September 22, 2010
Project No. 207247038

Ms. Alicia Ramos
County of Los Angeles Public Works
900 South Fremont Avenue, 5th Floor
Alhambra, California 91803-1331

Subject: Geotechnical Update Evaluation
Hall of Justice Repair and Reuse Project
211 West Temple Street
Los Angeles, California
Contract No. PW13097
Work Authorization No. ANMCP-00037

Dear Ms. Ramos:

In accordance with your request, Ninyo & Moore has performed a geotechnical update evaluation for planned improvements at the Hall of Justice located at 211 West Temple Street in Los Angeles, California (Figure 1). A geotechnical evaluation was previously performed for the Hall of Justice for a proposed parking structure at the site (Converse Consultants, 2003). The referenced previous report presented the results of their evaluation and included geotechnical recommendations pertaining to the design and construction of the parking structure. This report also stated that a separate report was in progress regarding adverse geologic bedding and temporary excavations. The supplemental report was not available for review. The purpose of our update evaluation was to evaluate the current site conditions relative to the previous geotechnical recommendations and to provide supplemental design recommendations, as appropriate. In addition, we also performed percolation testing at the site.

We understand that the proposed improvements will include a new parking structure and entry plaza. The new parking structure will be located on the north side of the existing Hall of Justice Building. The structure will have nine parking levels. Approximately half of the parking levels will be below the ground surface. The new entry plaza will be located on the east side of the Hall of Justice. The entry plaza will include concrete steps, hardscape and lawn areas. We also understand that the lawn areas of the plaza will include on-site infiltration systems for storm water runoff. At the time of our evaluation no plans or details for the parking structure, entry plaza or infiltration systems were available for review.

SCOPE OF SERVICES

Our scope of services included the following:

- Review of readily available background materials including State of California Seismic Hazard Zone maps, State of California Earthquake Fault Zone maps (Alquist-Priolo Special Studies Zones maps), topographic maps, published geologic maps and literature, and review of the referenced project geotechnical report by Converse Consultants.

- Geotechnical site reconnaissance to mark boring locations and to coordinate with on-site personnel and Underground Service Alert for underground utility location.

- Subsurface exploration consisting of excavation, logging, and sampling of three small-diameter hollow-stem auger borings drilled to depths ranging from approximately 16½ to feet to 81½ feet. The borings were logged by a representative from our firm, and bulk and relatively undisturbed soil samples were collected at selected intervals for laboratory testing.

- Cone penetrometer testing (CPT) at three locations on site. The CPT holes were advanced to refusal, which occurred at depths between approximately 35.9 to 72.6 feet. At one location down-hole shear wave testing was also performed.

- Field percolation testing at two boring locations to depths of approximately 16½ feet.

- Laboratory testing of selected samples to evaluate in-situ moisture and dry density, Atterberg Limits, direct shear strength, expansion index, R-value, pH, soluble sulfate content, resistivity, and chloride content.

- Data compilation and geotechnical analysis of the field and laboratory data.

- Preparation of a letter report to present our findings, conclusions, and recommendations for the project.

SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface evaluation was performed on August 4 and 5, 2010, and consisted of the drilling, logging, and sampling of three small-diameter borings and three CPT holes. The borings were drilled with a truck-mounted drill rig utilizing 8-inch-diameter, hollow-stem augers. Borings B-1 and B-2 were drilled to a depth of approximately 16½ feet and boring B-3 was drilled to a depth of approximately 81½ feet. Cone penetrometer testing was performed using a 30 ton CPT rig.

CPT holes, C-1, C-2 and C-3, were advanced to refusal at approximate depths of 72.6, 50.2 and 35.9 feet, respectively. Down-hole shear wave testing was performed in C-2. A representative from our firm logged the borings and obtained bulk and relatively undisturbed soil samples at selected depths for laboratory testing. The approximate locations of our borings and CPT holes are presented on Figure 2. Logs of the borings are presented in Appendix A. The results of the CPT exploration are presented in Appendix B

Geotechnical laboratory testing of selected soil samples included tests to evaluate in-situ moisture and density, Atterberg Limits, shear strength, expansion index, R-value, and soil corrosivity characteristics. Laboratory test results are presented on the boring log in Appendix A and in Appendix C.

GEOLOGY AND SUBSURFACE CONDITIONS

Based upon the results of our geologic background review and our subsurface evaluation, the site is generally underlain by fill soil overlying weathered sedimentary deposits of the Puente Formation.

Fill soil was encountered in our exploratory borings B-1 and B-2 to depths of approximately 5½ and 2 feet, respectively. The fill material was comprised of firm silty clay. The previous geotechnical evaluation reported fill depths ranging from approximately 2½ to 15 feet on site.

Puente Formation material was encountered beneath the fill in borings B-1 and B-2 and below the pavement in boring B-3 to the depths explored up to approximately 81½ feet. The Puente Formation generally consisted of thinly bedded, weakly indurated, soft to moderately hard, claystone and siltstone with occasional sandstone layers. The previous geotechnical exploration at the site by Converse Consultants included down-hole logging of large diameter borings. Geologic data reported indicates that the geologic structure includes bedding that strikes east-west and dips approximately 40 to 55 degrees to the south. Detailed descriptions are presented on the boring logs presented in Appendix A and in the referenced geotechnical report (Converse Consultants, 2003).

GROUNDWATER

At the time of our subsurface evaluation, seepage was encountered in boring B-3 at depths of approximately 30, 35½, 40 and 50 feet. Groundwater was also measured at a depth of approximately 27½ feet in an on-site piezometer previously installed (BH-2 by Converse Consulting, 2003). The previous geotechnical report also indicated multiple zones of seepage in exploratory borings ranging from approximately 16 to 65 feet deep. Review of readily available literature indicates that the historical groundwater elevation in the vicinity of the site is approximately 20 feet below the ground surface. Variations in groundwater depths due to various factors, including seasonal variations, groundwater pumping, and irrigation, will occur.

PERCOLATION TESTING

Percolation testing was performed in borings B-1 and B-2 on August 4 and 5, 2010. The testing was performed through slotted 2-inch-diameter polyvinyl chloride (PVC) pipe placed to the bottom of each boring (16½ feet) and backfilled with No. 3 Monterey sand. The borings were then filled of water to pre-soak the adjacent soils. After a pre-saturation period of approximately 24 hours, percolation testing was performed. Percolation testing consisted of filling the borings with water and measuring the drop in the water level through the perforated pipe. The percolation test data as well as the calculated percolation rates in accordance with County of Orange On-Site Sewage Guidelines (County of Orange) are presented in Tables 1 and 2.

Table 1 – Percolation Test Data - Boring B-1

Time Interval (minutes)	Drop in Water Level (inches)	Percolation Rate (gallons/ft^2/day)
58	1.33	0.5
59	0.66	0.3
57	0.40	0.2
60	0.32	0.1

Table 2 – Percolation Test Data - Boring B-2

Time Interval (minutes)	Drop in Water Level (inches)	Percolation Rate (gallons/ft^2/day
66	0.78	0.3
54	2.06	1.0
60	0.79	0.4
60	2.48	1.1
60	0.85	0.4

CONCLUSIONS

As requested by the County of Los Angeles, our update geotechnical study was intended to provide data to evaluate the site conditions compared to the conditions reported in the previous project geotechnical report (Converse Consultants, 2003). Based on the results of our current geotechnical evaluation, is our opinion that the site subsurface conditions are generally similar to the conditions presented in the previous project geotechnical report. Furthermore, it is our opinion that the planned improvements are feasible from a geotechnical perspective, provided the recommendations in the report prepared by Converse Consultants (2003) and as updated herein are incorporated into the design and construction of the project. Based on our update geotechnical evaluation the following conclusions were reached.

- The site is underlain by variable depths of undocumented older fill soils and formational deposits of the Puente Formation. The Puente Formation is comprised of interbedded, soft to moderately hard siltstone, claystone and sandstone to the depths explored.

- Undocumented fill soils are not considered suitable for support of foundations. We anticipate the below grade parking structure will bear on formational deposits of the Puente Formation. Existing fill soils should be removed and re-compacted for support of other new foundations.

- The Puente Formation is reported to dip approximately 40 to 55 degrees to the south, which is considered an adverse geologic structure with respect to some temporary excavations on site.

- Groundwater was encountered at variable depths during the current and previous site exploration ranging from approximately 16 to 65 feet deep. During the previous exploration the groundwater was observed occurring as seepage along bedding planes. The depth to groundwater was measured in a piezometer on site at approximately 27½ feet. A groundwater depth of approximately 15 feet should be considered for construction dewatering and for design of below grade structures.

- The more clayey soils on site have a medium potential for expansion.
- The subject site is not located within a State of California Earthquake Fault Zone (formerly known as an Alquist-Priolo Special Studies Zone) (Hart and Bryant, 1997). The potential for fault rupture on site is considered low.
- The probabilistic PGA_{MCE} for the site was calculated as 0.82 g using the United States Geological Survey (USGS, 2008) ground motion calculator (web-based). The design PGA was estimated to be 0.55 g using the USGS ground motion calculator.

SUPPLEMENTAL RECOMMENDATIONS

In general, the recommendations presented in the previous geotechnical report are considered applicable for the project (Converse Consultants, 2003). At the time this report was prepared detailed plans for the parking structure and other site improvements were not available for review. The project plans should be reviewed by our office as they become available and based on the plan review the geotechnical recommendations for the project may be updated as appropriate. The following supplemental recommendations are presented based on our update evaluation and project understanding.

Earthwork

The earthwork recommendations presented in the previous project report generally remain applicable for the project along with the following supplemental recommendations. Existing undocumented fill soils, which are present after planned excavations are made, should be removed and re-compacted to provide suitable support for new foundations. We anticipate that excavations for the new parking structure will expose undisturbed formational materials. Re-compaction of existing fills should be performed for new foundations that may be associated with entry plaza improvements, such as garden walls, retaining walls or other structural improvements. The fill soils should be compacted to 90 percent or more in accordance with American Society for Testing and Materials (ASTM) D 1557.

In hardscape areas we recommend that existing fill soils and/or loose natural soils be excavated and re-compacted to a depth of approximately three feet below the planned finish grades. In addition, to mitigate the potential impacts of expansive soils on site, we recommend that the upper approximately 18 inches of soil beneath exterior slabs-on-grade consist of compacted, non-expansive (Expansion Index of 20 or less) granular on site or imported soil. Imported soil should be evaluated by the geotechnical consultant prior to importing to the site.

Foundations

Foundations for the parking structure should bear in undisturbed formational material and may be designed in accordance with the recommendations of the previous project geotechnical report (Converse Consultants, 2003). Shallow footing foundations for new walls that may be associated with the entry plaza should be founded in engineered compacted fill soil or undisturbed formational material a depth of approximately 24 inches or more below the planned finish grades. Footings should have a width of 24 inches or more. Footings founded as recommended may be designed for an allowable soil bearing pressure of 2,500 pounds per square foot (psf). The allowable bearing pressure may be increased by up to one-third when considering loads of short duration, such as wind and seismic forces.

