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

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
111 E. MILL STREET
CITY OF SAN BERNARDINO, CALIFORNIA

PROJECT NO. 63243.1
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Table of Contents

Page No.

INTRODUCTION 1

PROJECT CONSIDERATIONS 2

REVIEW OF PREVIOUS REPORTS 2

REVIEW OF AERIAL PHOTOGRAPHS 2

EXISTING SITE CONDITIONS 3

SUBSURFACE FIELD INVESTIGATION 4

LABORATORY TESTING PROGRAM 5

GEOLOGIC CONDITIONS 5
- Regional Geologic Setting 5
- Site Geologic Conditions 6
- Fills 6
- Alluvium 6
- Groundwater Hydrology 6
- Surface Runoff 8
- Mass Movement 8
- Faulting 8
- Historical Seismicity 10
- Secondary Seismic Hazards 11
  - Liquefaction 11
  - Seiches/Tsunamis 12
  - Flooding (Water Storage Facility Failure) 12
  - Seismically-Induced Landsliding 12
  - Rockfalls 13

SOILS AND SEISMIC DESIGN CRITERIA (California Building Code 2010) 13
- CBC Earthquake Design Summary 13

INFILTRATION TESTING AND TEST RESULTS 14

CONCLUSIONS 16
- General 16
- Foundation Support 16

Table of Contents

Page No.

Geologic Mitigations 17
Seismicity 17

RECOMMENDATIONS 18
Geologic Recommendations 18
General Site Grading 18
Initial Site Preparation 18
Preparation of Fill Areas 19
Preparation of Building Pad Areas 19
Engineered Compacted Fill 20
Short Term Excavations 20
Slope Construction 21
Slope Protection 21
Foundation Design 21
Settlement 22
Building Area Slab-on-Grade 23
Exterior Flatwork 23
Wall Pressures 24
Preliminary Pavement Design 25
Sulfate Protection 26
Infiltration 26
Construction Monitoring 27

LIMITATIONS 28

TIME LIMITATIONS 29

CLOSURE 29

REFERENCES 30

REFERENCES (Continued) 31

APPENDICES

Appendix A - Index Map, Boring Location Maps, Regional Geologic Map and Historical Seismicity Maps
Appendix B - Field Investigation Program and Boring and Trench Logs
Appendix C - Laboratory Testing Program and Test Results
Appendix D - Liquefaction Calculations
Appendix E - Infiltration/Percolation Testing
Appendix F - Settlement Calculations

INTRODUCTION

During March of 2016, a Preliminary Geotechnical Investigation was performed by LOR Geotechnical Group, Inc., for a proposed commercial development of 111 E. Mill Street in the City of San Bernardino, California. 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 1969;
- 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/percolation testing to establish a clear water rate for the proposed infiltration basin;
- 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 four parcels of land that comprise approximately 10.6 acres located at 111 E. Mill Street in the City of San Bernardino, California. The property is currently developed with several structures, a railroad spur,

and paved areas. At the time of our investigation, the site was unoccupied. Proposed development within the property is currently scheduled to include a truck repair and wash structure with the remainder of the development to consist of parking and driveway areas. Much of the existing structures are proposed to be razed. In addition, an infiltration basin is proposed within the southwest corner of the site.

A map of the site, prepared by W.J. McKeever, Inc., dated February 2, 2016, illustrating the current topography and existing structures as well as the proposed development, was provided for our use. 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 are anticipated, excluding removals and over-excavations, to reach the proposed grades.

REVIEW OF PREVIOUS REPORTS

In 2001, this firm conducted a geotechnical investigation for the commercial project located adjacent to the subject site on the south-southwest (LOR, 2011). At that time, the far southwest portion of the subject site was a part of that project. One exploratory boring was advanced to a maximum depth of 50 feet in this area at that time and encountered approximately 4 feet of fill materials overlying native alluvial soils. No groundwater was encountered in that boring.

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, 1953, 1955, 1959, and 1969.

The aerial photographs reviewed at the CSBFCD consisted of vertical aerial stereographic 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.

As noted in the earliest photographs, a small natural drainage traversed the site from the southwest to the northeast. By 1953, this drainage was channelized south of the site. Extensive channelization projects of the natural drainage were noted at the site and adjoining areas to the northeast of the site in 1959. This channelization resulted in heavy impacts to the site in and adjacent to the drainage. By 1969, the channelization projects were complete with the site in a similar condition as seen today.

EXISTING SITE CONDITIONS

The buildings onsite include a former retail store with office space and associated warehouse/storage space in three adjoining buildings in the northeast corner; a receiving building and associated smaller building just east of the adjoining buildings in the northeast corner; a metal and wooden building with pole supports (former sawmill) located along the east side, south of the adjoining buildings in the northeast corner; three pole sheds (canopy structures) located south of the former sawmill building along the east side; and two adjoining buildings in the south end which include a former machine shop, saw milling, a hydraulic oil room, and maintenance area. Aboveground silos or tanks are present at the former sawmill buildings along the east side and in the south end of the site.

Another building was present in the north central portion of the site and was of concrete block wall, exterior stucco, wood/metal roof construction. The building has been partially destructed, likely as a result of vandalism. A tan-colored, relatively small storage unit or trailer on concrete blocks to the southeast.

Other structures and features include a railroad spur track within associated asphalt pavement, entering the site along the east side, in line with the southernmost pole shed, in a southwesterly direction, turning to the east; two wood platforms located in the southeast portion towards the south end, a double wide trailer/mobile structure just south of the wood platforms; and remnant slabs.

The southwest corner of the site was vacant.

Most of the remainder of the site area was paved with asphalt concrete and/or concrete, with the western portion unpaved with some gravel covering. Where, present, the pavement was in poor condition.

The topography of the site is relatively planar, generally falling gently to the southwest. However, the parking area along Mill Street is graded to drain northwesterly to Mill Street.

The site is mostly free of vegetation, but includes some weeds and grasses and trees primarily in the southwest corner of the site as well as along the periphery to the east, south, and west.

Power poles and lines are present along the north side (right-of-way for Mill Street), west side, and along the west side of the main part of the site, dividing it and the area to the west.

SUBSURFACE FIELD INVESTIGATION

Our subsurface field exploration program was conducted on March 1, 2016 and consisted of drilling 8 exploratory borings with a truck-mounted Mobile B61 drill rig equipped with an 8-inch diameter hollow stem augers. The borings were advanced to depths ranging from approximately 6.5 to 50 feet below the existing ground surface. The approximate locations of our exploratory borings are presented on the attached Boring Location Map, Enclosure A-2 within Appendix A.

In addition, two shallow and one deep exploratory trenches were excavated within the southwest corner of the site in the area of the proposed basin. The shallow trenches were utilized for infiltration testing. A John Deere 410J backhoe equipped with a 24-inch bucket was utilized to excavate the trenches to depths of 5 to 10.5 feet.

The subsurface conditions encountered in the exploratory borings and trench were logged by a 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 and trench 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, 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 2.1 kilometers (1.3 miles) to the southwest, the San Andreas fault located approximately 7.9 kilometers (4.9 miles) to the northeast, the Cucamonga fault approximately 17.9 kilometers (11.1 miles) to the northwest, and the Elsinore fault located approximately 39 kilometers (24.2 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: As encountered within our exploratory borings and trench, fill materials were encountered to depths of 3 to 15 feet. In general, the fill materials consisted of silty sand with minor units of sandy silt and well graded sand which were in a loose state. As noted within our exploratory trench, the fill materials in that area contained abundant, large pieces of concrete debris up to 5 feet in length. Refusal due to the abundant, large pieces of concrete was experienced within the exploratory trench.

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. The alluvial materials were noted to be dry to damp and in a to medium dense to dense state.

It should be noted that various authors have divided the local alluvial soils into different units, mainly based upon age and composition. Two 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.

No groundwater was encountered in any our borings which extended to depths of approximately 50 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.

The nearest wells to the subject site with groundwater level data in the Cooperative Well Measuring Program, Fall 2015 (Western Municipal Water District and San Bernardino Valley Municipal Water District, 2016) include State Well Nos. 01S04W15A (Well #66, Drake Dr & Allen), 01S04W15C (Well #03, Mill & Arrowhead), 01S04W15F (Well #63, Central/Arrowhead), 01S04W15F006S, 01S04W15F007S, and 01S04W15F008S (Orange Show Wells), 01S04W15G (Well #65, Washington/Central). These wells are located within 0.5 mile of the subject site. State Wells 01S04W15A, 01S04W15F and 01S04W15G were 15- to 22-foot wells which were dry from April of 1993 until November of 1998 when they were destroyed. Water levels in State Well No. 01S04W15C, a 23.8-foot well, were measured from April of 1993 to March of 2002 at levels ranging from approximately 13 to 23.8 feet bgs. This well has been dry or has had no readings from Fall of 2002 to Fall of 2014. The three Orange Show Wells, State Well Nos. 01S04W15F006S, 01S04W15F007S, and 01S04W15F008S, have water level measurements from April of 2004 to October of 2014, ranging from approximately 32 to 97 feet bgs, but were mostly in the 40- to 70-foot depth range, with levels dropping over time.

The California Department of Water Resources (DWR) online Water Data Library was searched for groundwater level information. This search revealed similar groundwater levels to those discussed above for the Cooperative Well Measuring Program data.

Based on the data available at the online State of California Geotracker database, the leaking underground storage tank (LUST) site ARCO #5267, located at 495 South Waterman Avenue, approximately 0.33 mile east-northeast of the subject site, groundwater depths in the local area ranged from approximately 34 to 80 feet bgs from 2001 to 2007. Based on the ARCO groundwater data, the local groundwater flow direction is estimated to range from westerly to southwesterly.

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 just southwest 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 crossing the far 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 2.1 kilometers (1.3 miles) 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 7.9 kilometers (4.9 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, located approximately 17.9 kilometers (11.1 miles) to the northwest 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 39 kilometers (24.2 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. (Reeder, 2010). 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-5, 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 35 ± 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-6, 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, iindicates 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 = 0.89g and Depth to Groundwater = 10 feet)	
Boring	**Depths of Liquefiable Soil Layers from the Existing Ground Surface**
B-1	from 10 to 15 feet from 25 to 30 feet from 40 to 45 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 D.

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 (2013 California Building Code)

Section 1613 of Chapter 16 of the 2013 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.

It should be noted that the classification of use and occupancy of all proposed structures at the site, and thus design requirements, shall be the responsibility of the structural engineer and the building official.

CBC Earthquake Design Summary

The following earthquake design criteria have been formulated for the site utilizing the source referenced above.

However, these values should be reviewed by the building official (Risk Category) and structural engineer and the final design should be performed by a qualified structural engineer familiar with the region.

CBC 2013 SEISMIC DESIGN SUMMARY (ASCE 7-10)* Site Location (USGS WGS84) 34.091, -117.2851, Risk Category III	
Site Class Definition Chapter 20 ASCE 7	D
S_s Mapped Spectral Response Acceleration at 0.2s Period, (Figure 1613.3.1(1))	2.302
S_1 Mapped Spectral Response Acceleration at 1s Period, (Figure 1613.3.3(2))	1.051
F_a Short Period Site Coefficient at 0.2s Period, (Table 1613.3.3(1))	1.0
F_v Long Period Site Coefficient at 1s Period,(Table 1613.3.3(2))	1.5
S_{MS} Adjusted Spectral Response Acceleration at 0.2s Period, (eq .16-37)	2.302
S_{M1} Adjusted Spectral Response Acceleration at 1s Period, (eq .16-38)	1.576
S_{DS} Design Spectral Response Acceleration at 0.2s Period,(eq .16-39)	1.534
S_{D1} Design Spectral Response Acceleration at 1s Period, (eq .16-40)	1.051

CBC 2013 SEISMIC DESIGN SUMMARY (ASCE 7-10)* Site Location (USGS WGS84) 34.091, -117.2851, Risk Category III	
Seismic Design Category - Short Period (Table 1613.3.5(1))	E
Seismic Design Category - Long Period (Table 1613.3.5(2))	E
*Values obtained from U.S.G.S. online U.S. Seismic Design Maps tool	

INFILTRATION TESTING AND TEST RESULTS

Two double ring infiltration tests were conducted at the approximate locations illustrated on Enclosure A-2. Test pits were excavated to depths of approximately 5 to 5.5 feet and a 12-inch diameter casing was installed within the center of the test location with a 24-inch diameter casing centered around it. Each casing was imbedded to a depth approximately 3.5 inches. These liners extended approximately 16.5-inches above the bottom of the test location. The test locations were tested immediately after the casings were installed by filling both the inside and outside casings and maintaining a water level to a height of approximately 1.0 to 1.5-inches.

The testing procedure was as follows:

Both the inside and outside area of the casings were filled with water to a level of approximately 1.0 to 1.5-inches above the ground surface. Water was then metered to maintain this water level within both rings. The volume of water use in a given time period was recorded at various time intervals to establish the infiltration rate of the inner ring. See the attached Infiltration Test Data sheets, Enclosures E-1 and E-2 within Appendix E for the test information and measurements.

The infiltration rate is measured as the drop in water level compared to the permeability of the bottom surface area soils in the bottom of the test hole. If casing is not used, the water column in the test hole is allowed to seep into both the bottom and sidewalls of the hole, for which the drop in water level must be corrected and reduced for the volume of water seeping into the sidewall and for the diameter of the test hole. As described above, the tests described herein were conducted using a 12-inch diameter inner casing and 24-inch diameter outer casing.

