PRELIMINARY GEOTECHNICAL INVESTIGATION AND PERCOLATION TESTS, PROPOSED RESIDENTIAL AND COMMERCIAL DEVELOPMENTS, 1640 MONROVIA AVENUE, CITY OF COSTA MESA, CALIFORNIA

Prepared for:

NEXUS COMPANIES

1 MacArthur Place, Suite 300
Santa Ana, California 92707

Project No. 012184-001

July 16, 2007

Leighton and Associates, Inc.

A LEIGHTON GROUP COMPANY

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

July 16, 2007

Project No. 012184-001

To: Nexus Companies
1 MacArthur Place, Suite 300
Santa Ana, California 92707

Attention: Mr. Douglas A. Burroughs, Manager of Preconstruction

Subject: Preliminary Geotechnical Investigation and Percolation Tests, Proposed Residential and Commercial Developments, 1640 Monrovia Avenue, City of Costa Mesa, California

In response to your request, Leighton and Associates, Inc. (Leighton) has performed a preliminary geotechnical investigation and percolation tests for the proposed residential and commercial developments located at 1640 Monrovia Avenue, Costa Mesa, California.

Based on the information provided by you, we understand that the proposed development will include construction of six (6) one- to four-story commercial buildings, four-story residential condominiums consisting of 156 units, one (1) 4.5-level parking structure, five (5) three-story live/work/loft lots, at-grade parking lots and driveways, recreation areas, and other associated facilities. All the structures are planned to be on-grade.

Based upon the results of this study, the proposed project is considered to be feasible from a geotechnical standpoint. The proposed buildings may be supported by conventional spread footing foundations with a slab-on-grade. Specific recommendations for the geotechnical aspects of the project are presented in this report.

17781 Cowan ▪ Irvine, CA 92614-6009
949.250.1421 ▪ Fax 949.250.1114 ▪ www.leightongeo.com

We appreciate the opportunity to work with you on this project. If you have any questions, or if we can be of further service, please call us at your convenience.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

REGISTERED PROFESSIONAL ENGINEER
ZAFAR AHMED
No. 2720
EXP. 06/30/09
GEOTECHNICAL
STATE OF CALIFORNIA

Zafar Ahmed, GE 2720
Senior Project Engineer

REGISTERED PROFESSIONAL GEOLOGIST
EDWARD LOUIS BURROWS
No. 1750
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Edward L. Burrows, CEG 1750
Principal Geologist

ZA/ELB/lr

Distribution: (4) Addressee

TABLE OF CONTENTS

Leighton

TABLE OF CONTENTS (Continued)

Leighton

1.0 INTRODUCTION

1.1 Purpose and Scope

The purpose of our geotechnical investigation was to explore the subsurface conditions with respect to the proposed improvements, to evaluate the geologic hazards of the site, and to provide geotechnical recommendations for design and construction of the proposed residential and commercial developments. The scope of work of our current study included the following tasks:

- Perform site reconnaissance to locate and mark the exploratory boring locations.
- Review of available site-specific information, including review of various publications, and documents (see Appendix A, References).
- Perform logging and sampling of nine (9) hollow-stem auger borings to depths of approximately 16.5 to 51.5 feet below the existing grade.
- After completion of logging and sampling, perform percolation tests at three (3) of the above boreholes to depths of approximately 16.5 feet below the existing grade.
- Collect representative soil samples at selected depth intervals and transport to our laboratory for testing.
- Conduct relevant geotechnical engineering laboratory tests on select representative samples to characterize the engineering properties of the soils.
- Perform geotechnical evaluation of collected data and relevant engineering analyses.
- Prepare this report summarizing our findings, conclusions, and recommendations.

1.2 Site Location and Proposed Development

The site is located at 1640 Monrovia Avenue in the City of Costa Mesa, California, bounded by Monrovia Avenue on the east and Babcock Street on the West as shown on Figure 1, *Site Location Map*. The site encompasses an area of approximately 6.7 acres and is currently occupied by three one-story building structures and paved parking lots and driveways. Boundaries of the existing buildings are shown in Plate 1, Boring

Location Map. The site is surrounded by commercial buildings all around and a trailer park along the northwestern boundary.

Based on the conceptual site plan (Architects Orange, 2007) provided by you, we understand that the proposed developments will include construction of the six (6) one- to four-story commercial buildings, four-story residential condominiums consisting of 156 units, one (1) 4.5-level parking structure, five (5) three-story live/work/loft lots, at-grade parking lots and driveways, recreation areas, and other associated facilities.

The above buildings (see Plate 1) are anticipated to be steel, tilt-up concrete, cast-in-place concrete and parking structures. All the buildings will be at-grade. Accordingly, no major site grading is expected for this development. No grading plans, structural plans or foundation plans are available at the time of this report.

1.3 Field Investigation

Prior to the field investigation, we performed a site reconnaissance to determine access issues for heavy equipment, access to water sources to conduct the proposed percolation tests, and to mark the proposed boring locations. Underground Surface Alert (USA) was then notified of the marked locations for utility clearance.

On May 23, 2007, the subsurface investigation was performed using hollow-stem auger boring techniques. Nine borings were advanced to depths ranging from approximately 16.5 to 51.5 feet below the current grade at an average elevation of approximately 110 feet above the mean sea level (msl). The drilling rig used for the exploration consisted of a truck-mounted CME-75 hollow-stem auger rig equipped with 8-inch-diameter augers. The approximate boring locations are shown on Plate 1, *Boring Location Map.*

The hollow-stem borings were logged by a staff geologist from our office. Relatively undisturbed soil samples were obtained at selected intervals within the borings using a California Ring sampler. Standard Penetration Tests (SPT) were also conducted at selected intervals within the borings. Bulk samples of representative soil types were also collected. Upon completion of drilling, four of the above nine borings were backfilled with soil cuttings and surfaces patched with cold mixed asphalt on the same day of drilling. Borings B-1 and B-6 were backfilled at first with 32 to 34 feet of Bentonite chips, then subsequently with soil cuttings add patched with cold mixed asphalt. Three borings (B-2, B-3 and B-7) were left open for percolation tests to be conducted on May

30, 2007. Excess soil was generated from Borings B-1 through B-3, B-6, and B-7 due to Bentonite backfill and dry well construction for percolation tests. The excess soil was drummed and properly labeled for disposal by others. A brief description of percolation test procedures is discussed below.

1.4 Field Percolation Tests

The percolation tests were performed on the subject site by a representative of Leighton. On May 29, 2007, a 4-inch-diameter perforated PVC pipe was installed in each of the three pre-drilled test borings (B-2, B-3 and B-7; see Plate 1), excavated to approximate depths of 16.5 feet below the existing grade, and the space between the test hole and the pipe was backfilled with ¾ inch gravel. The pipes were then filled with water to saturate the soils and left for pre-soaking overnight. After the presaturation, the field tests were conducted on May 30, 2007. During the tests the pipes were filled with water to the top and, and drop of water level was measured at intervals of one hour. The pipes were refilled to the top after each measurements and the procedure was repeated until the difference in water level at the end of the two consecutive intervals varied less than ten percent. After completion of the percolation tests, the test holes were backfilled with the native soil cuttings and surface patched with cold asphalt mix. Percolation test results are briefly summarized in Section 3.9, *Field Percolation Test Results*.

1.5 Laboratory Testing

Laboratory tests were performed on selected soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate the physical and engineering characteristics of the onsite soil. Tests performed during this investigation include:

- In-situ moisture and density;
- Percent finer than No. 200 Sieve;
- Atterberg Limits;
- Expansion Index;
- Maximum density and optimum moisture content (Modified Proctor);
- Direct shear;
- One dimensional consolidation;

- R-value; and
- Corrosion suite (pH, resistivity, chloride, and sulfate contents).

All laboratory tests were performed in general conformance with the ASTM or State of California Standard Methods. The results of the in-situ moisture and density tests are presented in Appendix B, *Boring Logs*. The results of other laboratory tests are presented in Appendix C of this report.

2.0 GEOLOGY AND SEISMICITY

2.1 Regional Geology

The project site is located within the west portion of the Newport Mesa, which is intermediate in elevation between the San Joaquin Hills (approximately 5 miles to the east) and the lower flood plain of the Santa Ana River, west of the site. The Mesa is capped by nearly horizontal alluvial and terrace deposits (ancient near shore marine and terrestrial deposits), which are underlain by sediments of the Tertiary-aged Monterey Formation.

The Newport Inglewood fault zone, which forms an important element of the regional geologic structure, results in the broad up-arching and disruption of the subsurface formations, extending as a southeast trending band from south-central Los Angeles Basin through Signal Hill in the Long Beach area, to the Huntington Beach and Newport-Costa Mesa area, then trends offshore.

2.2 Subsurface Soil and Groundwater Conditions

Subsurface soils underneath the existing pavements, as encountered during our investigation, consist of artificial fills up to a maximum depth of 3 feet which is underlain by Quaternary alluvium (Qal) up to maximum explored depths of 51.5 feet below the existing ground surface. Alluvial deposits consist of light grey to red-brown color, moist, primarily silty and clayey soils of stiff to very stiff consistency, and medium dense sandy soils with relatively high fine contents within upper approximately 20 feet. Alluvium below 20 feet became increasingly more granular and dense with depth. Detailed descriptions of the encountered subsurface materials are provided in the boring logs included in Appendix B.

Expansive Soil – Onsite soils within the upper 5 feet consist of primarily clayey sand to sandy clay soils. Laboratory tests of two selective samples indicated very low expansion potential (per UBC, 1997) with tested Expansion Index (EI) values ranging from 5 to 19 (Appendix C). However, based on our field observations during exploration and laboratory test results, near surface soils are anticipated to contain very low to low expansion potential (per UBC, 1997). Upon completion of rough grading, finish pad subgrade soils samples should be tested to determine actual potential for expansion.

Groundwater Conditions - Groundwater was encountered in two of our borings, B-1 and B-6 (see Figure 2), at depths approximately 45 feet below the current ground surface corresponding to an approximate elevation of 65 feet above msl. Based on the seismic hazard zone report for the Anaheim and Newport Beach 7.5 minute Quadrangles, the historically shallowest groundwater depths in the vicinity of the site are on the order of 30 feet below existing ground surface. Due to the depth to groundwater at the site, groundwater is not anticipated to be a constraint for design and construction of the proposed developments.

2.3 Faulting

Our review of available in-house literature indicates that no known active faults have been mapped across the site, and the site is not located within an Alquist-Priolo Earthquake Fault Zone (Hart and Bryant, 1999).

The nearest know active fault to the site is the L.A. basin segment of Newport-Inglewood faults, which is located approximately 1.4 kilometers from the site. The known regional active faults that could produce the most significant ground shaking at the site include the Newport-Inglewood (L.A. Basin and Offshore segments) and Palos Verdes faults that are within 20 kilometer radius from the site and are classified as seismic source type B (according to UBC/CBC). The nearest Type A seismic source is the Cucamonga fault which is approximately 58 kilometers away from site and has little effect to this site's seismicity.

2.4 Seismic Hazards Evaluation

Seismic hazards for sites in the region could include surface ground rupture, soil liquefaction, seismically-induced settlement, lateral spreading; earthquake induced flooding (tsunamis and seiches). The potential for these seismic hazards at the site is discussed below.

2.4.1 Surface Ground Rupture

Surface ground rupture is generally considered most likely to occur along pre-existing active faults. There are no active or potentially active faults mapped or known to traverse the subject site. The subject site does not lie within an earthquake fault zone as created by the Alquist-Priolo Earthquake Zoning Act (Hart and Bryant, 1999). Based on no active or potentially active faults mapped or known to cross the subject site, the potential for surface ground rupture at the subject site is considered low.

2.4.2 Liquefaction

Due to relatively shallow historic high groundwater level, the site is mapped within an area showing liquefaction opportunity in the Seismic Hazard Zones Map for the Anaheim and Newport Beach Quadrangles, (DMG, 1997). The historic high groundwater level for this site is at approximately 30 feet below the ground surface (DMG, 1997). Groundwater was encountered at depths approximately 45 feet below the existing grade during our exploration. However, due to clayey soil layers, silts and sands with relatively high fine contents above the anticipated groundwater level, and dense sandy soils below, the liquefaction potential at the site is considered low.

2.4.3 Seismically-Induced Settlement

Seismically-induced settlement is due to densification of loose granular soil during or shortly after ground shaking. Due to predominately dense nature of the soils encountered at the site, and anticipated moderate level of peak ground acceleration (see Section 3.1), seismically-induced settlement at the site is considered insignificant.

2.4.4 Lateral Spreading

Liquefaction may also cause lateral spreading. For lateral spreading to occur, the liquefiable zone must be continuous, unconstrained laterally, and free to move along gently sloping ground toward an unconfined area. Since the potential for liquefaction at the site is low, the potential for lateral spread at the site is also considered low.

2.4.5 Seismically-Induced Landslides

Since the site is relatively flat and is not located within a zone of potential seismic landslides based on the CDMG Seismic Hazard Zone Map for the Anaheim and Newport Beach Quadrangles (DMG, 1997), therefore seismically-induced landslides is not considered a hazard for this site.

2.4.6 Earthquake-Induced Flooding

Earthquake-induced flooding can result from the failure of dams or other water-retaining structures resulting from earthquakes. Due to the absence of such structures near the site, the potential for earthquake-induced flooding of the site is considered low.

2.4.7 Seiches and Tsunamis

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Based on the absence of retained bodies of water near the site, its inland location, and elevation seiches and tsunamis are not considered to be a hazard for this site.

2.5 Flood Hazard

Based on the Flood Insurance Rate Maps, prepared by The Federal Emergency Management Agency (FEMA, 2004), the site is not located within a flood hazard zone. The impact of potential flooding on the site should be addressed by the civil engineer.

3.0 RECOMMENDATIONS

The conclusions and recommendations have been developed based on the exhibited engineering properties of the soils and their anticipated behavior both during and after construction. Recommendations are specifically provided for design of foundations and at-grade floor slabs, retaining walls, concrete flatwork, and pavement. The geotechnical engineer should review the grading plan, foundation plans, and specifications when they are available to verify that the recommendations presented in this report have been properly interpreted and incorporated.

Based upon this study, we conclude that the proposed development is feasible from a geotechnical standpoint, provided that the recommendations presented in this report are properly incorporated in the design and construction of the project.

3.1 Seismic Design Parameters

This site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone. However, strong ground shaking due to seismic activity is anticipated at the site. The following seismic design values in Table 1 are based on the International Building Code (IBC), 2006 guidelines and should be considered as the minimum for the seismic analysis of the subject site. Additional seismic analyses may be necessary based on structural requirements.

