Appendix E:
Geology and Soils

LOR GEOTECHNICAL GROUP, INC.
Soil Engineering ▲ Geology ▲ Environmental

PRELIMINARY GEOTECHNICAL INVESTIGATION PROPOSED COMMERCIAL/WAREHOUSE DEVELOPMENT APN'S 0136-472-01, -02, AND 0136-492-01, EAST PARKING LOT FOR THE NATIONAL ORANGE SHOW CITY OF SAN BERNARDINO, CALIFORNIA

PROJECT NO. 12798.1
JUNE 6, 2011

Prepared For:

Lewis Retail Centers
1156 N. Mountain Avenue
Upland, California 91786

Attention: Mr. Timothy Reeves

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June 6, 2011

Lewis Retail Centers
1156 N. Mountain Avenue
Upland, California 91786

Project No. 12798.1

Attention: Mr. Timothy Reeves

Subject: Preliminary Geotechnical Investigation, Proposed Commercial/Warehouse Development, APN's 0136-472-01, -02, and 0136-492-01, East Parking Lot for the National Orange Show, City of San Bernardino, California.

LOR Geotechnical Group, Inc., is pleased to present this report summarizing our geotechnical investigation for the subject project. In summary, it is our opinion that the proposed development is feasible from a geotechnical perspective, provided the recommendations presented in the attached report are incorporated into design and construction.

The project site is underlain by a variable thickness of undocumented fill materials overlying alluvial materials which were loose to medium dense upon first encounter, becoming medium dense to dense at depth. Areas of deeper loose to medium dense soils were identified locally. It is our opinion that existing fill and upper loose alluvial materials will not provide uniform and/or adequate support for the proposed development. Thus, we recommend a compacted fill mat be constructed beneath footings and slabs. The fill mat should be a minimum of 24 inches thick below the bottom of the footings. The construction of this compacted fill mat will incorporate the removal of the existing, uncontrolled fills and upper loose alluvium. All on-site soils should be suitable for use as engineered compacted fill provide they are cleaned of any debris. Removals on the order of 10 to 12 feet with localized areas requiring removals of approximately 15 feet are anticipated to be required within the proposed building pad areas.

Very low expansive soils and good R-value quality soils were encountered on the site. A negligible sulfate content was found for the soils tested. Near completion and/or at the completion of site grading, additional tests of foundation and subgrade soils should be conducted to verify their soluble sulfate content and R-value quality.

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INTRODUCTION

During April and May of 2011, a Preliminary Geotechnical Investigation was performed by LOR Geotechnical Group, Inc., for a proposed commercial/warehouse development within Assessor's Parcel Numbers 0136-472-01, -02, and 0136-492-01 in the City of San Bernardino, California. These parcels generally comprise what is referred to as the east parking lot for the National Orange Show. The purpose of this investigation was to provide a technical evaluation of the geologic setting of the site and to provide geotechnical design recommendations for the proposed development. The scope of our services included:

- Review of available geotechnical literature, reports, maps, and agency information pertaining to the study area;
- A review of stereo aerial photographs of the site and immediate surrounding region from 1938 to 2005;
- A field reconnaissance by personnel from this firm 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;
- Infiltration testing to establish a clear water rate for the proposed infiltration swales and underground infiltration chamber system;
- Laboratory testing of selected soil samples obtained during the field investigation;
- Development of geotechnical recommendations for site development to include site grading, foundation design, and pavement;
- 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

The irregularly shaped property consists of approximately 38 acres located southeast of the intersection of Arrowhead Avenue and Mill Street in the City of San Bernardino, California. Proposed development within the property is currently scheduled to include a 616,000 square foot building in the central portion, a 78,960 square foot building in the southwest portion and a 27,810 square foot building in the northeast portion. An additional building was proposed for an area near the northeast corner of the site but this area, as well as associated nearby areas that were proposed for parking, have been eliminated from the current site area of investigation. These not-a-part areas are identified on our Enclosure A-2, Boring Location Map. A Site Plan which was prepared by HPA Inc., and dated April 11, 2011, was used as a base for presenting our geotechnical data.

The Site Plan indicates that loading docks, paved parking, and access areas and bio-swale areas, used for onsite infiltration of rain water runoff, are also proposed. No grading plans were available at the time of this report. However, based on the topography of the site and adjacent proportions, minimal cut and fills of less than 10 feet are anticipated, excluding removals and over-excavations, to reach the proposed grades.

REVIEW OF AERIAL PHOTOGRAPHS

A search was conducted for available stereo pairs of aerial photographs of the area on file at the County of San Bernardino Flood Control District (CSBFCD) collection by a representative from this firm. The search provided aerial photographs taken of the subject site and surrounding area in 1938, 1955, 1969, 1972, 1978, 1986, 1991, 1996, 2001, and 2005, the latest photographs available.

The aerial photographs reviewed at the CSBFCD consisted of vertical aerial stereo-graphic photograph pairs of varying scales, as available on file. These photographs were viewed using stereoscopes with magnifications of 2X and 4X for three-dimensional enhancement. Due to the relatively large photographic scales involved, the analysis and subsequent interpretation of detail from aerial photographs sometimes required a degree of subjective judgement. The degree of certainty for the interpretation of details depends upon such factors as the scale and the quality of the photograph. However, an analysis of aerial photographs will reveal the general site history as to the relative use of the land, possible ground disturbance, activities, etc.

A brief summary of the site and surrounding conditions during the various times, as reflected in the photographs, is given below.

1. **May 27, 1938, Photo Nos. AXL-60-105 & 106, Scale 1" = 1,700'**

In these early aerial photographs, the site mainly consists of vacant land that appears to be used as dry-land farmland and a few rural residential developments are located within the southwest portion of the site. What appears to be a couple of residences with garages and other small detached structures are located at the northeast corner Arrowhead Avenue and Central Avenue and what appears to be a couple of similar dwellings and structures are located in the southwest portion of the site parcel located at the southeast corner of Arrowhead Avenue and Central Avenue. There are also a few residences with associated detached structures (garages, sheds, animal pens, etc.) located in the northern portion of the site, southeast of the extension of Huff Street as well as a residence with one to two outbuildings located east of the intersection of Esperanza Street and Mountain View Avenue.

The above mentioned residential developments in the north portion of the site are located on the north side of a tributary to Warm Creek that apparently is incorrectly plotted on the 1901 and 1942 USGS topographic maps. This stream channel is locally lined with dense growths of trees/brush, particularly in the northeast potion and traverses the site from northeast to southwest, exiting the property and passing under Arrowhead Avenue approximately 400 to 500 feet south of Esperanza Street This segment of Warm Creek varies from about 20 to 50 feet in width and is of indeterminate depth but appears to be roughly about 5 to 10 feet in depth.

Other than the use as farmland, and minor residential development, a few faint dirt roads, likely used during harvesting/plowing, etc., traverse the property. In addition, a large area in the west portion of the site shows evidence for having been flooded during the historic 1938 flood event. The area of impact extends from north of Esperanza Street, to a maximum of approximately 400 feet east of Arrowhead Avenue and south to a couple hundred feet north of Central Avenue.

The main streets in the area - Arrowhead Avenue, Central Avenue and Mill Street exist as well established roads while Esperanza Street, Huff Street and Mountain View Avenue appear to be dirt roads through rural residential properties. Most of the properties around the site consist of similar rural farmland developments with some commercial development and what appears to be a school located along Mill Street

to the north. A main tributary to Warm Creek has been channelized recently, a couple hundred feet west of and parallel to Arrowhead Avenue, west of the site. What appears to be one of the original National Orange Show structures is located approximately one-quarter mile west-northwest of the site.

In the region, the Santa Ana River and associated tributaries impact the landscape. The Santa Ana River is located approximately 1.0 miles southeast of the subject site. The site is located at a transition from more developed areas to the north and west, to less developed areas to the south and east. Approximately 1.5 miles northwest of the site is a large railroad hub with numerous structures and tracks. Most of the agriculture in the region is dry farm land with some irrigated crops and relatively few orchards.

2. **November 9, 1955, Flight No. F-34, Photo Nos. 4-83 & -84, Scale 1" = 1,300'**

The residences and structures that were located at the northeast corner of Arrowhead Avenue and Central Avenue have been removed and it appears that the dwellings/structures southeast of this intersection have recently been demolished. The natural drainage course that traversed the northwest portion of the site has been filled in and the area graded flat. The northeast portion of the original, natural creek remains natural. From a point located approximately 200 feet southeast of the intersection of Esperanza Street and Mountain View Avenue, the drainage is now an open unlined linear ditch that trends south-southeast about 200 feet and then heads southwest along the approximate present day drainage alignment. The new drainage is crossed by two dirt roads.

A few more residences have been built in nearby offsite areas particularly northwest and northeast of the intersection of Esperanza Street and Mountain View Avenue. Commercial developments exist on the east side of Arrowhead Avenue between Mill Street and Esperanza Street and along the south side of Mill Street east of Arrowhead Avenue. Although the above mentioned National Orange Show building has been demolished, several new buildings and improvements have been made to the National Orange Show fairgrounds to the west. In general, significant expansion of commercial and residential development has occurred throughout the local area with relatively small areas of agricultural use left.

3. **February 1969, Flight No. C-295, Photo Nos. 55 & 56, Scale 1" = 2,200'**

Mountain View Avenue now extends across the site as a dirt or gravel lined road and crosses the formerly channelized but now backfilled portion of Warm Creek drainage (southwest of Mountain View Avenue). The natural drainage area that previously remained in and beyond the northeast portion of the site has been abandoned, cleared of vegetation and partially or possibly completely filled in. The previously channelized onsite portion of the Warm Creek tributary now appears to be backfilled and scars from channelization/installation of the storm drain pipe extend straight across the site toward a new major flood control channel that has been built northeast and east of the site. Beyond this area and east of the northeast portion of the site, a lumber yard is now present. In addition, a commercial building is now present near the far northeast corner of the site at the southwest corner of the intersection of Mill Street and Sierra Way. Approximately 150 to 200 feet southwest of the intersection of the storm drain and Mountain View Avenue is a dirt road that connects a new racetrack area with a parking area. The racetrack consists of an irregular shaped dirt loop in the area between Esperanza Street, Arrowhead Avenue, the storm drain and Mountain View Avenue. The parking area is located on the east side of Mountain View Avenue, south the storm drain and it also contains a double-wide modular type building and a second building to the east of the modular building. The southern portion of the site, south of Central Avenue, has been cleared of all trees and improvements. Central Avenue has been widened and a commercial/industrial development (steel fabrication) has been built at the northeast corner of Arrowhead Avenue and Esperanza Street. Continued development is evident in the region. The 215 Freeway and the Inland Center Mall are now located west of the subject site.

4. **October 15, 1972, Flight No. C-194, Photo No. 37, Scale 1" = 2,000'**

Stereo coverage was not available for the subject site and surrounding properties. The racetrack is no longer in use and is barely visible. The majority of the site appears as flat, barren ground. The area of the tributary of Warm Creek that had been channelized/put in underground pipe across the site has recently been cleared and smoothed. Commercial development continues in the area, particularly to the south and southeast.

5. **January 21, 1978, Flight No. C-279, Photo Nos. 121 & 122, Scale 1" = 2,000'**

The racetrack area is no longer distinguishable. The building located east of the modular building in the southeast portion of the site has been removed. Vegetation is growing roughly along the approximate alignment of the storm drain and there is now a faint dirt road that extends east-west across the central portion of he site. A small trailer type structure is located in the far southwest corner of the property. Otherwise, the site and vicinity appear much as described for the previous photographs.

6. **February 25, 1986, Flight No. C-450, Photo Nos. 123 & 124, 1" = 2,200'**

There is now lumber stored south of the commercial building that is located at the southwest corner of Mill Street and Arrowhead Avenue and it appears that the residences that were located just southwest of this area may have been removed and/or converted for use as commercial and/or storage purposes. What appears to be a large rain puddle is located at the northwest corner of the intersection of Mountain View Avenue and the onsite storm drain. The far southwest portion of the site, south of Central Avenue, now contains a possible structure along its eastern side and a commercial type business development has been built just east of this portion of the site. The remainder of the site and nearby areas generally appear as described for the previous photographs.

7. **July 1, 1991, Flight No. C-487, Photo Nos. 101 & 102, 1" = 2,200'**

Northwest of the intersection of the onsite storm drain and Mountain View Avenue, the ground appears disturbed, possibly in relation to the modifications to the drainage/storm drain in this area. The modular building that was located southeast of the Mountain View Avenue/storm drain crossing is no longer present but a new modular type building, with a few apparently semi-trucks/trailers near it, is located in the approximate area of the present day Department of Motor Vehicles facility. The structure that may have been present along the east side of the portion of the site located south of Central Avenue is no longer present.

8. **May 31, 1996, Flight No. C-528, Photo Nos. 153 & 154, 1" = 2,200'**

It appears that the residence located east of the intersection of Esperanza Street and Mountain View Avenue has been removed and there are a few more semi-trucks and

trailers located west-northwest of the Department of Motor Vehicles facility. Otherwise, the subject site and immediate surrounding region generally appear as described for the previous photographs.

9. June 15, 2001, Flight No. C-541, Photo Nos. 167 & 168, 1" = 2,300'

There are a few irregularly shaped blotches that range in diameter from approximately 20 to 50 feet, visible in the area just north of Central Avenue, east of Arrowhead Avenue and west of Mountain View Avenue (now barely discernible). These are likely puddles or patches of vegetation. The residential/possible commercial area adjacent to the northern portion of the site has been modified and now appears to be part of an expansion area for the adjacent lumber storage yard. A small structure, possibly a truck, is now situated in the south-central portion of the parcel south of Central Avenue. Otherwise, the site and vicinity appear much as described for the previous photographs.

10. January 18, 2005, Flight No. C-553, Photo Nos. 12-40 & -41, 1" = 1,100'

The southwest parcel, south of Central Avenue, now contains numerous stored items of unknown type and has a modular building along its east side. A combination of puddles and patches of vegetation exists across the site, mainly adjacent to the formerly channelized Warm Creek tributary. The remaining open channel portion of this Warm Creek tributary appears to have been largely filled in northeast of former Mountain View Avenue but it appears that a small portion of this channel that has not been completely filled in still remains just northeast of Mountain View Avenue. Numerous miscellaneous items and truck trailers are stored mainly within the northwest portion of the site, south of Esperanza Street and south of the lumber yard. There is an additional modular building as well as several cars and trucks located in the vicinity of the Department of Motor Vehicles facility in the southeast portion of the property. Lumber storage has increased in the northeast portion of the site and there is a portable water tower/tank located onsite at the northeast corner of Arrowhead Avenue and Central Avenue.

EXISTING SITE CONDITIONS

The subject site area consists largely of open, vacant, relatively flat land of which portions are currently being used for equipment storage. Heavy equipment for auction is currently located within the southwest portion while truck trailers, construction

equipment and some vehicles are also stored within the northern portion, generally south of the intersection of Esperanza Street and former Mountain View Avenue. Almost all of the site is covered by a layer of base materials and, excluding the far northeastern portion where there is a moderate growth of annual grasses and weeds, there is virtually no vegetation within the property. In the southeast corner is a modular building for the Department of Motor Vehicles that serves a truck driver testing center staged in this area of the site.

The property is bordered on the west by Arrowhead Avenue, on the northwest by Esperanza Street, on the south by commercial/industrial developments, on the east by a large open flood control channel and by a lumber yard on the northeast. As discussed within the above review of aerial photographs, a storm drain and associated easement for the San Bernardino County Flood Control District traverses the central portion of the site from northeast to southwest. The area of the storm drain is slightly elevated relative to adjacent areas of the site as crushed rock was recently placed over the easement area, reportedly to reduce the pressures from heavy equipment traffic across the storm drain pipe.

SUBSURFACE FIELD INVESTIGATION

Our subsurface field exploration program was conducted on April 29 and May 2, 2011 and consisted of drilling 11 exploratory borings with a truck-mounted Mobile B61 drill rig equipped with an 8-inch diameter hollow stem auger. The borings were advanced to depths ranging from 12.0 to 51.5 feet below the existing ground surface. In addition, two shallow borings for the sampling of R-value soils and two shallow borings for conducting infiltration tests were drilled. The approximate locations of our exploratory borings are presented on the attached Enclosure A-2 within Appendix A.

The subsurface conditions encountered in the exploratory borings were logged by an engineering geologist from this firm. Relatively undisturbed and bulk samples were obtained at typical maximum depth intervals of 5 feet and returned to our geotechnical laboratory in sealed containers for further testing and evaluation. A detailed description of the field exploration program and the boring logs are presented within 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, laboratory compaction, direct shear, sieve analysis, percent passing #200 sieve, sand equivalent, R-value, and soluble sulfate content. A detailed description of our laboratory testing program and the test results are presented within Appendix C.

GEOLOGIC CONDITIONS

Regional Geologic Setting

Regionally, the site lies within a natural geomorphic province known as the Peninsular Ranges. This province is characterized by northwest trending valleys and mountains that are, in part, due to the tectonic framework of this area, which is also dominated by a northwest trending structure.

Locally, the site is located within a structural, sedimentary mass known as the Bunker Hill - San Timoteo basin. This basin formed as a rift zone between the active San Andreas fault on the northeast and the active San Jacinto fault on the southwest. The southern boundary of the basin is less well defined, but is generally believed to coincide with the Banning fault zone. Sediment within the basin is derived mainly from Warm Creek, Twin Creek, City Creek and the Santa Ana River and also from lesser tributaries. Geologic mapping and subsurface exploration within the area indicates that the site is underlain by Pleistocene and Holocene alluvial sediments overlying crystalline bedrock at depth. The alluvial sediments consist mainly of sand, gravel, cobbles and boulders with interspersed layers of silts and clays. Within the vicinity of the site, the sediments are approximately 1,200 feet in thickness.

Discussion of the tectonic setting of the site can be found in following sections of this report. As mentioned above, the site lies relatively close to several well known active faults. These include the San Jacinto fault located approximately 1.6 kilometers (1 mile) to the southwest, the San Andreas fault located approximately 8.3 kilometers (5.2 miles) to the northeast, the Cucamonga fault approximately 17 kilometers (10.2 miles) to the northwest, and the Elsinore fault located approximately 24 kilometers (14.4 miles) to the southwest.

Site Geologic Conditions

Virtually all of the subject property is mantled with a variable thickness of fill and the entire site is underlain by deposits of alluvium. These units are described in further detail in the following sections.

Fills: Base materials averaging about six inches in thickness and consisting of ground asphalt and soil are present within the portion of the site at the southeast corner of Arrowhead Avenue and Central Avenue while base materials of similar thickness but consisting more often of crushed rock and sand are located across most other portions of the site. Thin remnants of asphalt pavement are present locally, as in the northwest area just southeast of the intersection of Arrowhead Avenue and Esperanza Street. In addition, crushed rock of unknown thickness overlies the storm drain easement that traverses the site from northeast to southwest. Beneath the surficial base, artificial fill soils of variable thickness appear to underlie most areas of the site. The fill soils encountered typically consisted of silty sand and, as encountered within our borings, ranged from less than 2 feet to approximately 9 feet in thickness. The fill soils appear to average approximately 4 feet in thickness, but, as mentioned above, fill thickness across the site is anticipated to be variable. The fill soils were not observed to contain significant quantities of deleterious materials, however, our exploratory borings exposed only a very small fraction of the near surface materials and it is possible that trash and/or debris is present within some of the onsite fill soils. The fills soils sampled in our borings were found to typically be loose to medium dense. Minor amounts of soils are present in the form of stockpiles in the northeast portion of the property. These are mainly located just south of the storm drain easement where it appears they have been placed to fill a local depression in this area.

Alluvium: The fill materials were noted to be underlain by fine to coarse grained materials of native alluvium. The alluvial materials generally consist of poorly to well graded sands with variable percentages of gravel. A small percentage of large gravel/small cobble size materials were encountered, but typically at depths below about 10 feet. The near surface alluvial materials were noted to be damp and in a loose to medium dense state, becoming medium dense to dense at depths typically below approximately 10 to 12 feet.