The test holes were found to have the following measured clear water infiltration rates:

Infiltration Test No.	Depth (ft.)*	Infiltration Rate**	
		gal/sf/day	in/hr
DRI-1	5	10	0.6
DRI-2	5.5	15	0.8
* depth measured below existing ground surface ** average of final two readings			

The results of our percolation testing are attached as Enclosures E-1 and E-2.

Because of the poor infiltration rates obtained in these near surface, finer grained soils, additional testing was conducted at depth in the more granular materials.

For the additional testing at depth, two constant head percolation tests were conducted. Testing consisted of two test holes which were excavated using a hollow stem auger drill rig to depths of approximately 9 to 9.5 feet below the existing ground surface. The holes were 8-inches in diameter. Two inches of gravel was placed in the bottom of the holes and perforated plastic liner was placed into the holes. A 2-inch diameter PVC pipe with a preset water level of 0.5 feet was inserted into the liners. A 5-gallon, glass bottle was then inverted over the pipes with a vacuum seal in order to maintain a constant 0.5 feet of water within the holes. The volume of water used in a given time period was recorded at various time intervals to establish the percolation rate which was then converted to an infiltration rate.

Our test results are summarized in the following table:

Test No.	Depth (ft.)*	Percolation Rate ** gal/sf/day	Infiltration Rate** in/hr
P-1	9	51.5	3.5
P-2	9.5	54	3.6
* depth measured below existing ground surface ** average of final two readings			

The results of our percolation testing are attached as Enclosures E-3 and E-4.

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 and trench 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 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 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 greater than 50 feet, 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 5 to 15-foot 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 associated with existing development, 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 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 5 to 15 feet will be required within the proposed building area of the site to encounter competent alluvial materials. 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 Building Pad 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 feasible. 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.

Removals conducted adjacent to existing off-site structures (ie structure to the east) must consist of partial or alternating removals so as not to distress the existing foundations. A checkerboard system with 8 to 10 feet wide sections is recommended if the existing structure is supported by a continuous wall footing. If isolated footings exist along the wall where the excavation is required, then removals adjacent to only one-half of the footing should occur at one time. After the initial removal has been

replaced as engineered compacted fill the remaining half can then be removed and replaced.

During the removal process adjacent to existing structures, this building should be monitored for movements. Removals should occur in very short time frames. In no instance should removal areas be allowed to remain open overnight. Removals should not be conducted during rainy conditions.

To provide adequate support, concrete slabs-on-grade should bear on a minimum of 12 inches of compacted soil placed over competent alluvium. The final pad surfaces should be rolled to provide smooth, dense surfaces upon which to place the concrete.

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, if required, 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 3.6 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.

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	T.I.	DESIGN R-VALUE	PRELIMINARY SECTION
Parking and Driveway Areas (Light Vehicular Traffic and No Truck Traffic)	5.0	40	0.20' AC/0.30' AB or 0.45' AC/Compacted Soil or 0..33' PCC/Compacted Soil
Parking and Driveway Areas (Heavy Truck Traffic)	7.0	40	0.30' AC/0.65' AB or 0.65' AC/Compacted Soil or Parking Areas - 0.50' PCC/Compacted Soil Driveway Areas - 0.46' PCC/Compacted Soil for driveway areas
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 a majority of the on site soils is considered negligible by the CBC. However, one of the samples tested was found to have a moderate sulfate exposure, category S1. During grading, it is anticipated that mixing and blending of the soils will result in an overall negligible sulfate exposure. For this condition, no specific recommendations are given for concrete elements to be in contact with on-site soils. During grading, additional sampling and testing should be conducted to confirm the above. Should moderate sulfate exposure be found, the requirements of ACI 318, Table 4.3.1 for concrete should be incorporated into the design.

Infiltration

Based upon our field investigation and test data, a clear water absorption rate of 3 inches per hour appears to be applicable for the planned infiltration in the areas and depths tested by P-1 and P-2. In the area and depths of DRI-1 and DRI-2, the soils are not considered conducive to infiltration based on our test result of 0.2 inches per hour. A factor of safety of 3 should be applied as indicated by the San Bernardino County Stormwater Program, Technical Guidance Document for Water Quality Management Plans (2011).

To ensure continued infiltration capability of the infiltration area, a program to maintain the facility should be considered. This program should include periodic removal of accumulated materials, which can slow the infiltration and decrease the water quality. Materials to be removed from the infiltration areas typically consist of litter, dead plant matter, and soil fines (silts and clays). Proper maintenance of the system is critical.

A maintenance program should be prepared and properly executed. At a minimum, the program should be as outlined in the San Bernardino County Stormwater Program, Technical Guidance Document for Water Quality Management Plans (2011).

The program should also incorporate the recommendations contained within this report and any other jurisdictional agency requirements.

Systems should be set back at least 10 feet from foundations or as required by the design engineer.

Any geotextile filter fabric utilized should consist of such that it prevents soil piping but has greater permeability than the existing soil.

During site development, care should be taken to not disturb the area(s) proposed for infiltration as changes in the soil structure could occur resulting in a change of the soil infiltration characteristics. These undisturbed areas and any surrounding undisturbed areas with any improvements are potential subject to some settlement.

Due to the presence of fill materials, such materials should be completely removed to expose native materials, similar to those tested and considered suitable for infiltration. After removal of the fill materials, should the resultant grade be undesirable, any fill placed should consist of granular materials, which after placement, have infiltration rates equal to or greater than those tested.

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 Burrtec Waste Industries, Inc. 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.

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.

Andrew A. Tardie
Staff Geologist

Robert M. Markoff, CEG 2073
Engineering Geologist

John P. Leuer, GE 2030
President

AAT/RMM/JPL/ss

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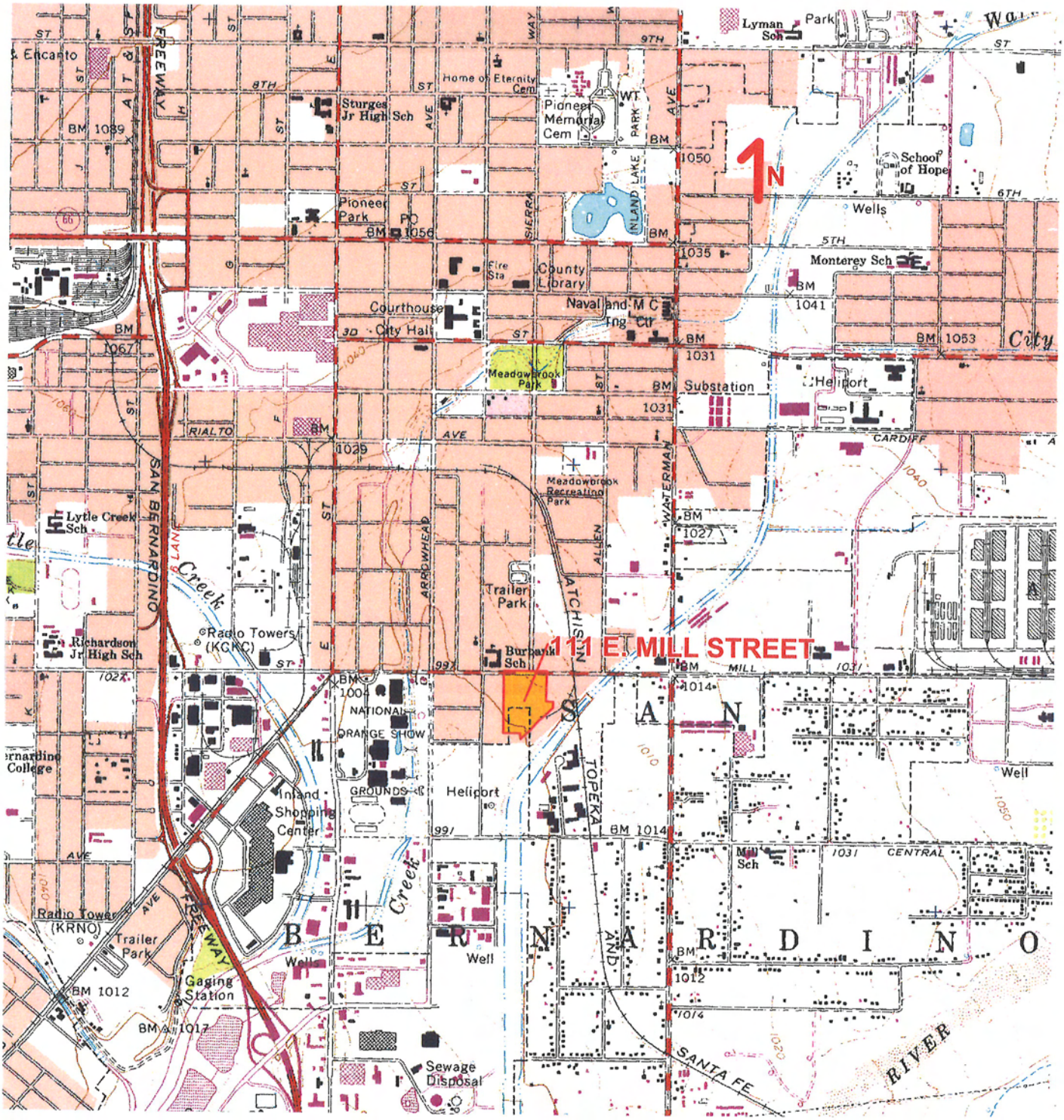
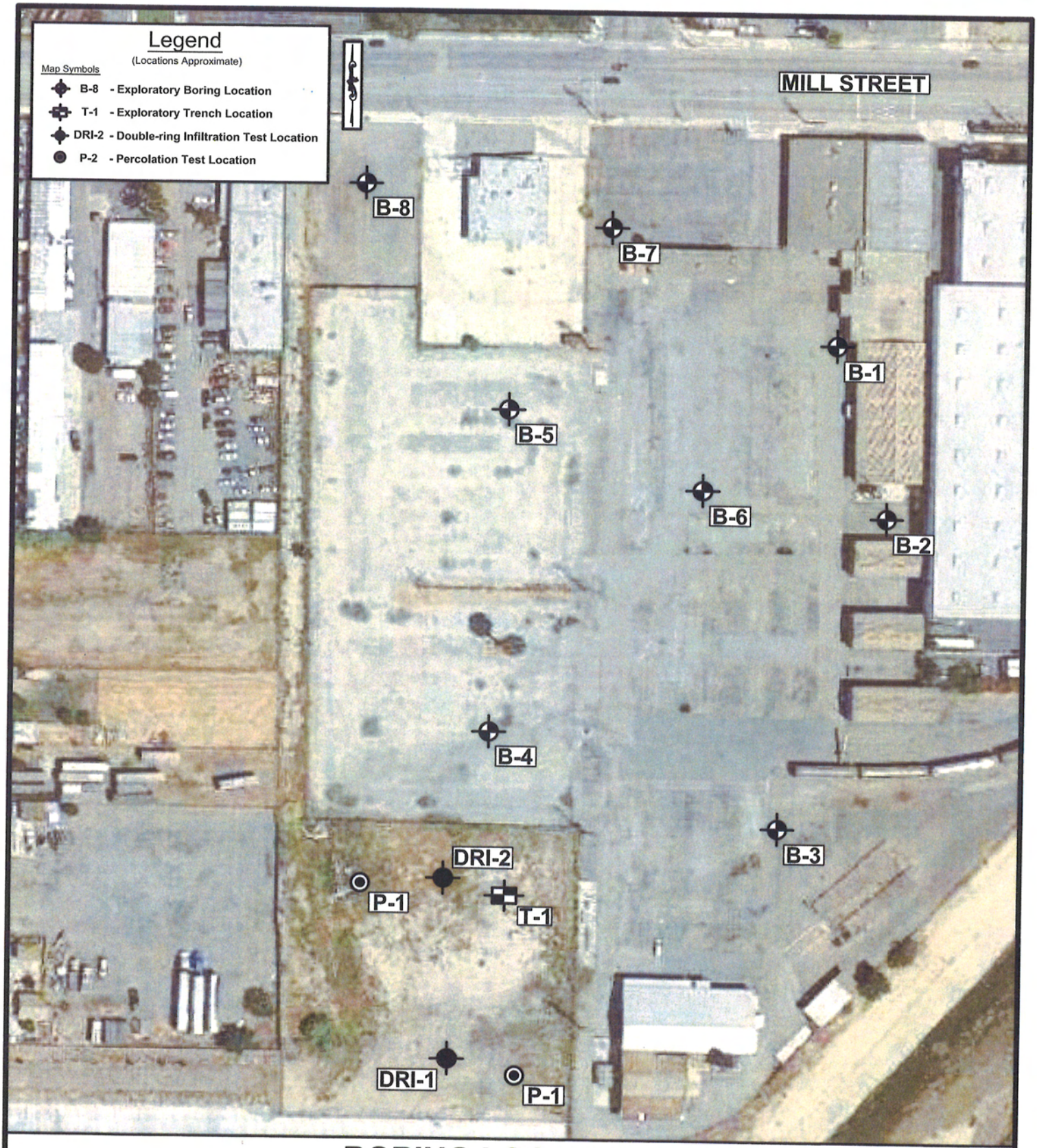
APPENDIX A