Table 1 - IBC Site Categorization and Site Coefficients

Categorization/Coefficient	Design Value
Site Class	D
Short Period (0.2 sec) Site Coefficient, F_a	1.0
Long Period (1.0 sec) Site Coefficient, F_v	1.5
Design (5% damped) spectral response accl. parameter at short period, S_{DS}	1.2g
Design (5% damped) spectral response accl. parameter at a period of 1 sec, S_{D1}	0.67g

3.2 Site Grading

The recommendations for earthwork and site preparation are based upon the assumption of minor cuts and fills, typically less than 1 to 2 feet to achieve planned site grades.

Site Preparation – Prior to construction, the site should be cleared of vegetation, trash, and debris, which should be disposed of offsite. Remnants of the existing development including all foundations, slabs, pavements and other unsuitable materials should be completely removed. Efforts should be made to locate any existing or abandoned utility lines in the area. Existing utility conduits should be removed or rerouted if they interfere with the proposed construction, and the resulting cavities should be properly backfilled and compacted.

Overexcavation and Recompaction – Depending on the encountered subsurface soil conditions and anticipated structural loads, we recommend remedial overexcavation limits for the proposed developments as discussed in the following.

- Building Footprints: As of the date of this report, no information on anticipated structural loads for the buildings were available to us. For preliminary design puprpose we considered maximum structural loads as presented in Table 2 below. Remedial overexcavation limits (both vertical and lateral) for the proposed building structures for different types of foundations are shown in the following Table 2.

Table 2 – Overexcavation Limits for the Proposed Structures

Proposed Structure	Maximum Structutal Loads	Depth of Overexcavation (feet)	Lateral Limit of Overexcvation[1] (feet)
Four-Story Office Building	Column load 150 kips; wall footing load 7.5 kips/foot.	10 (spread footing)	10 (spread footing)
		5 (mat foundation)	5 (mat foundation)
One to two-story commercial buildings	Column load 100 kips; wall footing load 5 kips/foot.	5	5
4.5-level parking strucutre	Column load 200 kips; wall footing load 10 kips/foot.	10 (spread footing)	10 (spread footing)
		5 (mat foundation)	5 (mat foundation)
Four-story residential condominiums[2]	Column load 200 kips; wall footing load 7.5 kips/foot.	10 (spread footing)	10 (spread footing)
		5 (mat foundation)	5 (mat foundation)
Three-story live/work/resident ial lofts[3]	Column load 100 kips; wall footing load 5 kips/foot.	5	5

[1] Lateral limit is from the edge of outermost foundation elements.

[2] Anticipated to be cast-in-place concrete structures.

[3] Anticipated to be wood freme structures.

Since the existing building will be demolished for the construction of the new buildings, additional overexcavation to remove the remnants of existing building may be required. The actual depth and extent of overexcavation should be evaluated at the time of construction by a representative of the geotechnical engineer. In-place testing of removal bottoms should be performed during grading to determine the competency of materials being left in place. A minimum criterion of 100 pounds-per-cubic-foot (pcf) or 85 percent relative compaction (ASTM D1557) should be considered for competent removal bottoms.

- Pavement and Concrete Flatwork Areas: In pavement and concrete flatwork areas, a minimum remedial removal and recompaction of 18 inches below the existing grade or finish grade, whichever is deeper, should be performed.

Subgrade Preparation – Exposed subgrade soil surfaces, including all excavation or removal bottoms, should be observed by a representative of the geotechnical engineer prior to placement of fill or construction of other improvements to verify that suitable soil is exposed. The exposed subgrade should be scarified to a depth of 6 inches, moisture-conditioned to near optimum-moisture content and then compacted to a minimum of 90 percent of the ASTM Test Method D1557 laboratory maximum density.

Fill Placement and Compaction – The onsite soil, free of organic material, oversize particles (cobbles, boulders, rubble, etc.) greater than 6 inches in largest dimension, is suitable to be used as general fill. Import soil should be evaluated and tested by the geotechnical consultant before delivering to the site. In general, fill material should be low in expansion potential (EI less than 51), non-organic and free of debris or other deleterious materials. All fill soil should be placed in thin, loose lifts no more than 8 inches thick, moisture-conditioned as necessary to approximately 2 to 3 percent above the optimum moisture content, and compacted using appropriate equipment to minimum of 90 percent relative compaction (ASTM D1557).

3.3 Foundation Design

Proposed residential and commercial buildings and the parking structure may be supported on shallow spread footings (continuous wall and/or column) or mat foundations.

3.3.1 Spread Footings

Spread footings (continuous wall and/or column) for the proposed structures should be bearing on a zone of newly placed properly compacted fill (see Section 3.2). Preliminary design parameters for spread footings are described in the following:

Minimum Footing Dimensions and Embedment - Footings for one- to two-story buildings should be embedded at least 18-inches below the lowest adjacent grade. Footings for three- to four-story buildings should be embedded at least 24-inches below the lowest adjacent grade. Footing embedments are measured from lowest adjacent finished grade, considered as the top of interior slabs-on-grade or the finished exterior grade, excluding landscape topsoil, whichever is lower. Footings located adjacent to utility trenches or vaults should be embedded below an imaginary 1:1 (horizontal:vertical) plane projected upward and outward from the bottom edge of the trench or vault, up towards the footing. Continuous/strip footings should have a minimum width of 18-inches for one- to two-story structures and 24-inches for three- to four-story structures, while column footings for all the structures should have a minimum width of 24-inches. All footing excavations should be observed by geotechnical engineer before reinforcing steel is placed.

Allowable Vertical Bearing - For footings founded on newly placed, properly compacted fill soil, an allowable vertical bearing capacity of 2,500 pounds-per-square-foot (psf) may be used for design for a minimum embedment of 18 inches below the lowest adjacent grade. This allowable bearing pressure may be increased by 500 psf for each additional foot of embedment and/or width, to a maximum vertical bearing value of 4,500 psf. These bearing values may be increased by one-third when considering short-term seismic or wind loads.

Lateral Loads - Lateral loads may be resisted by friction between the footings and the supporting subgrade. A maximum allowable frictional resistance of 0.35 may be used for design of concrete structures poured on properly compacted fill. In addition, lateral resistance may be provided by passive pressures acting against foundations poured neat against properly compacted granular fill. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds-per-cubic-foot (pcf) be used in design. These friction and passive values have already been reduced by a factor-of-safety of 1.5.

Settlement Estimates - For settlement estimates, we assumed maximum anticipated structural loads as presented in Table 2. If greater column or wall loads are required, we should re-evaluate our foundation recommendation, and re-calculate settlement estimates. Settlement of newly placed properly compacted fill materials is expected to predominantly occur during and within 60 days following fill placement. Buildings located on compacted fill soils should be designed in anticipation of 1 inch of total settlement and ½ inch of differential settlement within a 40 foot horizontal run. The majority of this settlement is anticipated to occur during construction as the load is applied. These short-term settlements and angular distortions are from imposed building loads and do not include dynamic settlements. Dynamic (dry sand and liquefaction) total and differential settlements are anticipated to be on the order ½ inch and ¼ inch over 40 horizontal feet, respectively, potentially in addition to the estimated static settlement.

3.3.2 Mat Foundations

As an alternative to spread footings, mat foundations may be recommended for the proposed structures. Mat foundations should be embedded minimum 2 feet below the lowest adjacent grade, minimum 6 inches thick, and founded on a minimum of 3 feet deep properly compacted fill subgrade. The above compacted subgrade may be designed for modulus of subgrade reaction of 150 pounds-per-cubic-inch (pci) and an allowable vertical bearing capacity of 2,000 psf. For lateral capacity of mat foundations, allowable passive earth pressure and frictional resistance between soil and concrete as discussed in Section 3.3.1 may be used. Allowable bearing capacity and passive resistance may be increased by one-third for short duration loadings such as seismic or wind loads.

The maximum anticipated total settlement for mat foundations may be on the order of 1 inch. Differential settlements may be estimated up to half of the maximum total settlement.

3.4 Concrete Slab-on-Grade

Slab-on-grade floors utilized with conventional foundations should be designed with a minimum thickness as indicated by the project structural engineer consistent with a modulus of subgrade reaction of 150 pounds-per-cubic-inch (pci) and reinforced in accordance with the structural engineer's recommendations. Slabs-on-grade should be reinforced with at least No. 3 reinforcing bars spaced no more than 18-inches on-center in two perpendicular directions ("each-way"). A slip-sheet or equivalent should be used if crack-sensitive floor coverings (such as ceramic tiles, etc.) are to be placed directly on the concrete slab-on-grade.

Interior slab-on-grade floors should be underlain by a 10-mil Visqueen moisture retarder (or equivalent). This moisture retarder can covered by a 2-inch layer of sand (SE of 30 or greater) to reduce curling, only if a hot weather concrete pour is anticipated. Visqueen sheets should overlap at least 6-inches. If long-term storage of moisture sensitive records (files) or floor coverings (e.g. vinyl tile, etc.) is to be used, additional moisture mitigation measures may be employed within or beneath concrete slab-on-grade floors. Moisture retarders do not completely eliminate moisture vapor movement from the underlying soils up through the slabs or from the unbonded water in the concrete. To reduce moisture vapor emissions that may result in delamination and other tile damage, we suggest the following, only for areas where moisture sensitive floor coverings are anticipated:

- Vapor Barrier: A 15-mil vapor retarder should be placed directly onto the properly compacted subgrade. If a laser screed or similar equipment is used during concrete placement, a more durable vapor barrier could be used such as Stego-Wrap™ 15-mil, or equivalent, to reduce the potential for tearing and/or ripping the vapor barrier. Concrete should be allowed to pour out uniformly across this vapor barrier, without a sand layer over the vapor barrier.

- Concrete: A concrete mix design with a low water to cement ratio (less than 0.45) should be used. Water should not be added to this mix during placement. The concrete should be cured in a manner to eliminate slab curling.

- Post Curing: Before floor coverings are placed, any bond breaker coating and all other contaminates should be removed from the slab-on-grade surface. Shot blasting the slab surface may be required. Once the building has been enclosed, and environmental controls (heating and air conditioning) are installed and operational, the slab-on-grade should then be tested for moisture vapor emission, in accordance with ASTM E 1907-97.

- Floor Coverings: We should review the proposed floor covering and adhesive products and placement procedures to be used. Adhesives and coverings should be compatible, and the manufacture's requirements should be followed. The tested moisture vapor emission rate (MVER) should be below the specified rate for the floor covering products used (e.g. MVER<5), before the product is placed. Expansion gaps hould be provided where floor tiles are placed adjacent walls under molding, and along appropriate grids for large expanses of tile. Carpet strips or expansion joint flashing plates can be used in open areas at these joints.

Construction Considerations - Cracking of concrete is normal as it cures due to drying and shrinkage, and should be expected. However, cracking is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low slump concrete can reduce the potential for shrinkage cracking. Concrete placement during hot weather should be minimized due to the potential for slab curling. Slabs should be designed and constructed as promulgated by the Portland Cement Association.

To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or weakened plane joints at frequent intervals. Joints should be laid out to form approximately square panels.

3.5 Retaining Walls

Based on the conceptual site plan, no above-graded retaining wall is planned at this site. However, retaining walls may be anticipated for elevator pits, utility vaults, aesthetical grade separation in landscape areas, etc. Any type of retaining walls should be designed for lateral earth pressures. The magnitude of these pressures depends on the amount that the wall can yield horizontally under load. If the wall can yield enough to mobilize full shear strength of backfill soils, then the wall can be designed for "active" pressure. If the wall cannot yield under the applied load, the shear strength of the soil cannot be mobilized and the earth pressure will be higher. Such walls should be designed for "at rest" conditions. If a structure moves toward the soils, the resulting resistance developed by the soil is the "passive" resistance. Retaining walls backfilled with non-expansive soils (EI values less than 51) should be designed using the following equivalent fluid pressures:

Table 3 - Retaining Wall Design Earth Pressures (Static, Drained)

Loading Conditions	Equivalent Fluid Density for Level Backfill (pcf)
Active	35
At-Rest	55
Passive[1]	300

[1] Maximum passive pressure not to exceed 3,000 psf at depth.

Unrestrained (yielding) cantilever walls should be designed for the active equivalent-fluid-weight value provided above for very low to low expansive soils that are free draining. In the design of walls restrained from movement at the top (non-yielding) such as basement walls, elevator pits, and utility vaults, the at-rest equivalent fluid pressure should be used. Total depth of retained earth for design of cantilever walls should be measured as the vertical distance below the ground surface measured at the wall face for stem design, or measured at the heel of the footing for overturning and sliding calculations. Should a sloping backfill other than a 2:1 (horizontal:vertical) be constructed above the wall (or a backfill is loaded by an adjacent surcharge load), the equivalent fluid weight values provided above should be re-evaluated on an individual case basis by Leighton.

In addition to the above lateral forces due to retained earth, surcharge due to above grade loads on the wall backfill, such as an adjacent structure, should be considered in design of the retaining wall. Vertical surcharge loads behind the retaining wall on or in the backfill within a 1:1 (horizontal:vertical) plane projection up and out from the retaining wall toe, should be considered as lateral and vertical surcharge. Unrestrained (cantilever) retaining walls should be designed to resist one-third of these surcharge loads applied as a uniform horizontal pressure on the wall. Braced walls should also be designed to resist an additional uniform horizontal-pressure equivalent to one-half of uniform vertical surcharge-loads.

Retaining wall foundations should be at least 18 inches wide and embedded a minimum of 18 inches below the lowest adjacent grade. Preliminary design parameters for retaining wall footings shown follow the recommendations in Section 3.3 of this report. Non-standard wall designs should be reviewed by Leighton. prior to construction to check that the proper soil parameters have been incorporated into the wall design.

All retaining walls should be provided with appropriate drainage. The outlet pipe should be sloped to drain to a suitable outlet. Typical wall drainage design is illustrated in Figure 2, *Retaining Wall Backfill and Subdrain Detail*, for non-expansive backfill. Wall backfill should be compacted by mechanical methods to a minimum of 90 percent relative compaction (ASTM D1557). Walls should not be backfilled until wall concrete attains the 28-day compressive strength and/or as determined by the Structural Engineer that the wall is structurally capable of supporting backfill. Lightweight compaction equipment should be used, unless other wise approved by the Structural Engineer.

3.6 Temporary Excavations and Shoring

Based on the materials encountered in the borings, sloped temporary excavations may be constructed according to the slope ratios presented in Table 4 below.