Foundations should be reinforced in accordance with the recommendations of the project structural engineer. We recommend that, as a minimum, continuous footings be reinforced with two No. 4 reinforcing steel bars, one placed near the top of the footing and one placed near the bottom. Due to the potential for corrosion, reinforcing bars should be covered by 3 or more inches of concrete.

Shallow foundations placed in compacted fill soils or formational material may be designed using a coefficient of friction of 0.35 (total frictional resistance equals the coefficient of friction times the dead load) at the concrete/soil interface. A design passive resistance value of 350 pounds per square foot of depth for level soil (with a maximum value of 3,500 pounds per square foot) may be used. The allowable lateral resistance can be taken as the sum of the friction resistance and passive resistance, provided the passive resistance does not exceed one-half the total allowable resistance. Passive resistance values may be increased by one-third when considering loads of short duration, such as wind or seismic loads.

Slabs-on-Grade

The recommendations provided in the previous project geotechnical report (Converse Consultants, 2003) are generally applicable for the design of slabs-on-grade. However, in areas where moisture-sensitive floor coverings are used, we recommend that a 10-mil thick polyethylene vapor barrier overlying a six-inch-thick open graded gravel (up to ¾ inch) be placed between the subgrade soil and the slab. In addition, a layer of two inches of sand should also be placed above the barrier to aid concrete curing.

Screen Walls and Retaining Walls

Screen walls and at grade retaining walls may be supported by foundations designed in accordance with the recommendations presented in the preceding section of this report. Lateral earth pressures recommended for design of yielding retaining walls are provided on Figure 4. Please note that the dynamic pressure presented on Figure 4 applies to retaining walls higher than 12 feet in accordance with the 2007 California Building Code (CBC). Passive pressures may be increased by one third when considering loads of short duration, including wind and seismic loads. Further, for sliding resistance, a friction coefficient of 0.35 may be used for the concrete and soil interface. The allowable resistance may be taken as the sum of the frictional and passive resistance, provided that the passive portion does not exceed one-half of the total allowable resistance.

Retaining walls should be backfilled with free-draining, granular, non-expansive imported soil (Expansion Index 20 or less). Measures should be taken to reduce the potential for build-up of moisture behind the retaining walls. Drainage design should include free-draining backfill materials and subsurface drainage provisions as shown on Figure 5.

Exterior Flatwork

Exterior flatwork should be supported on compacted non-expansive soils prepared in accordance with the earthwork recommendations presented in the preceding section of this report. Exterior flatwork should have a thickness of 4 inches or more. The flatwork should be reinforced with No. 4 steel reinforcing bars placed 24 inches on-center (each way) near the mid-height of the slab.

To reduce the potential for distress to exterior concrete flatwork due to movement of the underlying soil, we recommend that flatwork be installed with crack-control joints at an appropriate spacing as designed by the structural engineer. Exterior flatwork should be underlain by 2 inches of clean sand. We also recommend that exterior slabs be doweled to adjoining curbs, building walls, or other structures. Positive drainage should be established and maintained adjacent to flatwork.

Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 4 presents the seismic design parameters for the site in accordance with CBC (2007) guidelines and mapped spectral acceleration parameters (United States Geological Survey [USGS], 2008).

Table 4 – 2007 California Building Code Seismic Design Coefficients

Seismic Design Factors	Value
Site Class	C
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.3
Mapped Spectral Acceleration at 0.2-second Period, S_s	2.05g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.700g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	2.049g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.91g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.366g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.607g

Site-Specific Ground Motion Analysis

As a part of the seismic evaluation update, we also performed site-specific ground motion analysis in accordance with ASCE 7-05 Chapter 21 procedures. The following assumptions and procedures were used in our analysis: 1) The probabilistic MCE spectral response accelerations were taken as the mean (50th-percentile) values among the three attenuation relationships (Abrahamson and Silva, 1999, Campbell, 1997 and Sadigh, 1997) using the Open Seismic Hazard Analysis (OPENSHA) program (USGS, 2010); 2) The deterministic MCE response acceleration at each period was calculated as 150 percent of the largest median 5 percent damped spectral response acceleration computed with a maximum magnitude 7.3 and a site-to-fault distance of 5.22 km. These values were compared to the values computed in accordance with Figure 21.2-1 of ASCE 7-05 using Fa = 1.0 and Fv = 1.3. The larger value at each period was used as the deterministic MCE spectral response acceleration; 3) The site-specific MCE spectral response acceleration (SaM) at each period was taken as the lesser between values of 1) and 2); 4) The design spectral response acceleration at each period was taken as the maximum between two-third of SaM and 80 percent of Sa evaluated in accordance with ASCE 7-05 Section 11.4.5. Results of our analysis are shown on Figure 6.

Infiltration System Design Criteria

Based on the percolation testing, the percolation rate of the materials encountered to a depth of approximately 16.5 feet at boring locations B-1 and B-2 was approximately 0.1 to 1.0 gallons/ft^2/day. Due to variable subsurface conditions percolation rates will vary within the materials encountered at the site.

The design of on-site infiltration systems should consider that the interbedded formational materials are conducive to lateral migration of water along bedding planes and fracture systems. The design of the infiltration systems should include evaluation of existing and planned below grade structures, including the new parking structure, existing basements and existing tunnels with regard to lateral migration of infiltration water. We recommend that our office be consulted when designing on-site infiltration systems.

Corrosivity

Laboratory testing was performed on a representative sample of near-surface soil to evaluate soil pH, electrical resistivity, water-soluble chloride content, and water-soluble sulfate content. The soil pH and electrical resistivity tests were performed in general accordance with California Test Method (CT) 643. Chloride content tests were performed in general accordance with CT 422. Sulfate testing was performed in general accordance with CT 417. The laboratory test results are presented in Appendix C.

The soil pH and electrical resistivity were measured to be approximately 7.2 and 350 ohm-centimeters, respectively. The chloride content of the sample was approximately 100 ppm. The sulfate content of the tested sample was approximately 0.57 percent by weight (i.e., 5,700 ppm). Based on the laboratory test results and Caltrans (2003) corrosion criteria, the project site can be classified as a corrosive site, which is defined as having earth materials with greater than 500 ppm chlorides, greater than 0.20 percent sulfates (i.e., 2,000 ppm), or a pH of 5.5 or less.

Concrete Placement

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical and/or physical deterioration. Based on the ACI criteria (ACI, 2008), the potential for sulfate attack is severe for water-soluble sulfate contents in soil ranging from 0.2 to 2.0 percent by weight (2,000 to 20,000 ppm). As indicated above, the soil sample tested for this evaluation indicates a water-soluble sulfate content of 0.57 percent by weight (i.e., 5,700 ppm). Accordingly, the on-site soils are considered to have a severe potential for sulfate attack. Accordingly, we recommend that Type V cement with a water/cement ratio of 0.45 or less be used for the project.

In order to reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the concrete be placed with a slump of 4 inches based on ASTM C 143. The slump should be checked periodically at the site prior to concrete placement. We also recommend that crack control joints be provided in sidewalks and exterior hardscape in accordance with the recommendations of the project structural engineer to reduce the potential for distress due to minor soil movement and concrete shrinkage. The project structural engineer should be consulted for additional concrete specifications.

We appreciate the opportunity to be of service on this project.

Respectfully submitted,
NINYO & MOORE

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Attachments: References
Figure 1 – Site Location
Figure 2 – Boring Location
Figure 3 – Fault Location
Figure 4 – Lateral Earth Pressures for Yielding Retaining Walls
Figure 5 – Retaining Wall Drainage Detail
Figure 6 – Acceleration Response Spectra
Attachment A – Boring Logs
Attachment B – Cone Penetrometer Tests
Attachment C – Laboratory Testing

Distribution: (2) Addressee

REFERENCES

American Concrete Institute, 2008, Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary, dated January.

American Society of Civil Engineers (ASCE), 2006, Minimum Design Loads for Building and other Structures, Standard 7-05.

Blake, T.F., 2001, FRISKSP (version 4.00), A Computer Program for the Probabilistic Estimation of Peak Acceleration and Uniform Hazard Spectra Using 3-D Faults as Earthquake Sources.

Boore, D.M., Joyner, W.B., and Fumal, T.E., 1997, Equations for Estimating Horizontal Response Spectra and Peak Acceleration from Western North American Earthquakes: A Summary of Recent Work, Seismological Research Letters, Vol. 68, No. 1, pp. 128-153.

Bozorgnia, Y., Campbell, K.W., and Niazi, M., 1999, Vertical Ground Motion: Characteristics, Relationship with Horizontal Component, and Building Code Implications, Proceedings of the SMIP99 Seminar on Utilization of Strong Motion Data, September 15, Oakland, pp. 23-49.

California Building Standards Commission, 2007, California Building Code, California Code of Regulations, Title 24, Part 2, Volume 2 of 2.

California Department of Conservation, Division of Mines and Geology, State of California, 1998, Seismic Hazard Evaluation of the Los Angeles 7.5-Minute Quadrangle, Los Angeles County, California: Open-File Report 98-20.

California Department of Conservation, Division of Mines and Geology, State of California, 1999, Seismic Hazard Zones Official Map, Los Angeles Quadrangle, 7.5-Minute Series: Scale 1:24,000, Open-File Report 98-20, dated March 25

California Department of Conservation, Division of Mines and Geology, 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada: International Conference of Building Officials, dated February.

California Department of Transportation, 2003, Corrosion Guidelines, Version 1.0, dated September.

California Geological Survey, 2002, Revised 2002 California Seismic Shaking Analysis, Appendix A.

Campbell, K.W., 1997, Empirical Near-Source Attenuation Relationships for Horizontal and Vertical Components of Peak Ground Acceleration, Peak Ground Velocity, and Pseudo-Absolute Acceleration Response Spectra, Seismological Research Letters, Vol. 68, No. 1, pp. 154-179.

Cao, T.-Q., Bryant, W. A., Rowshandel, B., Branum, D., and Wills, C. J., 2003, The Revised 2002 California Probabilistic Seismic Hazard Maps, dated June.

City of Los Angeles, 2010, NavigateLA, 211 West Temple Street, Los Angeles, site accessed July 23, 2010, http://navigatela.lacity.org.

Converse Consultants, Inc., 2003, Geotechnical Investigation Report, Los Angeles County Hall of Justice, Northerly Corner of Temples Street and Spring Street, Los Angeles, California, Project No. 03-31-102-01, dated May 5.

Google Earth, 2010, Site Location 211 West Temple Street, Los Angeles, California, site accessed July 23, 2010, http://earth.google.com.

