =   34.8 ; Y =  150.0  and Radius,  113.8
      ***     1.176   ***
Failure Surface Specified By 12 Coordinate Points
  Point       X-Surf       Y-Surf
   No.         (ft)         (ft)
    1          33.45        35.22
    2          42.45        35.28
    3          51.41        36.08
```

```
    4         60.28       37.61
    5         68.99       39.87
    6         77.49       42.85
    7         85.71       46.51
    8         93.60       50.84
    9        101.10       55.81
   10        108.17       61.38
   11        114.76       67.52
   12        117.70       70.76
Circle Center At X =   37.4 ; Y =  143.6  and Radius,  108.4
      ***     1.176   ***
Failure Surface Specified By 10 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         36.55       36.78
    2         45.52       37.49
    3         54.39       39.05
    4         63.07       41.43
    5         71.48       44.63
    6         79.56       48.60
    7         87.22       53.32
    8         94.41       58.74
    9        101.05       64.81
   10        106.23       70.54
Circle Center At X =   33.6 ; Y =  131.7  and Radius,   95.0
      ***     1.176   ***
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         34.48       35.74
    2         43.38       37.11
    3         52.18       38.99
    4         60.86       41.37
    5         69.39       44.24
    6         77.74       47.60
    7         85.88       51.44
    8         93.79       55.73
    9        101.44       60.47
   10        108.80       65.65
   11        115.20       70.71
Circle Center At X =   15.1 ; Y =  191.2  and Radius,  156.7
      ***     1.177   ***
```

APPENDIX E

Report by Terra Geosciences

TERRA
GEOSCIENCES

SEISMIC REFRACTION SURVEY

682K DISTRIBUTION BUILDING PROJECT

CITY OF SAN BERNARDINO, CALIFORNIA

Project No. 272230-1

July 16, 2007

Prepared for:

LOR Geotechnical Group, Inc.
6121 Quail Valley Court
Riverside, CA 92507

Consulting Engineering Geology & Geophysics

P.O. Box 1099, Loma Linda, CA 92354 • 909-796-4667

LOR Geotechnical Group, Inc.
6121 Quail Valley Court
Riverside, CA 92507

Attention: Mr. Andrew Tardie

Regarding: Seismic Refraction Survey
682K Distribution Building Project
City of San Bernardino, California
LOR Project No. 22448.1

INTRODUCTION

As requested, this firm has performed a geophysical survey using the seismic refraction method for the above-referenced site, along selected areas as delineated by you. The purpose of this investigation was to assess the general seismic velocity characteristics of the underlying earth materials and to evaluate whether high velocity earth materials (non-rippable) are present along local areas which could possibly indicate areas of potential excavation difficulties, and also to aid in evaluating the subsurface structure and seismic velocity distribution. The local earth materials have been mapped by Miller, Matti, and Carson (2001) to consist of Mesozoic age metasedimentary rocks locally refereed to as the Pelona Schist which is typically fine-grained with the layering ranging from fine laminations to massive zones several meters thick. This rock also contains numerous layers of fine-gained quartzite and greenstone.

Representative Layer Velocity Profiles for each seismic line have been prepared and are presented in Appendix A, which indicates their respective "weighted average" subsurface velocities in generalized layers. In addition, associated Tomographic Models have also been prepared for comparative purposes, which generally indicate the relative structure and velocity distribution for each seismic line, and are presented within Appendix B. We understand that this report will be included as a technical appendix to your report, therefore as requested, the locations of our geophysical survey lines were transferred onto your field map for inclusion onto your final map.

As authorized by you, the following services were performed during this study:

- **Review of available published and unpublished geologic/geophysical data in our files pertinent to the site.**
- **Performing a seismic refraction survey by a State of California Professional Geophysicist, to include two seismic traverses along selected portions of the site.**
- **Preparation of this report, presenting our findings and conclusions with respect to the velocity characteristics and the rippability potentials of the subsurface earth materials.**

Accompanying Appendices

Appendix A - Layer Velocity Profiles
Appendix B - Tomographic Models
Appendix C - Excavation Considerations
Appendix D - References

SEISMIC REFRACTION SURVEY

Methodology

The seismic refraction method consists of measuring (at known points along the surface of the ground) the travel times of compressional waves generated by an impulsive energy source and can be used to estimate the layering, structure, and seismic acoustic velocities of subsurface horizons. Seismic waves travel down and through the soils and rocks, and when the wave encounters a contact between two earth materials having different velocities, some of the wave's energy travels along the contact at the velocity of the lower layer. The fundamental assumption is that each successively deeper layer has a velocity greater than the layer immediately above it. As the wave travels along the contact, some of the wave's energy is refracted toward the surface where it is detected by a series of motion-sensitive transducers (geophones). The arrival time of the seismic wave at the geophone locations can be related to the relative seismic velocities of the subsurface layers in feet per second (fps), which can then be used to aid in interpreting both the depth and type of materials encountered.

Field Procedures

One 12-channel and one 24-channel seismic refraction survey lines were performed being 195- and 300-feet in length with a target depth of 50- to 100±-feet, respectively. A 16-pound sledge-hammer was used as an energy source to produce the seismic waves and twelve to twenty-four, 14-Hz geophones (with 70% damping), were spaced at 12- to 15-foot intervals along the traverse lines to detect both the direct and refracted waves. The seismic wave arrivals were digitally recorded in SEG-2 format on a Geometrics StrataVisor™ NX model signal enhancement refraction seismograph. Seven shot points were utilized along each seismic line spread using forward, reverse, and intermediate locations, in order to obtain sufficient data for velocity analysis and depth modeling purposes. The data was acquired using a sampling rate of 0.25 milliseconds with a record length of 0.15 seconds. No acquisition filters were used. Each geophone and shot location was surveyed using a hand level and ruler for relative topographic correction. During acquisition, the seismograph provides both a hard copy and screen display of the seismic wave arrivals, of which are digitally recorded on the in-board seismograph computer.

Data Reduction

The data on the paper record and/or display screen were used to analyze the arrival time of the primary seismic "P"-waves at each geophone station, in the form of a wiggle trace, or wave travel-time curve, for quality control purposes in the field. All of the recorded data was transferred to our office computer for further processing, analyzing, and printing purposes, using the computer programs **SIP** (**S**eismic refraction **I**nterpretation **P**rogram) developed by Rimrock Geophysics, Inc. (1995), and **Rayfract™** (Intelligent Resources, Inc., 1996-2007). **SIP** is a ray-trace modeling program that evaluates the subsurface using layer assignments based on time-distance curves and is better suited for layered media, using the "Seismic Refraction Modeling by Computer" method (Scott, 1973). In addition, **Rayfract™** was also used for comparative purposes. **Rayfract™** is seismic refraction tomography software that models subsurface refraction, transmission, and diffraction of acoustic waves. Both computer programs perform their analysis using exactly the same input data, which includes first-arrival P-waves and line geometry.