Table 4 - Slope Ratios for Temporary Excavation

Maximum Depth of Cut (feet)	Maximum Slope Ratio* (horizontal:vertical)
0 – 5	Vertical
5 – 20	1:1

*Slope ratio assumed to be uniform from top to toe of slope.

Surfaces exposed in slope excavations should be kept moist but not saturated to retard raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods or rainfall. Surcharge loads should not be permitted within a horizontal distance equal to the depth of the cut from the top of slopes. Workers entering excavations should be protected from possible caving and raveling.

Based on the conceptual site plan, temporary shoring may be needed during construction. Temporary shoring may consist of shoulder piles and lagging. Preliminary geotechnical design parameters for temporary shoring will be provided after review of the final site plan.

3.7 Utility Trenches

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1.2 and 306-1.3 of the *Standard Specifications for Public Works Construction*, ("Greenbook"), 2003 Edition or corresponding sections in the later editions. Fill material should be placed in horizontal layers of thickness compatible to the type of equipment being used and should be compacted to at least 90 percent relative compaction (ASTM D 1557) by mechanical means only.

Where granular backfill is used in utility trenches adjacent to moisture sensitive subgrades and foundation soils, we recommend that a cut-off "plug" of impermeable material be placed in these trenches at the perimeter of buildings, and at pavement edges adjacent to irrigated landscaped areas. A "plug" can consist of a 5-foot long section of clayey soils with more than 35-percent passing the No. 200 sieve, or a Controlled Low Strength Material (CLSM) consisting of one sack of Portland-cement plus one sack of bentonite per cubic-yard of sand. CLSM should generally conform to Section 201-6 of the Standard Specifications for Public Works Construction, ("Greenbook"), 2003 Edition or corresponding sections in the later editions. Then CLSM plug is intended to reduce the likelihood of water migrating from landscaped areas, then seeping along permeable trench backfill into the building and pavement subgrades, resulting in wetting of moisture sensitive subgrade earth materials under buildings and pavements.

Excavation of utility trenches should be performed in accordance with the project plans, specifications and the California Construction Safety Orders (2003 Edition or more current). The contractor must be responsible for providing a "competent person" as defined in Article 6 of the California Construction Safety Orders. All safety precautions should be properly implemented at all times. Spoil piles from the excavation(s) and construction equipment should be kept away from the sides of the trenches. Leighton does not consult in the area of safety engineering

3.8 Site Soil Corrosivity

We conducted corrosion suite tests (pH, resistivity, chloride, and sulfate contents) for one representative bulk sample obtained from shallow depth at this site. The test results are included in Appendix C; a summary of the results and corresponding hazard levels are presented in the following Table 5. These limited test results indicate that that the near surface soil is anticipated to be moderately corrosive to buried ferrous metals. Water soluble sulfate and chloride contents of the onsite soils are found to be negligible (per Table 19-A-4 of CBC, 2001) and non-corrosive, respectively, to buried concrete.

Table 5 – Summary of the Corrosivity Test Results

Test Parameter	Test Results	General Classification of Hazard
Water-soluble sulfate content	0.01 percent by weight	Negligible sulfate exposure to buried concrete (per CBC, 2001)
Water-soluble chloride content	52 ppm	Non-corrosive to buried concrete (per Caltrans Specifications)
pH	8.19	Alkaline, relatively passive to buried metals
Minimum resistivity (in saturated condition)	2,870 ohm-cm	Moderately corrosive to buried ferrous pipes (per ASTM[1])

[1] ASTM STP 1013 titled *Effects of Soil Characteristics on Corrosion* (February, 1989).

Ferrous pipe buried in moist to wet site earth materials should be avoided by using high-density polyethylene (HDPE) or other non-ferrous pipe when possible. Ferrous pipe can be protected by polyethylene bags, tap or coatings, di-electric fittings or other means to separate the pipe from on-site soils. If buried ferrous pipes are planned for the project, further testing of soil samples corrosivity should be performed and specific recommendations for corrosion protection will need to be provided by a qualified corrosion engineer.

3.9 Field Percolation Test Results

Field percolation tests conducted on three borings on the north and south portions of the site (Boring B-2 and B-3 on the north and Boring B-7 on the south) indicate a fairly uniform percolation rate varying from 0.23 to 0.25 gallons/sq. feet/day averaged over depths up to approximately 16.5 feet below the existing grade. These rates are typical for silty soils. Based on our investigation, subsurface soils within upper 16.5 feet consist of primarily silt, clayey sand and sandy clay. Average percolation rates as mentioned above are consistent with the encountered soil types.

3.10 Pavement Design

3.10.1 Asphalt Concrete Pavements

Based on our field investigation findings, existing paved surface at the site consists of asphalt concrete overlying soil subgrade or aggregate base of variable thicknesses at different locations. Existing pavement sections are listed in Table 6 below (see Plate 1 for boring locations).

Table 6 – Existing Asphalt Pavement Sections

Boring No.	Asphalt Concrete (inches)	Aggregate Base (inches)
B-1	4.5	1.0
B-2	4.0	None
B-3	2.5	None
B-4	5.0	1.5
B-5	2.5	3.0
B-6	5.0	2.0
B-7	2.0	7.0
B-8	5.5	None
B-9	5.5	None

Our limited laboratory tests of one representative bulk sample of the existing pavement subgrade soils indicate an R-value of 36. Due to relative uniformity of the onsite soils within upper 5 feet as encountered during our field investigation, we assume an average R-value of 35 for preliminary design purpose. Considering this assumed R-value and following the Highway Design Manual (Caltrans, 2006) guidelines, minimum asphalt pavement sections for different Traffic Indices (TIs) ranging from 4 through 8 are listed in Table 7, below.

Table 7 - Asphalt Pavement Section Thickness

General Traffic Condition	Design Traffic Index (TI)	Asphalt Concrete (inches)	Aggregate Base[1] (inches)	Total Thickness (inches)
Automobile Parking	4.0	3.0	3.0	6.0
	4.5	3.0	4.0	7.0
Automobile Parking Lanes	5.0	3.0	5.0	8.0
	5.5	3.5	4.0	7.5
Truck Access & Parking Areas	6.0	3.5	6.0	9.5
	6.5	4.0	7.0	11.0
Public Roadway	7.0	4.0	8.0	12.0
	7.5	5.0	8.0	13.0
	8.0	5.0	9.0	14.0

[1] Minimum design R-value of aggregate base is 78.

Appropriate Traffic Index (TI) data should be selected by the project civil engineer or traffic engineering consultant and appropriate R-value of the subgrade soils will need to be determined after completion of rough grading to finalize the pavement design. Final pavement sections should be in general accordance with local, county and industry standards. Portland cement concrete may be used, rather than asphalt, in point and impact load areas such as trash truck bin loading areas.

Subgrade soils in the upper 18 inches of the driveways and parking areas should be properly compacted to at least 90 percent relative compaction (ASTM D1557) and should be moisture-conditioned to above optimum moisture contents, and kept in this condition until the pavement section is constructed. Minimum relative compaction requirements for aggregate base should be 95 percent of the maximum laboratory density (ASTM D1557).

Asphalt concrete and aggregate base should conform to Caltrans Standard Specifications (July 1995 Edition) Sections 39 and 26-1.02A, respectively. As an alternative, asphalt concrete can conform to Section 203-6 of the *Standard Specifications for Public Works Construction* (Green Book), 2003 Edition. Crushed aggregate base or crushed miscellaneous base can conform to Sections 200-2.2 and 200-2.4 of the *Standard Specifications for Public Works Construction* (Green Book), 2003 Edition, respectively.

3.10.2 Portland Cement Concrete Pavements

For preliminary planning purposes, proposed pavements at ground surface may be constructed of a minimum of 6 inches thick Portland Cement Concrete (PCC) overlying a minimum of 4 inches thick Class 2 aggregate base. These minimum sections are considered assuming an average daily truck traffic (ADTT) in both directions not exceeding 300. All PCC pavements should have a minimum 28-day concrete compressive strength of 3,000 psi and have appropriate joints and saw cuts in accordance with either Portland Cement Association (PCA) or American Concrete Institute (ACI) guidelines. Subgrade underneath the PCC pavements should be compacted to a minimum of 90 percent compaction within upper 18 inches. Use of concrete cutoff or edge barriers should be considered at the perimeter of the common parking or driveway areas when they are adjacent to either open (unfinished) or landscaped areas.

3.11 Surface Drainage

Ponding of water adjacent to structures should be avoided. During and after construction, positive drainage should be provided to direct surface water away from structures and towards suitable, nonerosive drainage devices. Locating planters adjacent to buildings or structures should be avoided. Where unavoidable, planters should be properly lined, such as with a membrane, to reduce penetration of irrigation water into the adjacent footing subgrades. Wherever possible, exposed soil areas should be above paved grades. Planters should not be depressed below adjacent paved grades unless drainage, such as catch basins and drains are provided.

3.12 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and laboratory testing. Our conclusions and recommendations presented in this report should be reviewed and verified by Leighton during site construction and revised accordingly if exposed geotechnical conditions vary from our preliminary findings and interpretations. The recommendations presented in this report are only valid if Leighton verifies the site conditions during construction. Geotechnical observation and testing should be provided during the following activities:

- Grading and excavation of the site;
- Overexcavation and compaction of all fill materials;
- Excavation and installation of foundations;
- After excavation of all slabs and footings and prior to placement of steel or concrete to confirm the slabs and footings are founded in firm, compacted fill;
- Utility trench backfilling and compaction;
- Pavement subgrade preparation and base course compaction; and
- When any conditions are encountered that vary significantly form the conditions described in this report.

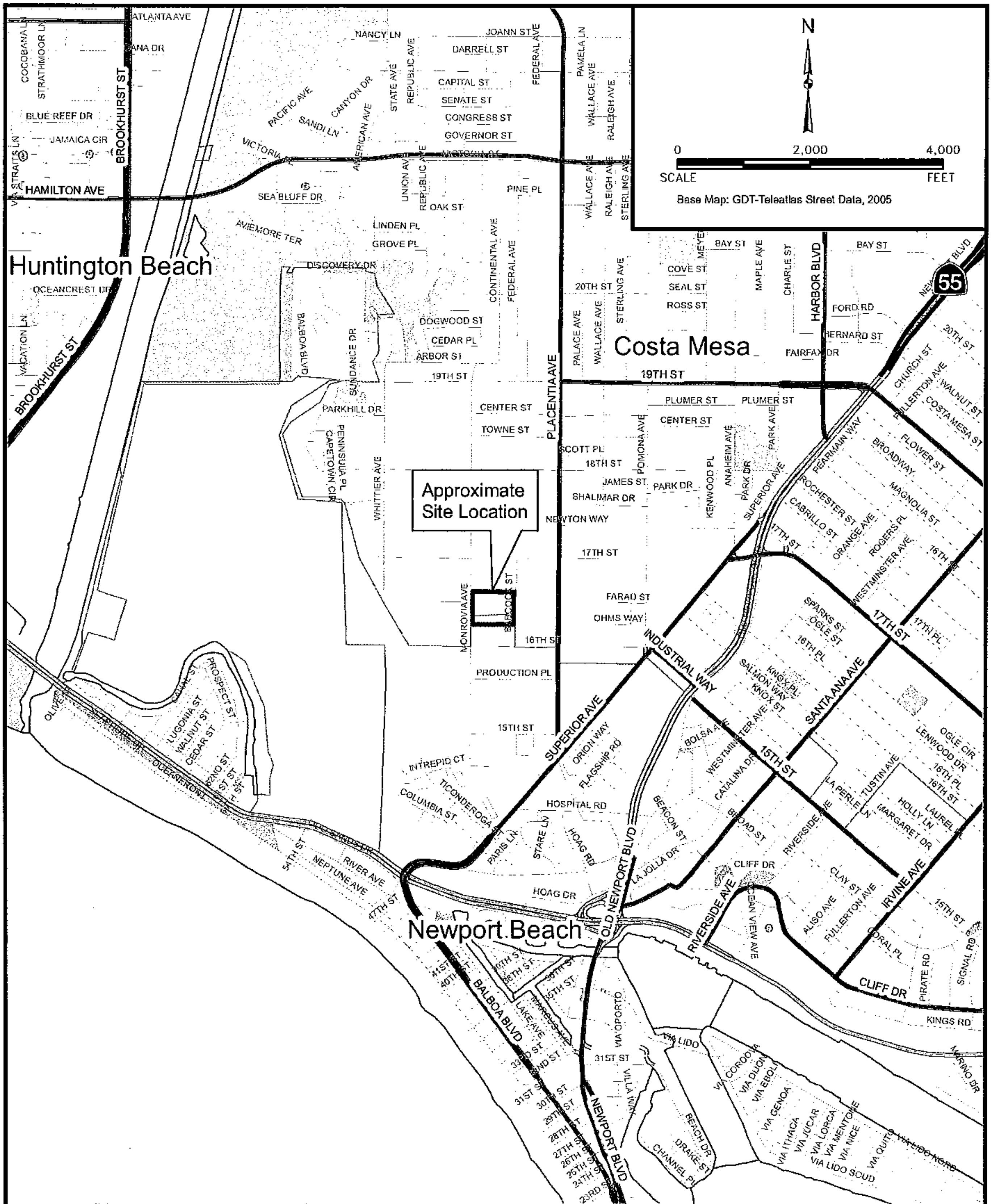
Leighton should review the grading and foundation plans and specifications, when available, to comment on the geotechnical aspects. Our recommendations should be revised, as necessary, based on future plans and incorporated into the final design plans and specifications.

4.0 LIMITATIONS

The conclusions and recommendations presented in this report have been based upon the generally accepted principles and practices of geotechnical engineering utilized by other competent engineers at this time and place. No other warranty is either expressed or implied.

The conclusions and recommendations presented in this report have been based upon the subsurface conditions encountered at discrete and widely spaced locations and at specific intervals below the ground surface. Due to the inherent variance in soils conditions, variability may be encountered during construction. Where encountered during construction, such variances should be brought to our attention to determine the impact upon the recommendations presented in this report.

This report has been prepared for the expressed use of our client and the design professionals assigned for this project. The report may not be used by others or for other projects without the expressed written consent of our client and our firm.