Hart, E.W., and Bryant, W.A., 1997, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps: California Department of Conservation, Division of Mines and Geology, Special Publication 42, with Supplements 1 and 2 added in 1999.

International Conference of Building Officials, 1997, Uniform Building Code: Volume 2.

Jennings, C.W., 1994, Fault Activity Map of California and Adjacent Areas: California Division of Mines and Geology, Geologic Data Map No. 6, Scale 1:750,000.

Mualchin, L., 1996a, California Seismic Hazard Detail Index Map 1996: Scale 1 inch = 50 km, dated July, revised March 15, 2006.

Mualchin, L., 1996b, A Technical Report to Accompany the Caltrans California Seismic Hazard Map 1996 (based on maximum credible earthquakes): California Department of Transportation Engineering Service Center, Office of Earthquake Engineering, Sacramento, California, dated July (Rev. 1).

Ninyo & Moore, 2010, Revised Proposal for Geotechnical Consulting Hall of Justice, 211 West Temple Street, Los Angeles, California, dated July 10.

Peterson, M.D., Bryant, W.A., Cramer, C.H., Cao, T., Reichle, M.S., Frankel, A.D., Lienkaemper, J.J., McCrory, P.A., and Schwartz, D.P., 1996, Probabilistic Seismic Hazard Assessment for the State of California: California Department of Conservation, Division of Mines and Geology, Open-File Report 96-08.

State of California, 1997, Special Studies Zones, Los Angeles Quadrangle, 7.5 Minute Series: Scale 1:24,000, dated January1.

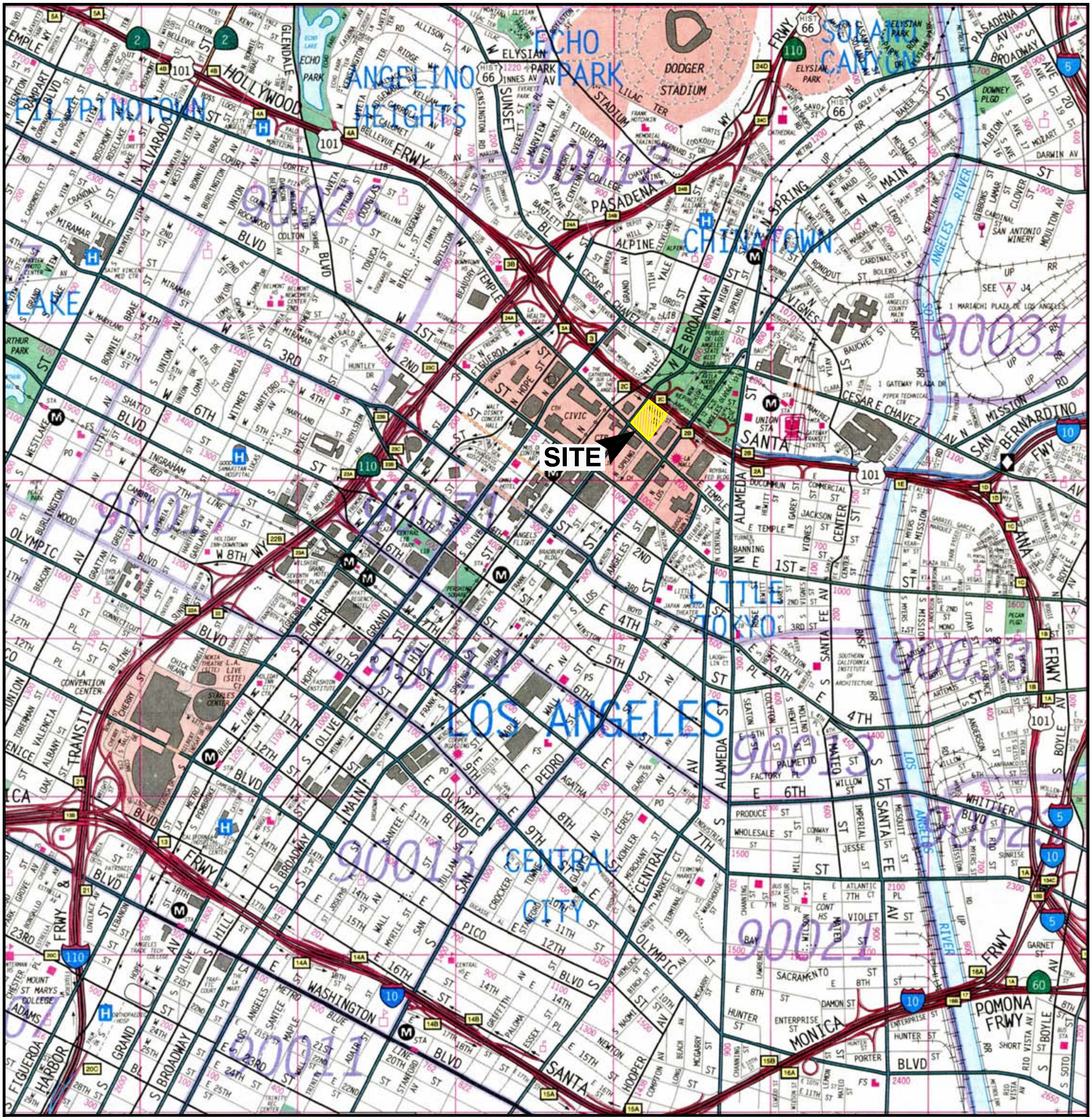
RBA Partners, Inc., 2007, Architectural Survey, Hall of Justice, Los Angeles, California, dated August 1.

Yerkes, R.F., and Campbell, R.H., 2005, Preliminary Digital Geologic Map of The Los Angeles 30’ x 60’ Quadrangle, Southern California, Version 1.0: United States Geological Survey Open-File Report 2005-1019, Scale 1:100,000.

United States Geological Survey, 1966 (Photorevised 1981), Los Angeles, California Quadrangle Map, 7.5 Minute Series: Scale 1:24,000.

United States Geological Survey, 2009, Earthquake Ground Motion Parameter Java Application, Java Ground Motion Parameter Calculator – Version 5.0.9a; http://earthquake.usgs.gov/research/hazmaps/design/.

United States Geological Survey, 2010, Open Seismic Hazard Analysis (OPENSHA), http://www.opensha.org/.



REFERENCE: 2007 THOMAS GUIDE FOR LOS ANGELES/ORANGE COUNTIES, STREET GUIDE AND DIRECTORY

APPROXIMATE SCALE IN FEET

0 2400 4800

N

NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

Map © Rand McNally, R.L.07-S-129

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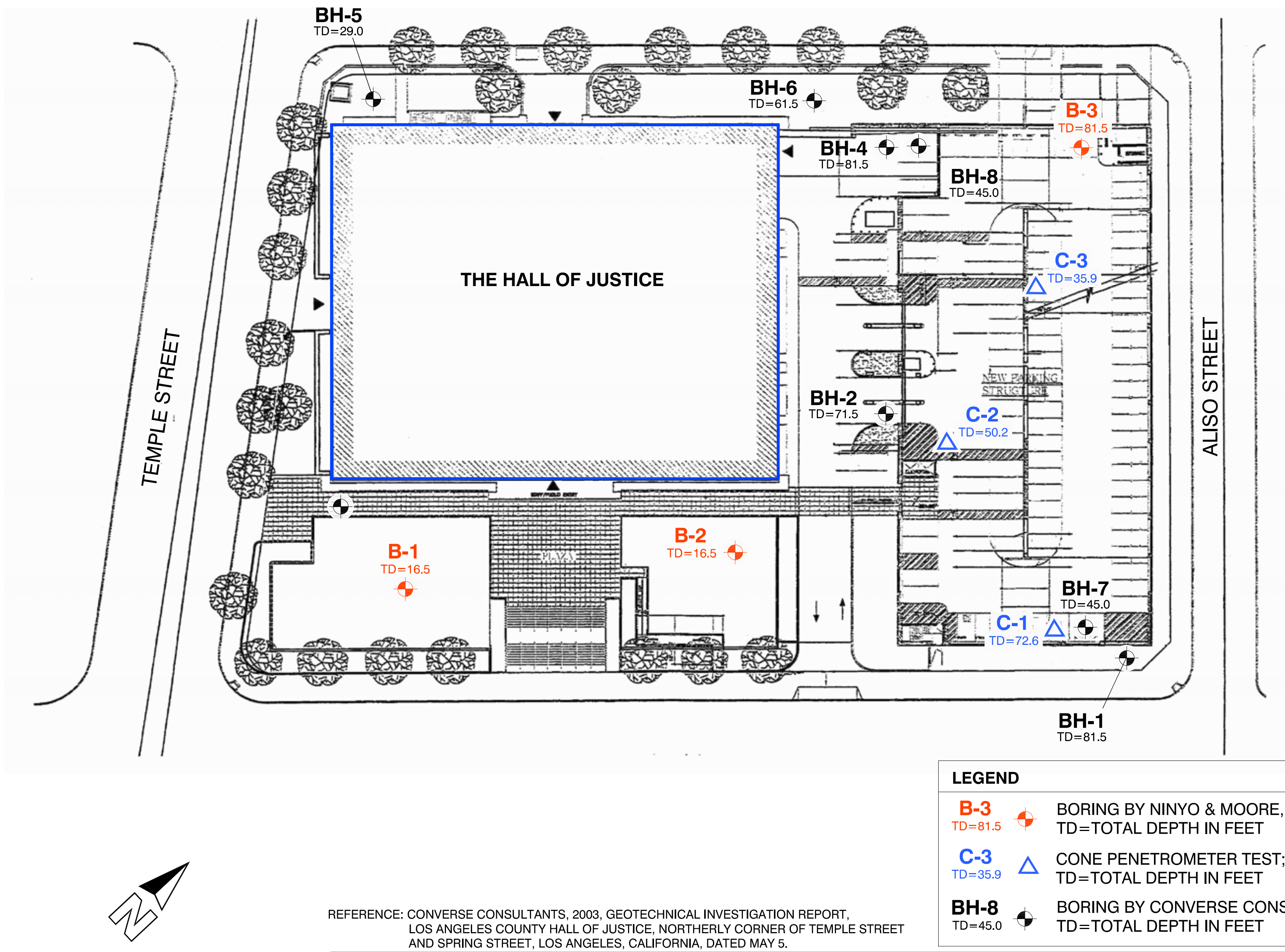
Ninyo & Moore		SITE LOCATION	FIGURE
PROJECT NO.	DATE	HALL OF JUSTICE 211 WEST TEMPLE STREET LOS ANGELES, CALIFORNIA	1
207247038	9/10		



LEGEND

B-3 TD=81.5	(red boring symbol)	BORING BY NINYO & MOORE, 2010. TD=TOTAL DEPTH IN FEET
C-3 TD=35.9	△	CONE PENETROMETER TEST; TD=TOTAL DEPTH IN FEET
BH-8 TD=45.0	(black boring symbol)	BORING BY CONVERSE CONSULTANTS, 2003. TD=TOTAL DEPTH IN FEET

REFERENCE: CONVERSE CONSULTANTS, 2003, GEOTECHNICAL INVESTIGATION REPORT, LOS ANGELES COUNTY HALL OF JUSTICE, NORTHERLY CORNER OF TEMPLE STREET AND SPRING STREET, LOS ANGELES, CALIFORNIA, DATED MAY 5.

NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

Ninyo & Moore		BORING LOCATIONS	FIGURE
PROJECT NO.	DATE	HALL OF JUSTICE 211 WEST TEMPLE STREET LOS ANGELES, CALIFORNIA	2
207247038	9/10		

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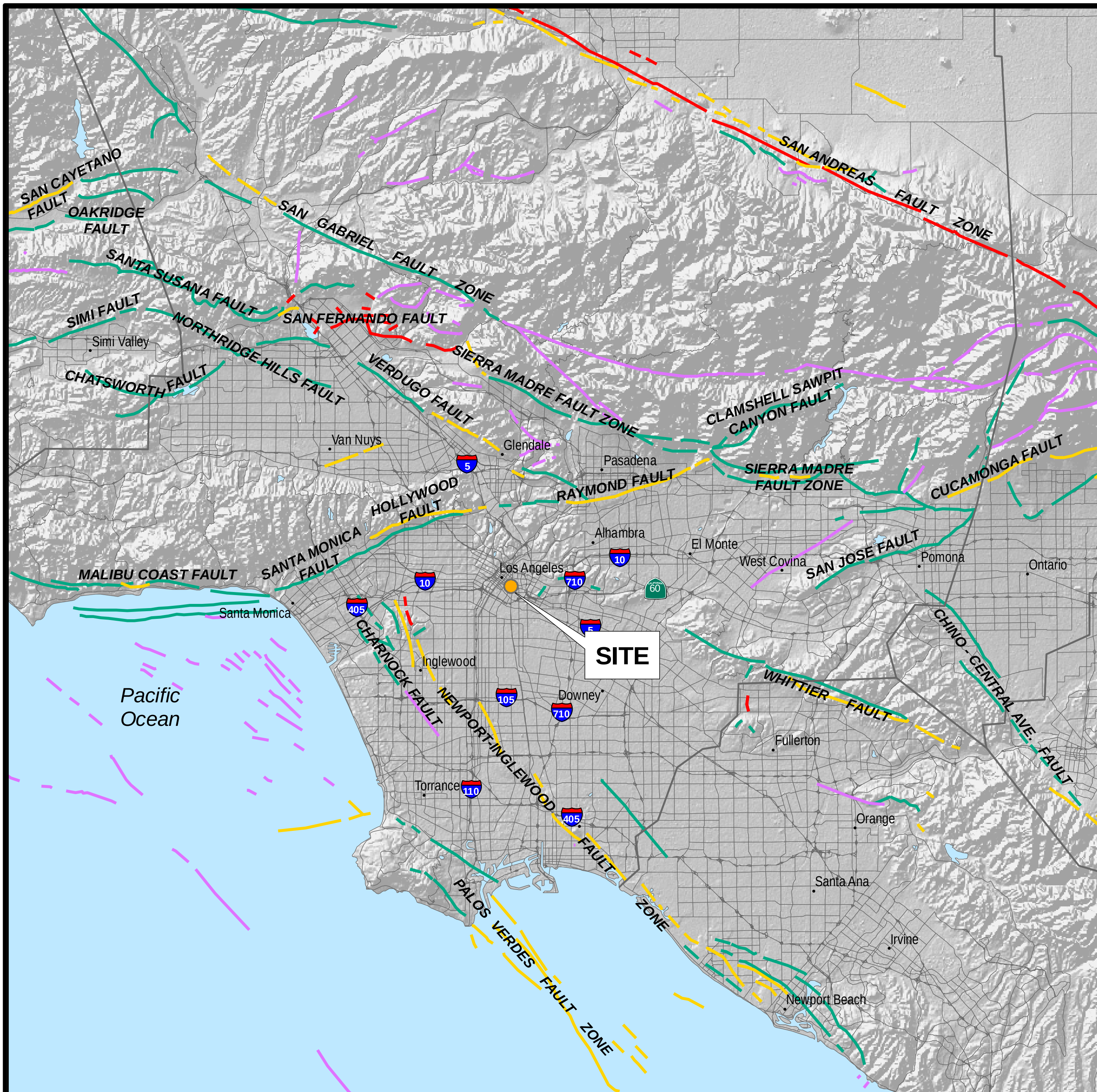


GIS DATA SOURCE: CALIFORNIA GEOLOGICAL SURVEY (CGS); ENVIRONMENTAL SYSTEMS RESEARCH INSTITUTE (ESRI)
REFERENCE: JENNINGS, 1994, FAULT ACTIVITY MAP OF CALIFORNIA AND ADJACENT AREAS

LEGEND

FAULT ACTIVITY:

- HISTORICALLY ACTIVE
- HOLOCENE ACTIVE
- COUNTY BOUNDARIES
- LATE QUATERNARY
- QUATERNARY

N

10 0 10

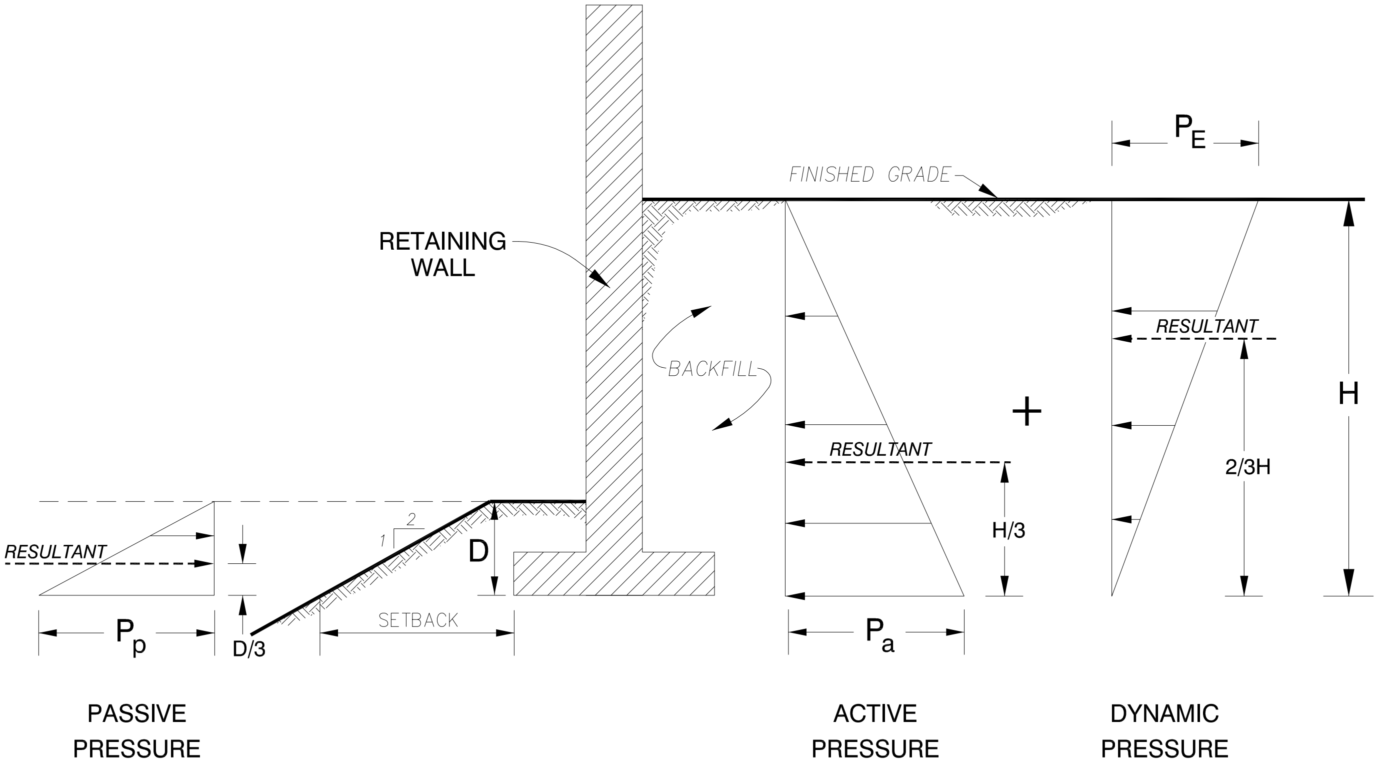
Miles

NOTE: ALL DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE

Ninyo & Moore

FAULT LOCATION MAP

PROJECT NO.	DATE	HALL OF JUSTICE 211 WEST TEMPLE STREET LOS ANGELES, CALIFORNIA	FIGURE
207247038	9/10		3



NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. STRUCTURAL, GRANULAR BACKFILL MATERIALS AS SPECIFIED IN GREENBOOK SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. DYNAMIC LATERAL EARTH PRESSURE IS BASED ON A PEAK GROUND ACCELERATION OF 0.55g
5. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
6. H AND D ARE IN FEET
7. SETBACK SHOULD BE IN ACCORDANCE WITH FIGURE 1805.3.1 OF THE CBC (2007)

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)(1)	
P_a	Level Backfill with Granular Soils (2)	2H:1V Sloping Backfill with Granular Soils (2)
	37 H	57 H
P_E	24 H	24 H
P_p	Level Ground	2H:1V Descending Ground
	350 D	350 D