SUMMARY OF GEOPHYSICAL INTERPRETATION

To begin with, it is important to understand that the data obtained during this survey represent an average of seismic velocities within any given layer. For example, within crystalline rocks, high seismic velocity boulders/dikes or local lithologic inconsistencies may be isolated within a low velocity matrix, thus yielding an average medium velocity for that layer. Therefore, in any given layer, a range of velocities could be anticipated, which can also result in a wide range of excavation characteristics.

In general, the upper 100±-feet of the hillside where locally surveyed was noted to be characterized by two to three major subsurface layers (**SIP** analysis, see Appendix A) with respect to seismic velocities and are generally described below:

- **Velocity Layer V1**: This uppermost velocity layer (V1) is most likely comprised of topsoil, colluvium, and/or highly weathered or intensely fractured bedrock materials. This layer has an average weighted velocity ranging from 1,660 to 2,091 fps, which is typical for these types of surficial-mantling materials.

- **Velocity Layer V2**: The second layer (V2) yielded weighted average seismic velocities of 3,666 to 4,461 fps, which most likely consists of highly to moderately weathered and fractured metamorphic bedrock. This velocity zone may also include abundant widely-scattered buried fresh large boulders, granitic dikes, and/or interbedded quartzite layers that are surrounded by a highly decomposed matrix.

- **Velocity Layer V3**: The third layer (V3) indicated a weighted average seismic velocity of 6,621 fps for the underlying local metamorphic bedrock. This velocity indicates the possibility of abundant widely-scattered buried fresh large boulders, interbedded quartzite layers, and/or granitic dikes within a moderately decomposed matrix, or possibly a relatively fresher crystalline bedrock matrix with wider-spaced fractures.

It is also important to note that the seismic velocities obtained in bedrock are influenced by the nature and character of the localized major structural discontinuities (bedding, foliation, fracturing, etc.). Generally, it is expected that higher (truer) velocities will be obtained when the seismic waves propagate along direction (strike) of the dominant structure, with a damping effect when the seismic waves travel in a perpendicular direction. Therefore, the seismic velocities obtained during our field study and as discussed below, should be considered minimum velocities at this time, as the structure of the bedrock locally is not known.

Using **Rayfract™**, a tomographic model for each seismic line was also prepared and analyzed for comparative purposes, as presented in Appendix B, which generally indicates the relative structure and velocity distribution. The models were prepared to display the same relative color intensities for the respective velocities so that they may be comparable across the site. Although no discrete velocity layers or boundaries are created, these models generally resemble the **SIP** analysis. **Rayfract™** allows imaging of subsurface velocity using first break energy propagation modeling. It can be seen in these models that the seismic velocity gradually increases with depth. It was noted that for the most part the seismic velocities on the Layer Velocity Profiles appear to generally correlate with the average of the velocity gradients as shown on the Tomographic Models.

GENERALIZED RIPPABILITY CHARACTERISTICS

Due to the variable lithologic nature of the Pelona Schist that comprises the hillsides and the localized granitic dikes that have been mapped within the subject property, excavations characteristics could vary significantly across the site even for similar seismic velocity ranges. The following summary is generally considered applicable for assessing the rippability characteristics of "granitic-type" bedrock, however, since granitic rock is not predominant within the subject site, the following discussion should be considered for comparative and informational purposes only. Due to the expected fracturing and foliation structures of the metamorphic rocks locally mapped at the site and as exposed, the rippability characteristics may be comparable to some types of granitic bedrock. This summary has been slightly modified for this project in an effort to help aid in evaluating potential excavation difficulties with respect to the seismic velocities that were obtained during our geophysical survey.

As indicated above, the generalized excavation characteristics of granitic bedrock as summarized below, is based on rippability performance charts prepared by Caterpillar, Inc. (2000 and 2004) and has been included to aid in evaluating potential excavation difficulties with respect to the seismic velocities obtained along the local areas surveyed. The velocity ranges described below are approximate and assume typical, good-working, heavy excavation equipment, such as single shank or D9R dozer, such as described by Caterpillar, Inc. (2000 and 2004); however, different excavating equipment (i.e., trenching equipment) may not correlate well with these velocity ranges. Trenching operations within bedrock materials with seismic velocities generally greater than 3,500 to 4,000±-fps, typically encounter very difficult to non-productable conditions.

In addition, a summary of excavation considerations has been included within Appendix C for reference in order to provide the client with a better understanding of the complexities of excavation in bedrock materials. These concepts should be understood so that proper planning and excavation techniques can be employed by the selected grading contractor.

- **Rippable Condition (0 - 4,000 ft/sec)**:

  This velocity range indicates rippable materials which may consist of alluvial-type deposits and decomposed metamorphic bedrock, with random hardrock floaters. These materials typically break down into silty sands (depending on parent lithologic materials), whereas floaters will require special disposal. Some areas containing numerous hardrock floaters may present utility trench problems. Large floaters exposed at or near finished grade may present problems for footing or infrastructure trenching.

- **Marginally Rippable Condition (4,000 - 8,000 ft/sec)**:

  This range of seismic velocities indicates materials which may consist of moderately weathered bedrock and/or large areas of fresh bedrock materials separated by weathered fractured zones. These bedrock materials are generally rippable with difficulty by a Caterpillar D9R or equivalent. Excavations may produce material that will partially break down into a coarse, silty to clean sand, with a high percentage of very coarse sand to pebble-sized material depending on the parent bedrock lithology. Less fractured or weathered materials will probably require blasting to facilitate removal.

- **Non-Rippable Condition (8,000 ft/sec or greater)**:

  This velocity range includes non-rippable material consisting primarily of moderately fractured bedrock at lower velocities and only slightly fractured or unfractured rock at higher velocities. Materials in this velocity range may be marginally rippable, depending upon the degree of fracturing and the skill and experience of the operator. Tooth penetration is often the key to ripping success, regardless of seismic velocity. If the fractures and joints do not allow tooth penetration, the material may not be ripped effectively; however, pre-blasting or "popping" may induce sufficient fracturing to permit tooth entry. In their natural state, materials with these velocities are generally not desirable for building pad grade, due to difficulty in footing and utility trench excavation. Blasting will most likely produce oversized material, requiring special disposal.