Geotechnical Investigation
Residential and Commercial Developments
1640 Monrovia Avenue
Costa Mesa, CA

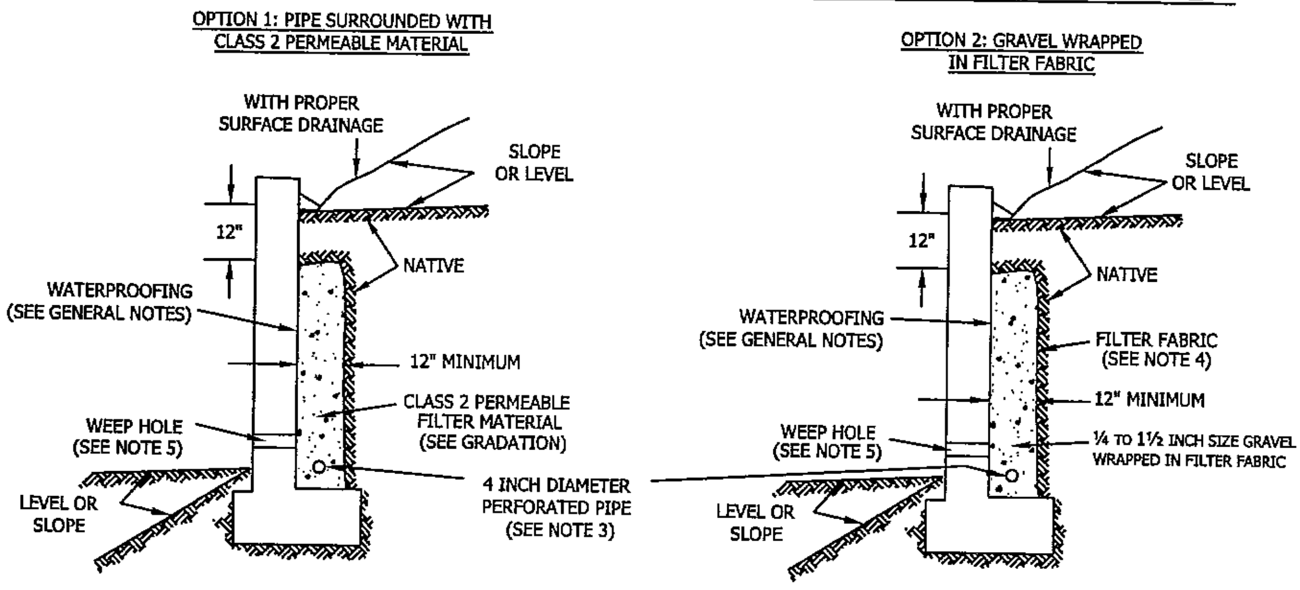
SITE LOCATION MAP

Project No.
012184-001

Date
July 2007

Figure 1

SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

GENERAL NOTES:

* Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
* Water proofing of the walls is not under purview of the geotechnical engineer
* All drains should have a gradient of 1 percent minimum

*Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)

*Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:

1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
4) Filter fabric should be Mirafi 140NC or approved equivalent.
5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL FOR WALLS 6 FEET OR LESS IN HEIGHT

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton

P:Drafting\templates\details\retain-wall-backfill-and subdrain.dwg (7/00)

FIGURE 2

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

APPENDIX A

References

Abrahamson, N. A and Silva, W. J., 1997, "Empirical Response Spectral Attenuation Relations for Shallow Crustal Earthquakes," *Seismological Research Letters*, Vol. 68, No. 1, January/February, pp. 94-127.

Architects Orange, 2007, Conceptual Site Plan, Nexus Development, 1640 Monrovia Avenue, Costa Mesa, California, Scale 1:360, dated May 31, 2007.

Blake, T. F., 2000a, UBCSEIS, A Computer Program for Evaluating the Seismic Parameters in Accordance with the 1997 UBC, Version 1.03, User's Manual.

Blake, T. F, 2000b, FRISKSP, A Computer Program for the Probabilistic Estimation of Peak Acceleration and Uniform Hazard Spectra Using 3-D Faults as Earthquake Sources, Version 4.00, User's Manual.

Bozorgnia, Y., Campbell, K. W., Niazi, M. (1999), "Vertical ground motion: Characteristics, relationships with horizontal component, and building code implications," Proc. SMIP99 Seminar on Utilization of Strong Motion Data, California Strong Motion Instrumentation Program, Sacramento, CA, 23-50.

______, 2000, CD-ROM containing digital images of Official Maps of Alquist-Priolo Earthquake Fault Zones that affect the Southern Region, DMG CD 2000-003 2000.

California Department of Conservation, Division of Mines And Geology (DMG), Seismic Hazard Zone Report for the Anaheim and Newport Beach 7.5-Minute Quadrangles, Orange County, Seismic Hazard Zone Report 03, 1997.

California Geological Survey (CGS; previously known as the California Division of Mines and Geology), 1998, *Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada,* to be used with the 1997 Uniform Building Code, International Conference of Building Officials, February, 1998.

Caltrans, 2006, Flexible Pavement Design, Chapter 630 of the Highway Design Manual, revised on September 1, 2006.

FEMA, 2004, Flood Insurance Rate Map, Orange County, California, Map No. 06059C0264H, revised on February 18, 2004.

APPENDIX A (Continued)

References

Hart, E.W. and Bryant, W.A., 1999, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps, California Division of Mines and Geology Special Publication 42, 38 p.

Hart, E.W., 1973, revised 1999, Fault Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps, California Division of Mines and Geology Special Publications 42

Sadigh, K., Chang, C. Y., Egan, J. A., Makdisi, F., and Youngs R. R., 1997, "Attenuation Relations for Shallow Crustal Earthquakes Based on California Strong Motion Data" *Seismological Research Letters*, Vol. 68, No. 1, January/February, pp. 180-189.

United States Geologic Survey, 1965, Anaheim 7.5-Minute Series Topographic Quadrangle Map, Orange County, California, released 1965, photo revised 1972.

United States Geological Survey, 1999, Preliminary Digital Geological Map of the Santa Ana 30' X 60' Quadrangle, Southern California, Version 1.0, Open-File Report 99-172.

GEOTECHNICAL BORING LOG B-1

Date: 5-23-07 | Sheet 1 of 2
Project: West Side Lofts | Project No. 012184-001
Drilling Co.: ABC Liovine Drilling | Type of Rig: CME-75
Hole Diameter: 8" | Drive Weight: 140 lbs Auto-hammer | Drop: 30"
Elevation Top of Hole: 112' | Location: See plate 1, Boring Location Map

Logged By: CDL
Sampled By: CDL

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	0								@ Surface: Approximately 4.5- inches of asphalt concrete over 1- inch of aggregate base.	EI, MD, RV
110									**Quaternary alluvium (Qal):**	
				B1 R-1	7 12 10	123.7	12.1	SC	@ 2.5': Clayey SAND, red-brown, wet, medium dense, fine grained sand with medium grained sand, trace fine well rounded gravel, trace amount of pin-hole open pore spaces, poorly developed blocky structure lined with clay.	
	5			R-2	12 15 22	120.7	13.8	SC	@ 5': Clayey SAND, brown, wet, medium dense, fine to medium grained sand, some coarse grained sand, black stringers, 46.7% passing No. 200 sieve.	-200
105										
	10			R-3	6 12 15	97.7	25.6	ML	@ 10': SILT, tan brown, wet, very stiff, very finely bedded, .5mm thick manganese deposits, grades to silty SAND, brown, moist, medium dense, fine grained sand, 87.2% passing No. 200 sieve.	-200, DS MD
100				B2						
	15			R-4	6 9 11	97.3	27.3	ML	@ 15': SAND, grey-brown, wet, medium dense, fine grained sand, grades to very fine bedded SILTS and CLAYS with interbeds of sand, stiff, wet, .5mm thick bedding, grey-brown with intermittent severely oxidized beds, fine grained sand, 88.6% passing No. 200 sieve.	-200, CN
95										
	20			R-5	15 20 26	113.0	3.2	SP	@ 20': Silty SAND, light grey, damp, medium dense, fine grained sand, grades to SAND with silt, dark orange brown, damp, medium dense, fine to medium grained sand, shell fragments.	
90										
	25			S-1	5 9 11			ML	@ 25': Sandy SILT, tan brown mottled with medium grey, moist, very stiff, fine grained sand, shell fragments, 51.3% passing No. 200 sieve.	-200
85										
	30									

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- -200 200 WASH
- AL ATTERBERG LIMITS
- EI EXPANSION INDEX
- RV R-VALUE

GEOTECHNICAL BORING LOG B-1

Date 5-23-07 **Sheet** 2 **of** 2

Project West Side Lofts **Project No.** 012184-001

Drilling Co. ABC Liovine Drilling **Type of Rig** CME-75

Hole Diameter 8" **Drive Weight** 140 lbs Auto-hammer **Drop** 30"

Elevation Top of Hole 112' **Location** See plate 1, Boring Location Map

Logged By CDL

Sampled By CDL

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	30			R-6	8 16 24	108.9	18.4	CL	@ 30': Silty SAND, orange-brown, moist, medium dense, fine grained sand, grades to silty CLAY with sand, brown interbedded with grey and discontinuous richly oxidized brown red beds, wet, very stiff, fine grained sand localized on bedding faces of very finely to laminated silty CLAY beds, 69.4% passing No. 200 Sieve.	-200
	35			S-2	7 12 14			SP-SM	@ 35': SAND with silt, light grey, damp, medium dense, fine grained sand, 6.5% passing No. 200 sieve.	-200
	40			R-7	15 45 48	106.5	4.6	SP-SM	@ 40': SAND with silt, grey-brown grades to richly oxidized red-brown, damp to moist, very dense, fine to medium grained sand.	
									@ 42': Encounter 4-inch thick bed of gravel.	
	45			S-3	5 11 14			SP-SM	@ 45': SAND with silt, grey-brown, wet, medium dense, fine to medium grained sand.	
	50			R-8	13 30 42			SP-SM	@ 50': SAND with silt, orange-brown grades to medium grey interbedded with brown sand lenses, wet, dense, fine to medium grained sand, 7.3% passing No. 200 sieve.	-200
	55									
	60									

Total depth of boring: 51.5 feet
Groundwater was encountered at 45 feet below ground surface
The boring was backfilled with 32 feet of bentonite chips and the remainder with soil cuttings.

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- -200 200 WASH
- AL ATTERBERG LIMITS
- EI EXPANSION INDEX
- RV R-VALUE

GEOTECHNICAL BORING LOG B-2

Date 5-23-07 **Sheet** 1 **of** 1
Project West Side Lofts **Project No.** 012184-001
Drilling Co. ABC Liovine Drilling **Type of Rig** CME-75
Hole Diameter 8" **Drive Weight** 140 lbs Auto-hammer **Drop** 30"
Elevation Top of Hole 112' **Location** See plate 1, Boring Location Map

Logged By CDL
Sampled By CDL

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	0								@ Surface: Approximately 4-inches of asphalt-concrete over no base.	
110									**Quaternary alluvium (Qal):**	
				B1 R-1	3 4 6	115.4	18.0	CL	@ 2.5': CLAY with sand, dark red-brown, wet, medium stiff, fine grained sand with medium grained sand, poorly developed blocky structure with clay lined ped faces, grades to fine to medium grained sand with depth, trace coarse grained sand.	
105	5			R-2	8 19 23	126.7	12.2	CL	@ 5': CLAY with sand, brown, wet, hard, medium grained sand, fine to coarse grained sand.	
100	10			R-3	10 19 27			ML	@ 10': Sandy SILT, tan-brown, moist, very stiff, fine grained sand, trace manganese.	
95	15			R-4	14 27 34			SP	@ 15': SAND, light orange-brown, dense.	
									Total depth of boring: 16.5 feet. No groundwater was encountered. The boring was converted to a dry well, total depth of dry well: 13 feet.	
90	20									
85	25									
	30									

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- -200 200 WASH
- AL ATTERBERG LIMITS
- EI EXPANSION INDEX
- RV R-VALUE

GEOTECHNICAL BORING LOG B-3

Date: 5-23-07 | Sheet 1 of 1
Project: West Side Lofts | Project No. 012184-001
Drilling Co.: ABC Liovine Drilling | Type of Rig: CME-75
Hole Diameter: 8" | Drive Weight: 140 lbs Auto-hammer | Drop: 30"
Elevation Top of Hole: 111' | Location: See plate 1, Boring Location Map

Logged By: CDL
Sampled By: CDL

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
110	0							@ Surface: Approximately 2.5-inches of asphalt-concrete over no base. **Quaternary alluvium (Qal):**	
			B1 R-1	4 8 10	117.7	15.0	CL	@ 2.5': CLAY with sand, dark red-brown, wet, stiff, fine grained sand, trace medium grained sand.	
105	5		R-2	8 16 23	123.6	12.9	CL/SC	@ 5': CLAY with sand/clayey SAND, dark red-brown, to brown with depth, wet, very stiff, fine grained sand, decomposed rootlets, trace poorly developed clay faced blocky structure.	
100	10		R-3	5 8 14			SC	@ 10': Clayey SAND, tan-brown, moist, medium dense, fine grained sand, grades to silty SAND, tan-brown, moist, medium dense, fine grained sand.	
95	15		R-3	9 15 20			SP	@ 15': SAND, light grey-tan-brown, damp, medium dense, fine to medium grained sand.	
								Total depth of boring: 16.5 feet. No groundwater was encountered. The boring was converted to a dry well, total depth of dry well: 13 feet.	
90	20								
85	25								
	30								

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- -200 200 WASH
- AL ATTERBERG LIMITS
- EI EXPANSION INDEX
- RV R-VALUE

GEOTECHNICAL BORING LOG B-4

Date: 5-23-07 | Sheet 1 of 1
Project: West Side Lofts | Project No. 012184-001
Drilling Co.: ABC Liovine Drilling | Type of Rig: CME-75
Hole Diameter: 8" | Drive Weight: 140 lbs Auto-hammer | Drop: 30"
Elevation Top of Hole: 111' | Location: See plate 1, Boring Location Map

Logged By: CDL
Sampled By: CDL

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
110	0								@ Surface: Approximately 5-inches of asphalt-concrete over 1.5-inches of aggregate base	
									Quaternary alluvium (Qal):	
				B1 R-1	4 8 12	121.0	14.4	CL	@ 2.5': CLAY with sand, dark red-brown, wet, stiff, fine grained sand, decomposed rootlets, trace poorly developed clay faced blocky structure.	
105	5			R-2	9 13 18	122.8	13.8	CL	@ 5': CLAY with sand, dark red-brown, wet, very stiff, fine grained sand, grades to a clayey SAND, tan-brown mottled with dark red-brown, moist, medium dense, fine to medium grained sand.	
100	10			R-3 B2	7 10 13	96.3	27.1	ML	@ 10': Sandy SILT with clay, tan-brown, moist to wet (free water on micaceous clay lined nearly horizontal discontinuous polynomial fractured poorly developed ped faces), stiff, fine grained sand, grades to silty SAND, tan-brown, very moist, medium dense, fine grained sand.	
95	15			R-4	13 31 45	112.4	11.2	ML	@ 15': Sandy SILT, dark grey-brown, moist, hard, fine grained sand, very finely bedded, grades to a SAND with silt, orange-brown, damp, dense, fine to medium grained sand.	
90	20			R-5	10 17 21	106.6	11.9	SM	@ 20': Silty SAND, grey-brown, moist, medium dense, fine grained sand, trace clay with depth.	
85	25			S-1	5 7 9			SC/SM	@ 25': Silty SAND/sandy CLAY, brown silty SAND mottled with grey CLAY, very moist, medium dense, fine grained sand, oxidized, grades to sandy CLAY with depth.	
	30								Total depth of boring: 26.5 feet. No groundwater was encountered. The boring was backfilled with soil cuttings.	