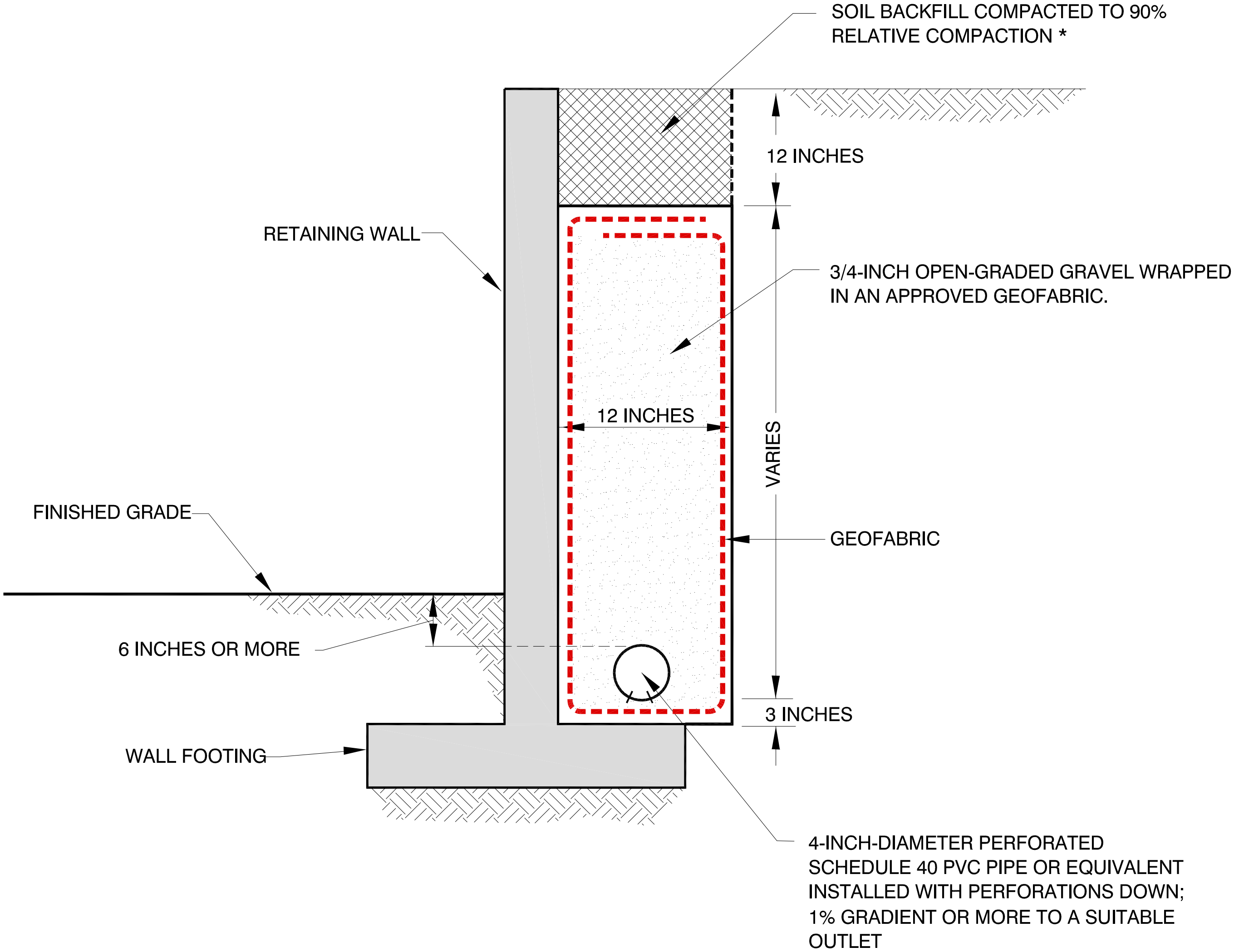
NOT TO SCALE

207247_A4.DWG.......-G.K.

Ninyo & Moore

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

PROJECT NO.	DATE		FIGURE
207247038	9/10	HALL OF JUSTICE 211 WEST TEMPLE STREET LOS ANGELES, CALIFORNIA	4



*BASED ON ASTM D1557

NOT TO SCALE

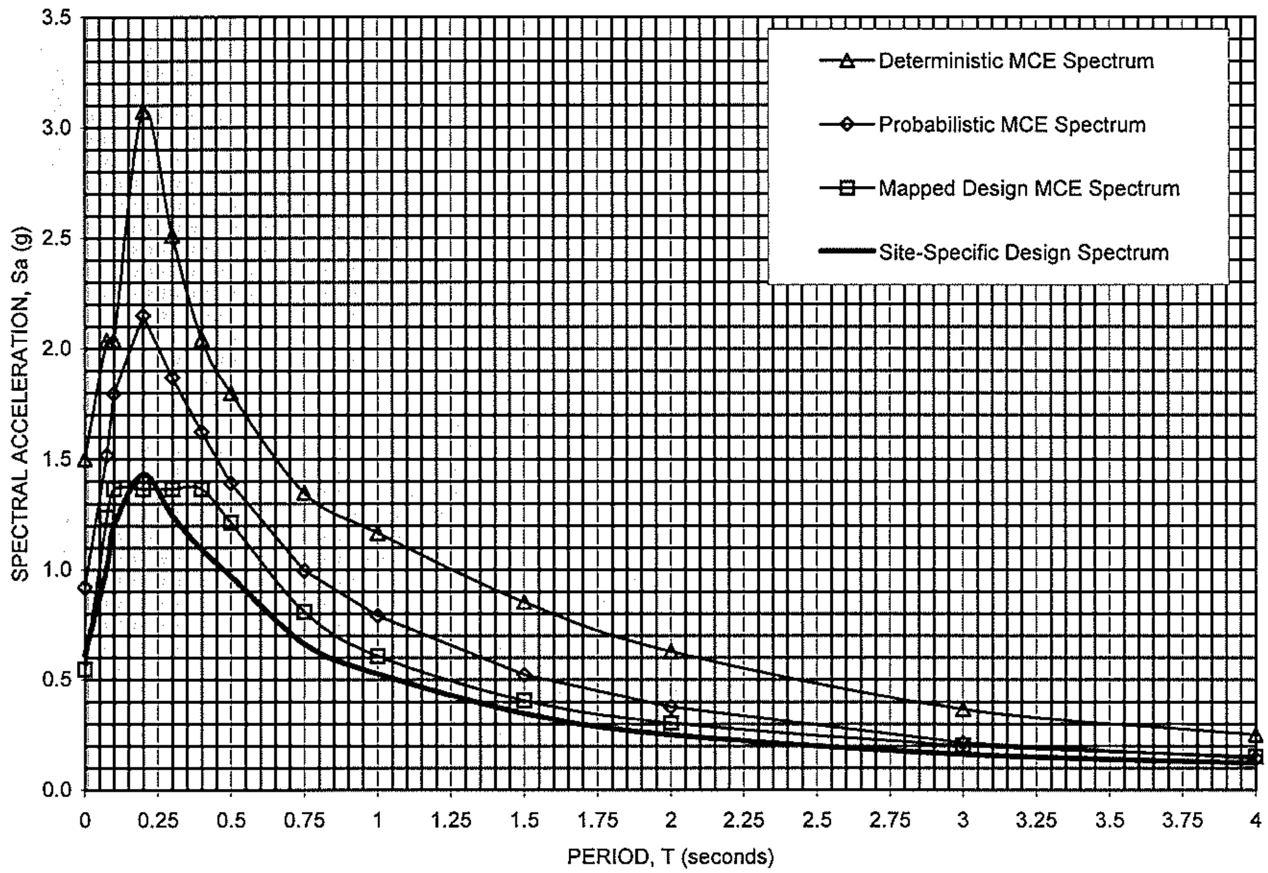
NOTE: AS AN ALTERNATIVE, AN APPROVED GEOCOMPOSITE DRAIN SYSTEM MAY BE USED.

207247_A5.DWG.......-G.K.

Ninyo & Moore		RETAINING WALL DRAINAGE DETAIL	FIGURE
PROJECT NO.	DATE	HALL OF JUSTICE 211 WEST TEMPLE STREET LOS ANGELES, CALIFORNIA	5
207789002	9/10		

PERIOD (seconds)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa, (g)
0.000	0.613
0.075	1.012
0.100	1.198
0.200	1.433
0.300	1.246
0.400	1.093
0.500	0.971
0.750	0.663

PERIOD (seconds)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa, (g)
1.000	0.527
1.500	0.348
2.000	0.250
3.000	0.162
4.000	0.121



NOTES:

1 Probabilistic Acceleration Response Spectrum (ARS) is for Maximum Considered Earthquake (MCE) with ground motion having 2% probability of exceedance In 50 years using the mean value among Abrahamson and Silva (1997), Campbell (1997) and Sadigh et al. (1997).

2 Deterministic ARS is 150% of the largest median values from the four attenuation relationships list above for a soft rock condition considering a Magnitude 7.3 event on the Puente Hills Blind Thrust fault located approximately 5.22 km from the site. Deterministic ARS conforms with the lower bound limit per ASCE 7-05 Section 21.2.2.

3 Site-Specific Design ARS is the lesser of spectral ordinates of deterministic and probabilistic ARS at each period per ASCE 7-05 Section 21.2.3. Site-Specific Design ARS conforms with lower bound limit per ASCE 7-05 Section 21.3.

4 Mapped Design ARS is computed from mapped spectral ordinates modified for Site Class C (soft rock profile) per ASCE 7-05 Section 11.4. It is presented for comparison.

5 ARS curves for horizontal ground motion assume 5% damping and do not include response modification factor or importance factor.

Ninyo & Moore		ACCELERATION RESPONSE SPECTRA	FIGURE
PROJECT	DATE	HALL OF JUSTICE 211 WEST TEMPLE STREET LOS ANGELES, CALIFORNIA	6
207247038	9/10		

APPENDIX A
BORING LOGS

Field Procedure for the Collection of Disturbed Samples
Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples
Bulk samples of representative earth materials were obtained from the exploratory excavations. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler
Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 18 inches with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples
Relatively undisturbed soil samples were obtained in the field using the following methods.

The Modified Split-Barrel Drive Sampler
The sampler, with an external diameter of 3.0 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer or bar, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

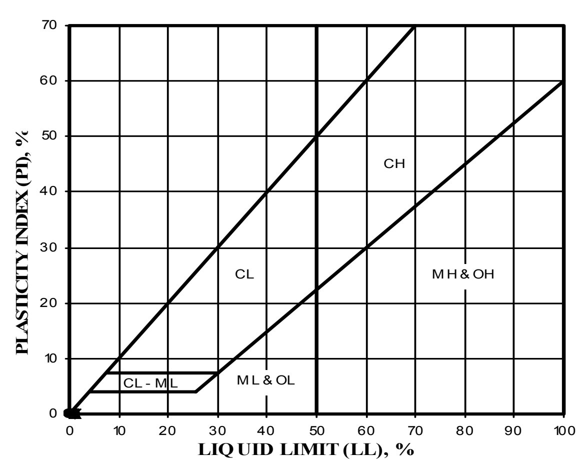
U.S.C.S. METHOD OF SOIL CLASSIFICATION

MAJOR DIVISIONS		SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (More than 1/2 of soil >No. 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > No. 4 sieve size)	GW	Well graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 1/2 of coarse fraction <No. 4 sieve size)	SW	Well graded sands or gravelly sands, little or no fines
		SP	Poorly graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (More than 1/2 of soil <No. 200 sieve size)	SILTS & CLAYS Liquid Limit <50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean
		OL	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS Liquid Limit >50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils

GRAIN SIZE CHART

CLASSIFICATION	RANGE OF GRAIN SIZE	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL **Coarse** **Fine**	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
SAND **Coarse** **Medium** **Fine**	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.075 4.76 to 2.00 2.00 to 0.420 0.420 to 0.075
SILT & CLAY	Below No. 200	Below 0.075

PLASTICITY CHART



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U.S.C.S. METHOD OF SOIL CLASSIFICATION

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	ALLUVIUM: Solid line denotes unit change.
								Dashed line denotes material change.
15								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Sheared Bedding Surface
20								The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG

EXPLANATION OF BORING LOG SYMBOLS

PROJECT NO.	DATE	FIGURE
	Rev. 01/03	

DATE DRILLED	8/4/10	BORING NO.	B-1
GROUND ELEVATION	319' ± (MSL)	SHEET	1 OF 1
METHOD OF DRILLING	8" Hollow-Stem Auger (Martini Drilling)		
DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"
SAMPLED BY	VAM	LOGGED BY	VAM
REVIEWED BY	LTJ		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 4 inches thick.
							CL	FILL: Yellowish brown, moist, firm, silty CLAY.
5			20	16.4	100.8			PUENTE FORMATION: Yellowish brown, moist, weakly indurated, CLAYSTONE and SILTSTONE; oxidation staining.
10			10					
15			32	36.1	79.8			Brown.
								Total Depth = 16.5 feet. Groundwater not encountered during drilling. Backfilled with soil cuttings and capped with concrete on 8/5/10. Note: Groundwater, though not encountered, at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
20								

Ninyo & Moore

BORING LOG

HALL OF JUSTICE, 211 WEST TEMPLE STREET
LOS ANGELES, CALIFORNIA

PROJECT NO.	DATE	FIGURE
207247038	8/10	A-1

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.