SUMMARY OF FINDINGS AND CONCLUSIONS

The raw field data was considered to be of only fair to good quality which had minor to moderate amounts of ambient "noise" that was introduced during our survey from nearby construction equipment and vehicular traffic from the adjacent roadways. Analysis of the data and picking of the primary "P"-wave arrivals was performed with moderate difficulty and some interpolation of data was necessary. Based on the results of our comparative seismic analyses of both **SIP** and **Rayfract™** (of which both software programs use exactly the same input data), the seismic refraction survey lines appear to generally coincide with one another, with some minor variances due to the methods that these programs process and integrate the input data. Anticipation of gradual increasing hardness with depth and lateral variations, with respect to excavation characteristics, should be anticipated across the subject site.

The anticipated excavation potentials of the **SIP** analysis velocity layers encountered locally during our survey are as follows:

- **Velocity Layer V1**:

  No major excavating difficulties are expected within the uppermost, low seismic velocity layer V1 (velocity range of 1,660 to 2,091 fps). This surficial layer is expected to be comprised of topsoil, colluvium, and/or highly weathered or intensely fractured bedrock materials. However, some fresher boulders could possibly be encountered based on surface exposures of bedrock outcrops in the general region.

- **Velocity Layer V2**:

  The second layer V2 is believed to consist of highly to moderately weathered and fractured metamorphic bedrock (velocity range of 3,666 to 4,461 fps) and should excavate with minor to moderate difficulty, assuming appropriate good-working equipment for the proposed type of excavation. Isolated floaters (i.e., boulders, corestones, etc.) could be present locally and could produce difficult excavation conditions locally. Placement of infrastructures in this material may also be difficult. Although not anticipated, localized blasting in these materials cannot be completely ruled out, due to the presence of any encountered fresh, buried boulders/dikes and/or thicker quartzite layers and granitic dikes.

- **Velocity Layer V3**:

  Hard excavation difficulties within the underlying higher velocity V3 layer (average weighted velocity of 6,621 fps) are anticipated, which is expected to consist of moderate to slightly weathered metamorphic bedrock materials with local granitic dikes. Hard excavating areas consisting of relatively fresher homogeneous bedrock with wide-spaced fracturing an/or localized buried boulders will most likely be encountered during both remedial grading and placement of infrastructures, which may require moderate to significant blasting to achieve desired grade.

It appears that there is a highly variable distribution of the less-weathered bedrock which can be as shallow as 35±-feet to greater than 80±-feet. Due to the interbedded nature of the schist, harder lithologic layers can be widely distributed at shallower depths, as well as the potential for the presence of quartzite and granitic dikes that will cause excavation difficulties. Based on the seismic velocity data as presented on the Tomographic Models, this variability across the site is expected.

It may be expected that when ground velocities on the order of 6,000±-fps or greater are encountered, increasing difficulties in excavation conditions and rippability will occur with respect to grading production. These increases may result in slower production rates from the cut excavation with a possible increase in the generation of oversized rock materials. This is also dependent upon the type and operating condition of the excavation equipment used, how hard the contractor is willing to work the equipment, and the structural discontinuities of the rock fabric. The decision for blasting of the rock for excavatability is sometimes made based upon economic production reasons and not solely on the rippability (velocity/hardness) characteristics of the bedrock.

To evaluate whether a particular bedrock material can be ripped or excavated, this geophysical survey should be used in conjunction with the geologic and/or geotechnical report for the subject project (if prepared) which should describe the physical properties of the bedrock. The physical characteristics of bedrock materials that favor ripping generally include the presence of fractures, faults and other structural discontinuities, weathering effects, brittleness or crystalline structure, stratification of lamination, large grain size, moisture permeated clay, and low compressive strength. If the bedrock is as thinly foliated and fractured at depth such as observed within surface exposures, this structure could aid in excavation production.

Unfavorable conditions can include such characteristics as massive and homogeneous formations, non-crystalline structure, absence of planes of weakness, fine-grained materials, and formations of clay origin where moisture makes the material plastic. Use of these physical bedrock conditions along with the subsurface velocity characteristics as presented within this report should aid in properly evaluating the type of equipment necessary and the production levels that can be anticipated for this project.

In summary, the results of this seismic refraction survey are to be considered as an aid in assessing the rippability potentials of the earth materials locally. This information should be carefully reviewed by the grading contractor and representative "test" excavations should be considered, so that they may be correlated with the data presented within this report.

CLOSURE

This survey was performed using "state of the art" geophysical techniques, computer processing, and equipment, in the localized areas delineated by you. We make no warranty, either expressed or implied. It should be noted that our data was obtained along only two specific areas; therefore, other local areas at the site beyond the limits of our seismic lines may contain different velocity layers and depths not encountered during our field survey. Estimates of velocity boundaries as presented in this report are generally considered to be within 10±-percent of the total depth of the contact. It should be understood that when using these theoretical geophysical principles and techniques, sources of error are possible in both the data obtained and in the interpretation.

If you should have any questions regarding this report or do not understand the limitations of this survey, please do not hesitate to contact our office.

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Principal Geophysicist
PGP 1002

PROFESSIONAL GEOPHYSICIST
DONN C.