It should be noted that various authors have divided the local alluvial soils into different units, mainly based upon age and composition. Depending upon the author, two to three alluvial units have been mapped within different portions of the site.

Examples of the alluvial unit separations are shown on the attached geologic maps, Enclosures A-3 and A-4 in Appendix A. While we do not disagree with these unit separations, we have not attempted to separate out the various alluvial units that may be present beneath the site. This is partly due to the fact that the surface of the site is covered by fill soils, thereby making mapping of the units unfeasible, but mainly because the various near surface alluvial soils encountered within the site display fairly uniform engineering characteristics from the geotechnical perspective.

A detailed description of the subsurface soil conditions, as encountered within our exploratory borings, is presented on the Boring Logs within Appendix B.

Groundwater Hydrology

The project site is located within the Bunker Hill sub-basin of the Upper Santa Ana Valley Groundwater Basin. This groundwater basin is bounded by the Loma Linda and San Jacinto faults on the southwest, by the San Bernardino Mountains on the northeast, by non-marine sedimentary hills (badlands) on the south, and by the Crafton Hills, which consist of metamorphic bedrock, on the east. In general, the depth to groundwater has increased since early development within the San Bernardino Valley. Data presented by Fife, et al (1976) indicates that artesian conditions or areas with upper confining layers existed in areas around and including the site in the early 1900's through the early 1960's and that the southeastern portion of the property was within, although along the margin of, a large area that contained bogs, swamps and marsh lands during the 1880's.

Although a few thin (approximately 1 to 3 inch thick) layers of very moist to wet soil were encountered at a couple of locations within our exploratory borings, no groundwater was encountered in any our three deeper borings which extended to depths of 51.5 feet beneath the surface.

A study conducted by the U.S Geological Survey (Matti and Carson, 1991) which addresses the liquefaction susceptibility in the San Bernardino Valley and vicinity included a 1:48,000 scale Contour Map Showing Minimum Depth to Ground Water for the period 1973-1983. This map indicates that the minimum depth to groundwater in the area of the site was as high as approximately 10 feet during that time period. Due to high rainfall and artificial recharge within the basin, the high groundwater conditions lasted until the mid-1980's after which time decreased precipitation, reduced artificial recharge and increased groundwater withdrawal in the area lowered the groundwater table.

Groundwater data prepared by the Department of Water Resources (May, 2011) and the Western Municipal Water District Cooperative Well Measuring Program (Spring, 2005), lists the depth to groundwater recorded in three wells located within one-quarter mile to the west of the site. These are identified as State Well Numbers 01S04W15F006S, 01S04W15F007S, and 01S04W15F008S. During the spring of 2004, the depth to groundwater in these wells ranged from approximately 32 to 38 feet below the surface. Following a steady decline, the depth to groundwater increased in each well through August of 2008. By August of 2008, the depth to groundwater within these wells ranged from 53 to 76 feet below the surface.

Based upon the above information, it appears that the depth to groundwater has been variable over the last century, mainly as a result of changes in demand, recharge and precipitation quantities. The recent declines may be attributed to the rise in population and consumption during this time period and although it appears unlikely that groundwater levels in the area might return to those of 30 years ago, this possibility cannot be precluded.

Surface Runoff

Current surface runoff at the site generally occurs as sheetflow to the southwest.

Mass Movement

The site and surrounding region is situated on a relatively flat surface. The occurrence of mass movement failures such as landslides, rockfalls, or debris flows within such areas is generally not considered common and no evidence of mass movement was observed on the site.

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 Loma Linda fault has been plotted as a concealed or poorly located and inferred fault on some maps (Fife, et al., 1976, for example) and plotted as possibly projecting through the southwest portion of the site (see Enclosure A-4). Another fault, possibly the Banning fault, has been similarly mapped as an inferred fault based upon seismic

data and shown as located just beyond the northeast corner of the site, running roughly parallel to the Loma Linda fault (Fife et al, 1976 and U.S. Geological Survey, 2005). However, surface expression for these faults is lacking and their activity level is uncertain. Therefore, as indicated above, these faults have not been classified as sufficiently active or well-defined and therefore are not included within current State of California Earthquake Fault Zones.

The nearest fault noted in our review of geologic documents is the San Jacinto fault which is located approximately 1.6 kilometers (1 mile) to the southwest. The San Jacinto fault is a sub-parallel branch of the San Andreas fault, 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. The average slip rate on this fault is thought to be on the order of 6 to 12 mm per year and, like the San Andreas fault, is thought to be capable of generating large magnitude events, on the order of 6.5 or greater.

The San Andreas fault itself is located approximately 8.3 kilometers (5.2 miles) to the northeast. The San Andreas fault is considered to be the major tectonic feature of California, separating the Pacific plate from the North American Plate. While estimates vary, the San Andreas fault is generally thought to have an average slip rate on the order of 24 mm per year and capable of generating large magnitude events on the order of 7.5 or greater.

The Cucamonga fault is generally considered to be 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, with San Antonio Canyon generally the dividing point. It is believed that the Sierra Madre/Cucamonga fault system may be capable of producing an earthquake on the order of 7.0 or greater.

The Whittier/Elsinore fault system is located approximately 24kilometers (14.4 miles) to the southwest. The Whittier/Elsinore fault zone is one of the largest in southern California. At its northern end it splays into two segments, named the Chino and Whittier faults, and at its southern end it is cut by the Yuba Wells fault. The primary sense of slip along the Elsinore fault is right lateral strike-slip. It is believed that the

Elsinore fault zone is capable of producing an earthquake magnitude on the order of 6.5 to 7.5.

Current standards of practice have included a discussion of all potential earthquake sources within a 100 kilometer (62 mile) radius. While there are other 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 greater distance and smaller 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. 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 December 31, 2010.

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 depicted on Enclosure A-4, within Appendix A, the site lies within a relatively active region associated with the San Jacinto fault to the southwest and the San Andreas fault to the northeast. Of these events, the closest was a magnitude 4.5 located approximately 3 kilometers (2 miles) to the northwest of the site on January 9, 2009.

In the second search, the micro seismicity of the area lying within a 15 kilometer (9.3 mile) radius of the site was examined by selecting an epicenter map listing events on the order of 1.0 and greater since 1978. In addition, only the "A" events, or most accurate events were selected. Caltech indicates the accuracy of the "A" events to be approximately 1 km. The result 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 depicted on this map, Enclosure A-5, there is distinct clustering located just to the northwest and southeast of the site. These are predominately associated with the San Jacinto fault.

In summary, the historical seismicity of the site entails numerous small to medium magnitude earthquake events occurring around the subject site. Therefore, any future developments at the subject site should anticipate that moderate to large seismic events could occur relatively close to the site.

Secondary Seismic Hazards

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 relatively loose sediments where the groundwater is usually less than 50-feet. Since our review of the seismicity of the site and groundwater data in the project area indicates that there is a potential for a strong ground motion seismic event to occur during the lifetime of the project, as well as groundwater levels as shallow as 10 feet below the existing ground surface, the susceptibility for soil liquefaction at the site was evaluated. This assessment was conducted via analysis of our boring logs, field blow counts, and laboratory soil data.

As displayed on the enclosed boring logs, much of the alluvial materials underlying the site consist of sands and silty sands which are in a loose to dense state. Liquefaction evaluation of these units, by means of the computer program LIQUEFY 2, indicates that some of these soils may be prone to liquefaction. The following table summarizes the results of our liquefaction analyses.

SUMMARY OF LIQUEFACTION ANALYSES (using a PGA = 0.87g and Depth to Groundwater = 10 feet)	
Boring	**Depths of Liquefiable Soil Layers from the Existing Ground Surface**
B-2	from 29 feet to 32 feet
B-7A	from 10 feet to 13 feet
B-10	from 10 feet to 15 feet, from 19 feet to 23 feet, from 44 feet to 47 feet

As noted in the table above, there is a potential for liquefaction to occur typically within relatively thin and scattered sandy alluvial layers located at depths ranging from 10 feet to 47 feet below the ground surface. These results generally agree with the study conducted by the USGS (Matti and Carson, 1991) which suggest that the site is lying within an area with a moderately high to high susceptibility for liquefaction. The results of the computer program analysis utilizing LIQUEFY2 is presented in Appendix E.

It should be explained that this liquefaction evaluation was conducted using the information obtained from our three deep borings (advanced to depths of 51.5 feet) placed at the site which collected SPT blow counts. This data is believed to provide a more reliable assessment of the site in regards to liquefaction than the data obtained from correlations with the California sampler utilized for the remaining, shallow borings.

It should be also pointed out that the above liquefaction calculations were performed using an earthquake ground motion at the site of 0.87g (PGA corresponding to a 475-year return period) caused by a 7.5 Magnitude seismic event and a historic groundwater level of 10 feet below the existing ground surface. The current management and demand of water of the San Bernardino basin will likely maintain water levels significantly below this level and thus help to minimize the liquefaction potential at the site.

Our data suggest that possible manifestations of liquefaction at the site could be in the form of ground settlements, sand boils/ground cracking, and bearing capacity failures of shallow foundations.

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

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

Seismically-Induced Landsliding: Due to the low relief of the site and surrounding region, the potential for landslides to occur at the site is considered nil.

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

SOILS AND SEISMIC DESIGN CRITERIA (California Building Code 2010)

Section 1613 in Chapter 16 of the 2010 California Building Code (CBC) contains the procedures and definitions for the calculations of the earthquake loads on structures and non structural components that are permanently attached to structures and their supports and attachments.

CBC Earthquake Design Summary

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.

CBC 2010 SEISMIC DESIGN SUMMARY Site Location: (USGS WGS84) 34.0877, -117.2878, Occupancy Category II	
Site Class Definition (Table 1613.5.2)	D
S_s Mapped Spectral Response Acceleration at 0.2s Period, (Figure 1613.5(1))	1.711
S_1 Mapped Spectral Response Acceleration at 1s Period, (Figure 1613.5(2))	0.612
F_a Short Period Site Coefficient at 0.2s Period, (Table 1613.5.3(1))	1.0

CBC 2010 SEISMIC DESIGN SUMMARY Site Location: (USGS WGS84) 34.0877, -117.2878, Occupancy Category II	
F_v Long Period Site Coefficient at 1s Period,(Table 1613.5.3(2))	1.5
S_{MS} Adjusted Spectral Response Acceleration at 0.2s Period, (eq .16-36)	1.711
S_{M1}Adjusted Spectral Response Acceleration at 1s Period, (eq .16-37)	0.918
S_{DS} Design Spectral Response Acceleration at 0.2s Period,(eq .16-38)	1.140
S_{D1} Design Spectral Response Acceleration at 1s Period, (eq .16-39)	0.612
Seismic Design Category, Short Period (Table 1613.5.6(1))	D
Seismic Design Category, Long Period (Table 1613.5.6(2))	D

PERCOLATION TESTING PROGRAM

Percolation testing was conducted in the general locations of the proposed bio-swale areas. Standard percolation tests for leach lines were performed.

For our testing, a test hole was excavated to the approximate anticipated proposed depth of the infiltration swale and chamber, approximately 5-feet. Holes were 8.5-inches in diameter. Two inches of gravel was placed in the bottom of the holes and perforated plastic liners were inserted. Test holes were pre-soaked with 5 gallons of water on the same day as excavated. Each test took place after pre-soaking. Measurements of the time for the water level to drop a given amount were taken.

Percolation test results are summarized in the following table. For the detailed field data, see the enclosed test data sheets within Appendix D.

TABLE OF PERCOLATION TEST RESULTS	
Test No.	**Infiltration Rate Gal/Sf/Day**
P-1	350
P-2	910

The clear water absorption rates obtained in our test locations ranged from 350 to 910 gallons per square foot per day. An infiltration rate of 350 gallons per square foot of area per day appears warranted for the site. The recommended rate of 350 gallons per square foot per day does not include a factor of safety. The actual test results are provided on the attached Percolation Test Results sheets, within Appendix D.

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 of the site for commercial/warehouse use is feasible from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into design and implemented during grading and construction.

It should be noted that the subsurface conditions encountered in our exploratory borings are indicative of the locations explored. The subsurface conditions may vary. If conditions are encountered during the construction of the project that differ significantly from those presented in this report, this firm should be notified immediately so we may assess the impact to the recommendations provided.

Foundation Support

Based upon our field investigation and laboratory test data, it is our opinion that the existing fills and upper loose alluvium will not, in their present condition, provide uniform and/or adequate support for the proposed structures. Our field observations, equivalent Standard Penetration Test (SPT) blow counts, and in-situ density test data indicated loose to medium dense in-situ conditions of the existing fill and upper alluvial

materials. These may cause unacceptable differential and/or overall settlements upon application of the anticipated foundation loads.

In addition, the site is located within an area which has been reported to have a moderately high to high potential for liquefaction. Current groundwater use within the region has lowered the water table at the site to a level below the liquefiable soils. While current groundwater is estimated to be present at depths of 53 feet or more, documentation reviewed during our study indicates that in the past this level has risen to about 10 feet from the ground surface. Based on this data and our analysis, if groundwater is allowed to rise to levels of 10 feet in the future, the site will have a potential for liquefaction to occur in the form of sand boils, ground fracturing, settlement, and temporary loss of soil bearing capacity for foundations. While the current use of the water aquifer will most likely sustain the present deep groundwater level, this condition can not be certain. Therefore, we would conclude that future structures at the site should be designed to withstand the potential effects of liquefaction.

To mitigate the consequences of liquefaction at the project, we recommend the use of reinforced foundations systems in combination with the suggested 10 to 12-feet deep removals of unsuitable soils.

Design parameters for these reinforced foundations are given the **RECOMMENDATIONS** section of this report.

Geologic Mitigations

No special mitigation methods are deemed necessary at this time, other than the geotechnical recommendations provided in the following sections.

Seismicity

Seismic ground rupture is generally considered most likely to occur along pre-existing active faults. Since no known active faults are known to exist at, or project into the site, the probability of ground surface rupture occurring at the site is considered nil. However, due to the site's close proximity to the previously described faults, it is reasonable to expect a relatively 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 faults described in this report, from a ground motion standpoint.

The effects of ground shaking anticipated at the subject site should 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).

RECOMMENDATIONS

Geologic Recommendations

No special geologic recommendations are deemed necessary at this time, other than the geotechnical recommendations provided in the following sections.

General Site Grading

All areas to be graded should be stripped of significant vegetation and other deleterious materials. Such materials may not be used as or within engineered fill.

All uncontrolled fills encountered during site preparation should be completely removed, cleaned of significant deleterious materials, and may then be reused as compacted fill. Uncontrolled fills were identified at the site during this study. Septic tanks and/or seepage pits could be encountered during removal operations in areas of previously existing residences, and, if encountered, these will require removal and/or proper abandonment.

It is our recommendation that all existing uncontrolled and/or undocumented fills and buried obstructions under any proposed flatwork and paved areas be removed and replaced with engineered compacted fill. If this is not done, premature structural distress (settlement) of the flatwork and pavement may occur.

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 existing, uncontrolled fill materials and loose to medium dense alluvium should be removed from structural areas and areas to receive structural fills. The data developed during this investigation indicate that removals on the order of 10 to 12 feet will be required across the majority of the site to encounter competent alluvial materials. However, areas requiring deeper removals, such as in the area of our exploratory boring B-3 where a layer/lense of loose sands was encountered, may be anticipated locally.

At boring location B-3, our research indicates that the loose soils encountered may be soils placed within and excavation mad in 1990 in this general area. The excavation and subsequent backfilling was conducted to remove a 10,000 gallon underground storage tank (UST) that previously existed in this general area (see our concurrent Phase 1 Investigation Report for more information pertaining to the UST). It should be noted that no evidence for contaminated soils was encountered within the soils sampled during drilling of this boring.

Competent alluvial materials are native materials with an in-place relative compaction of at least 85 percent.

The actual depths of removal should be verified during the grading operation by observation and in-place density testing.

Preparation of Fill Areas

After conducting the removals discussed above and prior to placing fill, surfaces of competent alluvial areas which are to receive fill should be throughly scarified to a minimum depth of 6 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).

Preparation of Foundation Areas

To lessen the effects of potential liquefaction, proposed site buildings should be safely founded on reinforced foundation systems to enable them to withstand temporary loss of soil bearing capacity, sand boils/ground fractures, and settlement. These systems

should bear on a minimum of 24 inches of engineered compacted fill. The site rough grading and recommended remedial removals are anticipated to provide the required 24-inch compacted fill mat upon which foundations can be placed. The compacted fill mat should extend at least 10 feet beyond the footing lines. Where deeper removals in excess of 10 feet are required, these removals should extend laterally at a 1:1 ratio. The bottom of this excavation should then be scarified to a depth of at least 6 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.

No structure should be placed across any areas where the ratio of the maximum depth of fill to minimum depth of fill is greater than a 3 to 1 ratio as measured from the bottom of the footing. For example, if one area of the building pad establishes 15 feet of fill, below the footing then the remaining pad should be over-excavated as needed to develop a minimum of 5 feet of compacted fill below all footings.

Engineered Compacted Fill

The on-site soils should provide adequate quality fill material, provided they are free from oversized and/or organic matter and other deleterious materials. Unless approved by the geotechnical engineer, rock or similar irreducible material with a maximum dimension greater than 8 inches should not be buried or placed in fills.

Import fill should be inorganic, non-expansive granular soils free from rocks or lumps greater than 8 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.

Short-Term Excavations

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

Short-term excavations of 5 feet deep and greater shall conform to Title 8 of the California Code of Regulations, Construction Safety Orders, Section 1504 and 1539 through 1547. Based on our exploratory borings, it appears that Type C soil is the predominant type of soil on the project and all short-term excavations should be based on this type of soil. Deviation from the standard short-term slopes are permitted using option 4, Design by a Registered Professional Engineer (Section 1541.1).

Short-term slope 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 two horizontal to one 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.

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.

Foundation Design

To lessen the effects of potential liquefaction, proposed buildings should be safely founded on reinforced foundation systems that will enable the buildings to withstand temporary loss of soil bearing capacity, sand boils/ground fractures, and the total and differential settlement indicated in the following section. These systems could consist of continuous spread footings having isolated footings interconnected with grade beams, post-tensioned slabs, or similar systems. A competent structural engineer

experienced in seismic design and liquefaction effects should propose the most adequate foundation system to be used at the site.

Foundation systems should bear on a minimum of 24 inches of engineered compacted fill. All foundations should have a minimum width of 12 inches and should be established a minimum of 12 inches below lowest adjacent grade.

For the minimum width and depth, spread foundations may be designed using an allowable bearing pressure of 1800 psf. This bearing pressure may be increased by 350 psf for each additional foot of width, and by 350 psf for each additional foot of depth, up to a maximum of 4000 psf. For example, a footing 3 feet wide and embedded 1.5 feet will have an allowable bearing pressure of 2,675 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 should not be increased for 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 allowable pressure. Foundations should be setback from slopes as detailed in the California Building Code.

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 should not be increased for seismic loading.

The recommendations for foundation design provided above are minimum. A competent structural engineer experienced in seismic design and liquefaction effects should propose the most adequate foundation systems to be used at the project.

Settlement

Total static settlement of individual foundations will vary depending on the width of the foundation and the actual load supported. Total static settlement of shallow foundations designed and constructed in accordance with the preceding recommendations are estimated to be on the order of 0.5 inch. Differential static settlements between adjacent footings should be about one-half of the total settlement. Settlement of all foundations is expected to occur rapidly, primarily as a result of elastic compression of supporting soils as the loads are applied, and should be essentially completed shortly after initial application of the loads.

Seismically-induced settlement, due to dynamic compression of the upper soils and liquefaction of the loose sandy layers beneath depths of 10 feet within the alluvium, is anticipated to be up to 3.7 inches. Differential seismically-induced settlement is expected to be one-half of the total seismically-induced settlement. Our settlement calculations are presented in Appendix F.