DATE DRILLED 8/14/10 BORING NO. B-2

GROUND ELEVATION 320' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"

SAMPLED BY VAM LOGGED BY VAM REVIEWED BY LTJ

DESCRIPTION/INTERPRETATION

Depth (feet)	Blows/Foot	Moisture (%)	Dry Density (PCF)	Classification U.S.C.S.	Description/Interpretation
0					ASPHALT CONCRETE: Approximately 2 1/2 inches thick.
				CL	PORTLAND CEMENT CONCRETE: Approximately 6 1/2 inches thick.
				GP-GM	AGGREGATE BASE: Brown, damp to moist, medium dense, poorly graded GRAVEL with sand and silt; approximately 4 inches thick.
				CL	FILL: Yellowish brown, moist, firm, silty CLAY.
					PUENTE FORMATION: Yellowish brown, moist, weakly indurated, CLAYSTONE and SILTSTONE; oxidation staining.
5	28	24.2	97.3		
10	20				
15	39	30.5	91.5		
					Total Depth = 16.5 feet. Groundwater not encountered during drilling. Backfilled with soil cuttings and capped with concrete on 8/5/10.
					Note: Groundwater, though not encountered, at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
20					

Ninyo & Moore

BORING LOG

HALL OF JUSTICE, 211 WEST TEMPLE STREET
LOS ANGELES, CALIFORNIA

PROJECT NO.	DATE	FIGURE
207247038	8/10	A-2

DATE DRILLED 8/4/10 BORING NO. B-3

GROUND ELEVATION 331' ± (MSL) SHEET 1 OF 5

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"

SAMPLED BY VAM LOGGED BY VAM REVIEWED BY LTJ

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 4 1/2 inches thick.
	X							PUENTE FORMATION: Yellowish brown, moist, weakly indurated, CLAYSTONE and SILTSTONE; oxidation staining.
5		X	40	27.0	92.5			
10		X	29					
15		X	50/5"	21.8	99.2			
20								

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BORING LOG

HALL OF JUSTICE, 211 WEST TEMPLE STREET
LOS ANGELES, CALIFORNIA

PROJECT NO.	DATE	FIGURE
207247038	8/10	A-3

DATE DRILLED	8/4/10	BORING NO.	B-3
GROUND ELEVATION	331' ± (MSL)	SHEET	2 OF 5
METHOD OF DRILLING	8" Hollow-Stem Auger (Martini Drilling)		
DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"
SAMPLED BY	VAM	LOGGED BY	VAM
REVIEWED BY	LTJ		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20			48					WEATHERED PUENTE FORMATION: (Continued) Olive gray to olive brown, moist, weakly indurated, CLAYSTONE, SILTSTONE, and SANDSTONE; oxidation staining.
25			72	23.0	100.6			
30			28	♀				@30': Seepage.
35			48	♀				@35.5': Seepage Saturated sandstone layer.
40								

Ninyo & Moore

BORING LOG

HALL OF JUSTICE, 211 WEST TEMPLE STREET
LOS ANGELES, CALIFORNIA

PROJECT NO.	DATE	FIGURE
207247038	8/10	A-4

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40		■	80	♀				PUENTE FORMATION: (Continued) Layers of olive gray to olive brown, moist, weakly indurated CLAYSTONE and SILTSTONE with weakly cemented SANDSTONE. @41': Seepage; saturated sandstone layer.
45		■	60					
50		■	32	♀				@50': Seepage (claystone layer) Saturated sandstone layer.
55		■	97					
60								

DATE DRILLED 8/4/10 BORING NO. B-3

GROUND ELEVATION 331' ± (MSL) SHEET 3 OF 5

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"

SAMPLED BY VAM LOGGED BY VAM REVIEWED BY LTJ

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
60			50/4"					PUENTE FORMATION: (Continued) Layers of olive brown, olive gray, brown, moist, weakly indurated CLAYSTONE, SILTSTONE, and weakly cemented SANDSTONE.
65			50/5"					
70			50/5"					
75			87					
80								

DATE DRILLED 8/4/10 BORING NO. B-3

GROUND ELEVATION 331' ± (MSL) SHEET 4 OF 5

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"

SAMPLED BY VAM LOGGED BY VAM REVIEWED BY LTJ

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 8/4/10 BORING NO. B-3 GROUND ELEVATION 331' ± (MSL) SHEET 5 OF 5 METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling) DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30" SAMPLED BY VAM LOGGED BY VAM REVIEWED BY LTJ **DESCRIPTION/INTERPRETATION**
80			50/5"					PUENTE FORMATION: (Continued) Layers of gray and dark gray, saturated, weakly to moderately indurated CLAYSTONE and weakly cemented SANDSTONE.
								Total Depth = 81.5 feet. Seepage encountered during drilling at approximately 30, 35.5, 41 and 50 feet. Backfilled with soil cuttings and capped with concrete on 8/5/10. Note: Groundwater may rise to a level higher than that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report. Please refer to the report for groundwater monitoring recommendations.
85								
90								
95								
100								

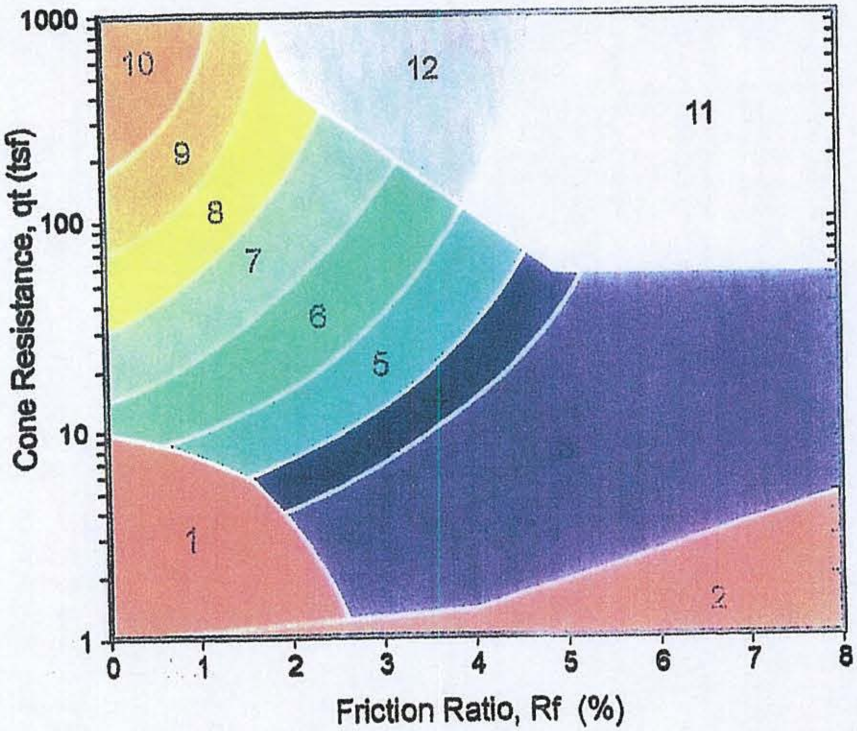
APPENDIX B

CONE PENETROMETER TESTING

KEHOE TESTING & ENGINEERING

CPT Classification Chart

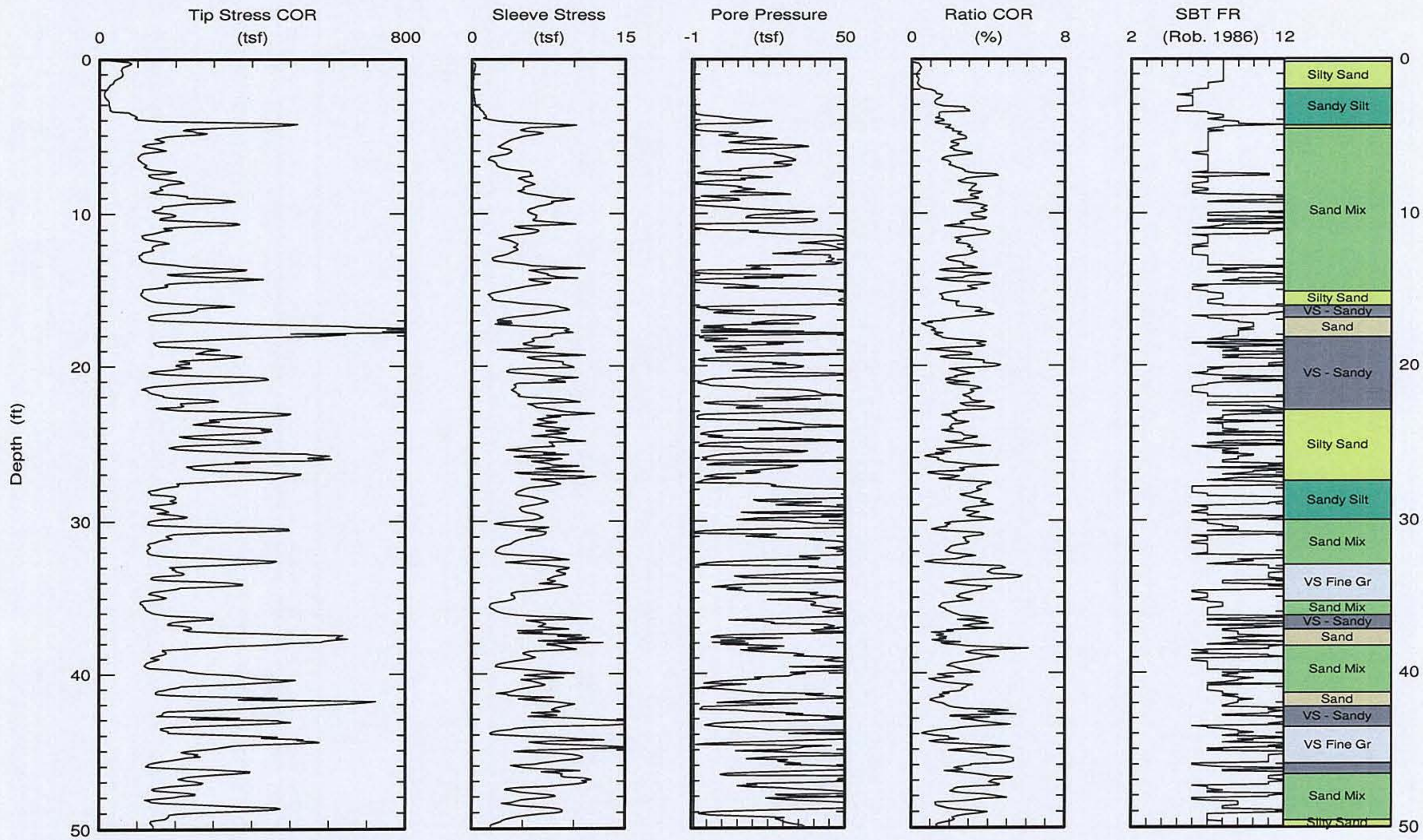
(after Robertson and Campanella, 1988)