SCHWARTZKOPF
No. 1002
STATE OF CALIFORNIA

APPENDIX A

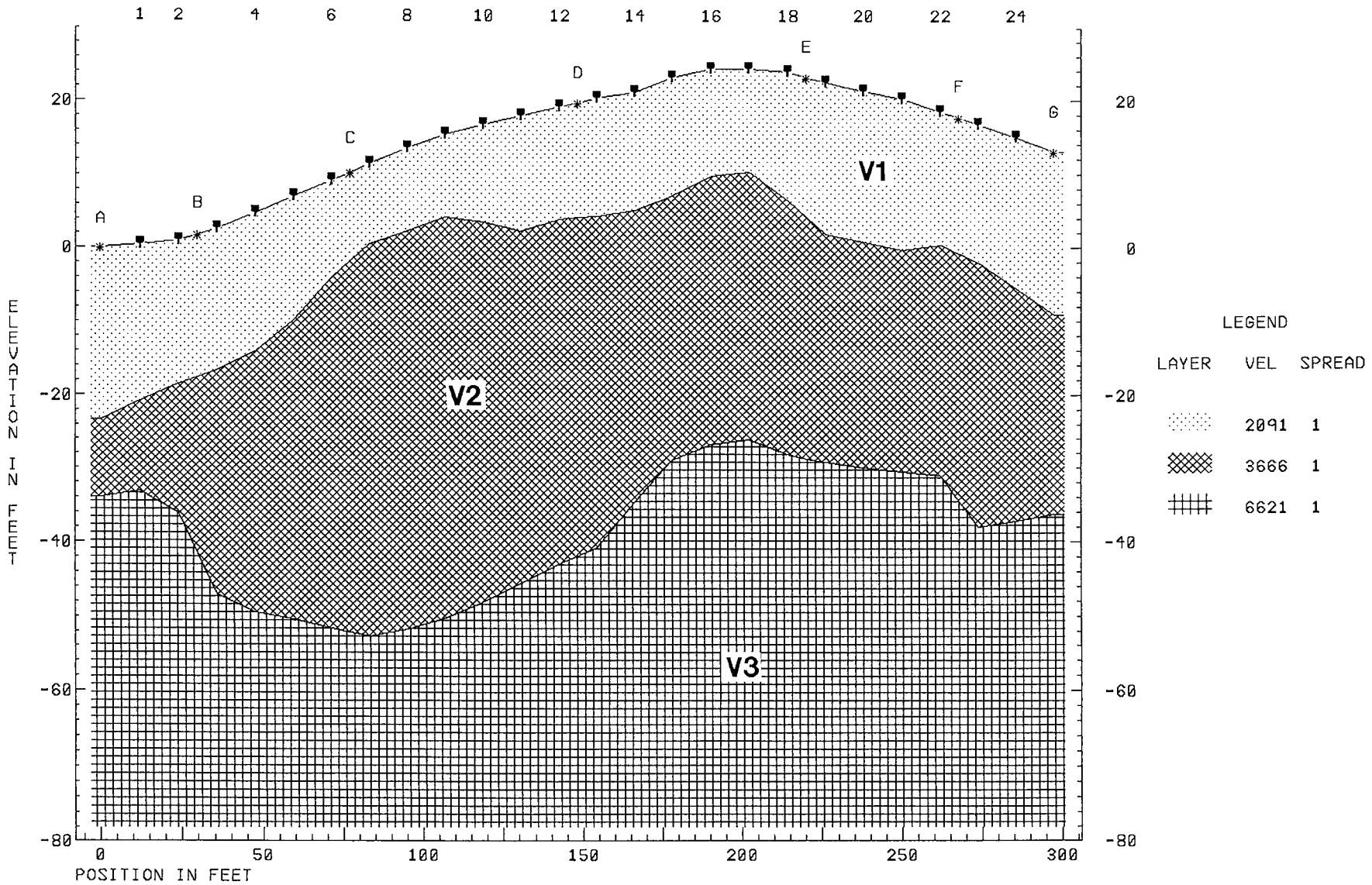
LAYER VELOCITY PROFILES

LAYER VELOCITY PROFILE S-1

South 35° East →

FILE 71207-1.SIP
SEISMIC LINE S-1

SPREAD 1

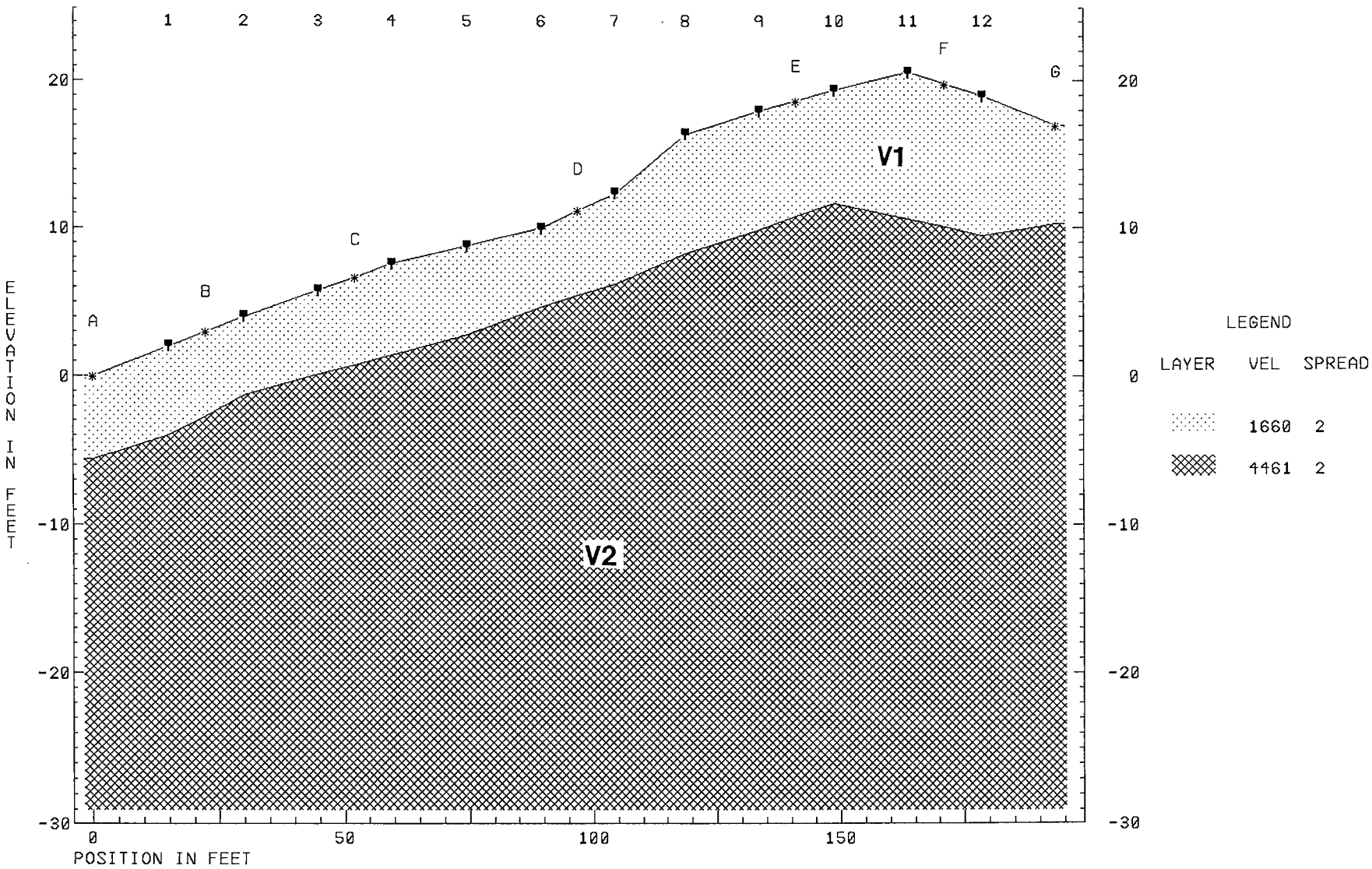


LAYER VELOCITY PROFILE S-2

North 60° West →

FILE 71207-2.SIP
SEISMIC LINE S-2

SPREAD 2

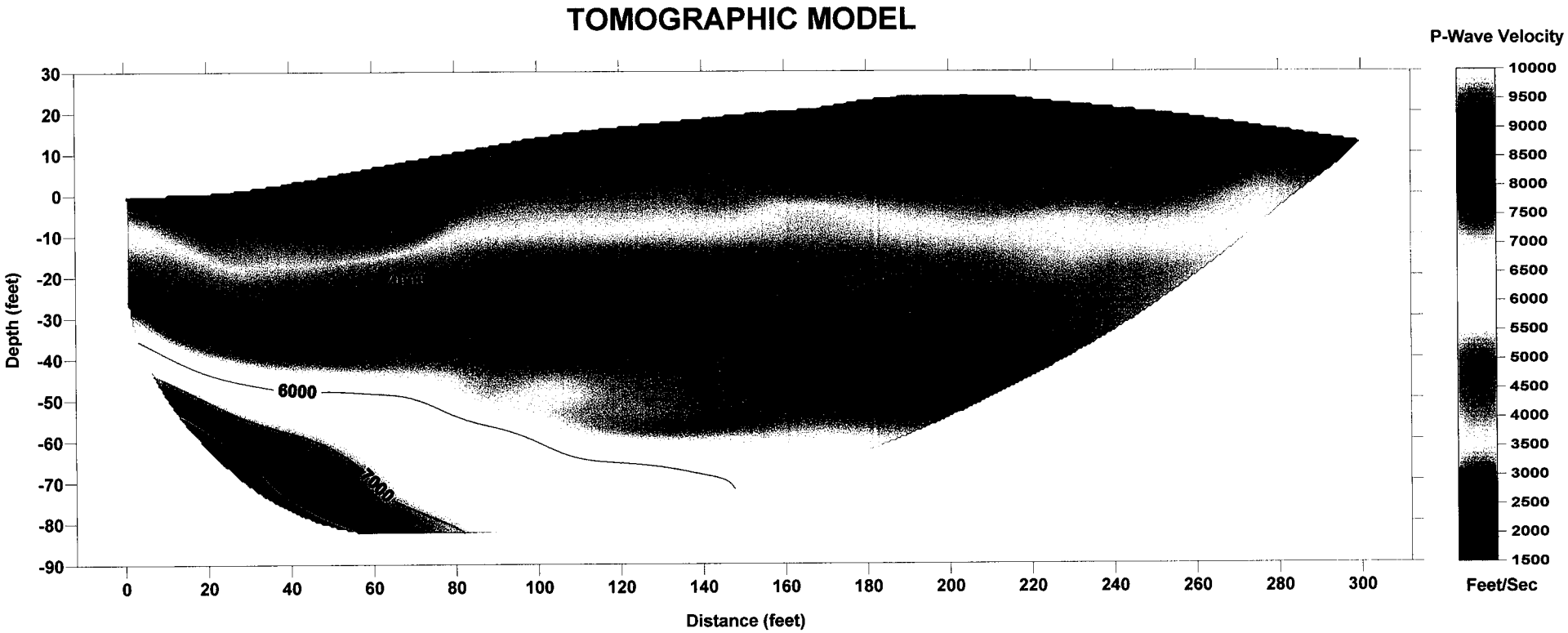


APPENDIX B

TOMOGRAPHIC MODELS

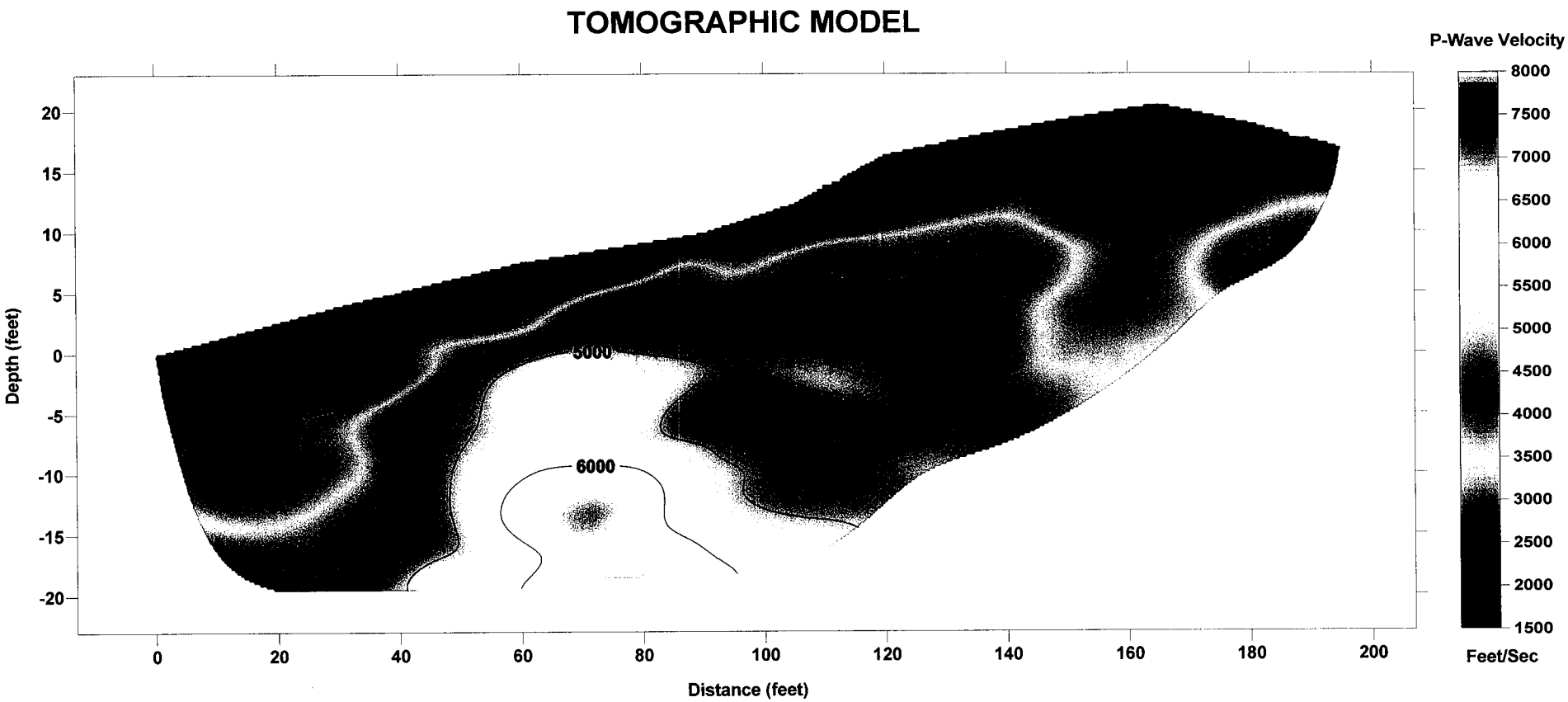
SEISMIC LINE S-1

South 35 East →



SEISMIC LINE S-2

North 60 West →



APPENDIX C

EXCAVATION CONSIDERATIONS

EXCAVATION CONSIDERATIONS

These excavation considerations have been included to provide the client with a brief overall summary of the general complexity of hard bedrock excavation. It is considered the clients responsibility to insure that the grading contractor they select is both properly licensed and qualified, with experience in hard-bedrock ripping processes. To evaluate whether a particular bedrock material can be ripped, this geophysical survey should be used in conjunction with the geologic or geotechnical report prepared for the project which describes the physical properties of the bedrock. The physical characteristics