Building Area Slab-on-Grade

To provide adequate support, concrete slabs-on-grade should bear on a minimum of 24 inches of compacted soil. During rough grading, the remedial removals recommended above will most likely provide the recommended 24 inches of compacted soil for adequate support of concrete slabs-on-grade. 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

To provide adequate support, exterior flatwork improvements should rest on a minimum of 12 inches of soil compacted to at least 90 percent (ASTM D 1557).

Wall Pressures

The design of footings for retaining walls should be performed in accordance with the recommendations described earlier under Preparation of Foundation Areas 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 35 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 0.30 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.

The above earth pressure parameters were developed based on the data obtained from testing the on-site granular soils. However, if the site grading requires import fills, the imported soils should be tested in order to modify the actual wall pressures, as necessary.

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.

Preliminary Pavement Design

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

AREAS	DESIGN LIFE (YR)	T.I.	DESIGN R-VALUE	PRELIMINARY SECTION
Parking and Driveway Areas (Light Vehicular Traffic and Occasional Truck Traffic)	10	4.0	50	0.20' AC/0.35' AB or 0.30' AC/Compacted Soil or 5.5" PCC/Compacted Soil
	20	5.0	50	0.20' AC/0.35' AB or 0.35' AC/Compacted Soil or 6.0" PCC/Compacted Soil
Parking and Driveway Areas (Heavy Truck Traffic)	10	6.0	50	0.25' AC/0.35' AB or 0.45' AC/Compacted Soil or 6.5" PCC/Compacted Soil
	20	7.0	50	0.35' AC/0.35' AB or 0.55' AC/Compacted Soil or 7.0" PCC/Compacted Soil

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 portland cement 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 12 to 15-foot intervals within 4 to 6 hours of concrete placement, or preferably 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 design was 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 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 CBC. Therefore, no specific recommendations are given for concrete elements to be in contact with on-site soils.

Percolation

Based upon our field investigation and percolation test data, a design absorption rate of 350 gallons per square foot of trench area per day for the planned infiltration areas appears applicable. This application rate is for clear water and does not incorporate a factor of safety.

To ensure continued percolation capability of the infiltration areas, a program to maintain this facility should be considered. This program should include removal of materials, which can slow the infiltration and decrease the water quality. Materials to

be removed from the catch basin areas typically consist of litter, dead plant matter, and soil fines (silts and clays).

Construction Monitoring

Post investigative services are an important and necessary continuation of this investigation. Project plans and specifications should be reviewed by the geotechnical consultant prior to construction to confirm that the intent of the recommendations presented herein have been incorporated into the design. Verification testing, including R-value and soluble sulfate content, should be performed during 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/or testing include, but are not necessarily limited to, the following:

1. Site preparation-stripping and removals.
2. Excavations, including approval of the bottom of excavation prior to filling.
3. Scarifying and recompacting prior to fill placement.
4. 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.
5. Subgrade preparation for pavements and slabs-on-grade.
6. Foundation excavations.

LIMITATIONS

This report contains geotechnical conclusions and recommendations developed solely for use by Lewis Retail Centers and their related entities 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.

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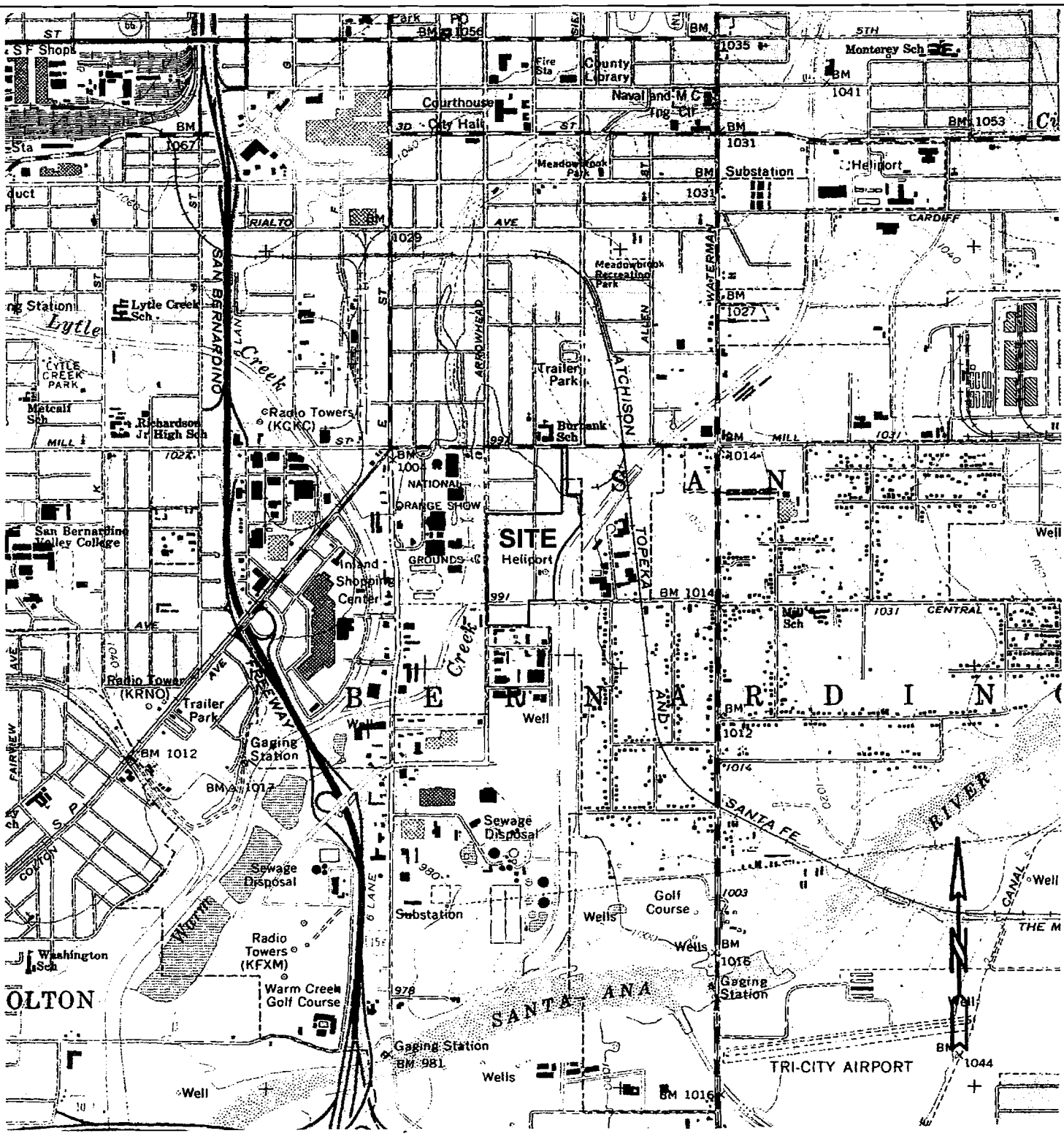
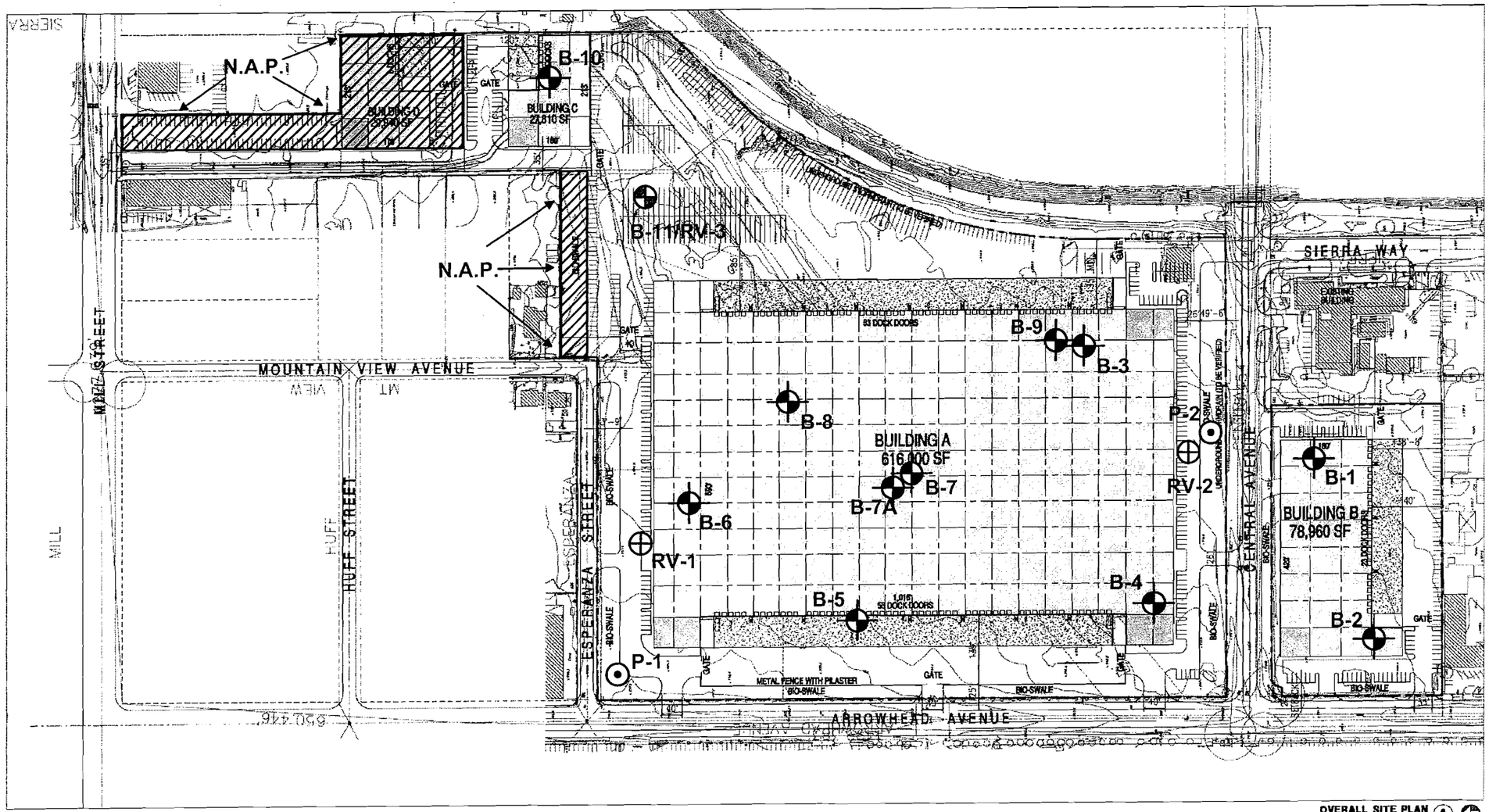
APPENDIX A

Index Map, Boring Location Map, Regional Geologic Map, and Historical Seismicity Maps



INDEX MAP

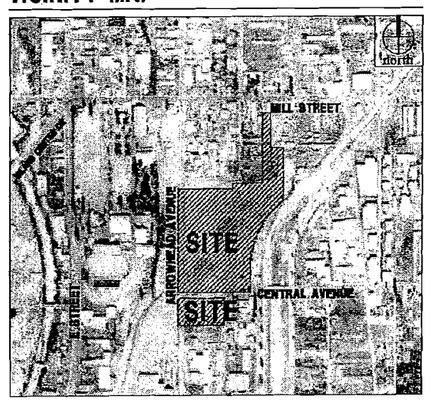
PROJECT:	NATIONAL ORANGE SHOW - SAN BERNARDINO, CALIFORNIA	PROJECT NO.:	12798.1
CLIENT:	LEWIS RETAILS CENTER	ENCLOSURE:	A-1
LOR Geotechnical Group, Inc.		DATE:	MAY, 2011
		SCALE:	1" = 2,000'



OVERALL SITE PLAN

PLAN NORTH

VICINITY MAP



Legend

(Locations Approximate)

Map Symbols

- **B-11** - Exploratory Boring Location
- **RV-3** - R-Value Sample Location
- **P-2** - Percolation Test Location

HPA

hpa, inc.
18851 bardeen avenue - ste. #100
irvine, ca
92612
tel: 949-863-1770
fax: 949-863-0851
email: hpa@hparchi.com

LICENSED ARCHITECT
C-29451
3-31-13
REGISTERED
STATE OF CALIFORNIA

Owner:

LEWIS GROUP OF COMPANIES

Lewis Retail Centers
1156 N. Mountain Avenue
Upland, CA 91786
tel: 909-579-1294
fax: 909-931-5564

Project:

NATIONAL ORANGE SHOW

Arrowhead Avenue
San Bernardino, CA

Consultants:

Civil:
Structural:
Mechanical:
Plumbing:
Electrical:
Landscape:
Fire Protection:
Soils Engineer:

Title: overall site plan

Project Number: 11100
Drawn by: UK
Date: 04-22-2011
Revision

Sheet:

A1.1

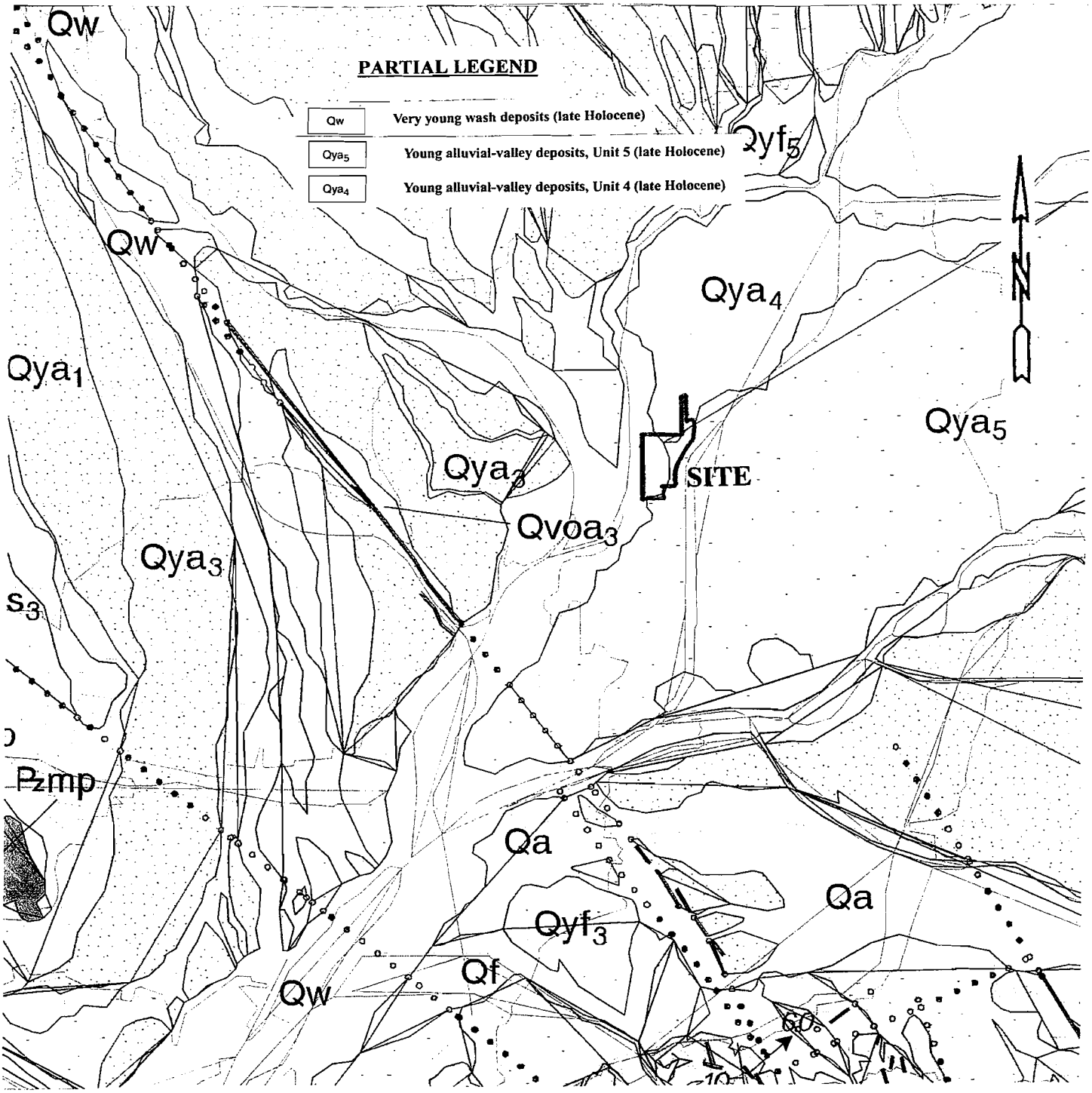
BORING LOCATION MAP

PROJECT: NATIONAL ORANGE SHOW, EAST PARKING LOT, SAN BERNARDINO, CALIFORNIA

CLIENT: LEWIS RETAIL CENTERS

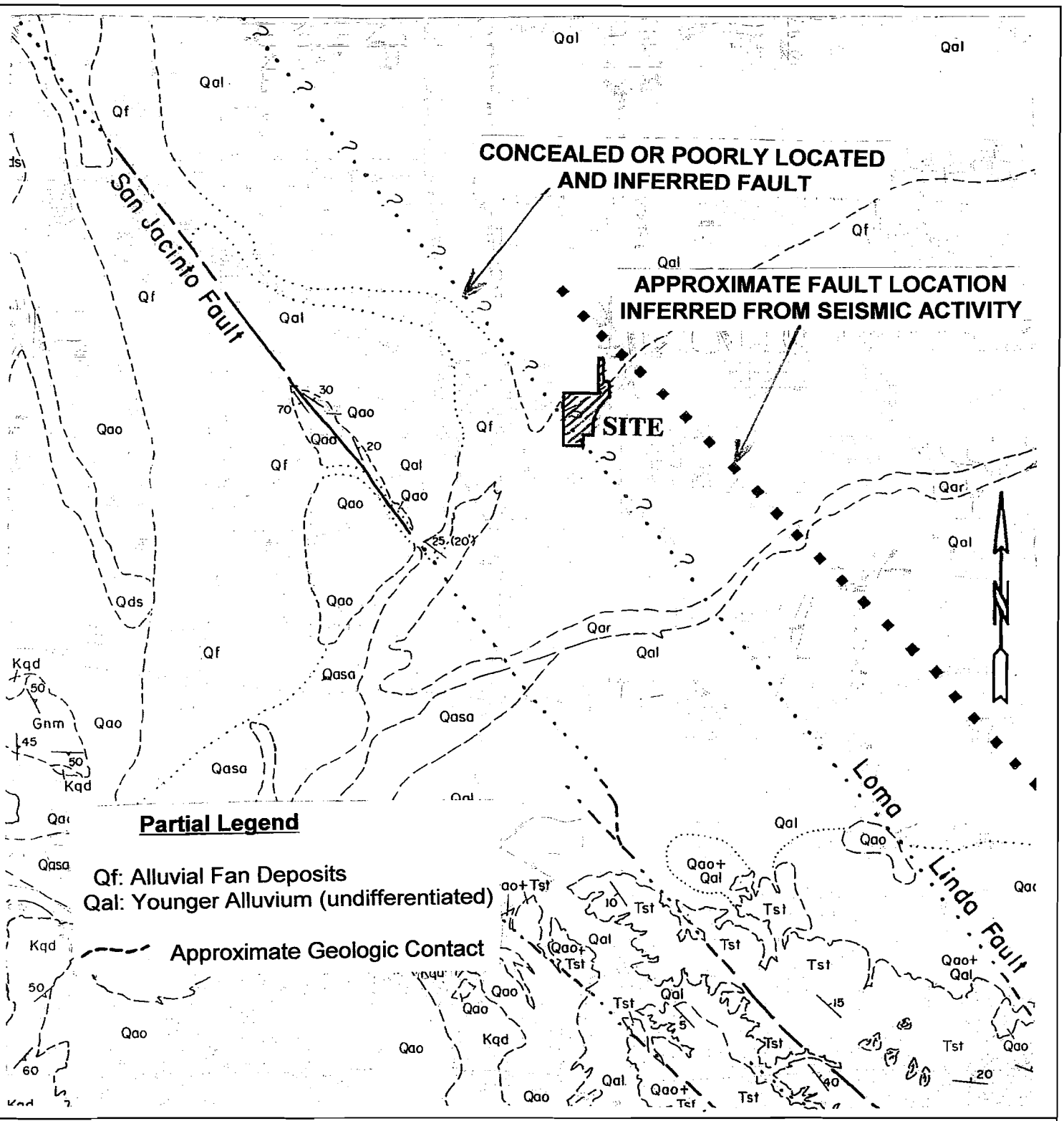
LOR Geotechnical Group, Inc.