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



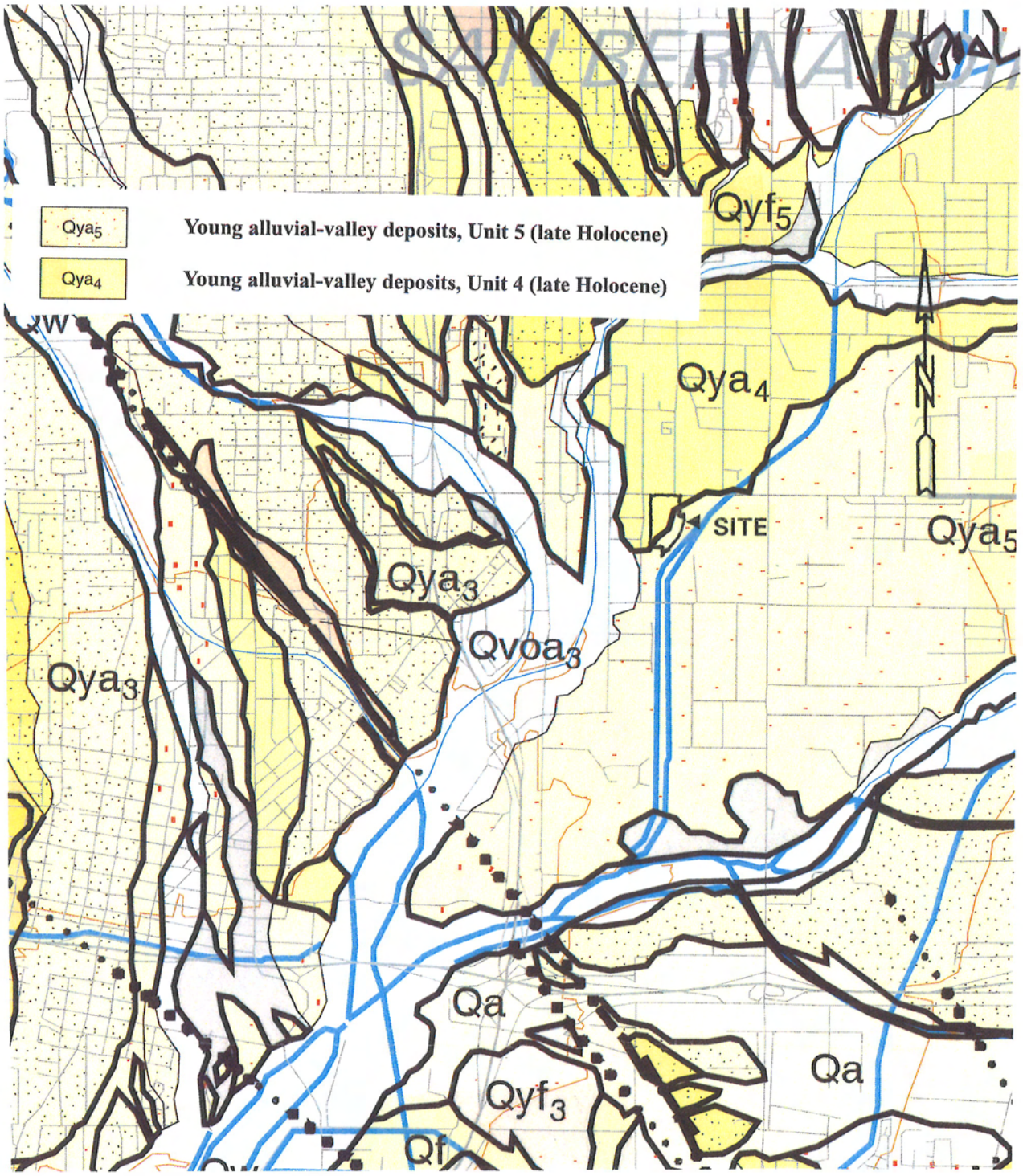
INDEX MAP

PROJECT:	111 EAST MILL STREET, SAN BERNARDINO, CA	**PROJECT NO.:**	63243.1
CLIENT:	BURRTEC WASTE INDUSTRIES, LLC	**ENCLOSURE:**	A-1
LOR Geotechnical Group, Inc.		**DATE:**	MARCH 2016
		SCALE:	1" = 2,000'



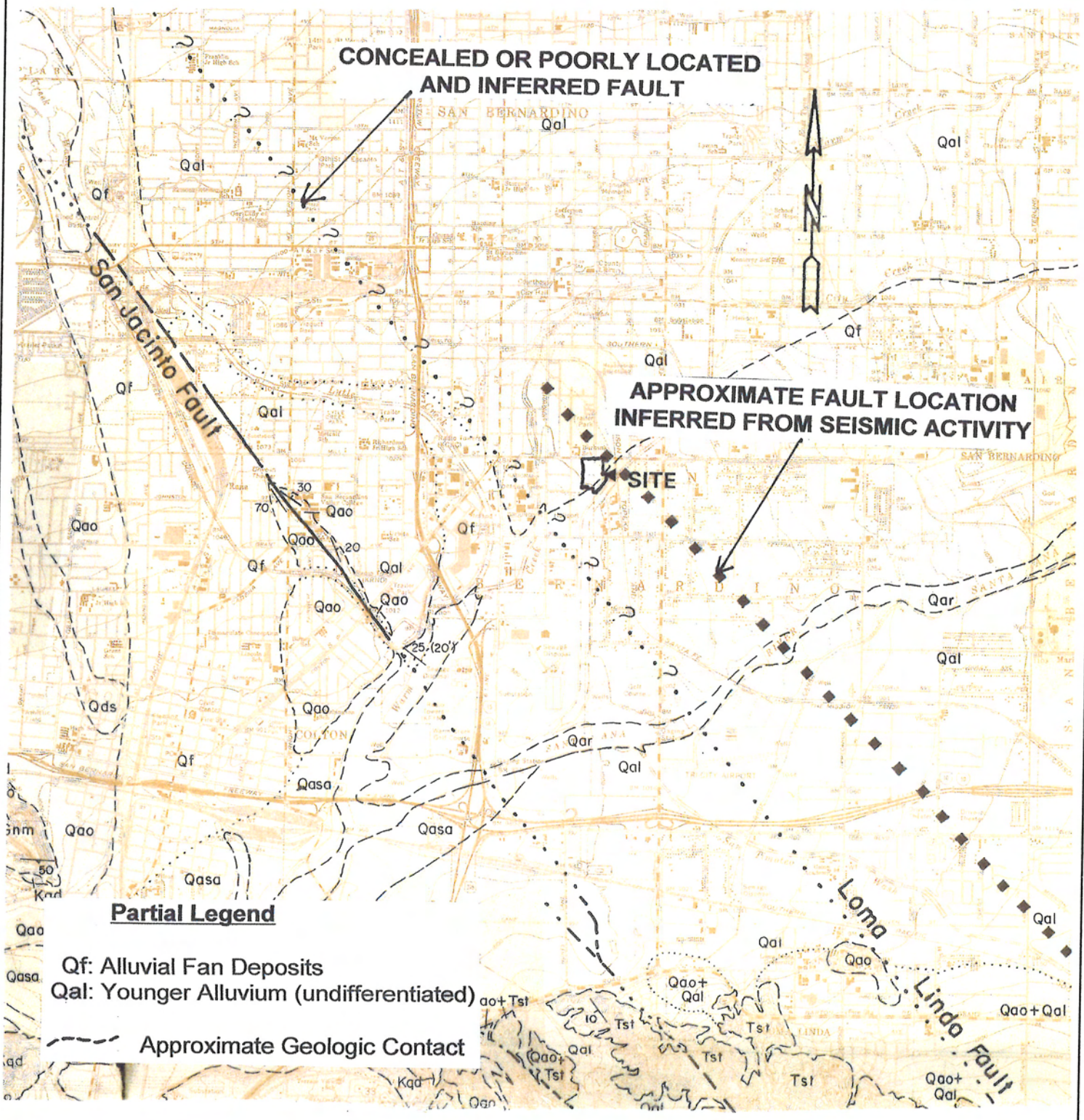
BORING LOCATION MAP

PROJECT:	111 EAST MILL STREET, SAN BERNARDINO, CA	**PROJECT NO:**	63243.1
CLIENT:	BURRTEC WASTE INDUSTRIES, LLC	**ENCLOSURE:**	A-2
LOR Geotechnical Group, Inc.		**DATE:**	MARCH 2016
		SCALE:	1" ≈ 120'



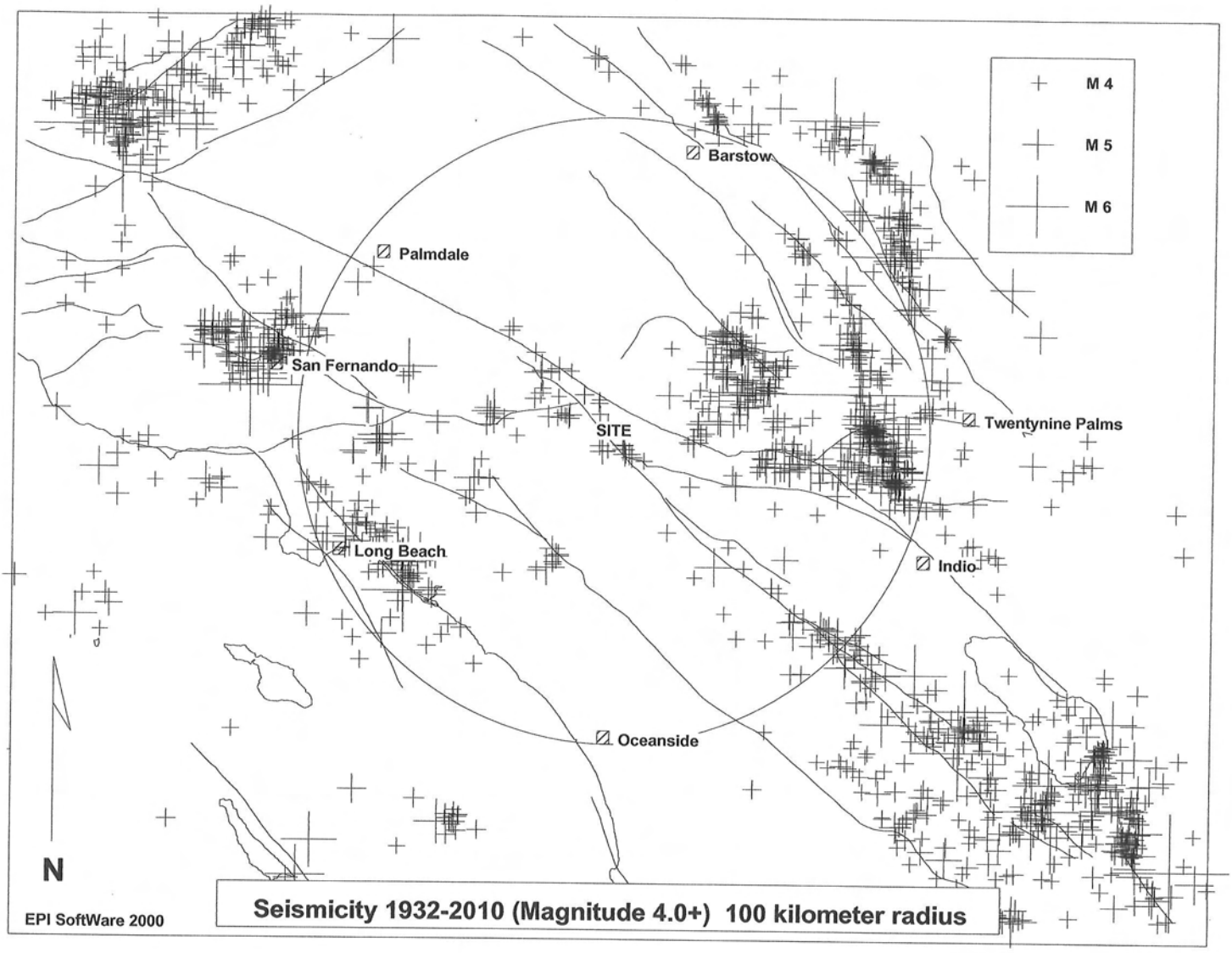
REGIONAL GEOLOGIC MAP (Morton and Miller, 2002)

PROJECT:	111 E. MILL STREET, SAN BERNARDINO, CA	PROJECT NO.:	63243.1
CLIENT:	BURRTEC WASTE INDUSTRIES, LLC	ENCLOSURE:	A-3
LOR Geotechnical Group, Inc.		DATE:	MARCH 2016
		SCALE:	1" = 3,333'



GEOLOGIC MAP, PORTIONS OF SW SAN BERNARDINO COUNTY (FiFe, 1976)

PROJECT:	111 EAST MILL STREET, SAN BERNARDINO, CA	PROJECT NO.:	63243.1
CLIENT:	BURRTEC WASTE INDUSTRIES, LLC	ENCLOSURE:	A-4
LOR Geotechnical Group, Inc.		DATE:	MARCH 2016
		SCALE:	1" = 3,350'



SITE LOCATION: 34.0910 LAT. -117.2851 LONG.