of bedrock materials that favor ripping generally include the presence of fractures, faults and other structural discontinuities, weathering effects, brittleness or crystalline structure, stratification of lamination, large grain size, moisture permeated clay, and low compressive strength. Unfavorable conditions can include such characteristics as massive and homogeneous formations, non-crystalline structure, absence of planes of weakness, fine-grained materials, and formations of clay origin where moisture makes the material plastic.

When assessing the potential rippability of the underlying bedrock of a given site, the above geologic characteristics along with the estimated seismic velocities can then be used to evaluate what type of equipment may be appropriate for the proposed grading. When selecting the proper ripping equipment there are three primary factors to consider, which are:

- **Down Pressure available at the tip, which determines the ripper penetration that can be attained and maintained,**
- **Tractor flywheel horsepower, which determines whether the tractor can advance the tip, and,**
- **Tractor gross-weight, which determines whether the tractor will have sufficient traction to use the horsepower.**

In addition to selecting the appropriate tractor, selection of the proper ripper design is also important. There are basically three designs, being radial, parallelogram, and adjustable parallelogram, of which the contractor should be aware of when selecting the appropriate design to be used for the project. The penetration depth will depend upon the down-pressure and penetration angle, as well as the length of the shank tips (short, intermediate, and long).

Also important in the excavation process is the ripping technique used as well as the skill of the individual tractor operator. These techniques include the use of one or more ripping teeth, up- and down-hill ripping, and the direction of ripping with respect to the geologic structure of the bedrock locally. The use of two tractors (one to push the first tractor-ripper) can extend the range of materials that can be ripped. The second tractor can also be used to supply additional down-pressure on the ripper. Consideration of light blasting can also facilitate the ripper penetration and reduce the cost of moving highly consolidated rock formations.