PROJECT NO:	12798.1
ENCLOSURE:	A-2
DATE:	MAY 2011
SCALE:	1" ≈ 230'



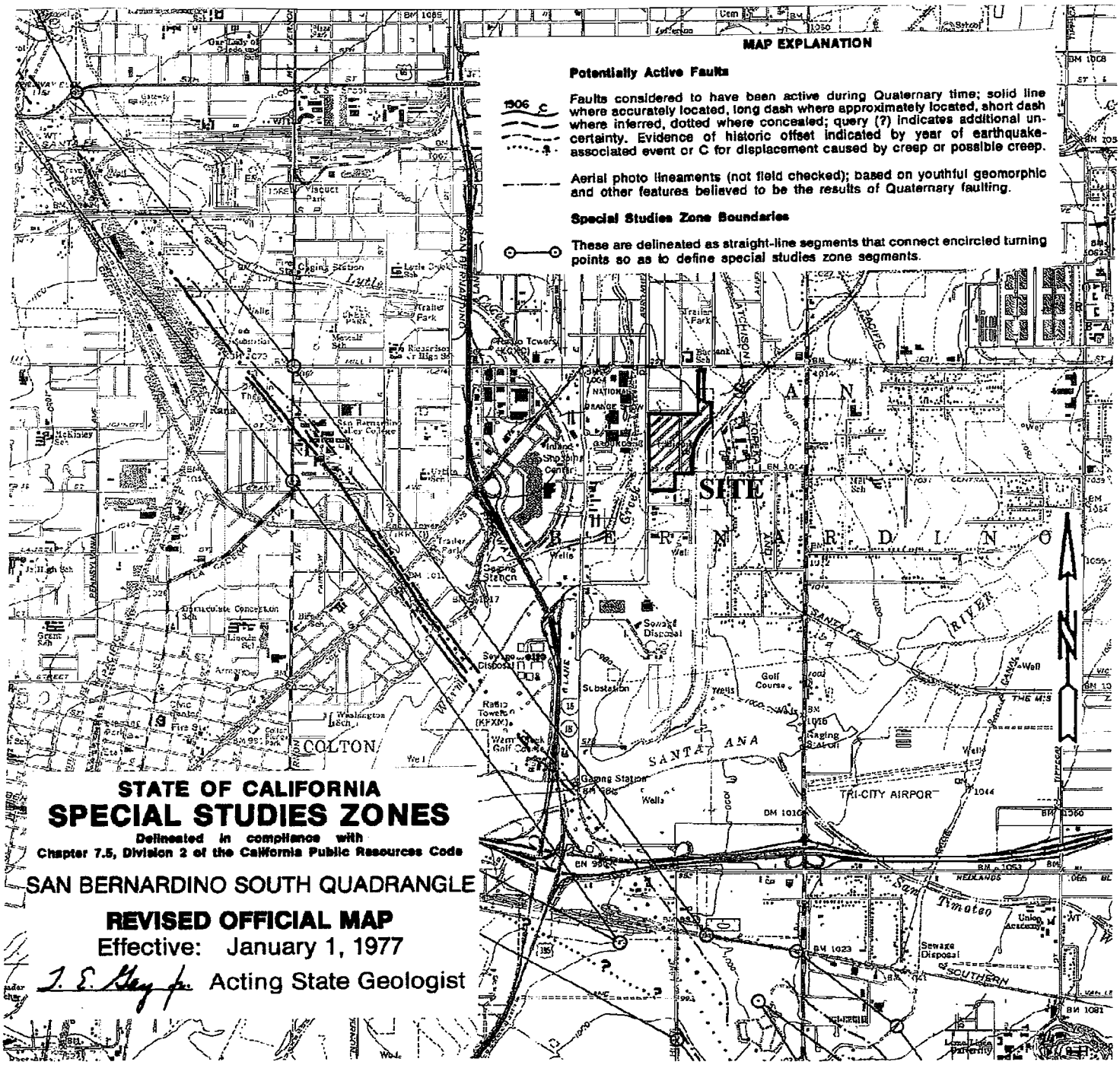
REGIONAL GEOLOGIC MAP

PROJECT:	NATIONAL ORANGE SHOW - SAN BERNARDINO, CALIFORNIA	PROJECT NO.:	12798.1
CLIENT:	LEWIS RETAILS CENTER	ENCLOSURE:	A-3
LOR Geotechnical Group, Inc.		DATE:	MAY, 2011
		SCALE:	1" = 3,700'



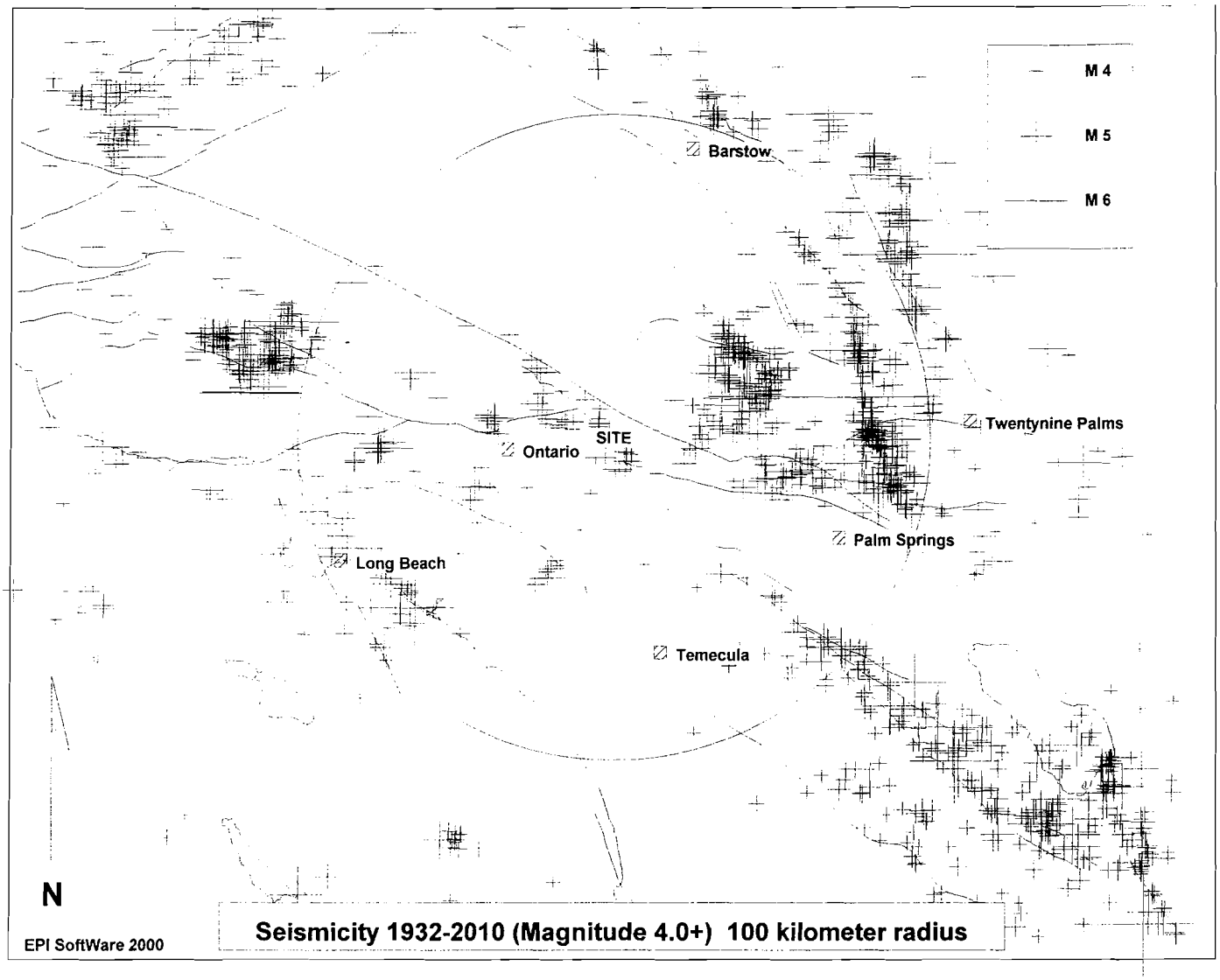
GEOLOGIC MAP, PORTION OF SW SAN BERNARDINO COUNTY

PROJECT:	NATIONAL ORANGE SHOW - SAN BERNARDINO, CALIFORNIA	PROJECT NO.:	12798.1
CLIENT:	LEWIS RETAILS CENTER	ENCLOSURE:	A-4
LOR Geotechnical Group, Inc.		DATE:	MAY, 2011
		SCALE:	1" = 3,850'



EARTHQUAKE FAULT ZONE MAP

PROJECT:	NATIONAL ORANGE SHOW - SAN BERNARDINO, CALIFORNIA	PROJECT NO.:	12798.1
CLIENT:	LEWIS RETAILS CENTER	ENCLOSURE:	A-5
LOR Geotechnical Group, Inc.		DATE:	MAY, 2011
		SCALE:	1" = 3,00'



SITE LOCATION: 34.0877 LAT. -117.2878 LONG.

MINIMUM LOCATION QUALITY: C

TOTAL # OF EVENTS ON PLOT: 1524

TOTAL # OF EVENTS WITHIN SEARCH RADIUS: 650

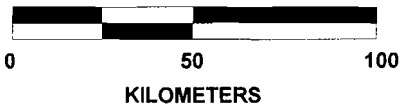
MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

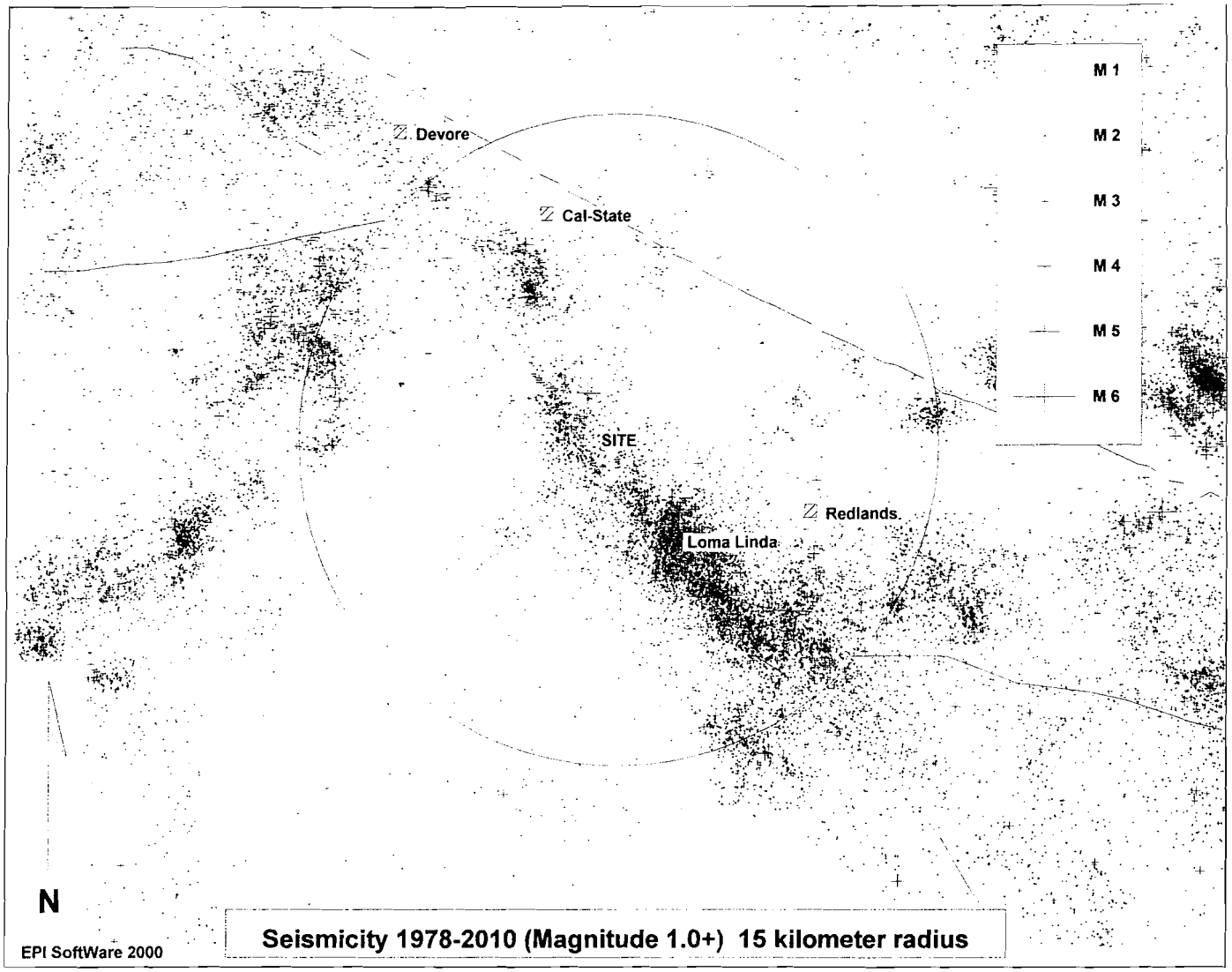
4.0- 4.9 : 588
5.0- 5.9 : 57
6.0- 6.9 : 4
7.0- 7.9 : 1
8.0- 8.9 : 0

CLOSEST EVENT: 4.5 ON FRIDAY, JANUARY 09, 2009 LOCATED APPROX. 3 KILOMETERS NORTHWEST OF THE SITE

LARGEST 5 EVENTS:

7.3 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 79 KILOMETERS EAST OF THE SITE
6.4 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 44 KILOMETERS EAST OF THE SITE
6.4 ON SATURDAY, MARCH 11, 1933 LOCATED APPROX. 81 KILOMETERS SOUTHWEST OF THE SITE
6.1 ON THURSDAY, APRIL 23, 1992 LOCATED APPROX. 90 KILOMETERS EAST OF THE SITE
6.0 ON SATURDAY, DECEMBER 04, 1948 LOCATED APPROX. 85 KILOMETERS EAST OF THE SITE





SITE LOCATION: 34.0877 LAT. -117.2878 LONG.

MINIMUM LOCATION QUALITY: A

TOTAL # OF EVENTS ON PLOT: 15642

TOTAL # OF EVENTS WITHIN SEARCH RADIUS: 6904

0 7 15
KILOMETERS

MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

1.0- 1.9 : 5873
2.0- 2.9 : 951
3.0- 3.9 : 72
4.0- 4.9 : 8
5.0- 5.9 : 0
6.0- 6.9 : 0
7.0- 7.9 : 0
8.0- 8.9 : 0

CLOSEST EVENT: 1.9 ON THURSDAY, NOVEMBER 12, 1992 LOCATED APPROX. .1 KILOMETER OF THE SITE

LARGEST 5 EVENTS:

4.8 ON WEDNESDAY, OCTOBER 02, 1985 LOCATED APPROX. 8 KILOMETERS SOUTHEAST OF THE SITE
4.5 ON FRIDAY, JANUARY 09, 2009 LOCATED APPROX. 2 KILOMETERS NORTHWEST OF THE SITE
4.5 ON WEDNESDAY, MARCH 11, 1998 LOCATED APPROX. 8 KILOMETERS SOUTHEAST OF THE SITE
4.4 ON MONDAY, FEBRUARY 21, 2000 LOCATED APPROX. 5 KILOMETERS SOUTHEAST OF THE SITE
4.3 ON THURSDAY, DECEMBER 28, 1989 LOCATED APPROX. 14 KILOMETERS NORTHWEST OF THE SITE

APPENDIX B

Field Investigation Program and Boring Logs

APPENDIX B
FIELD INVESTIGATION

Subsurface Exploration

Our subsurface exploration of the site consisted of drilling eleven exploratory borings to depths ranging from 12.0 to 51.5 feet below the existing ground surface using a CME drill rig on April 29, 2011 and May 2, 2011. In addition, two shallow borings for the sampling of R-value soils and two shallow borings for conducting infiltration tests were drilled. The approximate locations of the borings are shown on Enclosure A-2 within Appendix A.

The drilling exploration was conducted using a Mobile B61 drill rig equipped with 8-inch diameter hollow stem augers. The soils were continuously logged by a geologist from this firm who inspected the site, created 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 obtained at a typical maximum interval of 5 feet. The samples were recovered by using either a California split barrel sampler of 2.50-inch inside diameter and 3.25-inch outside diameter or a Standard Penetration Test (SPT) sampler. 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 provided in the boring logs, Enclosures B-1 through B-11. To aid in the liquefaction assessment of the project, three deep borings (B-2, B-7A and B-10) were conducted using SPT samplers to obtain directly SPT N-values. The field SPT N-values were further converted to SPT N-values corrected for field procedures ($\approx N_{60}$) which are included in the boring logs Enclosures B-2, B-7A, and B-10. Factors such as efficiency of the automatic trip hammer used during this investigation (80%), sampler type (SPT sampler without a liner with a correction factor of 1.2), and borehole diameter (4-inch inner diameter of the hollow stem auger used with a correction factor of 1.0) were applied to the raw SPT N-values to obtained SPT N_{60}.

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.

All samples obtained were taken to our geotechnical laboratory for storage and testing. Detailed logs of the borings are presented on the enclosed Boring Logs, Enclosures B-1 through B-11. A Boring Log Legend is presented on Enclosure B-i. A Soil Classification Chart is presented as Enclosure B-ii.

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

BORING LOG LEGEND

PROJECT:	APN's 0136-472-01, 02 and 0136-492-01	PROJECT NO.:	12798.1
CLIENT:	Lewis Investment Company, LLC	ENCLOSURE:	B-i
LOR Geotechnical Group, Inc.		DATE:	May 24, 2011

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:	APN's 0136-472-01-02, 0136-492-01	PROJECT NO.:	12798.1
CLIENT:	Lewis Investment Company, LLC.	ENCLOSURE:	B-ii
LOR Geotechnical Group, Inc.		DATE:	May 24, 2011

LOG OF BORING B-1

DEPTH IN FEET	TEST DATA: BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0	15		6.4		97.7			SM	@ 0' FILL BASE: ground asphalt & soil, brown, damp, loose. @ 0.5' ALLUVIUM: SILTY SAND, approximately 5% coarse grained sand, 10% medium grained sand, 65% fine grained sand, 20% silty fines, brown, damp, loose to medium dense.
5	14		7.6		87.3				@ 5.5' +/- thin silty layer
	14		6.5		100.2				
10	26		1.3					SP	@ 10'+/- GRAVELLY SAND, approximately 20% gravel, 10% coarse grained sand, 15% medium grained sand, 50% fine grained sand, 5% silty fines, yellowish brown, damp, medium dense.
15	33		1.5		106.6				@ 17'+/- gravel, possible cobbles.
20	22		15.6		85.8				@ 20' +/- thin silty sand layers included.
25	28		7.0		108.1			SM	@ 24' +/- SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 60% fine grained sand, 20% silty fines, dark grayish brown, moist, slight organic odor, occ, thin sand layers.
								SW	@ 27'+/- GRAVELLY SAND, approximately 15% gravel to (3+/-" diameter) 10% coarse grained sand, 25% medium grained sand, 45% fine grained sand, 5% silty fines, brown to yellowish brown, damp to moist, medium dense to dense.
30	50		1.2		109.4				
35									END OF BORING Fill at 0-0.5 feet No Groundwater No Bedrock

PROJECT:	APN's 0136-472-01,-02 and 0136-492-01	PROJECT NUMBER:	12798.1
CLIENT:	Lewis Investment Company, LLC	ELEVATION:	NI
		DATE DRILLED:	April 29, 2011
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	ENCLOSURE: B-1

LOR GEOTECHNICAL GROUP INC.