Zone	q_t / N	Soil Behavior Type	UCSCS
1	2	sensitive fine grained	OL-OH
2	1	organic material	Pt-OH
3	1	clay	CH
4	1.5	silty clay to clay	CL-CH
5	2	clayey silt to silty clay	ML-CL
6	2.5	sandy silt to clayey silt	MH-ML
7	3	silty sand to sandy silt	SM-ML
8	4	sand to silty sand	SP-SM
9	5	sand	SP
10	6	gravelly sand to sand	SW-SP
11	1	very stiff fine grained *	CL-MH
12	2	sand to clayey sand *	SP-SC

* overconsolidated or cemented

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 04/Aug/2010 Test ID: C-1 Project: LosAngeles
	Customer: Ninyo & Moore Job Site: COLA / Hall of Justice	



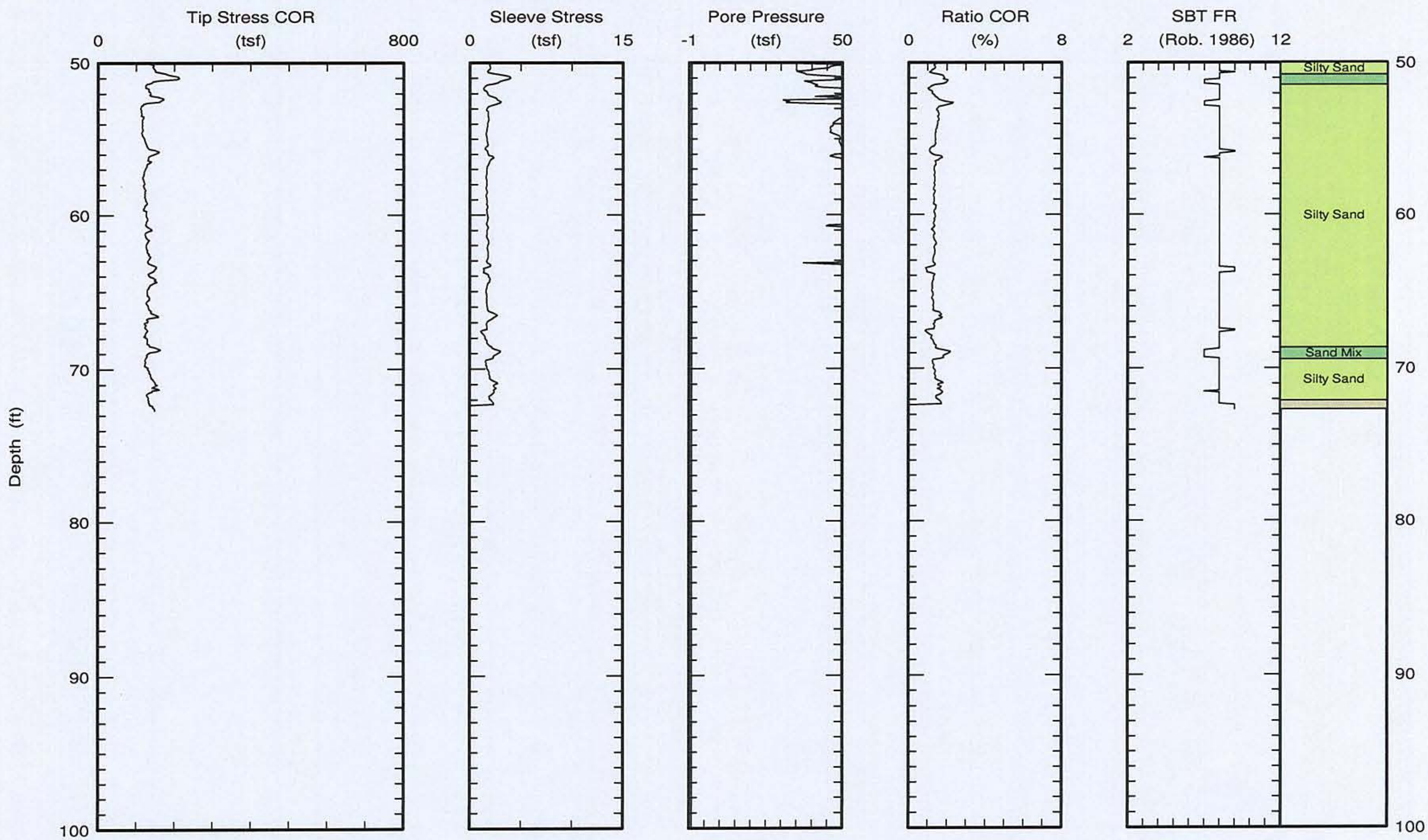
Maximum depth: 72.67 (ft)

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
rich@kehoetesting.com
www.kehoetesting.com

CPT Data
30 ton rig

Date: 04/Aug/2010
Test ID: C-1
Project: LosAngeles

Customer: Ninyo & Moore
Job Site: COLA / Hall of Justice



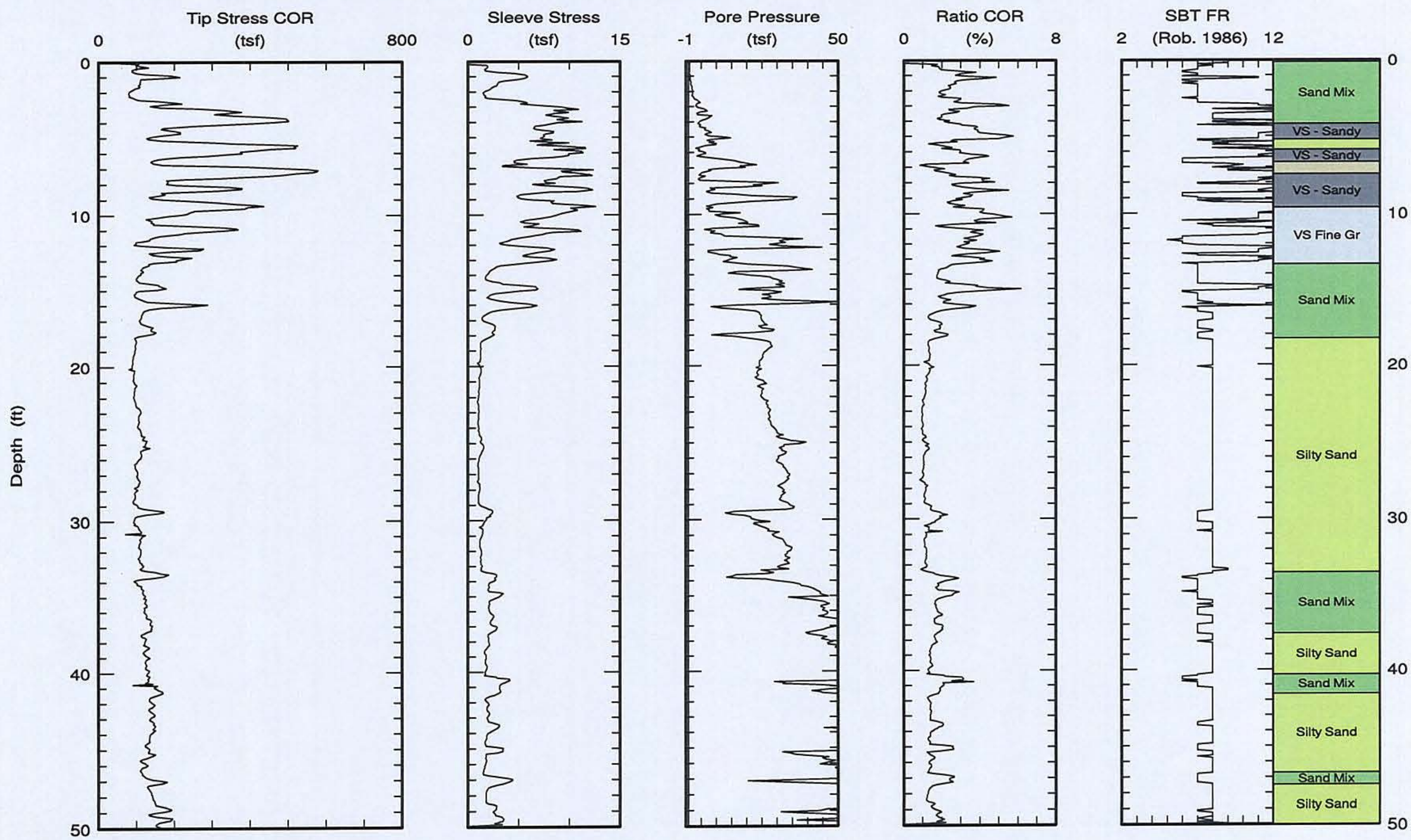
Maximum depth: 72.67 (ft)
Page 2 of 2

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
rich@kehoetesting.com
www.kehoetesting.com