All of the combined factors above should be considered by both the client and the grading contractor, to insure that the proper selection of equipment and ripping techniques are used for the proposed grading.

APPENDIX D

REFERENCES

REFERENCES

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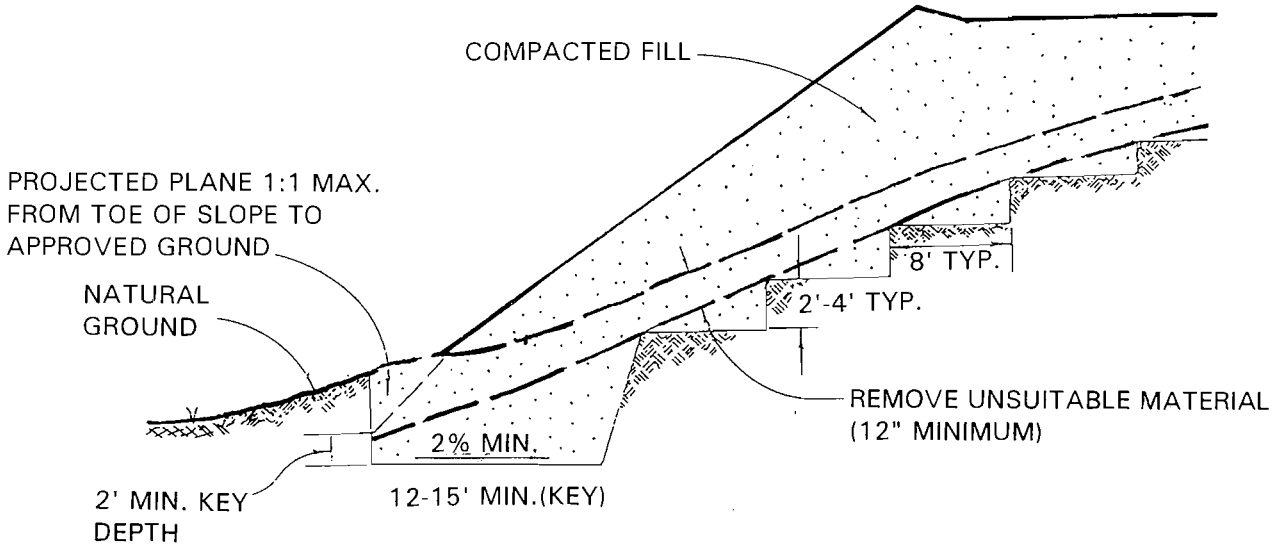
APPENDIX F

Geotechnical Sketches

TYPICAL KEYING AND BENCHING DETAIL



FILL SLOPE

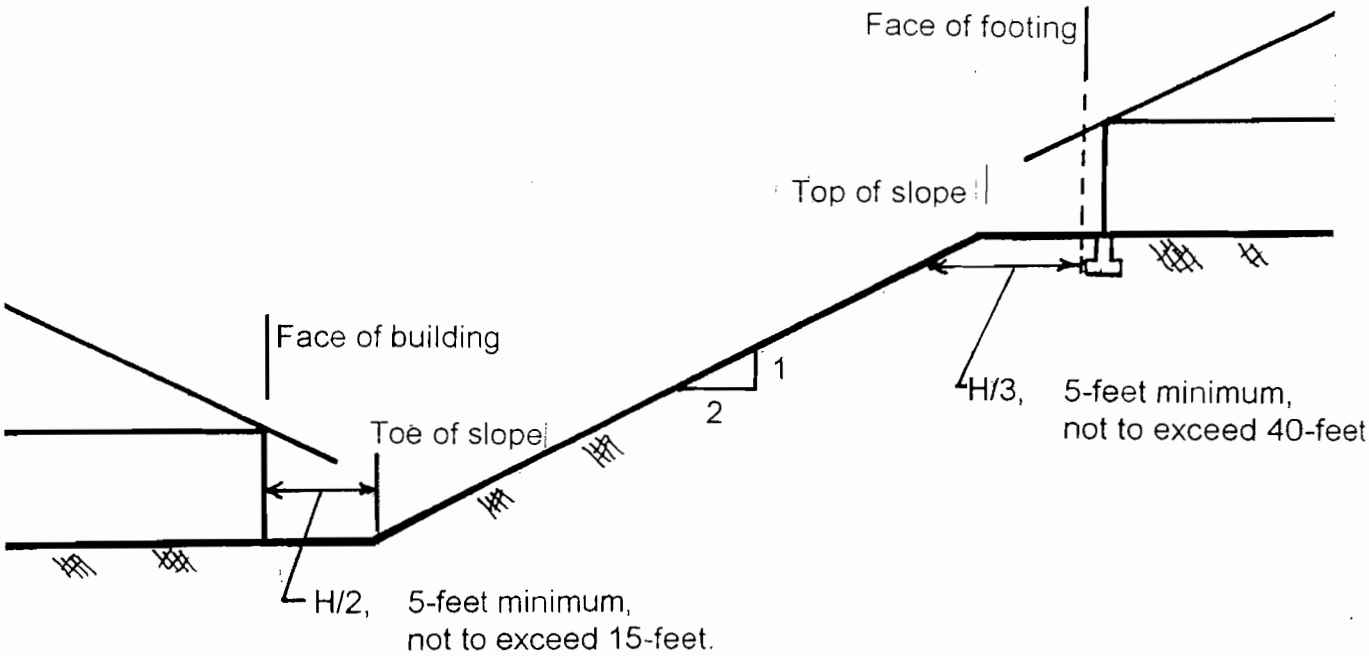
NOTES: 1) DIMENSIONS SHOWN SUBJECT TO FIELD CHANGE BASED ON ENGINEER'S JUDGEMENT

2) BENCHING REQUIRED WHEN FILLING OVER NATURAL GROUND STEEPER THAN 5H:1V

GEOTECHNICAL SKETCH

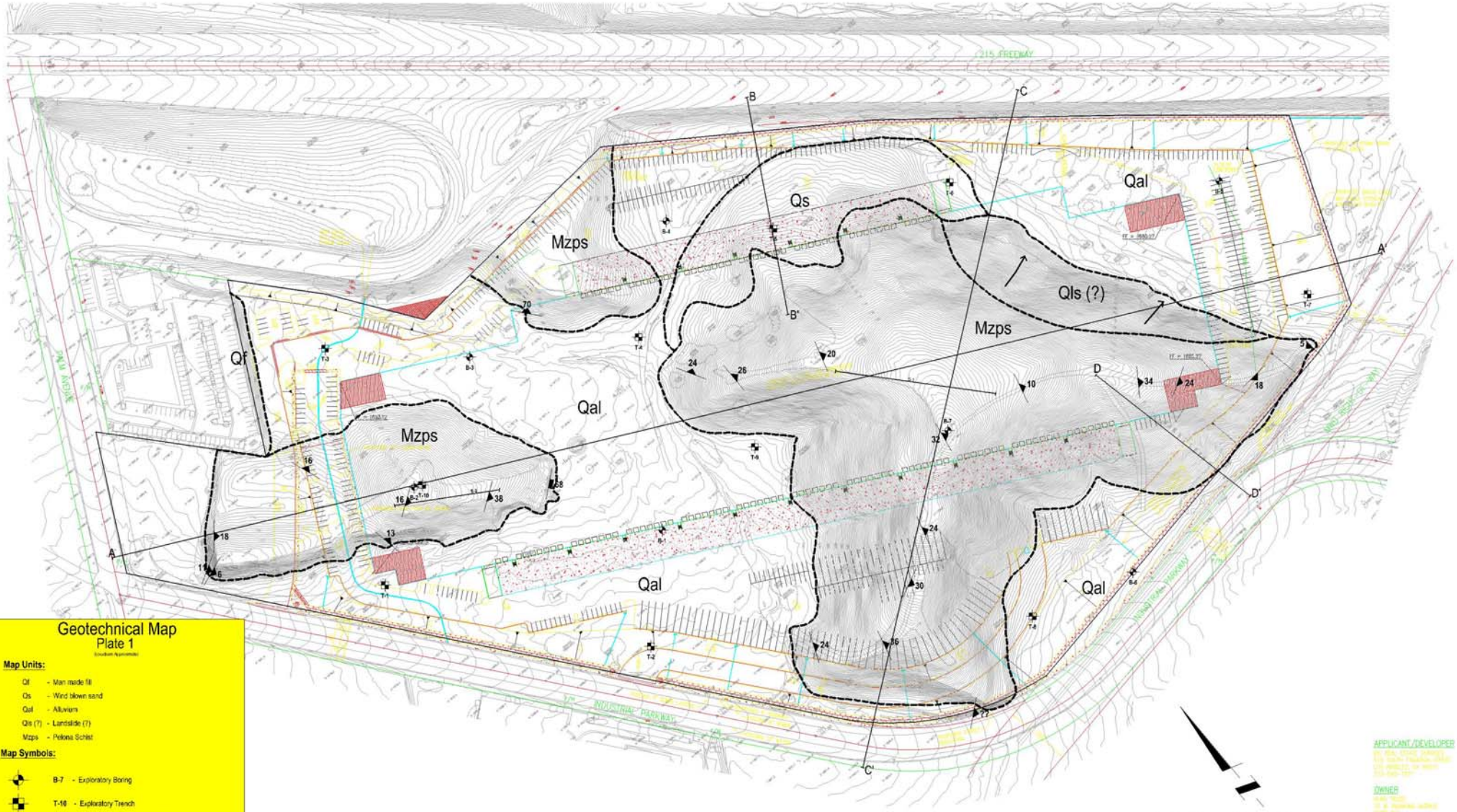
PROJECT:	682K DISTRIBUTION BUILDING, SAN BERNARDINO, CALIFORNIA	PROJECT NO.:	22448.1
CLIENT:	IDS REAL ESTATE	ENCLOSURE:	F-1
LOR Geotechnical Group, Inc.		DATE:	AUGUST 2007
		SCALE:	NOT TO SCALE

BUILDING SET-BACK REQUIREMENTS



GEOTECHNICAL SKETCH

PROJECT:	682K DISTRIBUTION BUILDING, SAN BERNARDINO, CALIFORNIA	PROJECT NO.:	22448.1
CLIENT:	IDS REAL ESTATE	ENCLOSURE:	F-2
LOR Geotechnical Group, Inc.		DATE:	AUGUST 2007
		SCALE:	NOT TO SCALE



Geotechnical Map
Plate 1

(Locations Approximate)

Map Units:

- Qf - Man made fill
- Qs - Wind blown sand
- Qal - Alluvium
- Qls (?) - Landslide (?)