MINIMUM LOCATION QUALITY: C

TOTAL # OF EVENTS ON PLOT: 1499

TOTAL # OF EVENTS WITHIN SEARCH RADIUS: 582

MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

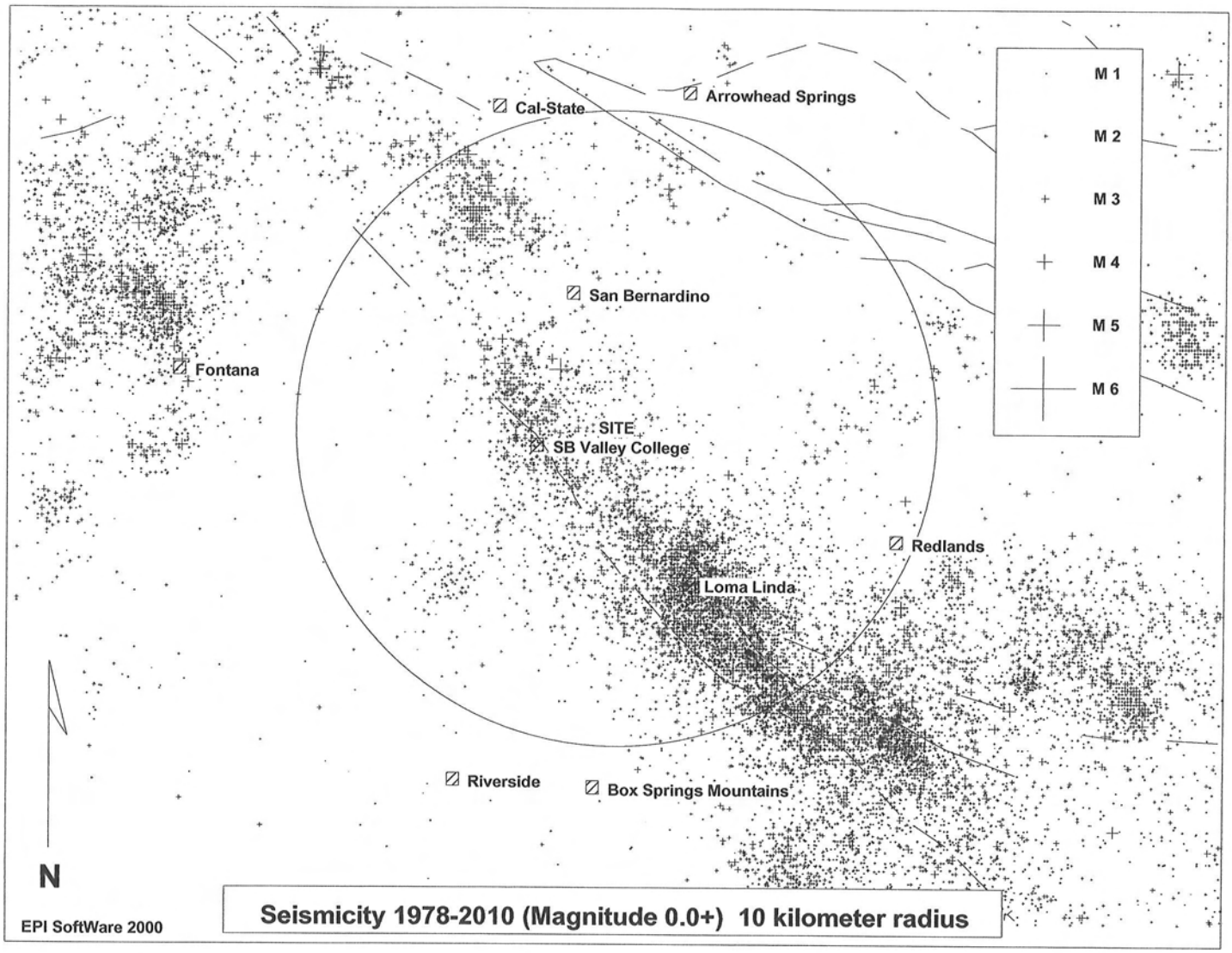
4.0- 4.9 : 522
5.0- 5.9 : 55
6.0- 6.9 : 4
7.0- 7.9 : 1
8.0- 8.9 : 0

0 50 100
KILOMETERS

CLOSEST EVENT: 4.5 ON FRIDAY, JANUARY 09, 200[illegible] LOCATED APPROX. 3 KILOMETERS NORTHWEST OF THE SITE

LARGEST 5 EVENTS:

7.3 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 78 KILOMETERS EAST OF THE SITE
6.4 ON SATURDAY, MARCH 11, 193[illegible] LOCATED APPROX. 83 KILOMETERS SOUTHWEST OF THE SITE
6.3 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 43 KILOMETERS EAST OF THE SITE
6.1 ON THURSDAY, APRIL 23, 199[illegible] LOCATED APPROX. 90 KILOMETERS EAST OF THE SITE
6.0 ON SATURDAY, DECEMBER 04, 194[illegible] LOCATED APPROX. 88 KILOMETERS EAST OF THE SITE



SITE LOCATION: 34.0910 LAT. -117.2851 LONG.

MINIMUM LOCATION QUALITY: A

TOTAL # OF EVENTS ON PLOT: 14680

TOTAL # OF EVENTS WITHIN SEARCH RADIUS: 6054

0 5 10
KILOMETERS

MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

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1.0- 1.9 : 4660
2.0- 2.9 : 957
3.0- 3.9 : 59
4.0- 4.9 : 6
5.0- 5.9 : 0
6.0- 6.9 : 0
7.0- 7.9 : 0
8.0- 8.9 : 0

CLOSEST EVENT: 1.8 ON TUESDAY, AUGUST 11, 1981 LOCATED APPROX. 0 KILOMETER OF THE SITE

LARGEST 5 EVENTS:

4.6 ON WEDNESDAY, OCTOBER 02, 198[illegible] LOCATED APPROX. 8 KILOMETERS SOUTHEAST OF THE SITE
4.5 ON FRIDAY, JANUARY 09, 200[illegible] LOCATED APPROX. 2 KILOMETERS NORTHWEST OF THE SITE
4.5 ON WEDNESDAY, MARCH 11, 199[illegible] LOCATED APPROX. 9 KILOMETERS SOUTHEAST OF THE SITE
4.4 ON MONDAY, FEBRUARY 21, 200[illegible] LOCATED APPROX. 5 KILOMETERS SOUTHEAST OF THE SITE
4.2 ON SATURDAY, JUNE 28, 1997 LOCATED APPROX. 9 KILOMETERS NORTHWEST OF THE SITE

APPENDIX B

Field Investigation Program, and Boring and Trench Logs

APPENDIX B
FIELD INVESTIGATION

Subsurface Exploration

The site was investigated on March 1, 2016 and consisted of advancing 8 exploratory borings to depths between approximately 6.5 and 50 feet below the existing ground surface and excavating 1 exploratory trench to a depth of approximately 10 feet. The approximate locations of the borings are shown on Enclosure A-2, within Appendix A.

The drilling exploration was conducted using a truck-mounted Mobile B61 drill rig equipped with 8-inch diameter hollow stem augers. The soils were continuously logged by our geologist 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 maximum interval of 5 feet. The samples were recovered by using a California split barrel sampler of 2.50 inch inside diameter and 3.25 inch outside diameter and a Standard Penetration Sampler (SPT) from the ground surface to the total depth explored. The samplers were driven by a 140 pound automatic trip hammer dropped from a height of 30 inches. The number of hammer blows required to drive the sampler into the ground the final 12 inches were recorded and further converted to an equivalent SPT N-value. Factors such as efficiency of the automatic trip hammer used during this investigation (80%), borehole diameter (8"), and rod length at the test depth were considered for further computing of equivalent SPT N-values corrected for field procedures (N60) which are included in the boring logs, Enclosures B-1 through B-5.

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 containers. Disturbed soil samples were obtained at selected levels within the borings and placed in sealed containers for transport to the laboratory.

The trenching exploration was conducted using a tractor-mounted John Deere 410J backhoe equipped with a 24-inch bucket. The soils were continuously logged by our geologist 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.

All samples obtained were taken to our geotechnical laboratory for storage and testing. Detailed logs of the borings are presented on the enclosed Boring and Trench Logs, Enclosures B-1 through B-9. A Boring and Trench Log Legend and Soil Classification Chart are presented on Enclosures B-i and B-ii, respectively.

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
14	Corrosion

BORING LOG LEGEND

PROJECT:	**111 EAST MILL STREET**	**PROJECT NO.:**	**63243.1**
CLIENT:	**BURRTEC WASTE INDUSTRIES, INC.**	**ENCLOSURE:**	**B-i**
LOR Geotechnical Group, Inc.		**DATE:**	**MARCH 2016**

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
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: 111 EAST MILL STREET	PROJECT NO.: 63243.1
CLIENT: BURRTEC WASTE INDUSTRIES, INC.	ENCLOSURE: B-ii
LOR Geotechnical Group, Inc.	DATE: MARCH 2016

LOG OF BORING B-1

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, ASPHALT CONCRETE, 0.16' thick, poor condition.
	17		8.2		111.8			CL	@ 0.16 feet, AGGREGATE BASE, 0.16' thick.
5									@ 0.32 feet, FILL:SILTY SAND, approximately 5% fine to coarse gravel to 1", 20% coarse-grained sand, 20% medium-grained sand, 35% fine-grained sand, 20% silty fines, dark brown, damp.
	33		3.5		112.7			SM	
	15		13.5		102.4			SW	@ 2 feet, LEAN CLAY with SAND, approximately 10% medium-grained sand, 30% fine-grained sand, 60% clayey fines of low plasticity, dark brown to black, damp.
10	14		9.7		107.1			SM	@ 5 feet, SILTY SAND with GRAVEL, approximately 10% fine to coarse gravel to 1.5", 25% coarse-grained sand, 25% medium-grained sand, 25% fine-grained sand, 15% silty fines, brown, damp.
15	34		1.6		102.7			SW	@ 7 feet, WELL GRADED SAND, approximately 5% fine to coarse gravel to 2", 30% coarse-grained sand, 30% medium-grained sand, 30% fine-grained sand, 5% silty fines, tan, dry.
20	41		2.9		107.2				@ 10 feet, SILTY SAND, approximately 5% fine gravel to 0.5", 20% coarse-grained sand, 30% medium-grained sand, 25% fine-grained sand, 20% silty fines, dark brown to black, damp.
25	14		11.6		100.6			SM	@ 15 feet, ALLUVIUM:WELL GRADED SAND, approximately 5% fine gravel to 0.5", 30% coarse-grained sand, 30% medium-grained sand, 30% fine-grained sand, white, dry.
									@ 20 feet, no gravel, slightly finer-grained.
30	54 for 5"		2.8		122.6			SW	@ 25 feet, SILTY SAND, approximately 5% coarse-grained sand, 35% medium-grained sand, 45% fine-grained sand, 15% silty fines, gray, damp.
35	26		3.4		108.6				@ 30 feet, WELL GRADED SAND, approximately 10% fine to coarse gravel to 2", 25% coarse-grained sand, 25% medium-grained sand, 35% fine-grained sand, 5% silty fines, tan, dry.
									@ 32 feet, increase in gravel, cobbles possibly present as difficult drilling was encountered.
									@ 35 feet, some iron oxide staining.
40	17		9.4		116.3			SM	@ 40 feet, SILTY SAND, trace fine gravel to 0.5", trace coarse-grained sand, approximately 20% medium-grained sand, 40% fine-grained sand, 40% silty fines, brown, some iron oxide staining, damp.
45	83 for 10"							SW	@ 45 feet, WELL GRADED SAND, approximately 20% coarse-grained sand, 35% medium-grained sand, 40% fine-grained sand, 5% silty fines, tan, dry, no recovery.
50	79 for 11"		2.1		109.1				@ 50 feet, trace fine to coarse gravel to 2".
									END OF BORING
55									Fill at 0-15 feet No Groundwater No Bedrock

TEST DATA

PROJECT:	111 East Mill Street	PROJECT NUMBER:	63243.1
CLIENT:	Burrtec Waste Industries, Inc.	ELEVATION:	
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	March 1, 2016
		EQUIPMENT:	Mobile B61
		HOLE DIA.: 8"	ENCLOSURE: B-1

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								SW	@ 0 feet, ASPHALT CONCRETE, 0.25' thick, poor condition.
	7		16.6		85.4			SM	@ 0.25 feet, FILL: WELL GRADED SAND, trace fine gravel to 0.5", approximately 25% coarse-grained sand, 30% medium-grained sand, 40% fine-grained sand, 5% silty fines, tan, moist.
5									@ 2 feet, SILTY SAND, approximately 5% coarse-grained sand, 30% medium-grained sand, 45% fine-grained sand, 20% silty fines, gray, damp.
	3		18.1		76.4			ML	@ 5 feet, ALLUVIUM: SANDY SILT, approximately 10% medium-grained sand, 30% fine-grained sand, 60% silty fines, gray, damp.
	14		9.5		97.9			SW	@ 7 feet, WELL GRADED SAND, approximately 20% coarse-grained sand, 35% medium-grained sand, 40% fine-grained sand, 5% silty fines, tan, dry, some iron oxide staining.
10	6		14.8		97.5				
								ML	@ 11 feet, SANDY SILT, approximately 45% fine-grained sand, 55% silty fines, gray, damp.
	30		1.6		106.7			SP	@ 12 feet, POORLY GRADED SAND, approximately 15% coarse-grained sand, 30% medium-grained sand, 50% fine-grained sand, 5% silty fines, white, dry.
15	34		1.4		107.0			SW	@ 15 feet, WELL GRADED SAND, approximately 5% fine gravel to 0.5", 30% coarse-grained sand, 30% medium-grained sand, 30% fine-grained sand, 5% silty fines, white, dry.
20	98		1.5		114.9				
25	58		2.7		108.2				END OF BORING Fill at 0-5 feet No Groundwater No Bedrock
30									