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- -200 200 WASH
- AL ATTERBERG LIMITS
- EI EXPANSION INDEX
- RV R-VALUE

GEOTECHNICAL BORING LOG B-5

Date 5-23-07 | Sheet 1 of 1
Project West Side Lofts | Project No. 012184-001
Drilling Co. ABC Liovine Drilling | Type of Rig CME-75
Hole Diameter 8" | Drive Weight 140 lbs Auto-hammer | Drop 30"
Elevation Top of Hole 111' | Location See plate 1, Boring Location Map

Logged By CDL
Sampled By CDL

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
110	0								@ Surface: Approximately 2.5-inches of asphalt-concrete over 3-inches of aggregate base	
									Quaternary alluvium (Qal):	
				B1 R-1	4 8 11	118.1	11.9	CL	@ 2.5': CLAY with sand, dark red-brown, wet, stiff, fine grained sand, trace medium grained sand, grades to a sandy CLAY, brown, moist, stiff, fine to medium grained sand, clay stringers.	
105	5			R-2	8 11 13	118.3	10.0	SM	@ 5': Silty SAND with clay, brown, moist, medium dense, fine to medium grained sand, black stringers, grades to silty SAND, brown, moist, medium dense, fine to coarse grained sand.	
100	10			R-3	8 12 15	95.9	25.4	ML	@ 10': Sandy SILT, tan-brown, moist, dense, stiff, fine grained sand, very finely bedded.	
95	15			R-4	11 22 33	102.7	9.3	SM/ML	@ 15': Silty SAND/sandy SILT, very finely bedded light orange-brown and medium grey, damp, dense/hard, fine grained sand, grades to SAND, light orange-brown, damp, dense, fine grained sand.	
90	20			R-5	11 20 22	111.3	6.3	SM	@ 20': Silty SAND, tan-brown, moist, medium dense, fine grained sand, grades in sampler tip to CLAY, bedded brown and medium grey, moist, hard, very finely bedded.	
85	25			S-1	4 5 7			SM	@ 25': Silty SAND, brown grades to brown mottled with grey, moist, medium dense, fine grained sand, silt content increases with depth, trace oxidization.	
	30								Total depth of boring: 26.5 feet. No groundwater was encountered. The boring was backfilled with soil cuttings.	

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
C CORE SAMPLE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
-200 200 WASH
AL ATTERBERG LIMITS
EI EXPANSION INDEX
RV R-VALUE

GEOTECHNICAL BORING LOG B-6

Date 5-23-07 **Sheet** 1 **of** 2

Project West Side Lofts **Project No.** 012184-001

Drilling Co. ABC Liovine Drilling **Type of Rig** CME-75

Hole Diameter 8" **Drive Weight** 140 lbs Auto-hammer **Drop** 30"

Elevation Top of Hole 111' **Location** See plate 1, Boring Location Map

Logged By CDL

Sampled By CDL

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
110	0							@ Surface: Approximately 5-inches of asphalt-concrete over 2-inches of aggregate base.	EI
								Quaternary alluvium (Qal):	
			B1 R-1	4 7 10	115.1	16.5	CL	@ 2.5': CLAY with sand, brown, wet, stiff, fine grained sand with black nodules, 63.9% passing No. 200 sieve.	-200
105	5		R-2	8 18 25	123.2	12.6	CL	@ 5': CLAY with sand, dark red-brown, wet, very stiff, fine grained sand.	AL
100	10		R-3	6 11 12	103.4	18.1	ML	@ 10': Sandy SILT, brown, moist, stiff, fine grained sand, very finely bedded, 81.4% passing No. 200 sieve.	-200, CN
95	15		R-4	7 8 14	93.7	27.4	ML	@ 15': Sandy SILT, tan-brown, wet, stiff, fine grained sand, grades to very finely bedded silty CLAY, medium grey with oxidized interbeds, wet, stiff, fine grained sand, 88.4% passing No. 200 sieve.	-200
90	20		R-5	10 20 28	100.3	5.7	SM	@ 20': Silty SAND, tan-brown, damp to moist, dense, fine grained sand, some medium grained sand.	
85	25		S-1	3 5 6			CL	@ 25': Sandy silty CLAY, mottled grey and red-brown, moist, stiff, fine grained sand, sandy SILT is mottled within silty CLAY matrix, 72.1% passing No. 200 sieve.	-200
	30								

SAMPLE TYPES:

- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:

- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- -200 200 WASH
- AL ATTERBERG LIMITS
- EI EXPANSION INDEX
- RV R-VALUE

GEOTECHNICAL BORING LOG B-6

Date 5-23-07 | Sheet 2 of 2
Project West Side Lofts | Project No. 012184-001
Drilling Co. ABC Liovine Drilling | Type of Rig CME-75
Hole Diameter 8" | Drive Weight 140 lbs Auto-hammer | Drop 30"
Elevation Top of Hole 111' | Location See plate 1, Boring Location Map

Logged By CDL
Sampled By CDL

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
80	30			R-6	7 16 24	110.8	8.5	SC	@ 30': Clayey SAND, tan grey-brown heavily mottled with richly oxidized red-orange, moist, medium dense, fine grained sand, grades to SAND, light tan mottled with orange, damp, medium dense, fine grained sand with medium grained sand.	
75	35			S-2	5 11 14			SC	@ 35': Clayey SAND, olive-grey, moist, medium dense, micaceous, 41.9% passing No. 200 sieve.	-200
70	40			R-7	18 39 47	103.7	3.4	SP-SM	@ 40': SAND with silt, medium grey mottled with oxidized red-orange, damp, very dense, fine grained sand with medium grained sand.	
65	45			S-3	9 12 14			SP-SM	@ 45': SAND with silt, brown, very moist, wet at 45.5', medium dense, fine grained sand, 10.2% passing No. 200 sieve.	-200
60	50			R-8	13 23 21			SP-SM	@ 50': SAND with silt, tan-brown, wet, medium dense, fine to medium grained sand, grades to gravelly SAND, medium grey, wet, fine to medium grained sand, medium angular siltstone gravel, strong chemical odor.	
55	55								Total depth of boring: 51.5 feet. Groundwater was encountered at 45.5 feet below ground surface. The boring was backfilled with 34 feet of bentonite chips and the remainder with soil cuttings.	
	60									

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- -200 200 WASH
- AL ATTERBERG LIMITS
- EI EXPANSION INDEX
- RV R-VALUE

GEOTECHNICAL BORING LOG B-7

Date 5-23-07 **Sheet** 1 **of** 1
Project West Side Lofts **Project No.** 012184-001
Drilling Co. ABC Liovine Drilling **Type of Rig** CME-75
Hole Diameter 8" **Drive Weight** 140 lbs Auto-hammer **Drop** 30"
Elevation Top of Hole 110' **Location** See plate 1, Boring Location Map

Logged By CDL
Sampled By CDL

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
110	0								@ Surface: Approximately 2-inches of asphalt-concrete over 7-inches of aggregate base.	
									Artificial fill (Af):	
									Quaternary alluvium (Qal):	
				B1 R-1	8 3 4	112.3	15.1	CL	@ 2.5': CLAY with sand, black, wet, medium stiff, fine grained sand, grades to sandy CLAY, brown mottled with red-brown, wet, medium stiff, fine grained sand.	
105	5			R-2	5 11 16	115.2	16.0	CL	@ 5': Clay with sand, dark red-brown, wet, very stiff, fine to medium grained sand, sand content increases with depth, finely bedded.	
100	10			R-3	7 12 16			CL	@ 10': Sandy silty CLAY, brown, moist, very stiff, fine grained sand, black decomposed rootlets, grades to CLAY, brown, moist, very stiff, micaceous.	
95	15			R-4	7 9 12			SM/ML	@ 15': Sandy SILT/silty SAND, tan-brown, very moist, stiff/medium dense, fine grained sand, grades in sampler tip to CLAY, tan-brown to brown, moist, stiff, very fine bedding.	
									Total depth of boring: 16.5 feet. No groundwater was encountered. The boring was converted to a dry well, total depth of dry well: 13 feet.	
90	20									
85	25									
80	30									

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
C CORE SAMPLE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
-200 200 WASH
AL ATTERBERG LIMITS
EI EXPANSION INDEX
RV R-VALUE

GEOTECHNICAL BORING LOG B-8

Date 5-23-07 **Sheet** 1 **of** 1
Project West Side Lofts **Project No.** 012184-001
Drilling Co. ABC Liovine Drilling **Type of Rig** CME-75
Hole Diameter 8" **Drive Weight** 140 lbs Auto-hammer **Drop** 30"
Elevation Top of Hole 111' **Location** See plate 1, Boring Location Map

Logged By CDL
Sampled By CDL

Elevation Feet	Depth Feet	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
110	0						@ Surface: Approximately 5.5-inches of asphalt-concrete over no base.	CR
							Artificial fill (Af):	
							Quaternary alluvium (Qal):	
		B1 R-1	9 12 15	119.6	13.6	CL	@ 2.5': CLAY with sand, brown, wet, very stiff, fine grained sand, some poorly developed clay faced blocky structure development.	
105	5	R-2	9 16 44	119.7	14.4	CL	@ 5': CLAY with sand, brown, wet, hard, fine grained sand, sand content increases with depth and grain size to fine to medium grained sand.	
100	10	R-3	11 15 17	108.7	15.7	SM	@ 10': Silty SAND, tan-grey-brown, moist, medium dense, fine grained sand, silt content increases with depth.	
95	15	R-4	9 11 16	88.5	33.3	ML	@ 15': Sandy SILT, tan-grey-brown, wet, very stiff, fine grained sand, very fine bedding, trace oxidized beds.	
90	20	R-5	10 18 27	110.1	15.7	SM	@ 20': Mottled light tan-brown fine grained SAND and brown fine grained silty SAND, moist, medium dense.	
85	25	S-1	5 8 12			CL	@ 25': Sandy CLAY, mottled medium grey-brown and red-brown, moist, stiff, fine grained sand, clay content increases with depth to CLAY.	
	30						Total depth of boring: 26.5 feet. No groundwater was encountered. The boring was backfilled with soil cuttings.	

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- -200 200 WASH
- AL ATTERBERG LIMITS
- EI EXPANSION INDEX
- RV R-VALUE

GEOTECHNICAL BORING LOG B-9

Date 5-23-07 — Sheet 1 of 1

Project: West Side Lofts — Project No. 012184-001

Drilling Co.: ABC Liovine Drilling — Type of Rig: CME-75

Hole Diameter: 8" — Drive Weight: 140 lbs Auto-hammer — Drop: 30"

Elevation Top of Hole: 111' — Location: See plate 1, Boring Location Map

Logged By: CDL

Sampled By: CDL

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Six Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
110	0								@ Surface: Approximately 5.5-inches of asphalt-concrete over no base.	
									Artificial fill (Af):	
				B1 R-1	6 11 15	118.7	15.2	CL	@ 2.5': CLAY with sand, brown, moist, stiff, fine grained sand, some black decomposed rootlet stringers.	
105	5			R-2	8 14 23	123.1	11.3	CL	@ 5': CLAY with sand, brown, moist, hard, fine grained sand, poorly developed clay lined ped faced blocky structure development, sand content and grain size increases with depth to fine to medium grained sand.	
100	10			R-3	7 16 20	101.0	23.7	CL	@ 10': CLAY, tan-brown, moist, very stiff, with fine grained sand, micaceous, finely bedded.	
95	15			R-4	6 12 20	90.2	33.1	CL	@ 15': CLAY, tan-grey-brown, moist, stiff, very fine bedding, trace micaceous and oxidization on bedding faces.	
90	20			R-5	9 12 20	118.2	7.1	SM	@ 20': Silty SAND, brown, moist, medium dense, fine to medium grained sand, .5-inch diameter vertical krotovia infilled with fine grey sand.	
85	25			S-1	4 6 11			CL	@ 25': Silty CLAY, grey, moist, stiff, 2-inch thick interbeds of fine grained tan SAND and brown fine grained sandy SILT.	
	30								Total depth of boring: 26.5 feet. No groundwater was encountered. The boring was backfilled with soil cuttings.	

SAMPLE TYPES:

- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:

- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- -200 200 WASH
- AL ATTERBERG LIMITS
- EI EXPANSION INDEX
- RV R-VALUE

APPENDIX C

Laboratory Test Procedures and Test Results

Moisture and Density Determination Tests: Moisture content and dry density determinations were performed, in general accordance with ASTM test method D2937, on relatively undisturbed samples obtained from the test borings and/or trenches. The results of these tests are presented in the boring logs (see Appendix A).

Percent Passing No. 200 Sieve: Selected samples were tested in accordance with the ASTM Standard D1140 to determine the amounts of materials finer than the U.S. Standard Sieve No. 200. Test results are presented in this appendix.

Atterberg Limits: The Atterberg Limits of selected soil samples were determined in accordance with ASTM Test Method D4318 for engineering classification of the fine-grained materials. Test results are presented in this appendix.

Expansion Index Tests: Expansion Index (EI) tests were performed on representative bulk samples of the onsite soil, in general accordance with the ASTM D 4829 Standard Test Method. Test results are presented in this appendix.

Maximum Density Tests: The maximum dry density and optimum moisture content of representative bulk soil samples were determined in accordance with ASTM Test Method D1557. Test results are presented on the *Modified Proctor Compaction Test* figures in this appendix.

Direct Shear Tests: Direct shear tests were performed on selected relatively undisturbed ring samples, which were soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. Samples and specimens were then transferred to the shear box, reloaded, and pore pressures set up in the sample (due to transfer) were allowed to dissipate for a period of approximately one-hour. Following pore pressure dissipation, samples were subjected to shearing forces. The samples were tested under various normal loads by a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of 0.05 inches per minute. Test results data and graphical plots are presented in this appendix.