LOG OF BORING B-2

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0' FILL BASE: ground asphalt and soil, brown, damp, loose .
	5		7.3		86.8				@ 0.5' ALLUVIUM: SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 45% fine grained sand, 35% silty fines, brown, mosit, loose to medium dense.
5	7								
	9		2.7 15.1		99.8 91.1				Below 6'+/-, increase in percentage of fine grained sand.
10	26		1.2					SW	@ 10'+/- WELL GRADED SAND, approximately 15% coarse grained sand, 40% medium grained sand, 40% fine grained sand, 5% silty fines, yellowish brown, damp, loose to medium dense.
15	35		2.6						@ 15' gravels, possible cobbles @ 17-18'.
20	24		20.1						@ 19' +/- SILTY SAND, approximately 5% coarse grained sand, 10% medium grained sand, 60% fine grained sand, 25% silty fines, yellowish brown, mosit, loose to medium dense.
								SP	@ 23'+/- SAND, approximately 10% medium grained sand, 85% fine grained sand, 5% silty fines, yellowish brown, damp, medium dense.
25	50		4.1						@ approximately 25-26' +/-, Thin dark, grayish brown, silty sand layer with organic odor.
30	11		22.7					SM	@ 29' +/-, SILTY SAND, approximately 5% coarse grained sand, 10% medium grained sand, 50% fine grained sand, 35% silty fines, yellowish brown to grayish-brown, moist (light seepage zone @ 30.5'), loose.
								SW	@ 32'+/- GRAVELLY SAND, approximately 25% gravel, 20% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 5% silty fines, brown, damp, dense.
35	80-3"		2.2						
									@ 38'+/- unit concsists of thin alternating layers of sand, silty sand to sandy silt and gravelly sand.
40	85		11.6						thin sm/ml layer, moist, included (@ 40.5+/-).
45	38		13.5						@ 45-46' sample shows gravel w/ sand, silty sand and sand layers.
50	18		15.5						@ 50.5'+/- light seepage zone
									END OF BORING
55									Fill at 0-5'+/- No Groundwater No Bedrock

PROJECT:	APN's 0136-472-01,-02 and 0136-492-01	PROJECT NUMBER:	12798.1
CLIENT:	Lewis Investment Company, LLC	ELEVATION:	NI
		DATE DRILLED:	May 2, 2011
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	ENCLOSURE: B-2

LOR GEOTECHNICAL GROUP INC.

LOG OF BORING B-3

DEPTH IN FEET	BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0' FILL: f-c sm with gravel (base), grayish bown.
	6		28.6		74.7				@ 0.5' FILL: SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 60% fine grained sand, 20% silty fines, brown, damp, loose.
								SP	@ 4' +/- ALLUVIUM:SAND, approximately 5% coarse grained sand, 10% medium grained sand, 80% fine grained sand, 5% silty fines, light brown, damp, very loose.
5	2		1.7		94.9				
	13		1.6		81.5				clean, fg sand (no cuttings surfacing)
10	4								clean sand (traces observed) still no cuttings
	7		1.4					SW	@ 12' GRAVELLY SAND, approximately 15% gravel, 10% coarse grained sand, 30% medium grained sand, 40% fine grained sand, 5% silty fines, light brown, damp, v-loose
15	10		2.7						@ 15"'GRAVELLY SAND, approximately 15% gravel, 10% coarse grained sand, 30% medium grained sand, 40% fine grained sand, 5% silty fines, light brown, damp, very loose.
	18		2.1		96.4				@ 17' contains traces of gravel (to 1"+/- diameter)
20	27		1.6		98.7			SP	clean, fine to medium grained sand.
								SW	@ 24'+/- WELL GRADED SAND, approximately 10% gravel, 20% coarse grained sand, 35% medium grained sand, 30% fine grained sand, 5% silty fines, light brown to yellowish brown, damp, medium dense to dense.
25	40		1.4						
30	29-4"								
									END OF BORING Fill at 0- 4'+/-feet No Groundwater No Bedrock
35									

PROJECT:	APN's 0136-472-01,-02 and 0136-492-01	PROJECT NUMBER:	12798.1
CLIENT:	Lewis Investment Company, LLC	ELEVATION:	NI
		DATE DRILLED:	May 2, 2011
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	ENCLOSURE: B-3

LOG OF BORING B-4

DEPTH IN FEET	BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0' FILL: SILTY SAND, approximately 5%gravel, 10% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 20% silty fines, brown, damp to moist, loose, contains traces of asphalt.
	18		0.9					SW	@ 2.5'+/- ALLUVIUM: WELL GRADED SAND, approximately 10% mostly fine gravel, 20% coarse grained sand, 35% medium grained sand, 30% fine grained sand, 5% silty fines, light brown, damp, loose to medium dense.
5	23		4.1		123.4				
	26		3.1						@ 7' minor gravel.
10	9		11.9		96.3				@ 10' minor iron-oxide staining, minor gravel to 1.5".
	14		3.2		99.2			SP	@ 12' +/- POORLY GRADED SAND, approximately 5% medium grained sand, 90% fine grained sand, 5% silty fines, yellowish brown, moist, loose to medium dense.
								SW	@ 14'+/- GRAVELLY SAND, approximately 20% gravel, 20% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 5% silty fines, yellowish brown, moist, medium dense.
15	26		5.4		105.2				Below 18'+/- contains occ. layers of silty sand.
20	14		23.7		96.7				
25	45		1.7		110.4				@ 25' sample is of clean, f-c sand, with only a trace of gravel.
30	49		2.4		108.6				@ 30' contains minor gravel
									END OF BORING Fill at 0-2.5+/- feet No Groundwater No Bedrock
35									

PROJECT:	APN's 0136-472-01,-02 and 0136-492-01	PROJECT NUMBER:	12798.1
CLIENT:	Lewis Investment Company, LLC	ELEVATION:	NI
		DATE DRILLED:	April 29, 2011
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	ENCLOSURE: B-4

LOR GEOTECHNICAL GROUP INC.

LOG OF BORING B-5

DEPTH IN FEET	BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0	14		1.6		96.6			SP	@ 0' FILL: POORLY GRADED SAND, approximately 5% coarse grained sand, 15% medium grained sand, 75% fine grained sand, 5% silty fines, light brown, damp, loose, contains traces of red brick/clay pipe and metal (nail).
5	8		9.1		89.1				
	2		31.7		77.5			SM	@ 6' +/- FILL: SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 5% fine grained sand, 30% silty fines, reddish brown, moist, loose.
10	12		4.0		96.5			SW	@ 9' +/- ALLUVIUM: approximately 5% gravel, 20% coarse grained sand, 30% medium grained sand, 40% fine grained sand 5% silty fines, yellowish brown, damp, loose, medium dense.
	28		1.1		108.4				@ 12' Increase in gravel content.
15	24		1.4						@ 15' increase in gravel.
	46		1.4		113.4				@ 17' gravelly sand layer.
20	49		2.0		114.1				@ 20' slightly coarser grained.
25	28		5.9		100.3			SP	@ 23'+/- POORLY GRADED SAND, approximately 5% coarse grained sand, 10% medium grained sand, 80% fine grained sand, 5% silty fines, yellowish brown, moist, medium dense to dense. sample at 25' contains thin sm/ml layers/lenses.
30	64		1.9					SW	@ 28' +/- GRAVELLY SAND, approximately 20% gravel, 15% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 5% silty fines, yellowish brown, damp, dense.
35									END OF BORING Fill at 0-9'+/- No Groundwater No Bedrock.

PROJECT:	APN's 0136-472-01,-02 and 0136-492-01	PROJECT NUMBER:	12798.1
CLIENT:	Lewis Investment Company, LLC	ELEVATION:	NI
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	April 29, 2011
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	ENCLOSURE: B-5

LOG OF BORING B-6

DEPTH IN FEET	BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0	8		22.6		95.1			SM	@ 0' FILL: SILTY SAND, approximately 5% coarse grained sand, 20% medium grained sand, 50% fine grained sand, 25% silty fines, brown, moist, loose.
								SP	@ 4'+/- ALLUVIUM: POOR GRADED SAND, approximately 5% coarse grained sand, 10% medium grained sand, 80% fine grained sand, 5% silty fines, yellowish brown, moist, loose.
5	4		38.7		80.6			SW	@ 6'+/- GRAVELLY SAND, approximately 15% gravel, 15% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 5% silty fines, yellowish brown, damp, loose to medium dense.
	16		5.4		96.1				
10	22		3.8		110.0				
	19		3.1		99.4				
								SM	@ 14' +/- SILTY SAND, approximately 5% coarse grained sand, 10% medium grained sand, 70% fine grained sand, 15% silty fines, yellowish brown to reddish brown, moist, loose.
15	7		34.8		79.8			SP	@ 16'+/- POORLY GRADED SAND, approximately 5% gravel, 10% coarse grained sand, 20% medium grained sand, 60% fine grained sand, 5% silty fines, yellowish brown, damp, medium dense to dense.
	40		2.9		104.1				
20	18		30.5		86.7				@ 20' increase in moisture content.
								SW	@ 23'+/- GRAVELLLY SAND, approximately 20% gravel, 15% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 5% silty fines, yellowish brown, moist, medium dense to dense.
25	25		2.0						below 27' minor to abundant gravel.
30	45		2.1		121.5				
									END OF BORING Fill at 0-4+/-feet No Groundwater No Bedrock.
35									

PROJECT:	APN's 0136-472-01,-02 and 0136-492-01	PROJECT NUMBER:	12798.1
CLIENT:	Lewis Investment Company, LLC	ELEVATION:	NI
		DATE DRILLED:	April 29, 2011
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	ENCLOSURE: B-6

LOG OF BORING B-7

DEPTH IN FEET	BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0	12		28.2		83.3			SM	@ 0' FILL: SILTY SAND, approximately 5% gravel, 10% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 25% silty fines, brown, damp, loose. @ 2' mostly fine to coarse grained sand with silty sand to sandy silty layers, dark gray-brown, sm/ml layer with organic odor @ 2.5'+/-, below 4' mostly f-c, yellowish brown sand with occ, thin silty sand layers.
5	20		3.3					SP	@ 5'+/- ALLUVIUM: POORLY GRADED SAND, approximately 15% medium-coarse grained sand, 80% fine grained sand, 5% silty fines, brown, damp, loose.
	2							SW	@ 7'+/- GRAVELLY SAND, approximately 25% gravel with possible cobbles, 15% coarse grained sand, 20% medium grained sand, 35% fine grained sand, 5% silty fines, brown damp, difficult drilling, spt attempted @ 12' encountered refusal on rocks.
10	10								
15									END OF BORING Fill at 0-5'+/- feet No Groundwater No Bedrock

PROJECT:	APN's 0136-472-01,-02 and 0136-492-01	PROJECT NUMBER:	12798.1
CLIENT:	Lewis Investment Company, LLC	ELEVATION:	NI
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	May 2, 2011
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	ENCLOSURE: B-7

LOG OF BORING B-7A

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0' FILL: SILTY SAND, approximately 5% gravel, 10% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 25% silty fines, brown, damp loose.
									@ 2' mostly fine to coarse grained sand with silt and occ silty sand to sandy silt layers, dark gray-brown sm/ml layer with organic odor @ 2.5+/-, below 4' mostly fine to coarse grained sand, yellowish brown sand with occ, thin silty sand layers.
5								SP	
	9		2.9		106.0			SW	@ 5' ALLUVIUM: POORLY GRADED SAND, approximately 15% medium to coarse grained sand, 80% fine grained sand, 5% silty fines, brown, damp, loose.
								SP	
10	1		11.3		88.2			SM	@ 7' GRAVELLY SAND, approximately 15% gravel, 20% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 5% silty fines, brown, damp, loose.
	12		16.5					SP	
15	23		3.1						@ 9' POORLY GRADED SAND, approximately 5% coarse grained sand, 10% medium grained sand, 80% fine grained sand, 5% silty fines, yellowish brown, damp, loose.
									@ 10.5 SILTY SAND, fine to medium grained, brown, moist, loose.
20	66		2.7						@ 13'+/- POORLY GRADED SAND, approximately 5% coarse grained sand, 15% medium grained sand, 75% fine grained sand, 5% silty fines, yellowish brown, damp, medium dense, to dense, gravel @ 16' 18' and at 22-23'.
25									
	41		4.0						@ 27' light grayish brown, clean, moist.
30	36		3.4						@ 30' Becomes yellowish brown, trace of gravel, increase in medium grained sand, gravels/possible cobbles @ 35.5'
									@ 32' abundant gravel.
35	80-5"		5.4						@ 35.5' gravel/possible cobbles.
								SW	@ 38'+/- GRAVELLY SAND, approximately 20% gravel, 20% coarse grained sand, 25% medium grained sand, 20% fine grained sand, 5% silty fines, yellowish brown, moist, medium dense to dense.
40	42		6.7						
45	32		4.1						@ 45' decrease in gravel & coarse grained sand.
50	70		10.6						Sample at 50' contains a thin, moist layer (1"+/- thick)
									END OF BORING
55									Fill at 0-5'+-/ No Groundwater No Bedrock

PROJECT:	APN's 0136-472-01,-02 and 0136-492-01	PROJECT NUMBER:	12798.1
CLIENT:	Lewis Investment Company, LLC	ELEVATION:	NI
		DATE DRILLED:	May 2, 2011
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	ENCLOSURE: B-7

LOG OF BORING B-8

DEPTH IN FEET	BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0' FILL: SILTY SAND, approximately 5% coarse grained sand, 20% medium grained sand, 40% fine grained sand, 35% silty fines, brown, moist, loose.
	13		935.2		9.9			SW	@ 1.5'+/- ALLUVIUM: GRAVELLY SAND, approximately 15% gravel, 20% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 5% silty fines, yellowish brown, damp, medium dense.
5	27		3.1		112.4				
	10		3.5		109.7				@ 8'-8.5' thin silty sand layer with mild organic odor.
10	13		7.4		105.8				Thin silty sand layer, 3-4" thick & very moist from 10-10.5'
	40		3.6		108.0				@ 12' clean, moist, gravelly sand.
15	56								below 15+/- increase in gravel, possibe cobbles.
20	29-5"								gravels, possible cobbles again @ 20-21'+/-.
	49		4.4		106.2				F-C sand with gravel.
25	52		3.5		114.6				@ 25' weakly mottled, brown, yellowish brown to reddish brown.
30	45		2.5		107.0				@30' clean, f-c sand with gravel, approximately 20% gravel, 25% coarse grained sand, 30% medium grained sand, 20% fine grained sand, 5% silty fines, grayish brown, moist, dense.
35									END OF BORING Fill at 0-1.5 feet No Groundwater No Bedrock

PROJECT:	APN's 0136-472-01,-02 and 0136-492-01	PROJECT NUMBER:	12798.1
CLIENT:	Lewis Investment Company, LLC	ELEVATION:	NI
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	May 2, 2011
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	ENCLOSURE: B-8

LOG OF BORING B-9

DEPTH IN FEET	BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0' FILL: F-C SILTY SAND w/ GRAVEL, approximately 5% coarse grained sand, 15% medium grained sand, 60% fine grained sand, 20% silty fines, brown, damp, loose.
5	16		2.0					SP	@ 4'+/- ALLUVIUM: POORLY GRADED SAND, approximately 10% gravel, 10% coarse grained sand, 30% medium grained sand, 45% fine grained sand, 5% silty fines, yellowish brown, damp, loose to medium dense.
	19		2.0						
10	32		1.5						@ 10' mostly fine to medium grained sand w/ occ. gravels.
	54		0.9					SW	@ 11.5' +/- GRAVELLY SAND, approximately 15% gravel, 15% coarse grained sand, 25% medium grained sand, 40% fine grained sand, 5% silty fines, yellowish brown.
15	40		1.3					SP	@ 15'+/- POORLY GRADED SAND, approximately 5% gravel, 10% coarse grained sand, 20% medium grained sand, 60% fine grained sand, 5% silty fines, yellowish brown, damp, medium dense to dense.
20	6		14.7						Thin silty sand layer @ 20' (sample = mix of f-m sand & sm).
25	43		2.6					SW	@ 24' +/- WELL GRADED SAND, approximately 10% gravel, 20% coarse grained sand, 35% medium grained sand, 30% fine grained sand, 5% silty fines, yellowish brown, damp, medium dense.
30									

END OF BORING

Fill at 0-4+/- feet
No Groundwater
No Bedrock

PROJECT:	APN's 0136-472-01,-02 and 0136-492-01	PROJECT NUMBER:	12798.1
CLIENT:	Lewis Investment Company, LLC	ELEVATION:	NI
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	May 2, 2011
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	ENCLOSURE: B-9

LOG OF BORING B-10

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0	13		8.0		111.2			SM	@ 0' FILL: SILTY SAND, approximately 5% gravel, 10% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 30% silty fines, brown, moist, loose to medium dense.
5	4		4.2		86.4			SP	@ 4'+/- ALLUVIUM: POORLY GRADED SAND, approximately 5% gravel, 10% coarse grained sand, 20% medium grained sand, 60% fine grained sand, 5% silty fines, yellowish brown, damp, loose.
	4		4.6		81.2				
10	7								
	16		3.4						@ 12' slight increase in coarse grained sand and gravel.
15	32		2.9						@ 15' less coarse grained sand and gravel, grayish brown in color.
20	21		15.6					SM	@ 19'+/- SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 50% fine grained sand, 30% silty fines, yellowish brown, moist, loose to medium dense, contains occ. thin (1-2" thick), orgainic rich layers.
25	34		1.5					SP	@ 23'+/- POORLY GRADED SAND, approximately 5% gravel, 10% coarse grained sand, 25% medium grained sand, 55% fine grained sand, 5% silty fines, light brown, damp, medium dense to dense.
								SW	@ 27'+/- WELL GRADED SAND W/ GRAVEL, approximately 10% gravel, 15% coarse grained sand, 30% medium grained sand, 40% fine grained sand, 5% silty fines, light brown, damp, dense.
30	54		1.7						
35	78		4.0						@ 35' includes thin(1" thick) silty sand layer.
40	45		2.9					SP	@ 38'+/- POORLY GRADED SAND, approximately 5% coarse grained sand, 15% medium grained sand, 25% fine grained sand, 5% silty fines, light brown, damp, medium dense to dense.
45	29		25.6					SM	@ 44'+/- SILTY SAND: approximately 5% coarse grained sand, 10% medium grained sand, 50% fine grained sand, 35% silty fines, brown to grayish brown, moist, medium dense.
								SW	@ 47'+/- GRAVELLY SAND, approximately 20% gravel, 15% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 5% silty fines, light brown, damp to moist, dense.
50	61		1.9						
55									END OF BORING Fill at 0-4 +/- feet No Groundwater No Bedrock

PROJECT:	APN's 0136-472-01,-02 and 0136-492-01	PROJECT NUMBER:	12798.1
CLIENT:	Lewis Investment Company, LLC	ELEVATION:	NI
		DATE DRILLED:	May 2, 2011
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	ENCLOSURE: B-10

LOG OF BORING B-11

DEPTH IN FEET	BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0' FILL: SILTY SAND, approximately 15% gravel, 15% coarse grained sand, 20% medium grained sand, 30% fine grained sand, 20% silty fines, brown, damp, loose (rocks @ 2-3').
	7		2.5		90.1				@ 3' ALLUVIUM: SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 50% fine grained sand, 30% silty fines, brown, damp, loose.
5	9		1.7					SP	@ 6' POORLY GRADED SAND, approximately 5% gravel, 10% coarse grained sand, 25% medium grained sand, 55% fine grained sand, 5% silty fines, brown, damp, loose.
	11		1.8						
10	12		1.3						@ 10' slight increase in gravel.
	28		11.3		100.5			SM	@ 11'+/- SILTY SAND, approximately 5% coarse grained sand, 10% medium grained sand, 50% fine grained sand, 35% silty fines, brown, moist, medium dense.
15	22		5.7		95.4			SP	@ 14'+/- POORLY GRADED SAND, approximately 5% coarse graded sand, 20% medium grained sand, 70% fine grained sand, 5% silty fines, yellowish brown, damp, medium dense.
20	28		7.8		99.2				@ 19'+/- becomes grayish brown, approximately 5% medium to coarse grained sand, 90% fine grained sand, 5% silty fines.
25	55		47.9		79.2			SW	@ 24'+/- GRAVELLY SAND, approximately, 25% gravel, 15% coarse grained sand, 30% medium grained sand, 25% fine grained sand, 5% silty fines, yellowish brown, damp, dense.
30	31		1.7						@ 30.5'+/- Thin lens of grayish brown, clayey silt encountered, otherwise, sample consists of gravelly sand.
35									END OF BORING Fill at 0-3+/- feet No Groundwater No Bedrock

PROJECT:	APN's 0136-472-01,-02 and 0136-492-01	PROJECT NUMBER:	12798.1
CLIENT:	Lewis Investment Company, LLC	ELEVATION:	NI
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	May 2, 2011
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	ENCLOSURE: B-11

APPENDIX C

Laboratory Testing Program and Test Results

APPENDIX C
LABORATORY TESTING

General

Selected soil samples obtained from the borings were tested in our geotechnical 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, sieve analysis, percent passing # 200 sieve, 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, in accordance with ASTM D 2921 and ASTM D 2216, respectively, and the results are shown on the boring logs, Enclosures B-1 through B-11, 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 compaction test method. The results are presented in the following table:

LABORATORY COMPACTION				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Maximum Dry Density (pcf)**	**Optimum Moisture Content (percent)**
B-2	3-7	(ML/SM) Sandy Silt/Silty Sand	110.0	15.5
B-9	4-7	(SP) Poorly Graded Sand	116.5	8.0

Direct Shear Tests

Shear tests are performed in general accordance with ASTM D 3080 with a direct shear machine at a constant rate-of-strain (0.04 inches/minute). The machine is designed to test a sample partially extruded from a sample ring in single shear. Sámples are tested at varying normal loads in order to evaluate the shear strength parameters, angle of internal friction and cohesion. Samples are tested in remolded condition (90 percent relative compaction per ASTM D 1557) and soaked, to represent the worse case conditions expected in the field.