TEST DATA

PROJECT:	111 East Mill Street	PROJECT NUMBER:	63243.1
CLIENT:	Burrtec Waste Industries, Inc.	ELEVATION:	
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	March 1, 2016
		EQUIPMENT:	Mobile B61
		HOLE DIA.: 8"	ENCLOSURE: B-2

LOG OF BORING B-3

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
	TEST DATA								
0									@ 0 feet, ASPHALT CONCRETE, 0.16' thick, fair condition.
									@ 0.16 feet, AGGREGATE BASE, 0.16' thick.
								SM	@ 0.32 feet, ALLUVIUM:SILTY SAND, approximately 5% fine gravel to 0.75", 10% coarse-grained sand, 30% medium-grained sand, 35% fine-grained sand, 20% silty fines.
	12		7.4		97.9				
5	10		19.8		83.2				@ 5 feet, approximately 10% medium-grained sand, 75% fine-grained sand, 15% silty fines, brown, moist.
									END OF BORING
									No Fill
									No Groundwater
									No Bedrock
10									

PROJECT:	111 East Mill Street	PROJECT NUMBER:	63243.1
CLIENT:	Burrtec Waste Industries, Inc.	ELEVATION:	
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	March 1, 2016
		EQUIPMENT:	Mobile B61
		HOLE DIA.: 8"	ENCLOSURE: B-3

LOG OF BORING B-4

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0									@ 0 feet, 0.75" rock.
								SM	@ 0.25 feet, ASPHALT CONCRETE, 0.08' thick.
									@ 0.33 feet, FILL: SILTY SAND, approximately 10% fine to coarse gravel to 1", 15% coarse-grained sand, 25% medium-grained sand, 30% fine-grained sand, 20% silty fines, tan, damp.
	11		7.6		113.7				
									@ 3 feet, some brick debris.
5	8		18.9		86.9			SM	@ 5 feet, ALLUVIUM: SILTY SAND, approximately 55% fine-grained sand, 45% silty fines, brown, moist, trace pinhole porosity.
									END OF BORING
									Fill at 0-5 feet
									No Groundwater
									No Bedrock
10									

PROJECT:	111 East Mill Street	PROJECT NUMBER:	63243.1
CLIENT:	Burrtec Waste Industries, Inc.	ELEVATION:	
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	March 1, 2016
		EQUIPMENT:	Mobile B61
		HOLE DIA.: 8"	ENCLOSURE: B-4

LOG OF BORING B-5

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0									@ 0 feet, 0.75" rock.
	13		19.6		95.8			ML	@ 0.5 feet, ALLUVIUM:SANDY SILT, approximately 5% coarse-grained sand, 10% medium-grained sand, 25% fine-grained sand, 60% silty fines, tan, moist, trace pinhole porosity, trace secondary calcite, trace iron oxide staining.
								SP SM	@ 3 feet, POORLY GRADED SAND with SILT, approximately 20% medium-grained sand, 70% fine-grained sand, 10% silty fines, grayish-brown.
5	17		7.0		98.2				
									END OF BORING No Fill No Groundwater No Bedrock
10									

PROJECT:	111 East Mill Street	PROJECT NUMBER:	63243.1
CLIENT:	Burrtec Waste Industries, Inc.	ELEVATION:	
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	March 1, 2016
		EQUIPMENT:	Mobile B61
		HOLE DIA.: 8"	ENCLOSURE: B-5

LOG OF BORING B-6

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0									@ 0 feet, ASPHALT CONCRETE, 0.33' thick, fair condition.
								SM	@ 0.33 feet, FILL: SILTY SAND, approximately 5% fine gravel to 0.5", 20% coarse-grained sand, 20% medium-grained sand, 20% fine-grained sand, 35% silty fines.
	15		11.7		107.2			SP	@ 2 feet, ALLUVIUM: POORLY GRADED SAND, approximately 10% coarse-grained sand, 30% medium-grained sand, 55% fine-grained sand, 5% silty fines, tan, moist.
5	23		15.4		111.4				@ 5 feet, thin, 1-2" thick occasional clay layers.
									END OF BORING Fill at 0-2 feet No Groundwater No Bedrock
10									

PROJECT:	111 East Mill Street	PROJECT NUMBER:	63243.1
CLIENT:	Burrtec Waste Industries, Inc.	ELEVATION:	
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	March 1, 2016
		EQUIPMENT:	Mobile B61
		HOLE DIA.: 8"	ENCLOSURE: B-6

LOG OF BORING B-7

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0									@ 0 feet, ASPHALT CONCRETE, 0.25' thick, poor condition.
	8		16.2		95.2			ML SM	@ 0.25 feet, ALLUVIUM:SILTY SAND/SANDY SILT, approximately 50% fine-grained sand, 50% silty fines, tan, moist.
5	10		13.4		85.3			SP	@ 5 feet, POORLY GRADED SAND, approximately 5% coarse-grained sand, 30% medium-grained sand, 60% fine-grained sand, 5% silty fines, tan, some iron oxide staining.
									END OF BORING No Fill No Groundwater No Bedrock
10									

PROJECT:	111 East Mill Street	PROJECT NUMBER:	63243.1
CLIENT:	Burrtec Waste Industries, Inc.	ELEVATION:	
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	March 1, 2016
		EQUIPMENT:	Mobile B61
		HOLE DIA.: 8"	ENCLOSURE: B-7

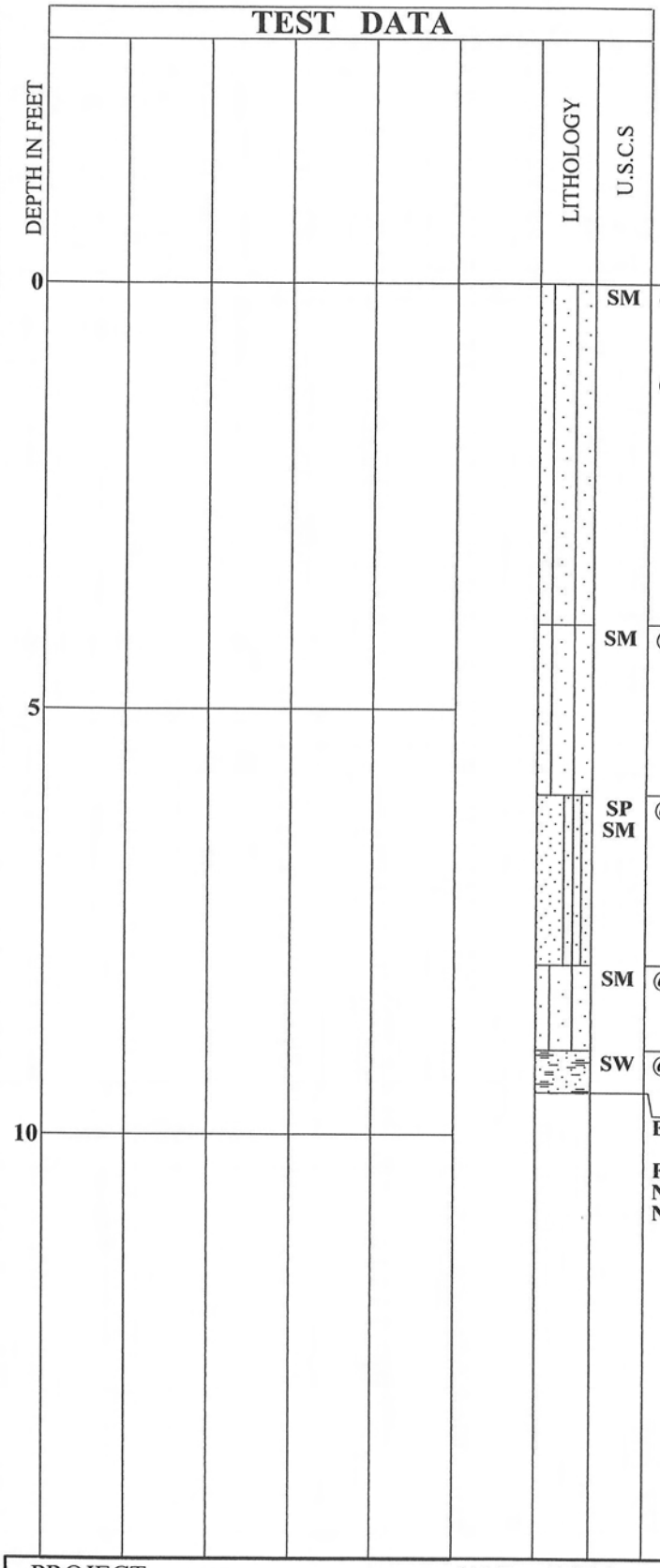
LOG OF BORING B-8

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0									@ 0 feet, 0.75" rock.
	7		20.6		90.1			ML	@ 0.25 feet, ALLUVIUM:SANDY SILT, approximately 10% medium-grained sand, 30% fine-grained sand, 60% silty fines, grayish-brown, damp, trace pinhole porosity.
5	10		24.6						
									END OF BORING No Fill No Groundwater No Bedrock
10									

TEST DATA

PROJECT:	111 East Mill Street	PROJECT NUMBER:	63243.1
CLIENT:	Burrtec Waste Industries, Inc.	ELEVATION:	
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	March 1, 2016
		EQUIPMENT:	Mobile B61
		HOLE DIA.: 8"	ENCLOSURE: B-8

LOG OF BORING P-1



U.S.C.S	DESCRIPTION
SM	@ 0 feet, FILL: SILTY SAND with GRAVEL, approximately 15% fine to coarse gravel to 3", 20% coarse-grained sand, 10% medium-grained sand, 20% fine-grained sand, 35% silty fines, tan, dry. @ 0.5 feet, some asphalt debris.
SM	@ 4 feet, ALLUVIUM: SILTY SAND, approximately 15% coarse-grained sand, 30% medium-grained sand, 30% fine-grained sand, 25% silty fines, brown, damp.
SP SM	@ 6 feet, POORLY GRADED SAND with SILT, approxiumately 5% coarse-grained sand, 25% medium-grained sand, 60% fine-grained sand, 10% silty fines, tan, dry.
SM	@ 8 feet, SILTY SAND, approximately 10% coarse-grained sand, 35% medium-grained sand, 35% fine-grained sand, 20% silty fines.
SW	@ 9 feet, WELL GRADED SAND, trace fine gravel to 0.5", approximately 30% coarse-grained sand, 30% medium-grained sand, 35% fine-grained sand, 5% silty fines, tan, dry.

END OF BORING

Fill at 0-4 feet
No Groundwater
No Bedrock

PROJECT:	**111 East Mill Street**	PROJECT NUMBER:	**63243.1**
CLIENT:	**Burrtec Waste Industries, Inc.**	ELEVATION:	
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	**March 1, 2016**
		EQUIPMENT:	**Mobile B61**
		HOLE DIA.: **8"**	ENCLOSURE: **B-9**

LOG OF BORING P-2

TEST DATA

DEPTH IN FEET	LITHOLOGY	U.S.C.S	DESCRIPTION
0		SM	@ 0 feet, FILL:SILTY SAND, approximately 5% fine gravel to 0.5", 10% coarse-grained sand, 30% medium-grained sand, 20% fine-grained sand, 35% silty fines, brown, dry.
			@ 3 feet, fine to coarse gravel to 3".
5		SM	@ 5 feet, ALLUVIUM:SILTY SAND, trace fine gravel to 0.5", approximately 5% coarse-grained sand, 20% medium-grained sand, 60% fine-grained sand, 15% silty fines, brown, damp.
			@ 7 feet, as above, approximately 15% coarse-grained sand, 30% medium-grained sand, 35% fine-grained sand, 20% silty fines.
		SP SM	@ 9 feet, POORLY GRADED SAND with SILT, approximately 5% coarse-grained sand, 30% medium-grained sand, 55% fine-grained sand, 10% silty fines, tan, dry.
10			END OF BORING Fill at 0-5 feet No Groundwater No Bedrock

PROJECT:	111 East Mill Street	PROJECT NUMBER:	63243.1
CLIENT:	Burrtec Waste Industries, Inc.	ELEVATION:	
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	March 1, 2016
		EQUIPMENT:	Mobile B61
		HOLE DIA.: 8"	ENCLOSURE: B-10

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, 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-8, 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	2-4	(SM) Silty Sand	125.0	10.0

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. Samples are tested at varying normal loads in order to evaluate the shear strength

parameters, angle of internal friction and cohesion. Samples are tested in 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.)**	**Apparent Cohesion (psf)**	**Angle of Internal Friction (degrees)**
B-2	2-4	(SM) Silty Sand	300	27

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 and C-2.

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 and C-2.