Consolidation Tests: Consolidation tests were performed on selected, relatively undisturbed ring samples. These samples were placed in a consolidometer and loads were applied in geometric progression. The percent consolidation for each load cycle was recorded as the ratio of the amount of vertical compression to the original 1-inch height. Consolidation results (curves) are presented on the One-Dimensional Consolidation figure in this appendix.

APPENDIX C (Continued)

Laboratory Test Procedures and Test Results

"R"-Value: The resistance "R"-value was determined by the California Standard Test Method No. 301 for representative pavement subgrade soils. Three specimens were prepared from one bulk sample and exudation pressure and "R"-value determined on each one. The graphically determined "R"-value at exudation pressure of 300 psi is reported in this appendix.

Chloride Content, Sulfate Content, Minimum Resistivity and pH Tests: Chloride content, sulfate content, minimum resistivity and pH tests were performed in general accordance with California Test Methods 422, 417, and 532. These results are presented in this appendix.

Boring No.	B-1	B-1	B-1	B-1	B-1	B-1	B-1	
Sample No.	R-2	R-3	R-4	S-1	R-6	S-2	R-8	
Depth (ft.)	5	10	15	25	30	35	50	
Sample Type	Drive	Drive	Drive	SPT	Drive	SPT	Drive	
Soil Identification	Brown clayey sand (SC)	Yellowish brown lean / silty clay (CL/CL-ML)	Light olive brown silt (ML)	Light olive brown sandy silt s(ML)	Light olive brown sandy silt s(ML)	Pale olive poorly graded sand with silt (SP-SM)	Pale olive poorly graded sand with silt (SP-SM)	
Moisture Correction								
Wet Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
Dry Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
Weight of Container (g)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
Moisture Content (%)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
Sample Dry Weight Determination								
Weight of Sample + Container (g)	535.60	352.17	347.40	515.05	380.70	547.16	469.60	
Weight of Container (g)	108.10	109.30	106.80	108.10	110.40	108.00	108.20	
Weight of Dry Sample (g)	427.50	242.87	240.60	406.95	270.30	439.16	361.40	
Container No.:								
After Wash								
Method (A or B)	B	B	B	B	B	B	B	
Dry Weight of Sample + Cont. (g)	335.90	140.40	134.30	306.20	193.10	518.70	443.25	
Weight of Container (g)	108.10	109.30	106.80	108.10	110.40	108.00	108.20	
Dry Weight of Sample (g)	227.80	31.10	27.50	198.10	82.70	410.70	335.05	
% Passing No. 200 Sieve	**46.7**	**87.2**	**88.6**	**51.3**	**69.4**	**6.5**	**7.3**	
% Retained No. 200 Sieve	53.3	12.8	11.4	48.7	30.6	93.5	92.7	

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: West Side Lofts
Project No.: 012184-001
Client Name: L & A / Irvine
Tested By: G. Berdy Date: 06/06/07

Boring No.	B-6	B-6	B-6	B-6	B-6	B-6		
Sample No.	R-1	R-3	R-4	S-1	S-2	S-3		
Depth (ft.)	2.5	10	15	25	35	45		
Sample Type	Drive	Drive	Drive	SPT	SPT	SPT		
Soil Identification	Olive brown sandy lean clay s(CL)	Brown silt with sand (ML)s	Olive brown lean clay (CL)	Pale olive silty clay with sand (CL-ML)s	Olive yellow clayey sand (SC)	Light olive brown poorly graded sand with silt (SP-SM)		
Moisture Correction								
Wet Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00	0.00		
Dry Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00	0.00		
Weight of Container (g)	1.00	1.00	1.00	1.00	1.00	1.00		
Moisture Content (%)	0.00	0.00	0.00	0.00	0.00	0.00		
Sample Dry Weight Determination								
Weight of Sample + Container (g)	673.90	356.42	617.30	491.79	457.41	463.78		
Weight of Container (g)	235.20	108.80	249.00	107.40	108.30	110.20		
Weight of Dry Sample (g)	438.70	247.62	368.30	384.39	349.11	353.58		
Container No.:								
After Wash								
Method (A or B)	B	B	B	B	B	B		
Dry Weight of Sample + Cont. (g)	393.40	154.80	291.80	214.50	311.00	427.80		
Weight of Container (g)	235.20	108.80	249.00	107.40	108.30	110.20		
Dry Weight of Sample (g)	158.20	46.00	42.80	107.10	202.70	317.60		
% Passing No. 200 Sieve	**63.9**	**81.4**	**88.4**	**72.1**	**41.9**	**10.2**		
% Retained No. 200 Sieve	36.1	18.6	11.6	27.9	58.1	89.8		

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: West Side Lofts
Project No.: 012184-001
Client Name: L & A / Irvine
Tested By: G. Berdy Date: 06/06/07

ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name:	West Side Lofts	Tested By:	G. Bathala	Date:	06/05/07
Project No. :	012184-001	Input By:	LF	Date:	06/06/07
Boring No.:	B-6	Checked By:	LF		
Sample No.:	R-2	Depth (ft.)	5.0		

Soil Identification: Olive brown lean clay with sand (CL)s

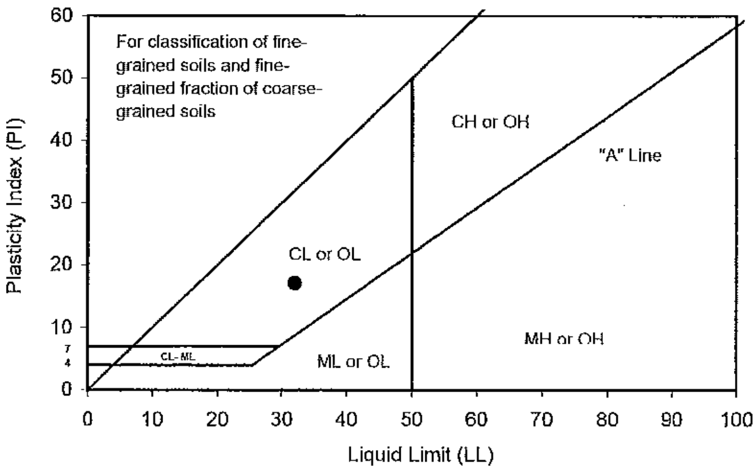
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			29	23	18	
Wet Wt. of Soil + Cont. (g)	10.91	11.82	17.38	16.25	15.93	
Dry Wt. of Soil + Cont. (g)	9.64	10.42	13.52	12.54	12.20	
Wt. of Container (g)	1.06	1.04	1.10	1.12	1.06	
Moisture Content (%) [Wn]	14.80	14.93	31.08	32.49	33.48	

Liquid Limit	**32**
Plastic Limit	**15**
Plasticity Index	**17**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 8.76

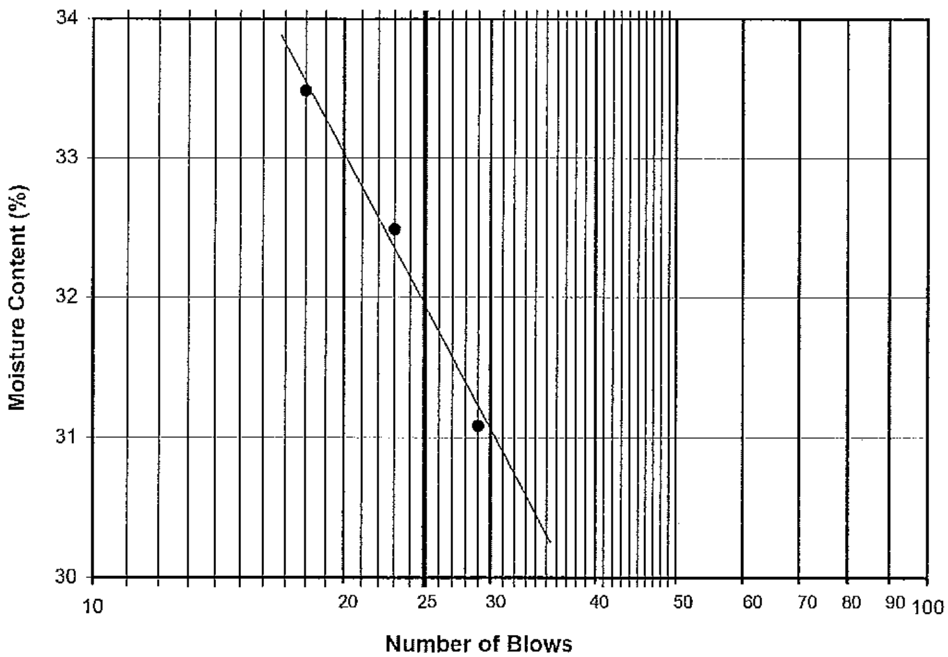
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.12}$



PROCEDURES USED

- [ ] Wet Preparation Multipoint - Wet
- [x] Dry Preparation Multipoint - Dry
- [x] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Leighton

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	West Side Lofts	Tested By:	G. Berdy	Date:	05/31/07
Project No. :	012184-001	Checked By:	LF	Date:	06/05/07
Boring No.:	B-1	Depth (ft.)	0-5		
Sample No. :	B1				
Soil Identification:	Dark yellowish brown clayey sand (SC)				

Dry Wt. of Soil + Cont.	(g)	1000.00
Wt. of Container No.	(g)	0.00
Dry Wt. of Soil	(g)	1000.00
Weight Soil Retained on #4 Sieve		0.00
Percent Passing # 4		100.00

MOLDED SPECIMEN		Before Test	After Test
Specimen Diameter	(in.)	4.01	4.01
Specimen Height	(in.)	1.0000	1.0040
Wt. Comp. Soil + Mold	(g)	632.20	451.50
Wt. of Mold	(g)	208.10	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	841.60	659.60
Dry Wt. of Soil + Cont.	(g)	779.30	600.80
Wt. of Container	(g)	0.00	208.10
Moisture Content	(%)	7.99	14.97
Wet Density	(pcf)	127.9	135.6
Dry Density	(pcf)	118.5	118.0
Void Ratio		0.423	0.429
Total Porosity		0.297	0.300
Pore Volume	(cc)	61.5	62.4
Degree of Saturation (%) [S_{meas}]		**51.0**	94.3

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
05/31/07	13:39	1.0	0	0.1500
05/31/07	13:49	1.0	10	0.1490
Add Distilled Water to the Specimen				
05/31/07	13:55	1.0	6	0.1510
06/01/07	6:54	1.0	1025	0.1540
06/01/07	8:22	1.0	1113	0.1540

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	5.0
Expansion Index (EI_{50}) = EI_{meas} - (50 - S_{meas})x((65+EI_{meas}) / (220-S_{meas}))	**5**

EXPANSION INDEX of SOILS

ASTM D 4829

Leighton

Project Name:	West Side Lofts	Tested By:	G. Berdy	Date:	05/31/07
Project No. :	012184-001	Checked By:	LF	Date:	06/05/07
Boring No.:	B-6	Depth (ft.)	0-5		
Sample No. :	B1				
Soil Identification:	Dark yellowish brown clayey sand (SC)				

Dry Wt. of Soil + Cont.	(g)	1000.00
Wt. of Container No.	(g)	0.00
Dry Wt. of Soil	(g)	1000.00
Weight Soil Retained on #4 Sieve		0.00
Percent Passing # 4		100.00

MOLDED SPECIMEN		Before Test	After Test
Specimen Diameter	(in.)	4.01	4.01
Specimen Height	(in.)	1.0000	1.0145
Wt. Comp. Soil + Mold	(g)	621.20	458.40
Wt. of Mold	(g)	192.00	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	820.50	650.40
Dry Wt. of Soil + Cont.	(g)	752.70	585.80
Wt. of Container	(g)	0.00	192.00
Moisture Content	(%)	9.01	16.40
Wet Density	(pcf)	129.5	136.3
Dry Density	(pcf)	118.8	117.1
Void Ratio		0.419	0.440
Total Porosity		0.295	0.305
Pore Volume	(cc)	61.2	64.1
Degree of Saturation (%) [S_{meas}]		**58.0**	100.7

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
05/31/07	11:15	1.0	0	0.2375
05/31/07	11:25	1.0	10	0.2370
Add Distilled Water to the Specimen				
05/31/07	11:34	1.0	9	0.2405
06/01/07	6:55	1.0	1170	0.2520
06/01/07	8:11	1.0	1246	0.2520

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	15.0
Expansion Index (EI_{50}) = EI_{meas} - (50 - S_{meas})x((65+EI_{meas}) / (220-S_{meas}))	**19**

MODIFIED PROCTOR COMPACTION TEST

Leighton

ASTM D 1557

Project Name:	West Side Lofts	Tested By :	R. Santos	Date:	06/05/07
Project No.:	012184-001	Input By :	LF	Date:	06/06/07
Boring No.:	B-1	Depth (ft.)	0-5		
Sample No. :	B1				
Soil Identification:	Dark yellowish brown clayey sand (SC)				

Preparation Method: [X] Moist [ ] Dry — [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03321 — *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3678.0	3795.0	3892.0	3818.0		
Weight of Mold (g)	1786.0	1786.0	1786.0	1786.0		
Net Weight of Soil (g)	1892.0	2009.0	2106.0	2032.0		
Wet Weight of Soil + Cont. (g)	552.80	516.30	478.30	488.20		
Dry Weight of Soil + Cont. (g)	534.50	488.70	443.40	444.40		
Weight of Container (g)	54.20	53.90	52.90	54.50		
Moisture Content (%)	3.81	6.35	8.94	11.23		
Wet Density (pcf)	125.6	133.4	139.8	134.9		
Dry Density (pcf)	121.0	125.4	128.3	121.3		