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

DIRECT SHEAR TESTS				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Cohesion (psf)**	**Angle of Internal Friction (degrees)**
B-2	3-7	(ML/SM) Sandy Silt/Silty Sand	280	30
B-9	4-7	(SP) Poorly Graded Sand	0	35

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 Enclosures C-1 through C-3.

Percent Passing No. 200 Sieve Tests

A quantitative determination of the percentage of soil passing the No. 200 sieve was performed for selected samples. The results indicate the percentage of fines in the soil. The results are presented in the following table:

PERCENT PASSING NO. 200 SIEVE TESTS			
Boring Number	Sample Depth (feet)	Soil Description	Percent by Weight Passing No. 200 Sieve (%)
B-2	5	(ML) Sandy Silt	61
B-3	2	(ML) Sandy Silt	90
B-5	7	(ML) Sandy Silt	54
B-7A	7	(SP/SW) Gravelly Sand	2
B-10	45	(SM) Silty Sand	48
B-11	12	(SM) Silty Sand	39

Sand Equivalent

The sand equivalent of selected 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 Enclosures C-1 through 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 Enclosures C-1 through 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			
Boring Number	Sample Depth (feet)	Soil Description (U.S.C.S.)	Sulfate Content (percent by weight)
B-2	3-7	(ML/SM) Sandy Silt/Silty Sand	<0.005
B-9	4-7	(SP) Poorly Graded Sand	0.009
B-11	0-3	(SM) Silty Sand with Gravel	0.015

APPENDIX D

Percolation Testing

LEACH LINE PERCOLATION TEST DATA/CONTINUOUS PRE-SOAK

Project:	APN's 0136-472-01, -02 & 0136-492-01	Test Date:	May 2, 2011
Project No.:	12798.1	Test Hole No.:	P-1
Soil Classification:	SP	Test Hole Size:	12"X10"
Depth of Test Hole:	5	Date Excavated/Pre-Soaked:	May 2, 2011

PRE-SOAK PERIOD

Time Interval		Amount of Water Used	
Start:	9:46:00 AM	5	Gal.
Stop:	10:17:00 AM		

TEST PERIOD					
TIME	TIME INTERVALS (Minutes)	INITIAL WATER VOLUME (Gallon)	FINAL WATER VOLUME (Gallon)	TOTAL WATER USED (Gallon)	PERCOLATION RATE (Gal/Sf/Day)
10:24:30 AM	6::5			5	350
10:31:00 AM					

LEACH LINE PERCOLATION TEST DATA/CONTINUOUS PRE-SOAK

Project:	APN's 0136-472-01, -02 & 0136-492-01	Test Date:	May 2, 2011
Project No.:	12798.1	Test Hole No.:	P-2
Soil Classification:	SP	Test Hole Size:	12"X10"
Depth of Test Hole:	5	Date Excavated/Pre-Soaked:	May 2, 2011

PRE-SOAK PERIOD

Time Interval		Amount of Water Used	
Start:	10:04:00 AM	5	Gal.
Stop:	10:06:00 AM		

TEST PERIOD					
TIME	TIME INTERVALS (Minutes)	INITIAL WATER VOLUME (Gallon)	FINAL WATER VOLUME (Gallon)	TOTAL WATER USED (Gallon)	PERCOLATION RATE (Gal/Sf/Day)
10:10:00 AM	2:50			5	910
10:12:30 AM					

APPENDIX E

Liquefaction Calculations

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*****************************
*                           *
*      L I Q U E F Y 2      *
*                           *
*       Version 1.50        *
*                           *
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EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: 12798.1 DATE: 05-17-2011

JOB NAME: Boring B-2

SOIL-PROFILE NAME: 12798b2.LDW

BORING GROUNDWATER DEPTH: 53.00 ft

CALCULATION GROUNDWATER DEPTH: 10.00 ft

DESIGN EARTHQUAKE MAGNITUDE: 7.50 Mw

SITE PEAK GROUND ACCELERATION: 0.870 g

BOREHOLE DIAMETER CORRECTION FACTOR: 1.00

SAMPLER SIZE CORRECTION FACTOR: 1.00

N60 HAMMER CORRECTION FACTOR: 1.00

MAGNITUDE SCALING FACTOR METHOD: Idriss (1997, in press)

Magnitude Scaling Factor: 1.000

rd-CORRECTION METHOD: NCEER (1997)

FIELD SPT N-VALUES ARE NOT CORRECTED FOR THE LENGTH OF THE DRIVE RODS.

Rod Stick-Up Above Ground: 5.0 ft

CN NORMALIZATION FACTOR: 1.044 tsf

MINIMUM CN VALUE: 0.6

NCEER [1997] Method

LIQUEFACTION ANALYSIS SUMMARY

PAGE 1

File Name: 12798b2.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	0.25	0.013	0.013	5	6.25	*	*	*	*	*	**
1	0.75	0.039	0.039	5	6.25	*	*	*	*	*	**
1	1.25	0.066	0.066	5	6.25	*	*	*	*	*	**
1	1.75	0.092	0.092	5	6.25	*	*	*	*	*	**
1	2.25	0.118	0.118	5	6.25	*	*	*	*	*	**
1	2.75	0.144	0.144	5	6.25	*	*	*	*	*	**
1	3.25	0.171	0.171	5	6.25	*	*	*	*	*	**
1	3.75	0.197	0.197	5	6.25	*	*	*	*	*	**
1	4.25	0.223	0.223	5	6.25	*	*	*	*	*	**
1	4.75	0.249	0.249	5	6.25	*	*	*	*	*	**
2	5.25	0.276	0.276	7	7.60	*	*	*	*	*	**
2	5.75	0.302	0.302	7	7.60	*	*	*	*	*	**
2	6.25	0.328	0.328	7	7.60	*	*	*	*	*	**
2	6.75	0.354	0.354	7	7.60	*	*	*	*	*	**
3	7.25	0.381	0.381	9	6.97	*	*	*	*	*	**
3	7.75	0.407	0.407	9	6.97	*	*	*	*	*	**
3	8.25	0.433	0.433	9	6.97	*	*	*	*	*	**
3	8.75	0.459	0.459	9	6.97	*	*	*	*	*	**
3	9.25	0.486	0.486	9	6.97	*	*	*	*	*	**
3	9.75	0.512	0.512	9	6.97	*	*	*	*	*	**
4	10.25	0.538	0.530	26	0.05	1.376	35.8	Infin	0.976	0.560	NonLiq
4	10.75	0.564	0.541	26	0.05	1.376	35.8	Infin	0.975	0.575	NonLiq
4	11.25	0.591	0.552	26	0.05	1.376	35.8	Infin	0.974	0.590	NonLiq
4	11.75	0.617	0.562	26	0.05	1.376	35.8	Infin	0.973	0.603	NonLiq
4	12.25	0.643	0.573	26	0.05	1.376	35.8	Infin	0.971	0.617	NonLiq
4	12.75	0.669	0.584	26	0.05	1.376	35.8	Infin	0.970	0.629	NonLiq
4	13.25	0.696	0.594	26	0.05	1.376	35.8	Infin	0.969	0.642	NonLiq
4	13.75	0.722	0.605	26	0.05	1.376	35.8	Infin	0.968	0.653	NonLiq
4	14.25	0.748	0.616	26	0.05	1.376	35.8	Infin	0.967	0.664	NonLiq
4	14.75	0.774	0.626	26	0.05	1.376	35.8	Infin	0.966	0.675	NonLiq
5	15.25	0.801	0.637	35	0.05	1.133	39.7	Infin	0.964	0.686	NonLiq
5	15.75	0.827	0.647	35	0.05	1.133	39.7	Infin	0.963	0.696	NonLiq
5	16.25	0.853	0.658	35	0.05	1.133	39.7	Infin	0.962	0.705	NonLiq
5	16.75	0.879	0.669	35	0.05	1.133	39.7	Infin	0.961	0.715	NonLiq
5	17.25	0.906	0.679	35	0.05	1.133	39.7	Infin	0.960	0.723	NonLiq
5	17.75	0.932	0.690	35	0.05	1.133	39.7	Infin	0.959	0.732	NonLiq
5	18.25	0.958	0.701	35	0.05	1.133	39.7	Infin	0.957	0.740	NonLiq
5	18.75	0.984	0.711	35	0.05	1.133	39.7	Infin	0.956	0.748	NonLiq
6	19.25	1.011	0.722	24	9.64	0.985	33.3	Infin	0.955	0.756	NonLiq
6	19.75	1.037	0.733	24	9.64	0.985	33.3	Infin	0.954	0.763	NonLiq
6	20.25	1.063	0.743	24	9.64	0.985	33.3	Infin	0.953	0.771	NonLiq
6	20.75	1.089	0.754	24	9.64	0.985	33.3	Infin	0.952	0.778	NonLiq
6	21.25	1.116	0.765	24	9.64	0.985	33.3	Infin	0.950	0.784	NonLiq

NCEER [1997] Method — LIQUEFACTION ANALYSIS SUMMARY — PAGE 2

File Name: 12798b2.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
6	21.75	1.142	0.775	24	9.64	0.985	33.3	Infin	0.949	0.791	NonLiq
6	22.25	1.168	0.786	24	9.64	0.985	33.3	Infin	0.948	0.797	NonLiq
6	22.75	1.194	0.797	24	9.64	0.985	33.3	Infin	0.947	0.803	NonLiq
7	23.25	1.221	0.807	50	0.06	0.883	44.2	Infin	0.946	0.809	NonLiq
7	23.75	1.247	0.818	50	0.06	0.883	44.2	Infin	0.945	0.814	NonLiq
7	24.25	1.273	0.829	50	0.06	0.883	44.2	Infin	0.943	0.820	NonLiq
7	24.75	1.299	0.839	50	0.06	0.883	44.2	Infin	0.942	0.825	NonLiq
7	25.25	1.326	0.850	50	0.06	0.883	44.2	Infin	0.941	0.830	NonLiq
7	25.75	1.352	0.860	50	0.06	0.883	44.2	Infin	0.940	0.835	NonLiq
7	26.25	1.378	0.871	50	0.06	0.883	44.2	Infin	0.939	0.840	NonLiq
7	26.75	1.404	0.882	50	0.06	0.883	44.2	Infin	0.938	0.844	NonLiq
7	27.25	1.431	0.892	50	0.06	0.883	44.2	Infin	0.936	0.849	NonLiq
7	27.75	1.457	0.903	50	0.06	0.883	44.2	Infin	0.935	0.853	NonLiq
7	28.25	1.483	0.914	50	0.06	0.883	44.2	Infin	0.934	0.857	NonLiq
7	28.75	1.509	0.924	50	0.06	0.883	44.2	Infin	0.933	0.861	NonLiq
8	29.25	1.536	0.935	11	6.73	0.807	15.6	0.170	0.932	0.865	0.20
8	29.75	1.562	0.946	11	6.73	0.807	15.6	0.170	0.931	0.869	0.20
8	30.25	1.588	0.956	11	6.73	0.807	15.6	0.170	0.928	0.871	0.20
8	30.75	1.614	0.967	11	6.73	0.807	15.6	0.170	0.924	0.872	0.19
8	31.25	1.641	0.978	11	6.73	0.807	15.6	0.170	0.920	0.873	0.19
8	31.75	1.667	0.988	11	6.73	0.807	15.6	0.170	0.916	0.873	0.19
9	32.25	1.693	0.999	80	0.07	0.748	59.9	Infin	0.912	0.874	NonLiq
9	32.75	1.719	1.010	80	0.07	0.748	59.9	Infin	0.907	0.874	NonLiq
9	33.25	1.746	1.020	80	0.07	0.748	59.9	Infin	0.903	0.874	NonLiq
9	33.75	1.772	1.031	80	0.07	0.748	59.9	Infin	0.899	0.874	NonLiq
9	34.25	1.798	1.042	80	0.07	0.748	59.9	Infin	0.895	0.874	NonLiq
9	34.75	1.824	1.052	80	0.07	0.748	59.9	Infin	0.891	0.874	NonLiq
9	35.25	1.851	1.063	80	0.07	0.748	59.9	Infin	0.887	0.874	NonLiq
9	35.75	1.877	1.073	80	0.07	0.748	59.9	Infin	0.883	0.873	NonLiq
9	36.25	1.903	1.084	80	0.07	0.748	59.9	Infin	0.879	0.873	NonLiq
9	36.75	1.929	1.095	80	0.07	0.748	59.9	Infin	0.875	0.872	NonLiq
9	37.25	1.956	1.105	80	0.07	0.748	59.9	Infin	0.871	0.871	NonLiq
9	37.75	1.982	1.116	80	0.07	0.748	59.9	Infin	0.867	0.870	NonLiq
9	38.25	2.008	1.127	80	0.07	0.748	59.9	Infin	0.863	0.870	NonLiq
9	38.75	2.034	1.137	80	0.07	0.748	59.9	Infin	0.859	0.869	NonLiq
9	39.25	2.061	1.148	80	0.07	0.748	59.9	Infin	0.855	0.867	NonLiq
9	39.75	2.087	1.159	80	0.07	0.748	59.9	Infin	0.851	0.866	NonLiq
10	40.25	2.113	1.169	85	16.71	0.701	76.3	Infin	0.846	0.865	NonLiq
10	40.75	2.139	1.180	85	16.71	0.701	76.3	Infin	0.842	0.864	NonLiq
10	41.25	2.166	1.191	85	16.71	0.701	76.3	Infin	0.838	0.862	NonLiq
10	41.75	2.192	1.201	85	16.71	0.701	76.3	Infin	0.834	0.861	NonLiq
10	42.25	2.218	1.212	85	16.71	0.701	76.3	Infin	0.830	0.859	NonLiq
10	42.75	2.244	1.223	85	16.71	0.701	76.3	Infin	0.826	0.858	NonLiq
10	43.25	2.271	1.233	85	16.71	0.701	76.3	Infin	0.822	0.856	NonLiq

NCEER [1997] Method — LIQUEFACTION ANALYSIS SUMMARY — PAGE 3

File Name: 12798b2.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
10	43.75	2.297	1.244	85	16.71	0.701	76.3	Infin	0.818	0.854	NonLiq
10	44.25	2.323	1.255	85	16.71	0.701	76.3	Infin	0.814	0.852	NonLiq
10	44.75	2.349	1.265	85	16.71	0.701	76.3	Infin	0.810	0.850	NonLiq
11	45.25	2.376	1.276	38	9.93	0.661	35.0	Infin	0.806	0.848	NonLiq
11	45.75	2.402	1.286	38	9.93	0.661	35.0	Infin	0.802	0.846	NonLiq
11	46.25	2.428	1.297	38	9.93	0.661	35.0	Infin	0.798	0.844	NonLiq
11	46.75	2.454	1.308	38	9.93	0.661	35.0	Infin	0.794	0.842	NonLiq
11	47.25	2.481	1.318	38	9.93	0.661	35.0	Infin	0.789	0.840	NonLiq
11	47.75	2.507	1.329	38	9.93	0.661	35.0	Infin	0.785	0.838	NonLiq
11	48.25	2.533	1.340	38	9.93	0.661	35.0	Infin	0.781	0.835	NonLiq
11	48.75	2.559	1.350	38	9.93	0.661	35.0	Infin	0.777	0.833	NonLiq
11	49.25	2.586	1.361	38	9.93	0.661	35.0	Infin	0.773	0.831	NonLiq
11	49.75	2.612	1.372	38	9.93	0.661	35.0	Infin	0.769	0.828	NonLiq
11	50.25	2.638	1.382	38	9.93	0.661	35.0	Infin	0.765	0.826	NonLiq
11	50.75	2.664	1.393	38	9.93	0.661	35.0	Infin	0.761	0.823	NonLiq
11	51.25	2.691	1.404	38	9.93	0.661	35.0	Infin	0.757	0.821	NonLiq

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*      L I Q U E F Y 2      *
*                           *
*       Version 1.50        *
*                           *
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EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: 12798.1 DATE: 05-17-2011

JOB NAME: Boring B-7

SOIL-PROFILE NAME: 12798b7.LDW

BORING GROUNDWATER DEPTH: 53.00 ft

CALCULATION GROUNDWATER DEPTH: 10.00 ft

DESIGN EARTHQUAKE MAGNITUDE: 7.50 Mw

SITE PEAK GROUND ACCELERATION: 0.870 g

BOREHOLE DIAMETER CORRECTION FACTOR: 1.00

SAMPLER SIZE CORRECTION FACTOR: 1.00

N60 HAMMER CORRECTION FACTOR: 1.00

MAGNITUDE SCALING FACTOR METHOD: Idriss (1997, in press)

Magnitude Scaling Factor: 1.000

rd-CORRECTION METHOD: NCEER (1997)

FIELD SPT N-VALUES ARE NOT CORRECTED FOR THE LENGTH OF THE DRIVE RODS.