- Mzps - Pelona Schist

Map Symbols:

- B-7 - Exploratory Boring
- T-10 - Exploratory Trench
- Seismic Line (Terra Geosciences, 2007)
- Cross Section
- Geologic Contact, Dashed Where Approximate
- Strike and Dip of Metamorphic Foliation
- Strike and Dip of Joint

Project No.: 22485.1 Date: August 2007
Client: IDS Real Estate
Project: 680K Distribution Building, San Bernardino, California
LGR Geotechnical Inc.

APPLICANT/DEVELOPER

OWNER

ZONING

DATE	BY	MARK	REVISIONS	APPR.	DATE

DESIGNED BY: – DRAWN BY: – CHECKED BY: –

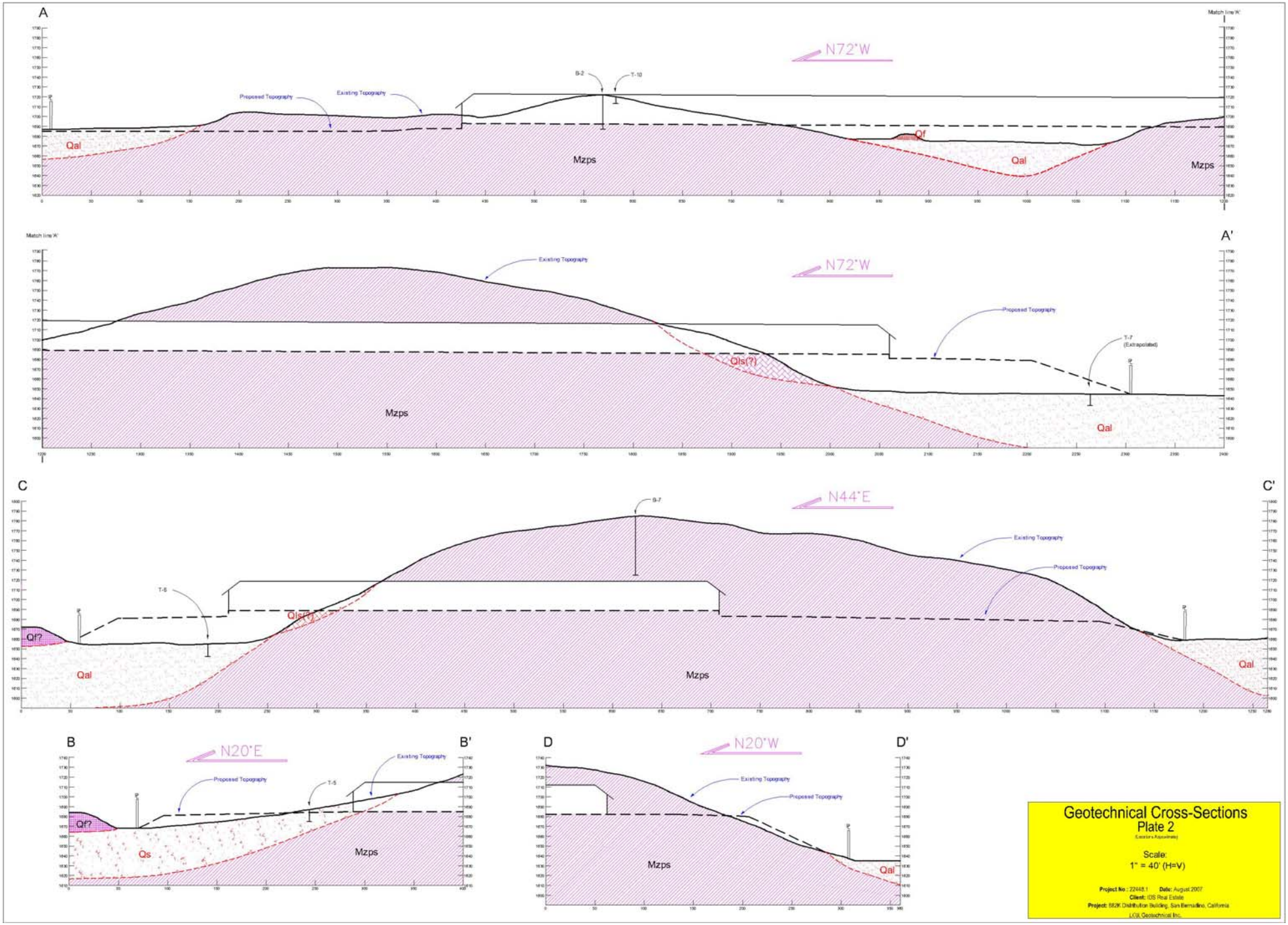
SCALE: 1" = 80'

DATE: 6/21/06

HURD TRUST – IDS REAL ESTATE GROUP

CITY OF SAN BERNARDINO DRC/ERC CASE:
PRE-APPLICATION REVIEW NO. 05-23
PRELIMINARY GRADING PLAN
PALM AVENUE AT 215 FREEWAY
A.P.N. 266-041-62

SHEET 1 OF 1



Geotechnical Cross-Sections
Plate 2
(Locations Approximate)

Scale:
1" = 40' (H=V)

Project No.: 22448.1 **Date:** August 2007
Client: IDS Real Estate
Project: 882K Distribution Building, San Bernardino, California
LGI Geotechnical Inc.