Maximum Dry Density (pcf) **128.5** **Optimum Moisture Content (%)** **8.5**

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

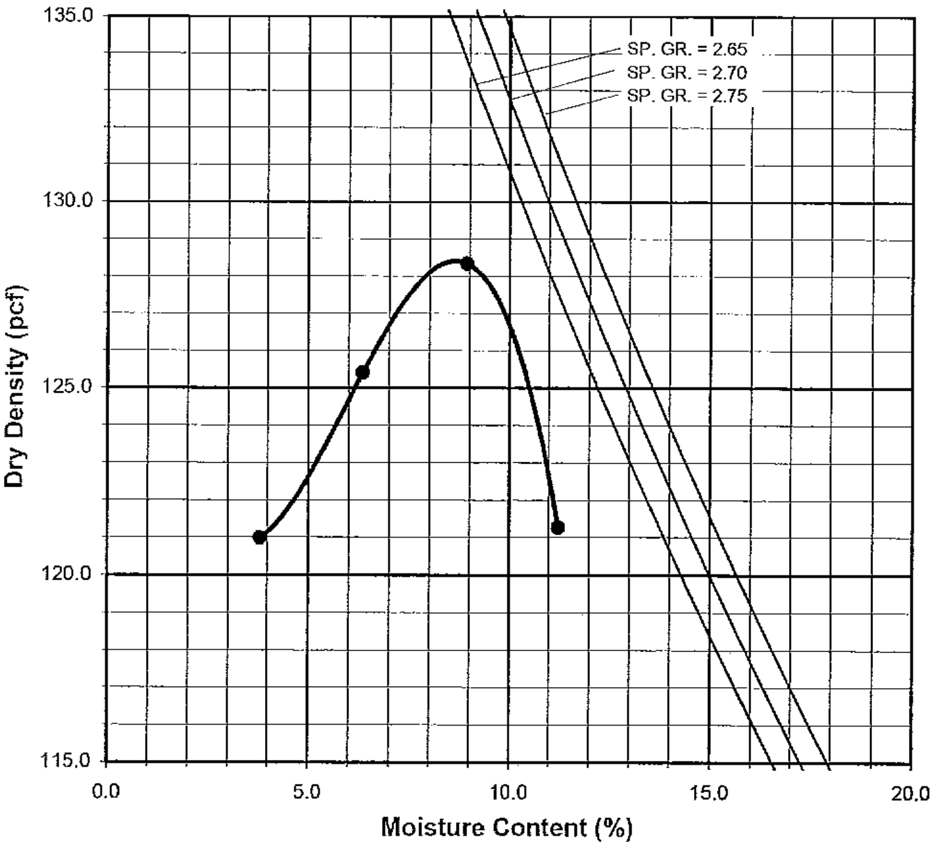
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



MODIFIED PROCTOR COMPACTION TEST

Leighton

ASTM D 1557

Project Name: West Side Lofts | Tested By : R. Santos | Date: 06/06/07

Project No.: 012184-001 | Input By : LF | Date: 06/07/07

Boring No.: B-1 | Depth (ft.) 10-15

Sample No. : B2

Soil Identification: Dark grayish brown lean clay with sand (CL)s

Preparation Method: [X] Moist [ ] Dry — [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft^3) 0.03321 — *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3687.0	3831.0	3849.0	3790.0		
Weight of Mold (g)	1786.0	1786.0	1786.0	1786.0		
Net Weight of Soil (g)	1901.0	2045.0	2063.0	2004.0		
Wet Weight of Soil + Cont. (g)	514.30	489.60	449.50	424.50		
Dry Weight of Soil + Cont. (g)	484.10	451.30	406.10	376.30		
Weight of Container (g)	55.30	54.10	51.20	53.90		
Moisture Content (%)	7.04	9.64	12.23	14.95		
Wet Density (pcf)	126.2	135.8	136.9	133.0		
Dry Density (pcf)	117.9	123.8	122.0	115.7		

Maximum Dry Density (pcf) 124.0 **Optimum Moisture Content (%)** 10.0

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

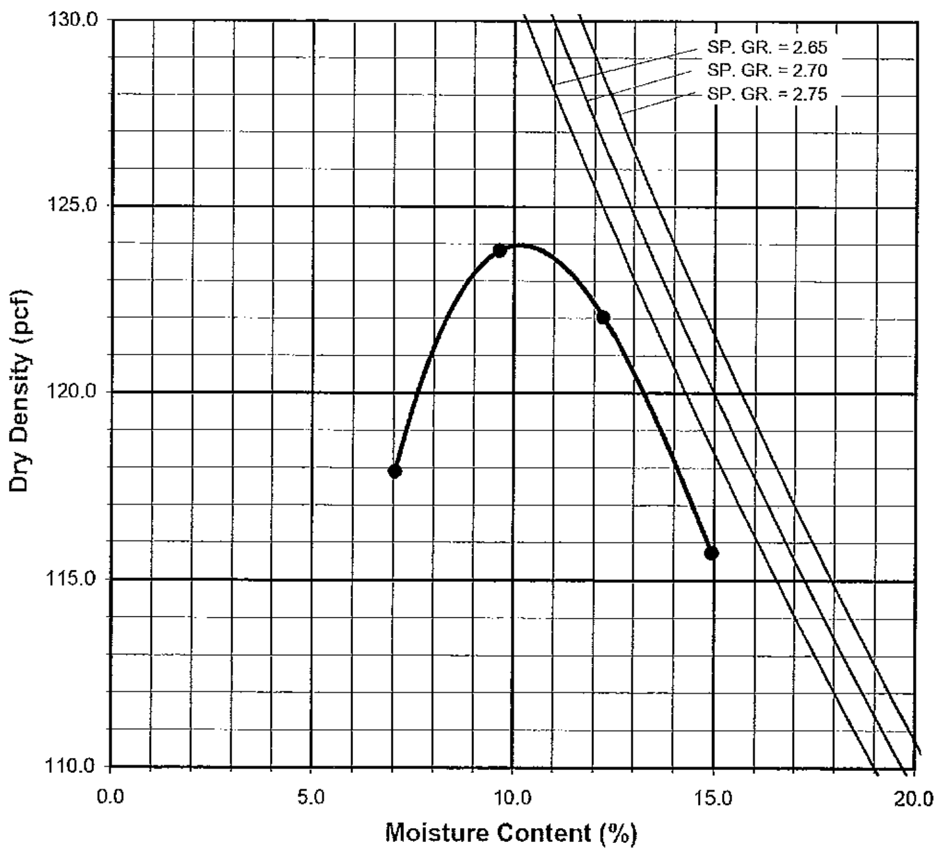
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

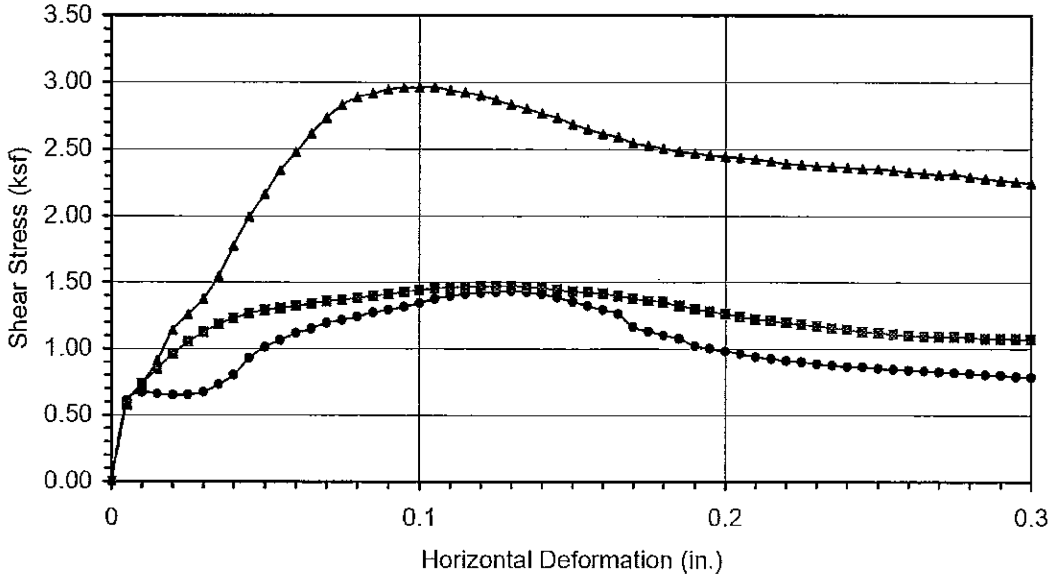
GR:SA:FI

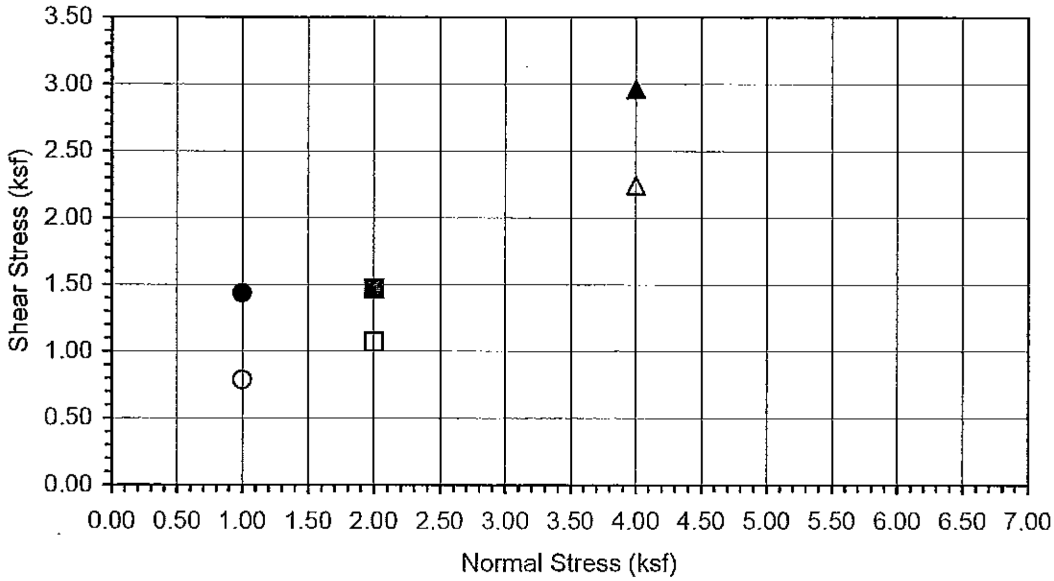
Atterberg Limits:

LL,PL,PI



MX B-1 B2





Boring No.	**B-1**
Sample No.	**R-3**
Depth (ft)	**10**

Sample Type:

Drive

Soil Identification:

Yellowish brown lean / silty clay (CL/CL-ML)

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 1.437	■ 1.474	▲ 2.965
Shear Stress @ End of Test (ksf)	○ 0.786	□ 1.072	△ 2.245
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	25.56	25.56	25.56
Dry Density (pcf)	97.7	100.1	100.5
Saturation (%)	95.2	101.0	101.9
Soil Height Before Shearing (in.)	0.9964	0.9866	0.9780
Final Moisture Content (%)	32.3	26.3	25.0

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Undrained

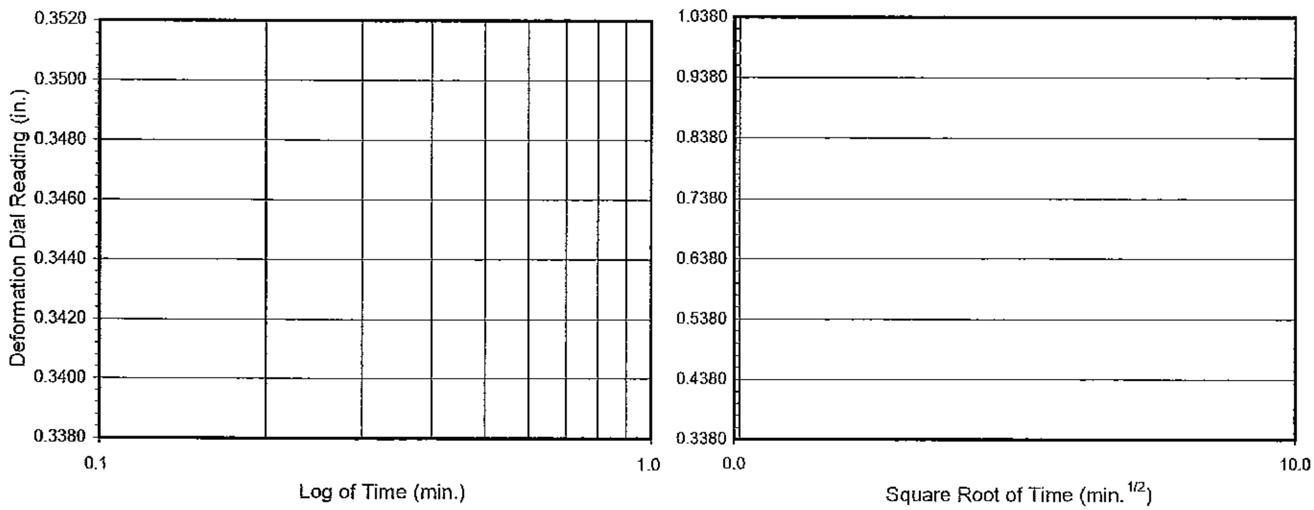
Project No.: 012184-001

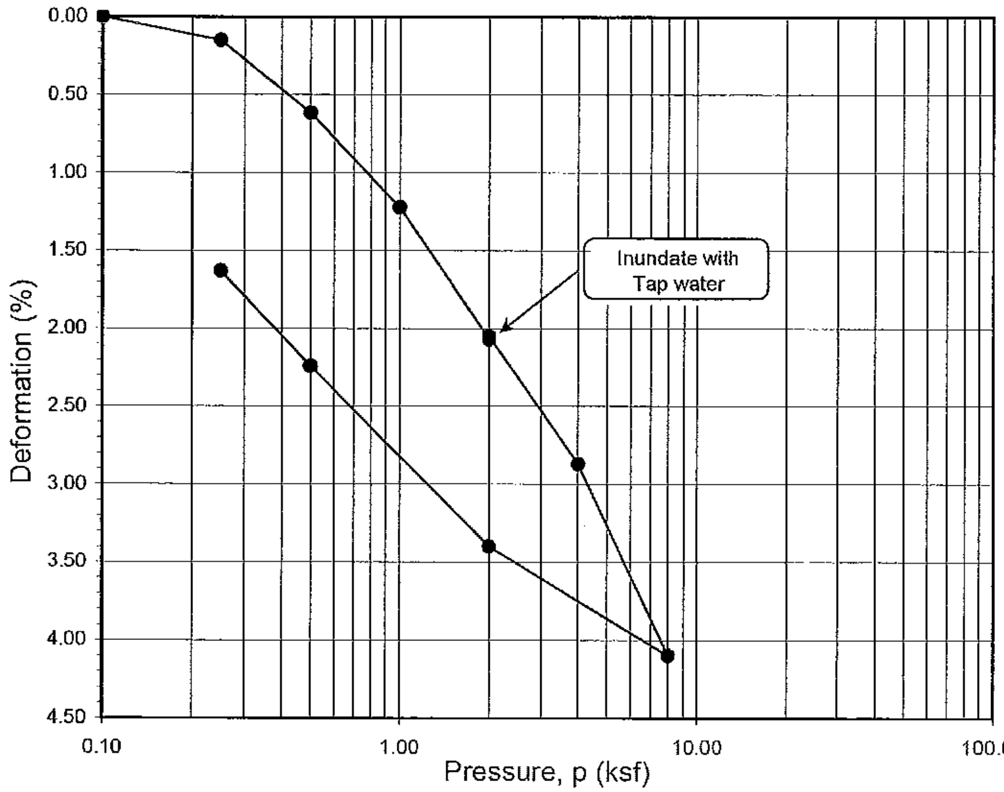
West Side Lofts

05-07

DS B-1 R-3

No Time Readings





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
B-1	**R-4**	**15**	**27.3**	**27.7**	**97.3**	**99.7**	**0.732**	**0.704**	**100**	**100**