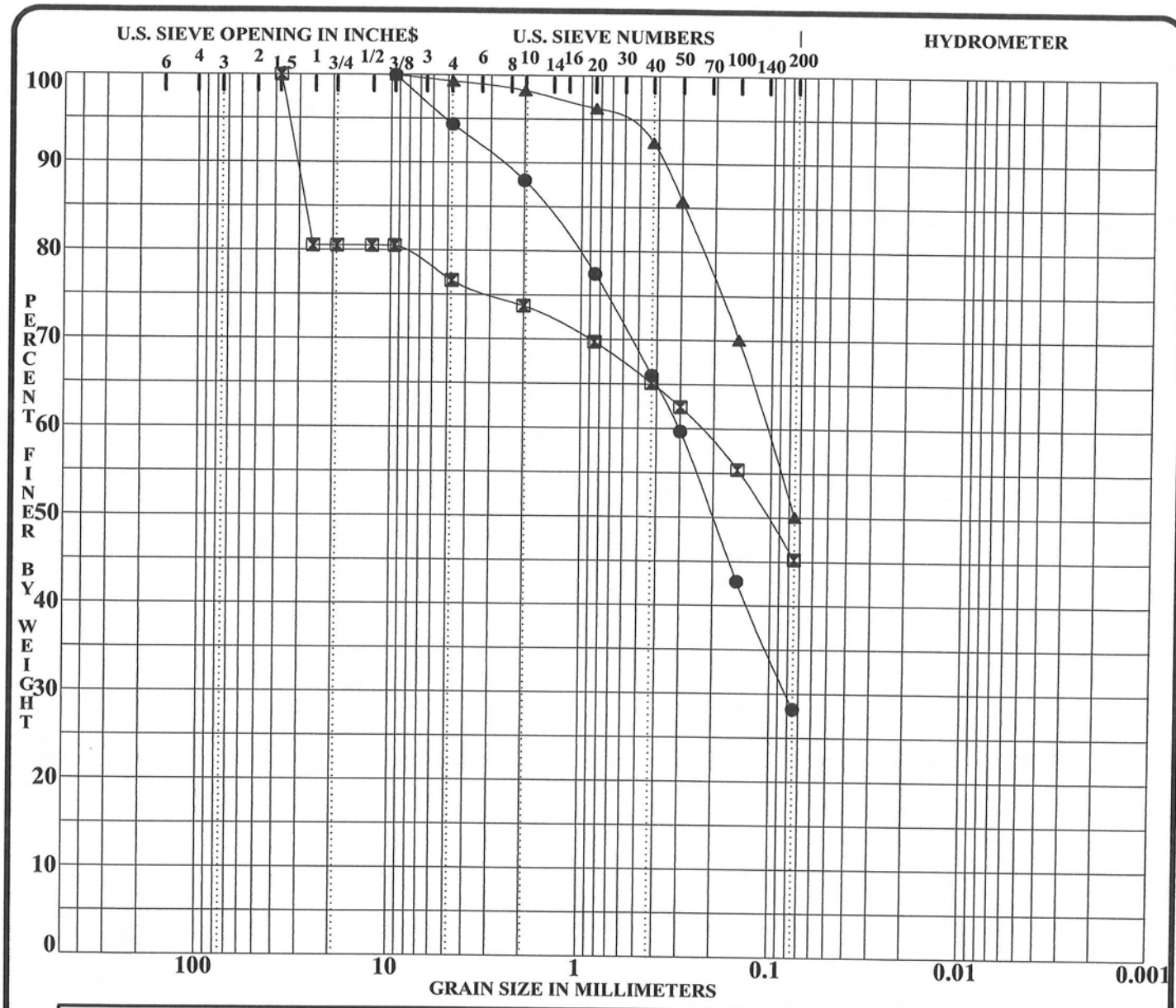
R-Value Test

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-1and C-2.

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	2-4	(SM) Silty Sand	0.13
B-4	0.33-3.33	(SM) Silty Sand with Gravel	0.015
B-5	0-5-3.5	(ML) Sandy Silt	<0.005
B-7	0.25-3.25	(SM) Silty Sand	<0.005



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification	Soil Classification	SE	RV			Cc	Cu
● B-3 @ 0.33-3.33'	(SM) Silty Sand	25	--				
⊠ B-4 @ 0.33-3.33'	(SM) Silty Sand with Gravel	7	--				
▲ B-5 @ 0.5-3.5	(ML) Sandy Silt	11	47				

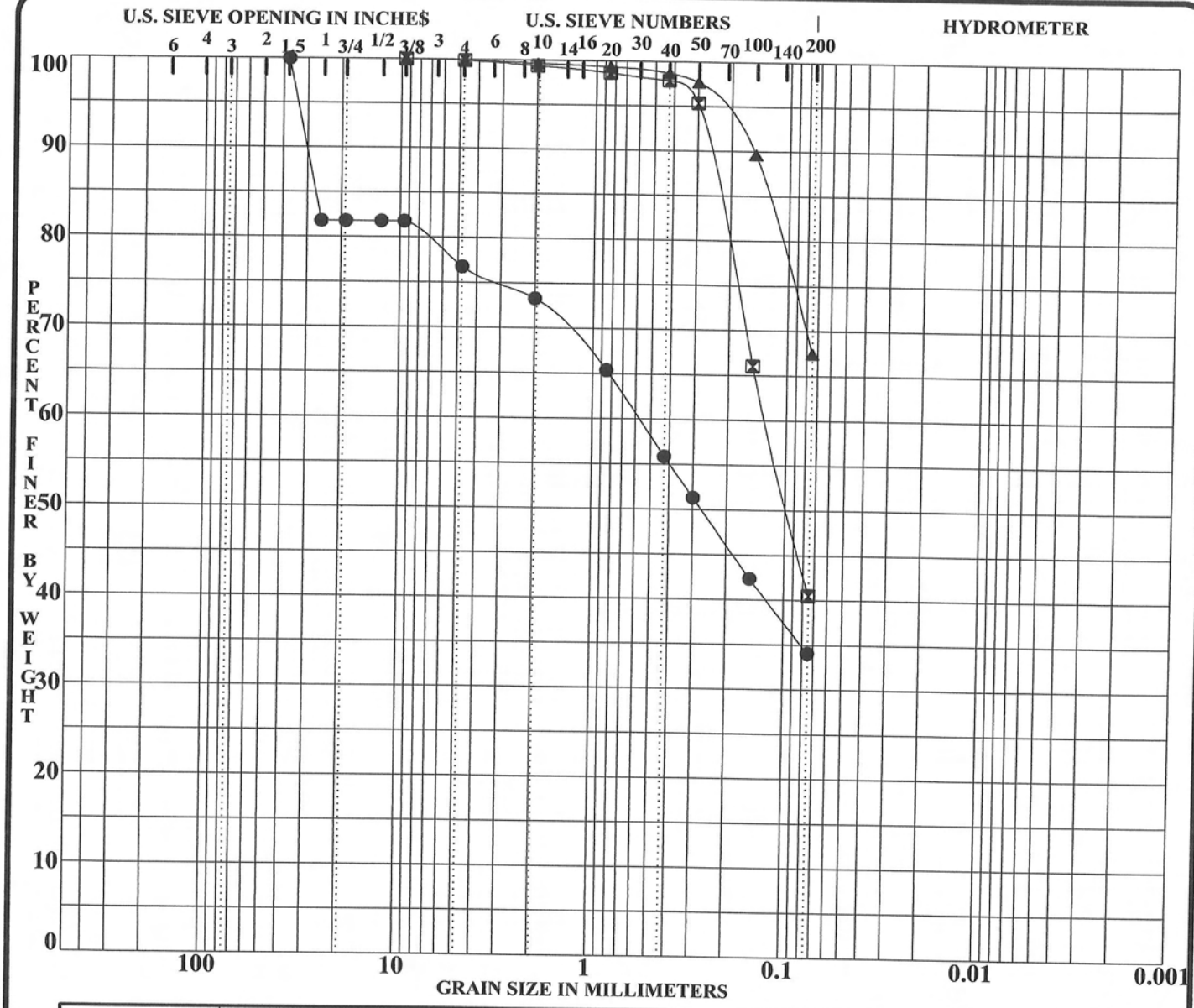
Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● B-3 @ 0.33-3.33'	9.50	0.31	0.082		5.6	66.1	28.3	
⊠ B-4 @ 0.33-3.33'	37.50	0.24			23.4	31.3	45.2	
▲ B-5 @ 0.5-3.5	9.50	0.11			0.7	49.2	50.1	

PROJECT *111 East Mill Street*

PROJECT NO. *63243.1*
DATE *3/28/16*

GRADATION CURVES
LOR Geotechnical Group, Inc.

ENCLOSURE C-1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification	Soil Classification	SE	RV			Cc	Cu
● B-6 @ 0.33-3.33'	(SM) Silty Sand with Gravel	16	--				
⊠ B-7 @ 0.33-3.33'	(SM) Silty Sand	13	56				
▲ B-8 @ 0.25-3.25'	(ML) Sandy Silt	7	40				

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● B-6 @ 0.33-3.33'	37.50	0.57			23.2	42.7	34.1	
⊠ B-7 @ 0.33-3.33'	9.50	0.13			0.1	59.4	40.5	
▲ B-8 @ 0.25-3.25'	9.50				0.1	32.4	67.5	

PROJECT 111 East Mill Street

PROJECT NO. 63243.1

DATE 3/28/16

GRADATION CURVES

LOR Geotechnical Group, Inc.

ENCLOSURE C-2

APPENDIX D

Liquefaction Calculations

```
*****************************
*                           *
*      L I Q U E F Y 2      *
*                           *
*       Version 1.50        *
*                           *
*****************************
```

EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: 9842-0000 DATE: 03-31-2016

JOB NAME: Sample Problem

SOIL-PROFILE NAME: LIQTEST.LDW

BORING GROUNDWATER DEPTH: 26.00 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 (1998, in press)

Magnitude Scaling Factor: 0.988

rd-CORRECTION METHOD: Idriss (1998, in press)

FIELD SPT N-VALUES ARE CORRECTED FOR THE LENGTH OF THE DRIVE RODS.

Rod Stick-Up Above Ground: 3.0 ft

CN NORMALIZATION FACTOR: 1.044 tsf

MINIMUM CN VALUE: 0.6

NCEER [1997] Method — LIQUEFACTION ANALYSIS SUMMARY — PAGE 1

File Name: LIQTEST.OUT

| CALC. | TOTAL | EFF. | FIELD | FC | | CORR. | LIQUE. | | INDUC. | LIQUE.

SOIL NO.	DEPTH (ft)	STRESS (tsf)	STRESS (tsf)	N (B/ft)	DELTA N1_60	C N	(N1)60 (B/ft)	RESIST RATIO	r d	STRESS RATIO	SAFETY FACTOR
1	0.25	0.014	0.014	17	~	*	*	*	*	*	**
1	0.75	0.043	0.043	17	~	*	*	*	*	*	**
1	1.25	0.072	0.072	17	~	*	*	*	*	*	**
1	1.75	0.101	0.101	17	~	*	*	*	*	*	**
2	2.25	0.131	0.131	17	~	*	*	*	*	*	**
2	2.75	0.161	0.161	17	~	*	*	*	*	*	**
2	3.25	0.191	0.191	17	~	*	*	*	*	*	**
2	3.75	0.221	0.221	17	~	*	*	*	*	*	**
2	4.25	0.252	0.252	17	~	*	*	*	*	*	**
2	4.75	0.282	0.282	17	~	*	*	*	*	*	**
3	5.25	0.311	0.311	33	~	*	*	*	*	*	**
3	5.75	0.341	0.341	33	~	*	*	*	*	*	**
3	6.25	0.370	0.370	33	~	*	*	*	*	*	**
3	6.75	0.399	0.399	33	~	*	*	*	*	*	**
4	7.25	0.428	0.428	15	~	*	*	*	*	*	**
4	7.75	0.457	0.457	15	~	*	*	*	*	*	**
4	8.25	0.486	0.486	15	~	*	*	*	*	*	**
4	8.75	0.515	0.515	15	~	*	*	*	*	*	**
4	9.25	0.544	0.544	15	~	*	*	*	*	*	**
4	9.75	0.573	0.573	15	~	*	*	*	*	*	**
5	10.25	0.602	0.595	14	6.55	1.333	20.5	0.224	0.990	0.567	0.39
5	10.75	0.632	0.608	14	6.55	1.333	20.5	0.224	0.988	0.580	0.38
5	11.25	0.661	0.622	14	6.55	1.333	20.5	0.224	0.986	0.592	0.37
5	11.75	0.691	0.636	14	6.55	1.333	20.5	0.224	0.984	0.604	0.37
5	12.25	0.720	0.650	14	6.55	1.333	20.5	0.224	0.982	0.615	0.36
5	12.75	0.749	0.664	14	6.55	1.333	20.5	0.224	0.980	0.626	0.35
5	13.25	0.779	0.677	14	6.55	1.333	20.5	0.224	0.978	0.636	0.35
5	13.75	0.808	0.691	14	6.55	1.333	20.5	0.224	0.976	0.645	0.34
5	14.25	0.837	0.705	14	6.55	1.333	20.5	0.224	0.974	0.654	0.34
5	14.75	0.867	0.719	14	6.55	1.333	20.5	0.224	0.972	0.663	0.33
6	15.25	0.895	0.731	38	~	~	~	~	~	~	~~
6	15.75	0.922	0.742	38	~	~	~	~	~	~	~~
6	16.25	0.949	0.754	38	~	~	~	~	~	~	~~
6	16.75	0.975	0.765	38	~	~	~	~	~	~	~~
6	17.25	1.002	0.776	38	~	~	~	~	~	~	~~
6	17.75	1.029	0.787	38	~	~	~	~	~	~	~~
6	18.25	1.056	0.799	38	~	~	~	~	~	~	~~
6	18.75	1.083	0.810	38	~	~	~	~	~	~	~~
6	19.25	1.110	0.821	38	~	~	~	~	~	~	~~
6	19.75	1.136	0.832	38	~	~	~	~	~	~	~~
6	20.25	1.163	0.843	38	~	~	~	~	~	~	~~
6	20.75	1.190	0.855	38	~	~	~	~	~	~	~~
6	21.25	1.217	0.866	38	~	~	~	~	~	~	~~