Rod Stick-Up Above Ground: 5.0 ft

CN NORMALIZATION FACTOR: 1.044 tsf

MINIMUM CN VALUE: 0.6

NCEER [1997] Method

LIQUEFACTION ANALYSIS SUMMARY

File Name: 12798b7.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	0.25	0.013	0.013	7	5.90	*	*	*	*	*	**
1	0.75	0.039	0.039	7	5.90	*	*	*	*	*	**
1	1.25	0.066	0.066	7	5.90	*	*	*	*	*	**
1	1.75	0.092	0.092	7	5.90	*	*	*	*	*	**
1	2.25	0.118	0.118	7	5.90	*	*	*	*	*	**
1	2.75	0.144	0.144	7	5.90	*	*	*	*	*	**
1	3.25	0.171	0.171	7	5.90	*	*	*	*	*	**
1	3.75	0.197	0.197	7	5.90	*	*	*	*	*	**
1	4.25	0.223	0.223	7	5.90	*	*	*	*	*	**
1	4.75	0.249	0.249	7	5.90	*	*	*	*	*	**
2	5.25	0.276	0.276	12	0.03	*	*	*	*	*	**
2	5.75	0.302	0.302	12	0.03	*	*	*	*	*	**
2	6.25	0.328	0.328	12	0.03	*	*	*	*	*	**
2	6.75	0.354	0.354	12	0.03	*	*	*	*	*	**
3	7.25	0.381	0.381	5	0.01	*	*	*	*	*	**
3	7.75	0.407	0.407	5	0.01	*	*	*	*	*	**
3	8.25	0.433	0.433	5	0.01	*	*	*	*	*	**
3	8.75	0.459	0.459	5	0.01	*	*	*	*	*	**
4	9.25	0.486	0.486	1	0.00	*	*	*	*	*	**
4	9.75	0.512	0.512	1	0.00	*	*	*	*	*	**
4	10.25	0.538	0.530	1	0.00	1.376	1.4	0.051	0.976	0.560	0.09
4	10.75	0.564	0.541	1	0.00	1.376	1.4	0.051	0.975	0.575	0.09
4	11.25	0.591	0.552	1	0.00	1.376	1.4	0.051	0.974	0.590	0.09
4	11.75	0.617	0.562	1	0.00	1.376	1.4	0.051	0.973	0.603	0.08
5	12.25	0.643	0.573	19	5.52	1.261	29.5	0.416	0.971	0.617	0.67
5	12.75	0.669	0.584	19	5.52	1.261	29.5	0.416	0.970	0.629	0.66
6	13.25	0.696	0.594	37	0.05	1.133	42.0	Infin	0.969	0.642	NonLiq
6	13.75	0.722	0.605	37	0.05	1.133	42.0	Infin	0.968	0.653	NonLiq
6	14.25	0.748	0.616	37	0.05	1.133	42.0	Infin	0.967	0.664	NonLiq
6	14.75	0.774	0.626	37	0.05	1.133	42.0	Infin	0.966	0.675	NonLiq
6	15.25	0.801	0.637	37	0.05	1.133	42.0	Infin	0.964	0.686	NonLiq
6	15.75	0.827	0.647	37	0.05	1.133	42.0	Infin	0.963	0.696	NonLiq
6	16.25	0.853	0.658	37	0.05	1.133	42.0	Infin	0.962	0.705	NonLiq
6	16.75	0.879	0.669	37	0.05	1.133	42.0	Infin	0.961	0.715	NonLiq
6	17.25	0.906	0.679	37	0.05	1.133	42.0	Infin	0.960	0.723	NonLiq
6	17.75	0.932	0.690	37	0.05	1.133	42.0	Infin	0.959	0.732	NonLiq
6	18.25	0.958	0.701	37	0.05	1.133	42.0	Infin	0.957	0.740	NonLiq
6	18.75	0.984	0.711	37	0.05	1.133	42.0	Infin	0.956	0.748	NonLiq
6	19.25	1.011	0.722	37	0.05	1.133	42.0	Infin	0.955	0.756	NonLiq
6	19.75	1.037	0.733	37	0.05	1.133	42.0	Infin	0.954	0.763	NonLiq
7	20.25	1.063	0.743	100	0.12	0.985	98.6	Infin	0.953	0.771	NonLiq
7	20.75	1.089	0.754	100	0.12	0.985	98.6	Infin	0.952	0.778	NonLiq
7	21.25	1.116	0.765	100	0.12	0.985	98.6	Infin	0.950	0.784	NonLiq

NCEER [1997] Method — LIQUEFACTION ANALYSIS SUMMARY — PAGE 2

File Name: 12798b7.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
7	21.75	1.142	0.775	100	0.12	0.985	98.6	Infin	0.949	0.791	NonLiq
7	22.25	1.168	0.786	100	0.12	0.985	98.6	Infin	0.948	0.797	NonLiq
7	22.75	1.194	0.797	100	0.12	0.985	98.6	Infin	0.947	0.803	NonLiq
7	23.25	1.221	0.807	100	0.12	0.985	98.6	Infin	0.946	0.809	NonLiq
7	23.75	1.247	0.818	100	0.12	0.985	98.6	Infin	0.945	0.814	NonLiq
7	24.25	1.273	0.829	100	0.12	0.985	98.6	Infin	0.943	0.820	NonLiq
7	24.75	1.299	0.839	100	0.12	0.985	98.6	Infin	0.942	0.825	NonLiq
8	25.25	1.326	0.850	80	0.09	0.883	70.7	Infin	0.941	0.830	NonLiq
8	25.75	1.352	0.860	80	0.09	0.883	70.7	Infin	0.940	0.835	NonLiq
8	26.25	1.378	0.871	80	0.09	0.883	70.7	Infin	0.939	0.840	NonLiq
8	26.75	1.404	0.882	80	0.09	0.883	70.7	Infin	0.938	0.844	NonLiq
9	27.25	1.431	0.892	66	0.07	0.850	56.2	Infin	0.936	0.849	NonLiq
9	27.75	1.457	0.903	66	0.07	0.850	56.2	Infin	0.935	0.853	NonLiq
9	28.25	1.483	0.914	66	0.07	0.850	56.2	Infin	0.934	0.857	NonLiq
9	28.75	1.509	0.924	66	0.07	0.850	56.2	Infin	0.933	0.861	NonLiq
9	29.25	1.536	0.935	66	0.07	0.850	56.2	Infin	0.932	0.865	NonLiq
9	29.75	1.562	0.946	66	0.07	0.850	56.2	Infin	0.931	0.869	NonLiq
10	30.25	1.588	0.956	58	0.06	0.807	46.9	Infin	0.928	0.871	NonLiq
10	30.75	1.614	0.967	58	0.06	0.807	46.9	Infin	0.924	0.872	NonLiq
10	31.25	1.641	0.978	58	0.06	0.807	46.9	Infin	0.920	0.873	NonLiq
10	31.75	1.667	0.988	58	0.06	0.807	46.9	Infin	0.916	0.873	NonLiq
10	32.25	1.693	0.999	58	0.06	0.807	46.9	Infin	0.912	0.874	NonLiq
10	32.75	1.719	1.010	58	0.06	0.807	46.9	Infin	0.907	0.874	NonLiq
10	33.25	1.746	1.020	58	0.06	0.807	46.9	Infin	0.903	0.874	NonLiq
10	33.75	1.772	1.031	58	0.06	0.807	46.9	Infin	0.899	0.874	NonLiq
10	34.25	1.798	1.042	58	0.06	0.807	46.9	Infin	0.895	0.874	NonLiq
10	34.75	1.824	1.052	58	0.06	0.807	46.9	Infin	0.891	0.874	NonLiq
11	35.25	1.851	1.063	80	0.07	0.748	59.9	Infin	0.887	0.874	NonLiq
11	35.75	1.877	1.073	80	0.07	0.748	59.9	Infin	0.883	0.873	NonLiq
11	36.25	1.903	1.084	80	0.07	0.748	59.9	Infin	0.879	0.873	NonLiq
11	36.75	1.929	1.095	80	0.07	0.748	59.9	Infin	0.875	0.872	NonLiq
11	37.25	1.956	1.105	80	0.07	0.748	59.9	Infin	0.871	0.871	NonLiq
11	37.75	1.982	1.116	80	0.07	0.748	59.9	Infin	0.867	0.870	NonLiq
11	38.25	2.008	1.127	80	0.07	0.748	59.9	Infin	0.863	0.870	NonLiq
11	38.75	2.034	1.137	80	0.07	0.748	59.9	Infin	0.859	0.869	NonLiq
11	39.25	2.061	1.148	80	0.07	0.748	59.9	Infin	0.855	0.867	NonLiq
11	39.75	2.087	1.159	80	0.07	0.748	59.9	Infin	0.851	0.866	NonLiq
12	40.25	2.113	1.169	67	0.06	0.701	47.0	Infin	0.846	0.865	NonLiq
12	40.75	2.139	1.180	67	0.06	0.701	47.0	Infin	0.842	0.864	NonLiq
12	41.25	2.166	1.191	67	0.06	0.701	47.0	Infin	0.838	0.862	NonLiq
12	41.75	2.192	1.201	67	0.06	0.701	47.0	Infin	0.834	0.861	NonLiq
12	42.25	2.218	1.212	67	0.06	0.701	47.0	Infin	0.830	0.859	NonLiq
12	42.75	2.244	1.223	67	0.06	0.701	47.0	Infin	0.826	0.858	NonLiq
12	43.25	2.271	1.233	67	0.06	0.701	47.0	Infin	0.822	0.856	NonLiq

NCEER [1997] Method

LIQUEFACTION ANALYSIS SUMMARY

PAGE 3

File Name: 12798b7.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
12	43.75	2.297	1.244	67	0.06	0.701	47.0	Infin	0.818	0.854	NonLiq
12	44.25	2.323	1.255	67	0.06	0.701	47.0	Infin	0.814	0.852	NonLiq
12	44.75	2.349	1.265	67	0.06	0.701	47.0	Infin	0.810	0.850	NonLiq
13	45.25	2.376	1.276	51	0.04	0.661	33.8	Infin	0.806	0.848	NonLiq
13	45.75	2.402	1.286	51	0.04	0.661	33.8	Infin	0.802	0.846	NonLiq
13	46.25	2.428	1.297	51	0.04	0.661	33.8	Infin	0.798	0.844	NonLiq
13	46.75	2.454	1.308	51	0.04	0.661	33.8	Infin	0.794	0.842	NonLiq
13	47.25	2.481	1.318	51	0.04	0.661	33.8	Infin	0.789	0.840	NonLiq
13	47.75	2.507	1.329	51	0.04	0.661	33.8	Infin	0.785	0.838	NonLiq
13	48.25	2.533	1.340	51	0.04	0.661	33.8	Infin	0.781	0.835	NonLiq
13	48.75	2.559	1.350	51	0.04	0.661	33.8	Infin	0.777	0.833	NonLiq
13	49.25	2.586	1.361	51	0.04	0.661	33.8	Infin	0.773	0.831	NonLiq
13	49.75	2.612	1.372	51	0.04	0.661	33.8	Infin	0.769	0.828	NonLiq
14	50.25	2.638	1.382	100	0.08	0.628	62.8	Infin	0.765	0.826	NonLiq
14	50.75	2.664	1.393	100	0.08	0.628	62.8	Infin	0.761	0.823	NonLiq
14	51.25	2.691	1.404	100	0.08	0.628	62.8	Infin	0.757	0.821	NonLiq

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*                           *
*      L I Q U E F Y 2      *
*                           *
*       Version 1.50        *
*                           *
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EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: 12798.1 DATE: 05-17-2011

JOB NAME: Boring B-10

SOIL-PROFILE NAME: 12798b10.LDW

BORING GROUNDWATER DEPTH: 53.00 ft

CALCULATION GROUNDWATER DEPTH: 10.00 ft

DESIGN EARTHQUAKE MAGNITUDE: 7.50 Mw

SITE PEAK GROUND ACCELERATION: 0.870 g

BOREHOLE DIAMETER CORRECTION FACTOR: 1.00

SAMPLER SIZE CORRECTION FACTOR: 1.00

N60 HAMMER CORRECTION FACTOR: 1.00

MAGNITUDE SCALING FACTOR METHOD: Idriss (1997, in press)

Magnitude Scaling Factor: 1.000

rd-CORRECTION METHOD: NCEER (1997)

FIELD SPT N-VALUES ARE NOT CORRECTED FOR THE LENGTH OF THE DRIVE RODS.

Rod Stick-Up Above Ground: 5.0 ft

CN NORMALIZATION FACTOR: 1.044 tsf

MINIMUM CN VALUE: 0.6

NCEER [1997] Method

LIQUEFACTION ANALYSIS SUMMARY

File Name: 12798b10.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	0.25	0.013	0.013	13	8.72	*	*	*	*	*	**
1	0.75	0.039	0.039	13	8.72	*	*	*	*	*	**
1	1.25	0.066	0.066	13	8.72	*	*	*	*	*	**
1	1.75	0.092	0.092	13	8.72	*	*	*	*	*	**
1	2.25	0.118	0.118	13	8.72	*	*	*	*	*	**
1	2.75	0.144	0.144	13	8.72	*	*	*	*	*	**
1	3.25	0.171	0.171	13	8.72	*	*	*	*	*	**
1	3.75	0.197	0.197	13	8.72	*	*	*	*	*	**
2	4.25	0.223	0.223	4	0.01	*	*	*	*	*	**
2	4.75	0.249	0.249	4	0.01	*	*	*	*	*	**
2	5.25	0.276	0.276	4	0.01	*	*	*	*	*	**
2	5.75	0.302	0.302	4	0.01	*	*	*	*	*	**
2	6.25	0.328	0.328	4	0.01	*	*	*	*	*	**
2	6.75	0.354	0.354	4	0.01	*	*	*	*	*	**
3	7.25	0.381	0.381	4	0.01	*	*	*	*	*	**
3	7.75	0.407	0.407	4	0.01	*	*	*	*	*	**
3	8.25	0.433	0.433	4	0.01	*	*	*	*	*	**
3	8.75	0.459	0.459	4	0.01	*	*	*	*	*	**
3	9.25	0.486	0.486	4	0.01	*	*	*	*	*	**
3	9.75	0.512	0.512	4	0.01	*	*	*	*	*	**
4	10.25	0.538	0.530	7	0.01	1.376	9.6	0.106	0.976	0.560	0.19
4	10.75	0.564	0.541	7	0.01	1.376	9.6	0.106	0.975	0.575	0.18
4	11.25	0.591	0.552	7	0.01	1.376	9.6	0.106	0.974	0.590	0.18
4	11.75	0.617	0.562	7	0.01	1.376	9.6	0.106	0.973	0.603	0.18
5	12.25	0.643	0.573	16	0.03	1.261	20.2	0.220	0.971	0.617	0.36
5	12.75	0.669	0.584	16	0.03	1.261	20.2	0.220	0.970	0.629	0.35
5	13.25	0.696	0.594	16	0.03	1.261	20.2	0.220	0.969	0.642	0.34
5	13.75	0.722	0.605	16	0.03	1.261	20.2	0.220	0.968	0.653	0.34
5	14.25	0.748	0.616	16	0.03	1.261	20.2	0.220	0.967	0.664	0.33
5	14.75	0.774	0.626	16	0.03	1.261	20.2	0.220	0.966	0.675	0.33
6	15.25	0.801	0.637	32	0.05	1.133	36.3	Infin	0.964	0.686	NonLiq
6	15.75	0.827	0.647	32	0.05	1.133	36.3	Infin	0.963	0.696	NonLiq
6	16.25	0.853	0.658	32	0.05	1.133	36.3	Infin	0.962	0.705	NonLiq
6	16.75	0.879	0.669	32	0.05	1.133	36.3	Infin	0.961	0.715	NonLiq
6	17.25	0.906	0.679	32	0.05	1.133	36.3	Infin	0.960	0.723	NonLiq
6	17.75	0.932	0.690	32	0.05	1.133	36.3	Infin	0.959	0.732	NonLiq
6	18.25	0.958	0.701	32	0.05	1.133	36.3	Infin	0.957	0.740	NonLiq
6	18.75	0.984	0.711	32	0.05	1.133	36.3	Infin	0.956	0.748	NonLiq
7	19.25	1.011	0.722	21	7.90	0.985	28.6	0.368	0.955	0.756	0.49
7	19.75	1.037	0.733	21	7.90	0.985	28.6	0.368	0.954	0.763	0.48
7	20.25	1.063	0.743	21	7.90	0.985	28.6	0.368	0.953	0.771	0.48
7	20.75	1.089	0.754	21	7.90	0.985	28.6	0.368	0.952	0.778	0.47
7	21.25	1.116	0.765	21	7.90	0.985	28.6	0.368	0.950	0.784	0.47

NCEER [1997] Method — LIQUEFACTION ANALYSIS SUMMARY — PAGE 2

File Name: 12798b10.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
7	21.75	1.142	0.775	21	7.90	0.985	28.6	0.368	0.949	0.791	0.47
7	22.25	1.168	0.786	21	7.90	0.985	28.6	0.368	0.948	0.797	0.46
7	22.75	1.194	0.797	21	7.90	0.985	28.6	0.368	0.947	0.803	0.46
8	23.25	1.221	0.807	34	0.04	0.883	30.1	Infin	0.946	0.809	NonLiq
8	23.75	1.247	0.818	34	0.04	0.883	30.1	Infin	0.945	0.814	NonLiq
8	24.25	1.273	0.829	34	0.04	0.883	30.1	Infin	0.943	0.820	NonLiq
8	24.75	1.299	0.839	34	0.04	0.883	30.1	Infin	0.942	0.825	NonLiq
8	25.25	1.326	0.850	34	0.04	0.883	30.1	Infin	0.941	0.830	NonLiq
8	25.75	1.352	0.860	34	0.04	0.883	30.1	Infin	0.940	0.835	NonLiq
8	26.25	1.378	0.871	34	0.04	0.883	30.1	Infin	0.939	0.840	NonLiq
8	26.75	1.404	0.882	34	0.04	0.883	30.1	Infin	0.938	0.844	NonLiq
9	27.25	1.431	0.892	54	0.05	0.807	43.7	Infin	0.936	0.849	NonLiq
9	27.75	1.457	0.903	54	0.05	0.807	43.7	Infin	0.935	0.853	NonLiq
9	28.25	1.483	0.914	54	0.05	0.807	43.7	Infin	0.934	0.857	NonLiq
9	28.75	1.509	0.924	54	0.05	0.807	43.7	Infin	0.933	0.861	NonLiq
9	29.25	1.536	0.935	54	0.05	0.807	43.7	Infin	0.932	0.865	NonLiq
9	29.75	1.562	0.946	54	0.05	0.807	43.7	Infin	0.931	0.869	NonLiq
9	30.25	1.588	0.956	54	0.05	0.807	43.7	Infin	0.928	0.871	NonLiq
9	30.75	1.614	0.967	54	0.05	0.807	43.7	Infin	0.924	0.872	NonLiq
9	31.25	1.641	0.978	54	0.05	0.807	43.7	Infin	0.920	0.873	NonLiq
9	31.75	1.667	0.988	54	0.05	0.807	43.7	Infin	0.916	0.873	NonLiq
9	32.25	1.693	0.999	54	0.05	0.807	43.7	Infin	0.912	0.874	NonLiq
9	32.75	1.719	1.010	54	0.05	0.807	43.7	Infin	0.907	0.874	NonLiq
9	33.25	1.746	1.020	54	0.05	0.807	43.7	Infin	0.903	0.874	NonLiq
9	33.75	1.772	1.031	54	0.05	0.807	43.7	Infin	0.899	0.874	NonLiq
9	34.25	1.798	1.042	54	0.05	0.807	43.7	Infin	0.895	0.874	NonLiq
9	34.75	1.824	1.052	54	0.05	0.807	43.7	Infin	0.891	0.874	NonLiq
10	35.25	1.851	1.063	78	0.07	0.748	58.5	Infin	0.887	0.874	NonLiq
10	35.75	1.877	1.073	78	0.07	0.748	58.5	Infin	0.883	0.873	NonLiq
10	36.25	1.903	1.084	78	0.07	0.748	58.5	Infin	0.879	0.873	NonLiq
10	36.75	1.929	1.095	78	0.07	0.748	58.5	Infin	0.875	0.872	NonLiq
10	37.25	1.956	1.105	78	0.07	0.748	58.5	Infin	0.871	0.871	NonLiq
10	37.75	1.982	1.116	78	0.07	0.748	58.5	Infin	0.867	0.870	NonLiq
11	38.25	2.008	1.127	45	0.04	0.701	31.6	Infin	0.863	0.870	NonLiq
11	38.75	2.034	1.137	45	0.04	0.701	31.6	Infin	0.859	0.869	NonLiq
11	39.25	2.061	1.148	45	0.04	0.701	31.6	Infin	0.855	0.867	NonLiq
11	39.75	2.087	1.159	45	0.04	0.701	31.6	Infin	0.851	0.866	NonLiq
11	40.25	2.113	1.169	45	0.04	0.701	31.6	Infin	0.846	0.865	NonLiq
11	40.75	2.139	1.180	45	0.04	0.701	31.6	Infin	0.842	0.864	NonLiq
11	41.25	2.166	1.191	45	0.04	0.701	31.6	Infin	0.838	0.862	NonLiq
11	41.75	2.192	1.201	45	0.04	0.701	31.6	Infin	0.834	0.861	NonLiq
11	42.25	2.218	1.212	45	0.04	0.701	31.6	Infin	0.830	0.859	NonLiq
11	42.75	2.244	1.223	45	0.04	0.701	31.6	Infin	0.826	0.858	NonLiq
11	43.25	2.271	1.233	45	0.04	0.701	31.6	Infin	0.822	0.856	NonLiq