Soil Identification: Light olive brown silt (ML)

Leighton

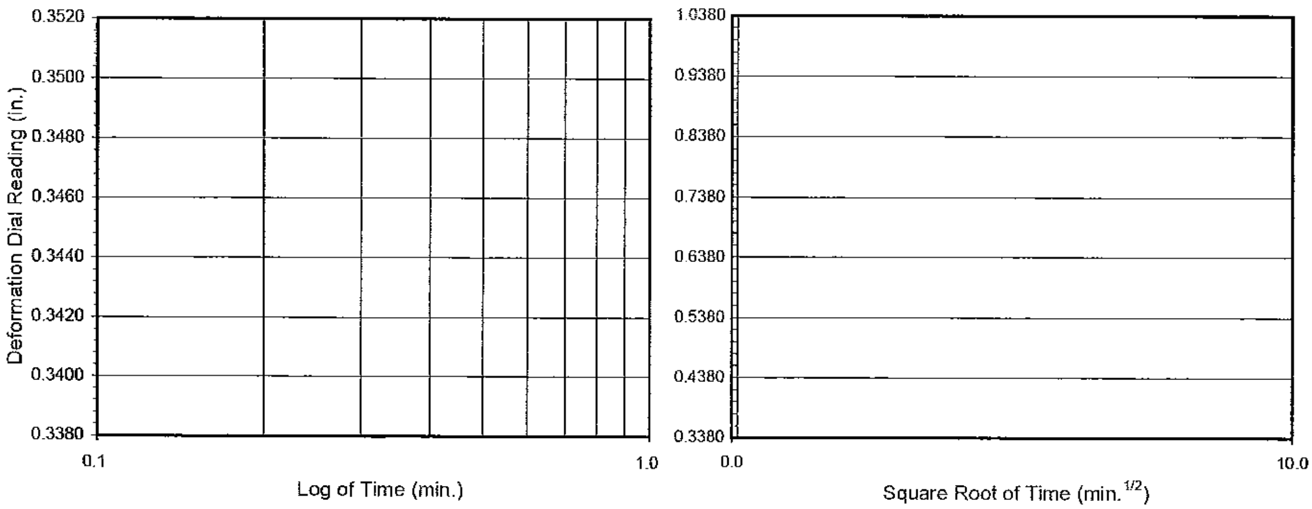
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
(ASTM D 2435)

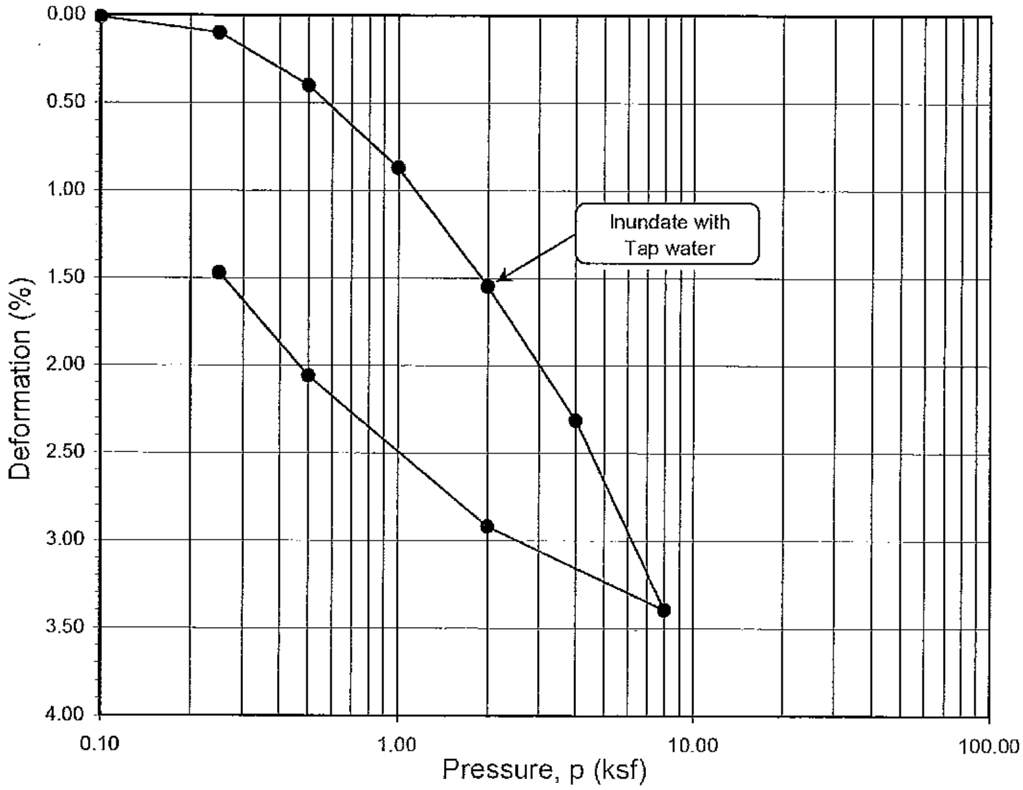
Project No.: 012184-001

West Side Lofts

06-07

No Time Readings





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
B-6	**R-3**	**10**	**18.1**	**22.2**	**103.4**	**104.6**	**0.630**	**0.606**	**78**	**98**

Soil Identification: Brown silt with sand (ML)s

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
(ASTM D 2435)

Project No.: 012184-001

West Side Lofts

06-07

Leighton

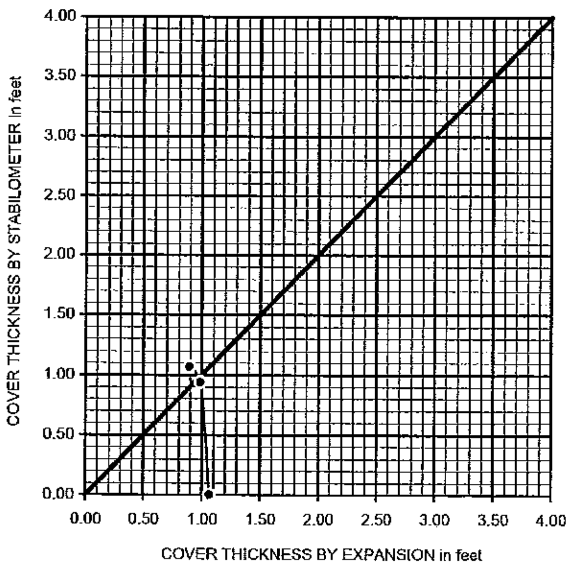
R-VALUE TEST RESULTS

PROJECT NAME:	West Side Lofts	PROJECT NUMBER:	012184-001
SAMPLE NUMBER:	B1	SAMPLE LOCATION:	B-1 @ 0-5'
SAMPLE DESCRIPTION:	SC	TECHNICIAN:	SCF
SAMPLED BY:	CDL	DATE COMPLETED	6/5/2007

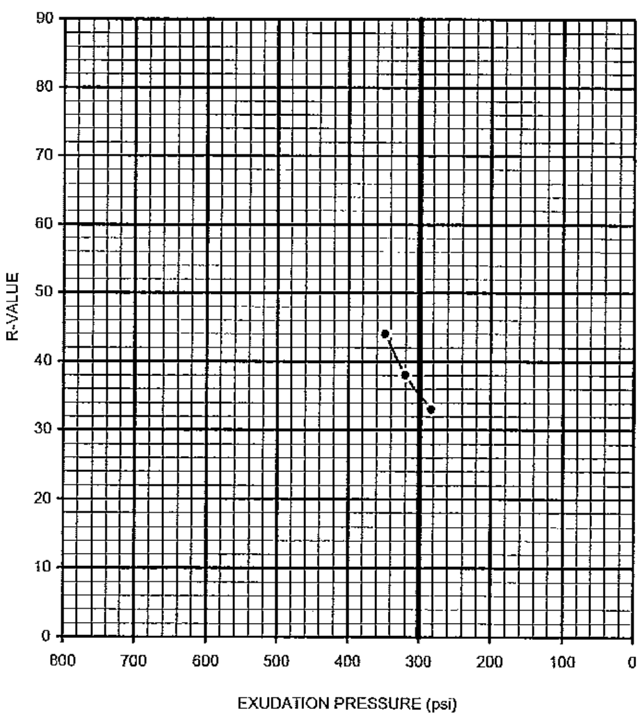
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	11.8	12.1	12.3
HEIGHT OF SAMPLE, Inches	2.49	2.47	2.51
DRY DENSITY, pcf	122.3	122.3	121.0
COMPACTOR PRESSURE, psi	75	50	50
EXUDATION PRESSURE, psi	348	320	282
EXPANSION, Inches x 10exp-4	32	28	0
STABILITY Ph 2,000 lbs (160 psi)	73	82	87
TURNS DISPLACEMENT	3.75	3.87	4.22
R-VALUE UNCORRECTED	44	38	33
R-VALUE CORRECTED	44	38	33

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.90	0.99	1.07
EXPANSION PRESSURE THICKNESS, ft.	1.07	0.93	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	39
R-VALUE BY EXUDATION:	36
EQUILIBRIUM R-VALUE:	36

SOIL RESISTIVITY TEST
DOT CA TEST 532 / 643

Leighton

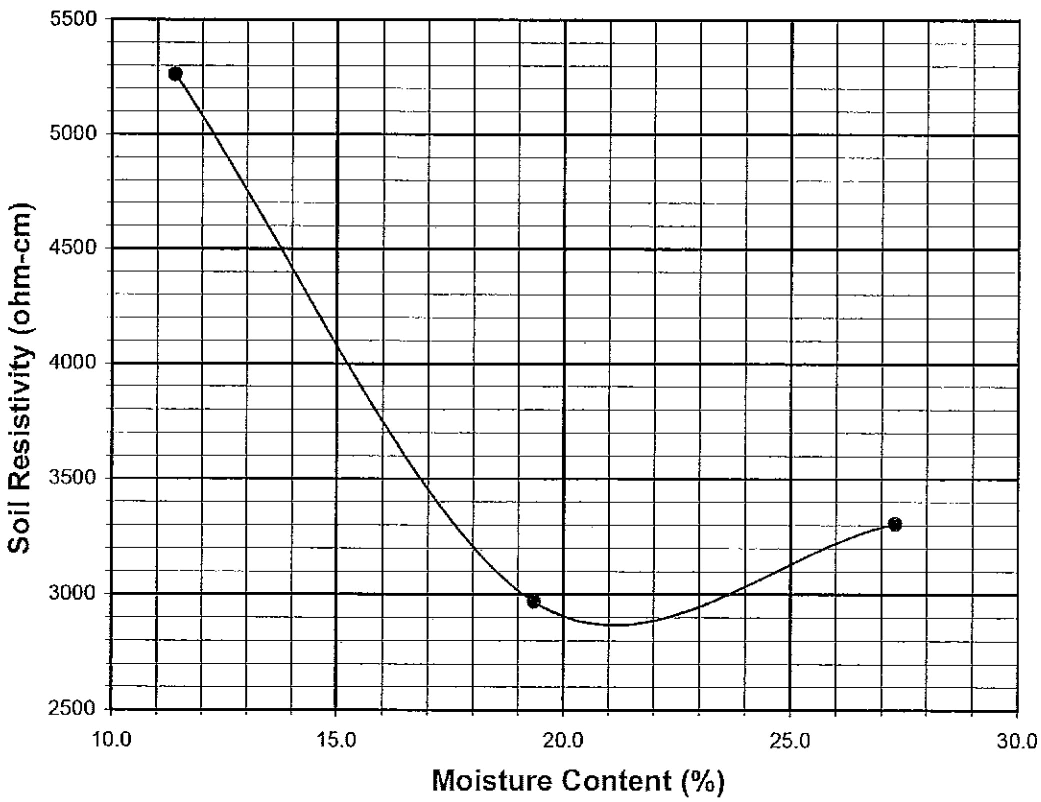
Project Name: West Side Lofts
Project No. : 012184-001
Boring No.: B-8
Sample No. : B1
Soil Identification: (SC)g

Tested By : VJ Date: 05/31/07
Data Input By: LF Date: 06/05/07
Depth (ft.) : 0-5

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	100	11.39	780	5262
2	200	19.35	440	2968
3	300	27.30	490	3306
4				
5				

Moisture Content (%) (MCi)	3.43
Wet Wt. of Soil + Cont. (g)	215.85
Dry Wt. of Soil + Cont. (g)	209.96
Wt. of Container (g)	38.41
Container No.	
Initial Soil Wt. (g) (Wt)	1300.00
Box Constant	6.746
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 532 / 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 532 / 643	
2870	**21.0**	**111**	**52**	**8.19**	**21.8**



Leighton

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: West Side Lofts — Tested By: VJ — Date: 05/31/07

Project No.: 012184-001 — Data Input By: LF — Date: 06/05/07

Boring No.	B-8			
Sample No.	B1			
Sample Depth (ft)	0-5			
Soil Identification:	(SC)g			
Wet Weight of Soil + Container (g)	215.85			
Dry Weight of Soil + Container (g)	209.96			
Weight of Container (g)	38.41			
Moisture Content (%)	3.43			
Weight of Soaked Soil (g)	100.74			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	3			
Crucible No.	20			
Furnace Temperature (°C)	840			
Time In / Time Out	7:25 / 8:10			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	21.2258			
Wt. of Crucible (g)	21.2232			
Wt. of Residue (g) (A)	0.0026			
PPM of Sulfate (A) x 41150	106.99			
PPM of Sulfate, Dry Weight Basis	**111**			

CHLORIDE CONTENT, DOT California Test 422

ml of Chloride Soln. For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.7			
PPM of Chloride (C -0.2) * 100 * 30 / B	50			
PPM of Chloride, Dry Wt. Basis	**52**			

pH TEST, DOT California Test 532/643

pH Value	**8.19**			
Temperature °C	21.8			