NCEER [1997] Method

LIQUEFACTION ANALYSIS SUMMARY

File Name: LIQTEST.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	1.244	0.877	38	~	~	~	~	~	~	~~
6	22.25	1.271	0.888	38	~	~	~	~	~	~	~~
6	22.75	1.297	0.900	38	~	~	~	~	~	~	~~
6	23.25	1.324	0.911	38	~	~	~	~	~	~	~~

6	23.75	1.351	0.922	38	~	~	~	~	~	~	~
6	24.25	1.378	0.933	38	~	~	~	~	~	~	~
6	24.75	1.405	0.944	38	~	~	~	~	~	~	~
7	25.25	1.432	0.956	14	6.25	0.858	18.1	0.196	0.930	0.787	0.25
7	25.75	1.460	0.969	14	6.25	0.858	18.1	0.196	0.927	0.791	0.25
7	26.25	1.488	0.981	14	6.25	0.858	18.1	0.196	0.925	0.794	0.24
7	26.75	1.516	0.994	14	6.25	0.858	18.1	0.196	0.923	0.797	0.24
7	27.25	1.544	1.006	14	6.25	0.858	18.1	0.196	0.921	0.800	0.24
7	27.75	1.572	1.019	14	6.25	0.858	18.1	0.196	0.919	0.802	0.24
7	28.25	1.601	1.031	14	6.25	0.858	18.1	0.196	0.917	0.805	0.24
7	28.75	1.629	1.044	14	6.25	0.858	18.1	0.196	0.915	0.807	0.24
7	29.25	1.657	1.056	14	6.25	0.858	18.1	0.196	0.913	0.810	0.24
7	29.75	1.685	1.069	14	6.25	0.858	18.1	0.196	0.911	0.812	0.24
8	30.25	1.714	1.082	40	~	~	~	~	~	~	~
8	30.75	1.744	1.096	40	~	~	~	~	~	~	~
8	31.25	1.773	1.110	40	~	~	~	~	~	~	~
8	31.75	1.803	1.125	40	~	~	~	~	~	~	~
8	32.25	1.833	1.139	40	~	~	~	~	~	~	~
8	32.75	1.863	1.153	40	~	~	~	~	~	~	~
8	33.25	1.892	1.167	40	~	~	~	~	~	~	~
8	33.75	1.922	1.181	40	~	~	~	~	~	~	~
8	34.25	1.952	1.196	40	~	~	~	~	~	~	~
8	34.75	1.982	1.210	40	~	~	~	~	~	~	~
8	35.25	2.012	1.224	40	~	~	~	~	~	~	~
8	35.75	2.041	1.238	40	~	~	~	~	~	~	~
8	36.25	2.071	1.252	40	~	~	~	~	~	~	~
8	36.75	2.101	1.267	40	~	~	~	~	~	~	~
8	37.25	2.131	1.281	40	~	~	~	~	~	~	~
8	37.75	2.161	1.295	40	~	~	~	~	~	~	~
8	38.25	2.190	1.309	40	~	~	~	~	~	~	~
8	38.75	2.220	1.323	40	~	~	~	~	~	~	~
8	39.25	2.250	1.338	40	~	~	~	~	~	~	~
8	39.75	2.280	1.352	40	~	~	~	~	~	~	~
9	40.25	2.311	1.367	17	6.38	0.750	19.1	0.199	0.866	0.828	0.24
9	40.75	2.342	1.383	17	6.38	0.750	19.1	0.199	0.864	0.828	0.24
9	41.25	2.374	1.399	17	6.38	0.750	19.1	0.199	0.862	0.827	0.24
9	41.75	2.406	1.416	17	6.38	0.750	19.1	0.199	0.860	0.826	0.24
9	42.25	2.438	1.432	17	6.38	0.750	19.1	0.199	0.858	0.826	0.24
9	42.75	2.470	1.448	17	6.38	0.750	19.1	0.199	0.855	0.825	0.24
9	43.25	2.501	1.464	17	6.38	0.750	19.1	0.199	0.853	0.824	0.24

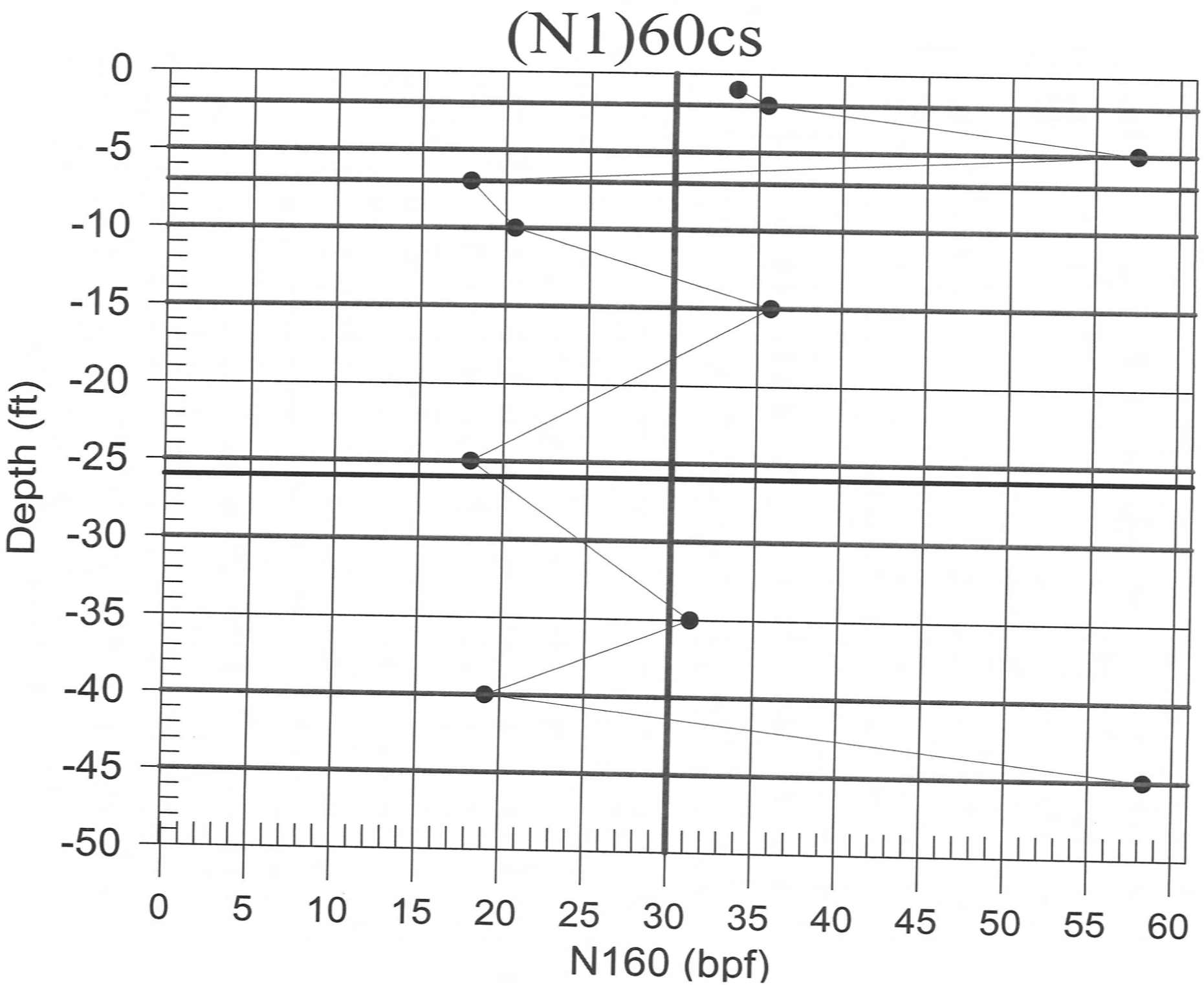
NCEER [1997] Method

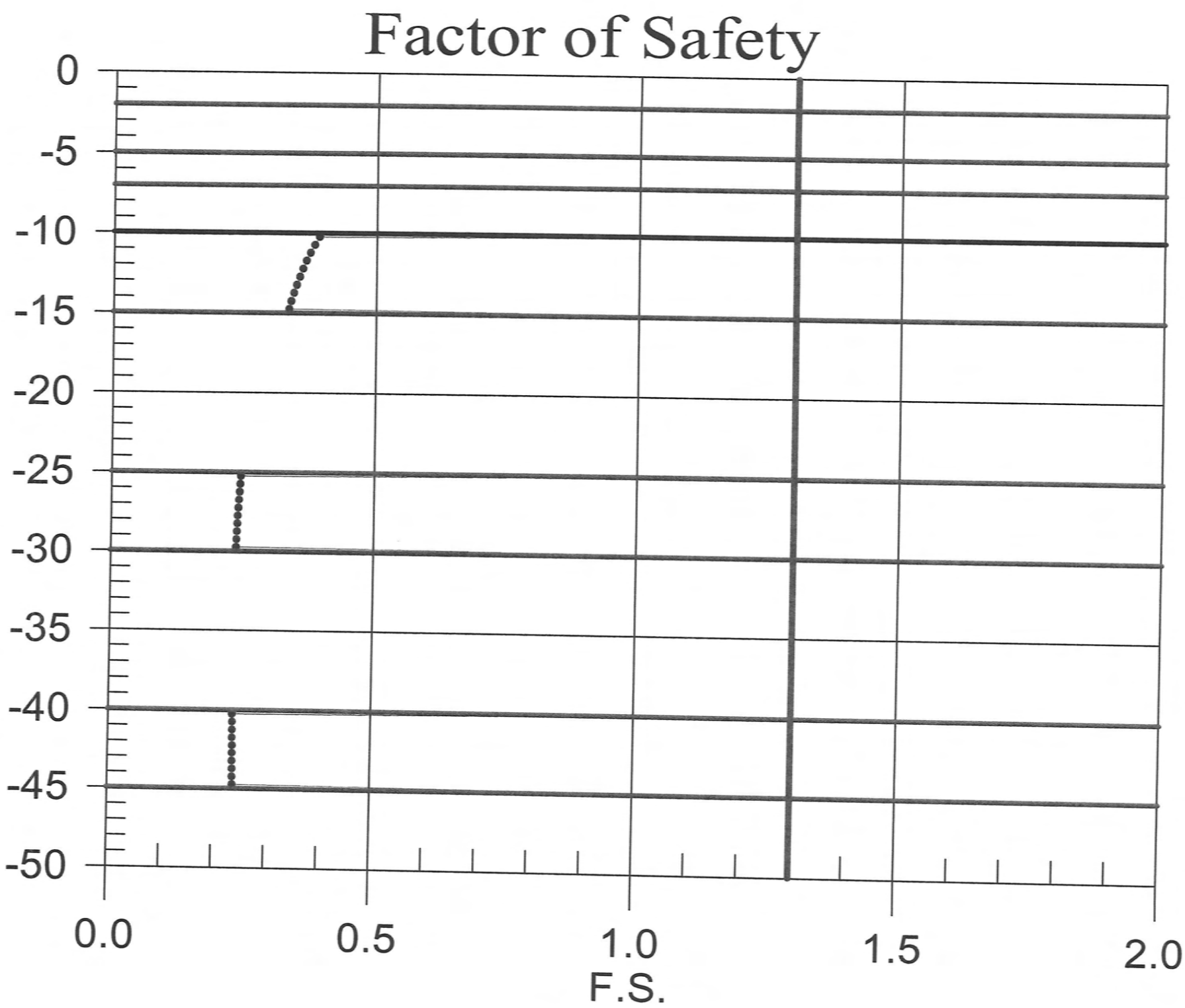
LIQUEFACTION ANALYSIS SUMMARY

File Name: LIQTEST.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
9	43.75	2.533	1.480	17	6.38	0.750	19.1	0.199	0.851	0.824	0.24
9	44.25	2.565	1.497	17	6.38	0.750	19.1	0.199	0.849	0.823	0.24
9	44.75	2.597	1.513	17	6.38	0.750	19.1	0.199	0.847	0.822	0.24
10	45.25	2.627	1.527	81	~	~	~	~	~	~	~
10	45.75	2.655	1.539	81	~	~	~	~	~	~	~
10	46.25	2.682	1.551	81	~	~	~	~	~	~	~
10	46.75	2.710	1.564	81	~	~	~	~	~	~	~
10	47.25	2.738	1.576	81	~	~	~	~	~	~	~
10	47.75	2.766	1.588	81	~	~	~	~	~	~	~
10	48.25	2.794	1.600	81	~	~	~	~	~	~	~

10	48.75	2.822	1.613	81						
10	49.25	2.850	1.625	81						
10	49.75	2.877	1.637	81						


(N1)60cs
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
-50
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
N160 (bpf)


Factor of Safety
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
-50
0.0
0.5
1.0
1.5
2.0
F.S.

APPENDIX E

Infiltration/Percolation Testing

DOUBLE RING INFILTROMETER TEST DATA

Project:	111 E. Mill Street	Test Date:	February 24, 2016
Project No.:	63243.1	Test Hole No.:	DRI-1
Soil Classification:	(SM) Silty Sand	Test Hole Diameter:	12-in inner 24-in annular
Depth of Test Hole:	5.5 ft	Date Excavated:	February 24, 2016
Liquid Used:	Tap Water	pH:	7.8
Area of Rings:	Inner = 0.785 ft^2 Annular = 2.36 ft^2	Depth of Water in Rings:	1"
Tested By:	RL	Ring Penetration:	3.5"
Liquid Level Maintained Using:	Vacuum Seal		
Depth to Water Table:	50+ ft.		