NCEER [1997] Method

LIQUEFACTION ANALYSIS SUMMARY

PAGE 3

File Name: 12798b10.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
11	43.75	2.297	1.244	45	0.04	0.701	31.6	Infin	0.818	0.854	NonLiq
12	44.25	2.323	1.255	29	8.76	0.661	27.9	0.335	0.814	0.852	0.39
12	44.75	2.349	1.265	29	8.76	0.661	27.9	0.335	0.810	0.850	0.39
12	45.25	2.376	1.276	29	8.76	0.661	27.9	0.335	0.806	0.848	0.39
12	45.75	2.402	1.286	29	8.76	0.661	27.9	0.335	0.802	0.846	0.40
12	46.25	2.428	1.297	29	8.76	0.661	27.9	0.335	0.798	0.844	0.40
12	46.75	2.454	1.308	29	8.76	0.661	27.9	0.335	0.794	0.842	0.40
13	47.25	2.481	1.318	61	0.05	0.628	38.3	Infin	0.789	0.840	NonLiq
13	47.75	2.507	1.329	61	0.05	0.628	38.3	Infin	0.785	0.838	NonLiq
13	48.25	2.533	1.340	61	0.05	0.628	38.3	Infin	0.781	0.835	NonLiq
13	48.75	2.559	1.350	61	0.05	0.628	38.3	Infin	0.777	0.833	NonLiq
13	49.25	2.586	1.361	61	0.05	0.628	38.3	Infin	0.773	0.831	NonLiq
13	49.75	2.612	1.372	61	0.05	0.628	38.3	Infin	0.769	0.828	NonLiq
13	50.25	2.638	1.382	61	0.05	0.628	38.3	Infin	0.765	0.826	NonLiq
13	50.75	2.664	1.393	61	0.05	0.628	38.3	Infin	0.761	0.823	NonLiq
13	51.25	2.691	1.404	61	0.05	0.628	38.3	Infin	0.757	0.821	NonLiq

APPENDIX F

Settlement Calculations

Estimation of Dynamic Settlement

Project Name: Lewis Retail Center
Project No.: 12798.1
Client:

Boring: ***B-2*** (Assumption: recompaction of upper 10' and gw @ 10')

Estimation of Settlement of Dry Sands

amax (g) =	0.87
Magnitude =	7.5
Nc =	15.16

Layer No.	Depth to Sample (ft)	Thickness (ft)	$(N_1)_{60cs}$	σ'_o (psf)	σ'_m (psf)	z (ft)	z_{center} (ft)	z_{center} (m)	r_d	τ_{av} (psf)	G_{max} (psf)	a	b	τ_{av}/G_{max}	γ	ε_{15}	ε_{NC}	Settlement (in)
1	2	5	30	287.5	192.6	5.0	2.5	0.76	1.00	162.48	862,174	0.13	26,063	0.0001885	0.003067	0.001886	0.00189	0.23
2	5	2	30	690.0	462.3	7.0	6.0	1.83	1.00	388.80	1,335,674	0.13	15,413	0.0002911	0.003292	0.002024	0.00203	0.10
3	7	3	30	977.5	654.9	10.0	8.5	2.59	0.99	548.82	1,589,770	0.14	12,507	0.0003452	0.003418	0.002101	0.00211	0.15
	Total Thickness =	*10*															***Settlement =***	***0.48***

Estimation of Settlement of Saturated Sands

amax (g) =	0.87
Magnitude =	7.5
rm =	1

Layer No.	Depth to Sample (ft)	Thickness (ft)	$(N_1)_{60}$	N_{corr}	$(N_1)_{60}$	σ_o (psf)	σ'_o (psf)	z (ft)	z_{center} (ft)	z_{center} (m)	r_d	$(\tau_{av}/\sigma'_o)_M$	$(\tau_{av}/\sigma'_o)_{7.5}$	$\varepsilon_{CM=7.5}$ (%)	Settlement (in)
4	10	5	33	0	33	1412.5	1256.5	15.0	12.5	3.81	0.98	0.626	0.626	0.500	0.3
5	15	4	40	0	40	1885.0	1448.2	19.0	17.0	5.18	0.97	0.715	0.715	0.000	0.0
6	20	4	34	0	34	2305.0	1618.6	23.0	21.0	6.40	0.96	0.771	0.771	0.000	0.0
7	25	6	44	0	44	2830.0	1831.6	29.0	26.0	7.92	0.94	0.818	0.818	0.000	0.0
8	30	3	9	4	13	3302.5	2023.3	32.0	30.5	9.30	0.91	0.845	0.845	2.200	0.8
9	35	8	60	0	60	3880.0	2257.6	40.0	36.0	10.97	0.89	0.861	0.861	0.000	0.0
10	40	5	75	0	75	4562.5	2534.5	45.0	42.5	12.95	0.85	0.862	0.862	0.000	0.0
11	45	5	34	0	34	5087.5	2747.5	50.0	47.5	14.48	0.82	0.855	0.855	0.000	0.0
	Total Thickness =	*40*												***Settlement =***	**1.1**

Estimated Total Settlement = 1.6 inches

Boring: *B-7* (Assumption: recompaction of upper 10' and gw @ 10')

Estimation of Settlement of Dry Sands

amax (g) =	0.87
Magnitude =	7.5
Nc =	15.16

Layer No.	Depth to Sample (ft)	Thickness (ft)	$(N_1)_{60cs}$	σ'_o (psf)	σ'_m (psf)	z (ft)	z_{center} (ft)	z_{center} (m)	r_d	τ_{av} (psf)	G_{max} (psf)	a	b	τ_{av}/G_{max}	γ	ε_{15}	ε_{NC}	Settlement (in)
1	2	5	30	287.5	192.6	5.0	2.5	0.76	1.00	162.48	862,174	0.13	26,063	0.0001885	0.003067	0.001886	0.00189	0.23
2	5	2	30	690.0	462.3	7.0	6.0	1.83	1.00	388.80	1,335,674	0.13	15,413	0.0002911	0.003292	0.002024	0.00203	0.10
3	7	3	30	977.5	654.9	10.0	8.5	2.59	0.99	548.82	1,589,770	0.14	12,507	0.0003452	0.003418	0.002101	0.00211	0.15
	Total Thickness =	*10*															***Settlement =***	***0.48***

Estimation of Settlement of Saturated Sands

amax (g) =	0.87
Magnitude =	7.5
rm =	1

Layer No.	Depth to Sample (ft)	Thickness (ft)	$(N_1)_{60}$	N_{corr}	$(N_1)_{60}$	σ_o (psf)	σ'_o (psf)	z (ft)	z_{center} (ft)	z_{center} (m)	r_d	$(\tau_{av}/\sigma'_o)_M$	$(\tau_{av}/\sigma'_o)_{7.5}$	$\varepsilon_{CM=7.5}$ (%)	Settlement (in)
4	10	2	1	0	1	1255.0	1192.6	12.0	11.0	3.35	0.99	0.588	0.588	10.000	2.4
5	12	1	23	1.5	25	1412.5	1256.5	13.0	12.5	3.81	0.98	0.626	0.626	1.200	0.1
6	15	7	41	0	41	1832.5	1426.9	20.0	16.5	5.03	0.97	0.707	0.707	0.000	0.0
7	20	5	105	0	105	2462.5	1682.5	25.0	22.5	6.86	0.95	0.788	0.788	0.000	0.0
8	25	2	72	0	72	2830.0	1831.6	27.0	26.0	7.92	0.94	0.818	0.818	0.000	0.0
9	27	3	57	0	57	3092.5	1938.1	30.0	28.5	8.69	0.92	0.835	0.835	0.000	0.0
10	30	5	47	0	47	3512.5	2108.5	35.0	32.5	9.91	0.90	0.852	0.852	0.000	0.0
11	35	5	60	0	60	4037.5	2321.5	40.0	37.5	11.43	0.88	0.862	0.862	0.000	0.0
12	40	5	46	0	46	4562.5	2534.5	45.0	42.5	12.95	0.85	0.862	0.862	0.000	0.0
13	45	5	33	0	33	5087.5	2747.5	50.0	47.5	14.48	0.82	0.855	0.855	0.500	0.3
14	50	1.5	66	0	66	5428.8	2886.0	51.5	50.8	15.47	0.80	0.846	0.846	0.000	0.0
	Total Thickness =	*41.5*												***Settlement =***	**2.8**

Estimated Total Settlement = 3.3 inches

Boring: ***B-10*** (Assumption: recompaction of upper 10' and gw @ 10')

Estimation of Settlement of Dry Sands

amax (g) =	0.87
Magnitude =	7.5
Nc =	15.16

Layer No.	Depth to Sample (ft)	Thickness (ft)	$(N_1)_{60cs}$	σ'_o (psf)	σ'_m (psf)	z (ft)	z_{center} (ft)	z_{center} (m)	r_d	τ_{av} (psf)	G_{max} (psf)	a	b	τ_{av}/G_{max}	γ	ε_{15}	ε_{NC}	Settlement (in)
1	2	5	30	287.5	192.6	5.0	2.5	0.76	1.00	162.48	862,174	0.13	26,063	0.0001885	0.003067	0.001886	0.00189	0.23
2	5	2	30	690.0	462.3	7.0	6.0	1.83	1.00	388.80	1,335,674	0.13	15,413	0.0002911	0.003292	0.002024	0.00203	0.10
3	7	3	30	977.5	654.9	10.0	8.5	2.59	0.99	548.82	1,589,770	0.14	12,507	0.0003452	0.003418	0.002101	0.00211	0.15
	Total Thickness =	*10*															***Settlement =***	***0.48***

Estimation of Settlement of Saturated Sands

amax (g) =	0.87
Magnitude =	7.5
rm =	1

Layer No.	Depth to Sample (ft)	Thickness (ft)	$(N_1)_{60}$	N_{corr}	$(N_1)_{60}$	σ_o (psf)	σ'_o (psf)	z (ft)	z_{center} (ft)	z_{center} (m)	r_d	$(\tau_{av}/\sigma'_o)_M$	$(\tau_{av}/\sigma'_o)_{7.5}$	$\varepsilon_{CM=7.5}$ (%)	Settlement (in)
4	10	2	9	0	9	1255.0	1192.6	12.0	11.0	3.35	0.99	0.588	0.588	2.750	0.7
5	12	3	20	0	20	1517.5	1299.1	15.0	13.5	4.11	0.98	0.649	0.649	1.550	0.6
6	15	4	36	0	36	1885.0	1448.2	19.0	17.0	5.18	0.97	0.715	0.715	0.000	0.0
7	20	4	21	2	23	2305.0	1618.6	23.0	21.0	6.40	0.96	0.771	0.771	1.350	0.6
8	25	4	30	0	30	2725.0	1789.0	27.0	25.0	7.62	0.94	0.811	0.811	0.800	0.4
9	30	8	44	0	44	3355.0	2044.6	35.0	31.0	9.45	0.91	0.847	0.847	0.000	0.0
10	35	3	58	0	58	3932.5	2278.9	38.0	36.5	11.13	0.88	0.861	0.861	0.000	0.0
11	40	6	31	0	31	4405.0	2470.6	44.0	41.0	12.50	0.86	0.863	0.863	0.650	0.5
12	45	3	18	4	22	4877.5	2662.3	47.0	45.5	13.87	0.83	0.859	0.859	1.450	0.5
13	50	4.5	36	0	36	5271.3	2822.1	51.5	49.3	15.01	0.80	0.850	0.850	0.000	0.0
	Total Thickness =	*41.5*												***Settlement =***	**3.2**

Estimated Total Settlement = 3.7 inches

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 this office at your convenience.

Respectfully submitted,
LOR Geotechnical Group, Inc.

Robert M. Markoff, CEG 2073
Engineering Geologist

John P. Leuer, GE 2030
President

REGISTERED PROFESSIONAL ENGINEER
JOHN P. LEUER
NO. 2030
EXPIRATION DATE
09/30/11
GEOTECHNICAL
STATE OF CALIFORNIA

RMM/GCB:JPL/amp

Distribution: Addressee (6)

MATCHLINE: SEE SHEET 2

ESPERANZA STREET

BUILDING A
616,000 SF

BUILDING A
616,000 SF

CENTRAL AVENUE

SIERRA WAY

BUILDING B
78,960 SF

ARROWHEAD AVENUE

ARROWHEAD AVENUE

GRAPHIC SCALE

(IN FEET)
1 inch = 60 ft.

EARTHWORK SUMMARY

RAW CUT: 43,900 CY (C)

RAW FILL: 78,800 CY (F)

10% SHRINKAGE: 7,880 CY (F)

TOTAL: TOTAL: 40,580 CY (F)

*ASSUMED 12" AVERAGE PAVEMENT SECTION

*ASSUMED 9" BUILDING PAD

Underground Service Alert

Call: TOLL FREE
1-800
422-4133

TWO WORKING DAYS BEFORE YOU DIG

RECOMMENDED FOR APPROVAL BY PLAN CHECKER

PLAN CHECK FIRM: XXX
XXX
XXX
TEL: XXX
FAX: XXX

PLAN CHECKER NAME:

PLAN CHECKER SIGNATURE:

PLAN CHECKER RCE No.: EXP. DATE:

ORC Engineering, Inc.
Civil Engineering/Land Surveying/Land Planning

160 S. Old Springs Road
Suite 210
Anaheim Hills, CA 92808
714-685-6860

MARK	REVISIONS	BY	APPR.	DATE

BENCH MARK:
XXX

APPROVED _____ 2011

SENIOR CIVIL ENGINEER XXX
REGISTERED CIVIL ENGINEER NO. XXX

DRAWN BY:

CHECKED BY:

RECOMMENDED BY:

CITY OF SAN BERNARDINO

DEVELOPMENT SERVICES—PUBLIC WORKS/ENGINEERING

CONCEPTUAL GRADING PLAN FOR NATIONAL ORANGE SHOW

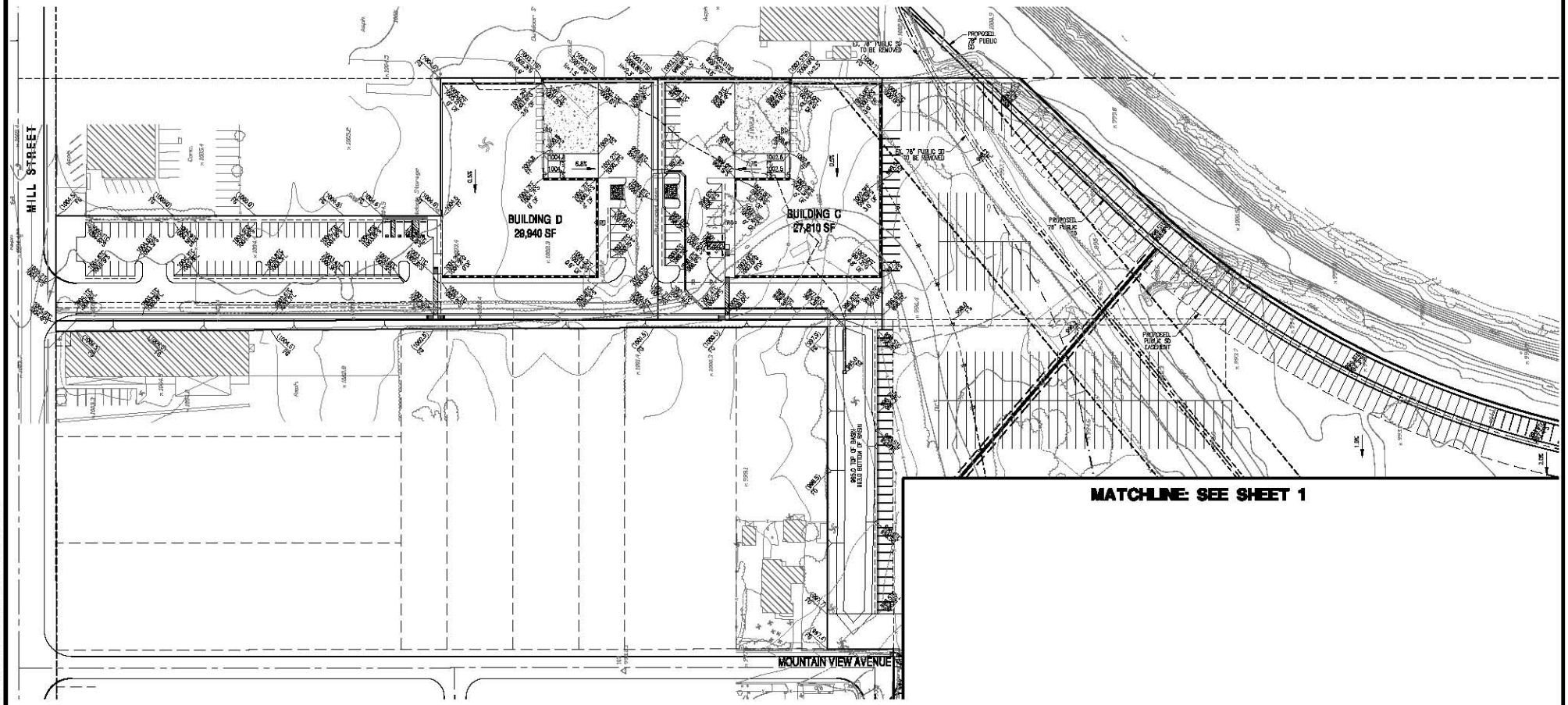
ARROWHEAD AVENUE

FOR CITY USE ONLY: FILE NO. W.O. NO.

DRAWING NO. XXXXX

SHEET 1 OF 2 SHEETS

EXTERNAL REFERENCES: 11420 MA 11420 CONA 11-420 BTOT 11-420bnd01 GRC_cb_plbmp
FILENAME: M:\2011\11-420 Lewis Foothill Orange Show SB\CD\11420 CG01.dwg LAST SAVED ON: Jun 16 2011 8:37am PLOTTED BY: LIANNE, ON: Jun 16 2011 9:00am, CFG



GRAPHIC SCALE

(IN FEET)
1 inch = 60 ft.

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PLAN CHECKER NAME:
PLAN CHECKER SIGNATURE:
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160 S. Old Springs Road
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Anaheim Hills, CA 92808
714-685-6860

MARK	REVISIONS	BY	APPR.	DATE

BENCH MARK:
XXX

APPROVED _____ 2011
SENIOR CIVIL ENGINEER XXX
REGISTERED CIVIL ENGINEER NO. XXX
DRAWN BY:
CHECKED BY:
RECOMMENDED BY:

CITY OF SAN BERNARDINO
DEVELOPMENT SERVICES—PUBLIC WORKS/ENGINEERING
CONCEPTUAL GRADING PLAN
FOR
NATIONAL ORANGE SHOW
ARROWHEAD AVENUE

FOR CITY USE ONLY: FILE NO. W.O. NO.

DRAWING NO.
XXXXX

SHEET 2 OF 2 SHEETS

EXTERNAL REFERENCES: 11420 MA 11420 CONA 11-420 BIT01 11-420bn-b01 GRC_ca_gdemp
FILENAME: M:\2011\11-420 Lewis Foothill Orange Show SB\CD\11420 CG01.dwg LAST SAVED ON: Jun 10 2011 8:37am PLOTTED BY: LIANNE, ON: Jun 10 2011 9:00am, CFG