TEST PERIOD

TRIAL NO.	INNER TIME		TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	WATER USED (lbs.) inner	WATER USED (lbs.) annular space	WATER USED (gal) inner	WATER USED (gal) annular space	INFILTRATION RATE (gal/sf./day) inner	INFILTRATION RATE (gal/sf./day) annular space	INFILTRATION RATE (in/hr) inner	INFILTRATION RATE (in/hr) annular space	REMARKS
1	S	9:19	15	15	2.47	16.73	0.30	2.01	36.3	81.7	2.4	5.5	Refilled Outer Bottle
	E	9:34											
2	S	9:36	15	30	2.61	8.83	0.31	1.06	38.3	43.1	2.6	2.9	
	E	9:51											
3	S	9:51	15	45	3.27	13.68	0.39	1.64	48.0	66.8	3.2	4.5	
	E	10:06											
4	S	10:09	15	60	3.41	6.87	0.41	0.82	50.0	33.6	3.4	2.2	Refilled Outer Bottle
	E	10:26											
5	S	10:26	30	90	3.75	16.53	0.45	1.98	27.5	40.4	1.8	2.7	
	E	10:56											

TEST PERIOD													
TRIAL NO.	INNER				WATER USED (lbs.)		WATER USED (gal)		INFILTRATION RATE (gal/sf./day)		INFILTRATION RATE (in/hr)		REMARKS
	TIME		TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	inner	annular space	inner	annular space	inner	annular space	inner	annular space	
6	S	10:59	30	120	.33	16.01	0.04	1.92	2.4	39.1	0.2	2.6	
	E	11:29											
7	S	11:29	30	150	.62	10.71	0.07	0.29	4.6	26.2	0.3	1.8	Refilled Outer Bottle
	E	11:59											
8	S	12:02	30	180	3.56	9.5	0.43	1.14	26.1	23.2	1.8	1.6	
	E	12:32											
9	S	12:35	30	210	3.14	9.85	0.38	1.18	23.1	24.1	1.5	1.6	Refilled Outer Bottle
	E	1:05											
10	S	1:05	30	240	2.87	13.27	0.34	1.59	21.1	32.4	1.4	2.2	Refilled Outer Bottle
	E	1:35											
11	S	1:36	30	270	2.54	16.49	0.31	1.98	18.6	40.3	1.2	2.7	
	E	2:06											
12	S	2:06	30	300	1.30	15.47	0.16	1.86	9.5	37.8	0.6	2.5	
	E	2:36											
13	S	2:36	30	330	1.5	14.26	0.18	1.71	11.0	34.8	0.7	2.3	
	E	3:06											

DOUBLE RING INFILTROMETER TEST DATA

Project:	111 E. Mill Street	Test Date:	February 24, 2016
Project No.:	63243.1	Test Hole No.:	DRI-2
Soil Classification:	(SM) Silty Sand	Test Hole Diameter:	12-in inner 24-in annular
Depth of Test Hole:	5.5 ft	Date Excavated:	February 24, 2016
Liquid Used:	Tap Water	pH:	7.8
Area of Rings:	Inner = 0.785 ft^2 Annular = 2.36 ft^2	Depth of Water in Rings:	1"
Tested By:	RL	Ring Penetration:	3.5"
Liquid Level Maintained Using:	Vacuum Seal		
Depth to Water Table:	50+ ft.		

TEST PERIOD

TRIAL NO.	INNER TIME		TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	WATER USED (lbs.) inner	WATER USED (lbs.) annular space	WATER USED (gal) inner	WATER USED (gal) annular space	INFILTRATION RATE (gal/sf./day) inner	INFILTRATION RATE (gal/sf./day) annular space	INFILTRATION RATE (in/hr) inner	INFILTRATION RATE (in/hr) annular space	REMARKS
1	S	9:48	15	15	3.21	9.99	0.39	1.20	47.1	48.9	3.2	3.3	
	E	10:03											
2	S	10:03	15	30	1.77	3.12	0.21	0.37	25.9	15.2	1.7	1.0	
	E	10:18											
3	S	10:18	15	45	.92	3.24	0.11	0.39	13.5	15.8	0.9	1.1	
	E	10:33											
4	S	10:33	15	60	1.09	2.34	0.13	0.28	16.0	11.4	1.1	0.8	
	E	10:48											
5	S	10:48	30	90	1.6	0.14	0.19	0.74	11.7	15.0	0.8	9.0	
	E	11:13											

TEST PERIOD													
TRIAL NO.	INNER				WATER USED (lbs.)		WATER USED (gal)		INFILTRATION RATE (gal/sf./day)		INFILTRATION RATE (in/hr)		REMARKS
	TIME		TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	inner	annular space	inner	annular space	inner	annular space	inner	annular space	
6	S	11:20	30	120	3.15	4.19	0.38	0.50	23.1	10.2	1.5	0.7	
	E	11:50											
7	S	11:50	40	160	2.57	7.94	0.31	0.95	14.1	14.5	0.9	1.0	
	E	12:30											
8	S	12:30	30	190	3.21	11.97	0.39	1.44	23.6	29.2	1.6	2.0	
	E	1:00											
9	S	1:00	30	220	2.14	4.59	0.26	0.55	15.7	11.2	1.1	1.8	
	E	1:30											
10	S	1:30	30	250	1.77	4.71	0.21	0.57	13.0	11.5	0.9	0.8	
	E	2:00											
11	S	2:00	30	280	1.52	6.04	0.18	0.73	11.2	14.8	0.8	1.0	
	E	2:30											
12	S	2:30	30	310	1.85	6.07	0.22	0.73	13.6	14.8	0.9	1.0	
	E	3:00											

MODIFIED BOTTLE INFILTROMETER TEST DATA

Project:	111 E. Mill Street	Test Date:	March 1, 2016
Project No.:	63243.1	Test Hole No.:	P-1
Soil Classification:	(SW) Well Graded Sand	Test Hole Diameter:	8"
Depth of Test Hole:	9 ft.	Date Excavated:	March 1, 2016
Tested By:	DAW		

TEST PERIOD

TRIAL NO.		TIME	TIME INTERVAL (minutes)	TOTAL ELAPSED TIME (minutes)	WATER USED (lbs.)	WATER USED (gal.)	INFILTRATION RATE (gal/sf/day)	INFILTRATION RATE (in/hr)	REMARKS
1	S	8:57	20	20	9.43	1.13	65.3	4.4	
	E	9:17							
2	S	9:17	10	30	3.35	0.40	46.4	3.1	Refilled
	E	9:27							
3	S	9:53	10	40	5.09	0.61	70.5	4.7	
	E	10:03							
4	S	10:03	10	50	4.77	0.57	66.0	4.4	
	E	10:13							
5	S	10:13	3	53	4.69	0.56	216.4	14.5	Refilled
	E	10:16							

TEST PERIOD									
TRIAL NO.	TIME		TIME INTERVAL (minutes)	TOTAL ELAPSED TIME (minutes)	WATER USED (lbs.)	WATER USED (gal.)	INFILTRATION RATE (gal/sf/day)	INFILTRATION RATE (in/hr)	REMARKS
6	S	10:40	10	63	4.75	0.57	65.8	4.4	
	E	10:50							
7	S	10:50	10	73	6.11	0.73	84.6	5.7	
	E	11:00							
8	S	11:00	10	83	8.96	1.08	124.0	8.3	Refilled
	E	11:10							
9	S	11:23	20	103	6.14	0.74	42.5	2.9	
	E	11:43							
10	S	11:43	20	123	6.05	0.73	41.9	2.8	
	E	12:03							
11	S	12:26	10	133	6.86	0.82	95.0	6.4	
	E	12:36							
12	S	12:36	10	143	3.49	0.42	48.3	3.2	Refilled
	E	12:46							
13	S	12:50	10	153	6.75	0.81	93.4	6.3	
	E	1:00							
14	S	1:30	10	163	5.56	0.67	77.0	5.2	
	E	1:40							

TEST PERIOD									
TRIAL NO.	TIME		TIME INTERVAL (minutes)	TOTAL ELAPSED TIME (minutes)	WATER USED (lbs.)	WATER USED (gal.)	INFILTRATION RATE (gal/sf/day)	INFILTRATION RATE (in/hr)	REMARKS
15	S	1:40	10	173	3.72	0.43	51.5	3.5	
	E	1:50							
16	S	1:50	10	183	3.74	0.45	51.8	3.5	
	E	2:00							

MODIFIED BOTTLE INFILTROMETER TEST DATA

Project:	111 E. Mill Street	Test Date:	March 1, 2016
Project No.:	63243.1	Test Hole No.:	P-2
Soil Classification:	(SP) Poorly Graded Sand	Test Hole Diameter:	8"
Depth of Test Hole:	9'6"	Date Excavated:	March 1, 2016
Tested By:	DAW		

TEST PERIOD

TRIAL NO.		TIME	TIME INTERVAL (minutes)	TOTAL ELAPSED TIME (minutes)	WATER USED (lbs.)	WATER USED (gal.)	INFILTRATION RATE (gal/sf/day)	INFILTRATION RATE (in/hr)	REMARKS
1	S	9:48	10	10	9.61	1.15	133.0	8.9	
	E	9:58							
2	S	9:58	5	15	6.32	0.76	175.0	11.7	Refilled
	E	10:03							
3	S	10:28	10	25	6.04	0.73	83.6	5.6	
	E	10:38							
4	S	10:38	10	35	7.95	0.95	110.1	7.4	Refilled
	E	10:48							
5	S	11:15	10	45	6.76	0.81	93.6	6.3	
	E	11:25							
6	S	11:25	10	55	5.05	0.61	69.9	4.7	Refilled
	E	11:35							

TEST PERIOD									
TRIAL NO.	TIME		TIME INTERVAL (minutes)	TOTAL ELAPSED TIME (minutes)	WATER USED (lbs.)	WATER USED (gal.)	INFILTRATION RATE (gal/sf/day)	INFILTRATION RATE (in/hr)	REMARKS
7	S	12:06	10	65	7.05	0.85	87.6	6.5	Refilled
	E	12:16							
8	S	1:15	10	75	4.11	0.49	56.9	3.8	
	E	1:25							
9	S	1:25	10	85	4.49	0.54	62.2	4.2	
	E	1:35							
10	S	1:35	10	95	3.92	0.47	54.3	3.6	
	E	1:45							
11	S	2:05	10	105	4.02	0.48	55.6	3.7	
	E	2:15							
12	S	2:15	10	115	3.98	0.47	53.9	3.6	
	E	2:25							

APPENDIX F

Settlement Calculations

Estimation of Settlement in Sands Dues to Earthquake Shaking

Project Name: 111 East Mill Street
Project No.: 63243.1
Client: Burrtec Waste Industries, Inc.

Boring: B-1

PGA: 0.87
Magnitude: 7.5
GW Depth (ft): 10

Settlement of Dry Sands

Layer No.	Soil Layer	From (ft)	To (ft)	Thickness (ft)	Zcenter (ft)	γd (pcf)	ω	γ (pcf)	σ (psf)	σ'm (psf)	Dr (%)	(N1)60	K2	Gmax (psf)	Zcenter (m)	rd	γeff(Geff/Gmax)	γeff	εcm 7.5	εcm 7.1	εcm	Settlement (in)
1	Asphalt	0	0.16	0.16	0.08	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2	Base	0.16	0.32	0.16	0.24	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
3	SM	0.32	2	1.68	1.16	111.8	3.2%	115.4	194	128	63%	17	132.2	1,495,604	0.4	1.00	7.3E-05	1.1E-04	0.02%	0.02%	0.04%	0.01
4	CL	2	5	3	3.5	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
5	SM	5	7	2	6	112.7	3.5%	116.6	1385	914	84%	33	205.8	6,221,133	1.8	0.98	1.2E-04	1.9E-04	0.01%	0.01%	0.02%	0.00
6	SW	7	10	3	8.5	102.4	13.5%	116.2	1734	1144	59%	15	121.6	4,114,777	2.6	0.97	2.3E-04	4.9E-04	0.08%	0.08%	0.16%	0.06
																				Settlement =		0.07

Settlement of Saturated Sands

Layer No.	Soil Layer	From (ft)	To (ft)	Thickness (ft)	Zcenter (ft)	γd (pcf)	ω	γ (pcf)	σ (psf)	σ' (psf)	Dr (%)	(N1)60	Zcenter (m)	rd	(τ/σ')	rm	ε	Settlement (in)
7	SM	10	15	5	12.5	107.1	9.7%	117.5	2321	2009	56%	14	3.8	0.96	0.63	1.012	1.9%	1.14
8	SW	15	25	10	20	105.0	2.3%	107.3	3394	2458	89%	38	6.1	0.94	0.73	1.012	0.1%	0.12
9	SM	25	30	5	27.5	100.6	11.6%	112.3	3956	2708	56%	14	8.4	0.92	0.76	1.012	2.0%	1.20
10	SW	30	40	10	35	115.6	3.1%	119.2	5147	3275	90%	40	10.7	0.89	0.79	1.012	0.0%	0.01
11	SM	40	45	5	42.5	116.3	9.4%	127.2	5784	3600	63%	17	13.0	0.87	0.79	1.130	1.8%	1.08
12	SW	45	50	5	47.5	109.1	2.1%	111.4	6341	3845	99%	81	14.5	0.86	0.80	0.000	0.0%	0.00
																	Settlement =	3.55

Total Settlement =	3.62 in