CPT Data
30 ton rig

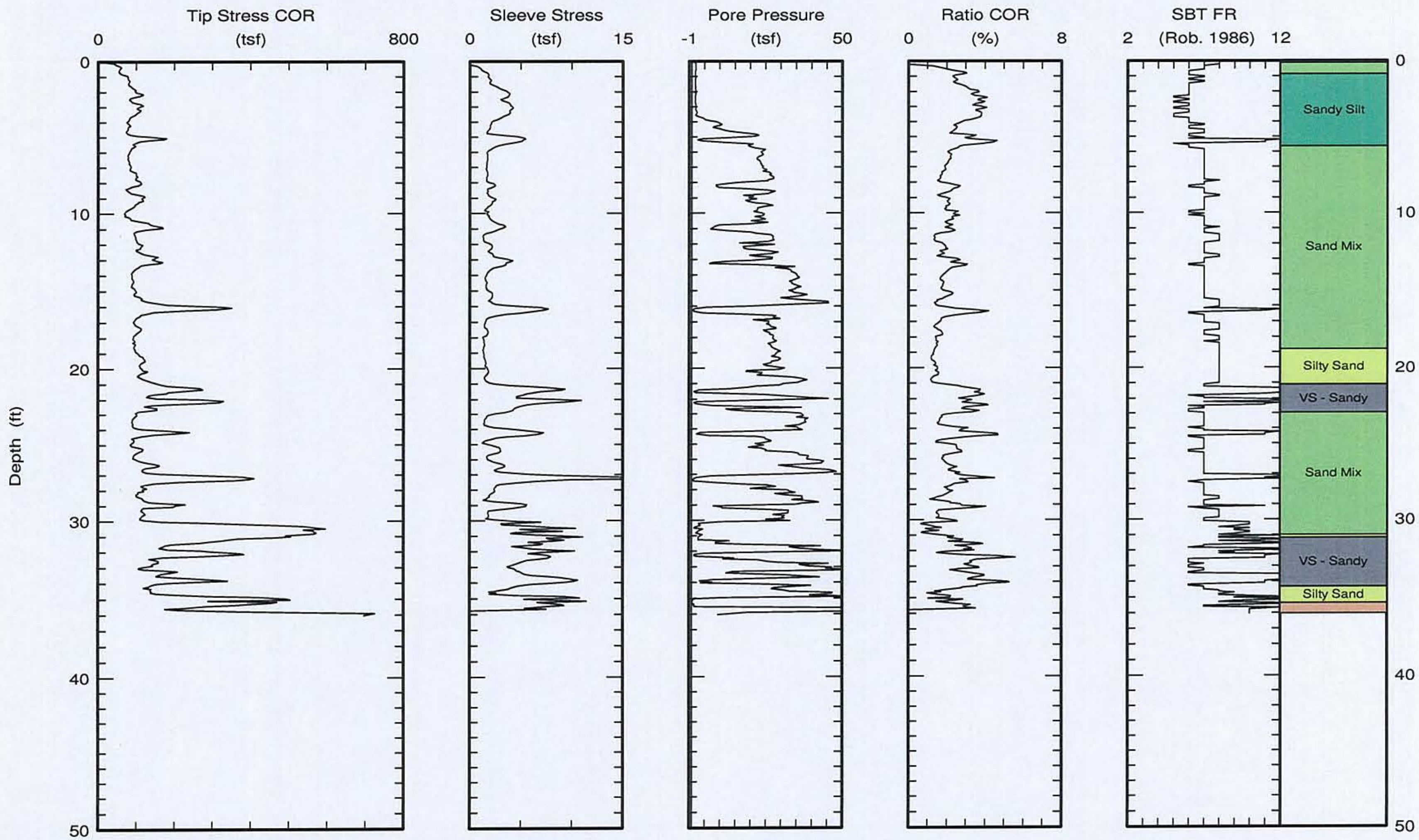
Date: 04/Aug/2010
Test ID: C-2
Project: LosAngeles

Customer: Ninyo & Moore
Job Site: COLA / Hall of Justice



Maximum depth: 50.21 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com www.kehoetesting.com	CPT Data 30 ton rig	Date: 04/Aug/2010 Test ID: C-3 Project: LosAngeles
	Customer: Ninyo & Moore Job Site: COLA / Hall of Justice	



Maximum depth: 36.05 (ft)

INPUT FILE: C:\temp\C-1.CSV

" Depth " (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	65.248	0.222	0.340	8	16	24	9E9
1.500	37.978	0.142	0.374	7	12	18	9E9
2.500	19.395	0.258	1.332	6	7	11	1.283
3.500	61.377	1.051	1.694	7	20	30	9E9
4.500	315.995	6.119	1.926	8	76	114	9E9
5.500	147.058	3.656	2.416	7	48	72	9E9
6.500	110.550	2.583	2.234	7	37	56	9E9
7.500	151.513	4.697	3.040	6	59	89	10.268
8.500	175.808	5.549	3.111	6	68	102	11.858
9.500	225.177	7.202	3.167	12	109	164	9E9
10.500	221.420	6.515	2.895	7	72	101	9E9
11.500	137.800	4.458	3.069	6	56	73	9.635
12.500	116.193	3.483	2.800	6	48	59	8.240
13.500	233.617	6.013	2.520	7	76	87	9E9
14.500	240.657	6.164	2.530	7	78	85	9E9
15.500	134.952	2.865	1.979	7	46	48	9E9
16.500	212.995	6.866	3.166	12	104	103	9E9
17.500	557.173	6.530	1.159	9	108	102	9E9
18.500	283.695	6.200	2.167	8	69	63	9E9
19.500	275.552	7.551	2.707	7	89	79	9E9
20.500	263.783	7.427	2.763	7	86	75	9E9
21.500	154.235	4.388	2.760	7	51	44	9E9
22.500	236.510	7.510	3.118	12	115	97	9E9
23.500	361.127	7.866	2.154	8	87	72	9E9
24.500	358.587	8.704	2.404	12	173	140	9E9
25.500	369.353	6.746	1.813	8	89	71	9E9
26.500	365.105	7.933	2.145	8	89	69	9E9
27.500	323.065	8.394	2.578	12	156	119	9E9
28.500	163.232	5.803	3.373	6	66	50	11.352
29.500	167.798	5.756	3.293	6	67	50	11.531
30.500	258.328	5.208	1.969	8	63	46	9E9
31.500	137.217	3.827	2.624	7	47	34	9E9
32.500	276.575	6.117	2.165	8	68	48	9E9
33.500	195.073	8.517	4.208	11	194	135	9E9
34.500	208.502	6.208	2.908	7	68	46	9E9
35.500	115.687	2.552	2.064	7	39	26	9E9
36.500	213.887	7.050	3.187	12	106	70	9E9
37.500	467.360	8.174	1.728	8	113	74	9E9
38.500	240.435	7.190	2.931	7	78	50	9E9
39.500	149.393	4.429	2.785	7	51	33	9E9
40.500	355.930	7.224	2.009	8	86	54	9E9
41.499	357.023	5.474	1.512	9	69	43	9E9
42.499	279.542	7.836	2.756	12	136	83	9E9
43.499	281.645	8.136	2.840	12	137	83	9E9
44.499	419.957	9.163	2.136	8	103	61	9E9
45.499	189.382	8.174	4.181	11	187	110	9E9
46.499	261.738	8.276	3.068	12	129	75	9E9
47.499	198.820	6.994	3.390	12	99	57	9E9
48.499	265.492	5.934	2.157	8	66	37	9E9
49.499	158.415	4.059	2.494	7	52	29	9E9

INPUT FILE: C:\temp\C-1.CSV

" Depth " (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
50.499	149.625	2.216	1.406	8	38	21	9E9
51.499	139.522	2.400	1.616	8	36	20	9E9
52.499	128.620	2.250	1.635	8	33	18	9E9
53.499	106.342	1.804	1.556	8	28	15	9E9
54.499	108.132	1.676	1.429	8	28	15	9E9
55.499	126.442	1.713	1.255	8	33	17	9E9
56.499	121.122	1.963	1.494	8	31	16	9E9
57.499	109.868	1.648	1.366	8	29	15	9E9
58.499	112.268	1.657	1.346	8	29	15	9E9
59.499	114.158	1.719	1.372	8	30	15	9E9
60.499	118.197	1.756	1.358	8	31	16	9E9
61.499	117.597	1.749	1.347	8	31	16	9E9
62.499	119.851	1.781	1.352	8	32	16	9E9
63.499	128.152	1.683	1.187	8	34	17	9E9
64.499	124.748	1.795	1.277	8	34	17	9E9
65.499	117.447	1.698	1.291	8	31	16	9E9
66.499	126.310	2.161	1.546	8	33	17	9E9
67.499	110.257	1.520	1.209	8	30	15	9E9
68.499	125.202	2.092	1.486	8	34	17	9E9
69.499	113.820	1.983	1.560	8	30	15	9E9
70.499	125.075	2.065	1.466	8	34	17	9E9
71.499	127.757	2.320	1.628	8	34	17	9E9
72.499	127.414	0.876	0.623	9	27	14	9E9

INPUT FILE: C:\temp\C-2.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	123.438	2.670	2.162	7	39	59	9E9
1.500	104.995	3.202	3.045	6	40	60	7.005
2.500	129.422	4.012	3.090	6	50	75	8.646
3.500	393.390	9.212	2.336	12	189	284	9E9
4.500	217.760	7.529	3.441	12	105	158	9E9
5.500	356.788	9.168	2.560	12	172	258	9E9
6.500	238.382	6.582	2.735	7	77	116	9E9
7.500	396.447	9.275	2.332	12	191	287	9E9
8.500	229.103	8.008	3.444	12	111	167	9E9
9.500	309.428	9.460	3.029	12	150	218	9E9
10.500	212.891	7.424	3.440	12	103	138	9E9
11.500	147.810	5.657	3.720	12	73	91	9E9
12.500	200.503	7.057	3.464	12	98	114	9E9
13.500	121.437	3.557	2.820	6	48	53	8.349
14.500	127.470	3.805	2.864	6	51	53	8.795
15.500	126.425	3.447	2.606	7	42	42	9E9
16.500	131.625	3.195	2.360	7	43	41	9E9
17.500	127.752	2.402	1.814	7	42	38	9E9
18.500	95.173	1.606	1.614	7	32	28	9E9
19.500	87.952	1.228	1.324	8	22	19	9E9
20.500	87.543	1.222	1.328	8	22	19	9E9
21.500	91.670	1.109	1.150	8	23	19	9E9
22.500	98.922	1.059	1.018	8	25	20	9E9
23.500	101.265	1.056	0.991	8	26	21	9E9
24.500	112.490	1.234	1.042	8	28	22	9E9
25.500	112.640	1.424	1.198	8	28	22	9E9
26.500	95.667	1.063	1.045	8	24	18	9E9
27.500	98.742	1.123	1.070	8	25	19	9E9
28.500	90.943	1.091	1.124	8	23	17	9E9
29.500	120.915	1.989	1.581	8	30	22	9E9
30.500	96.083	1.517	1.503	8	24	17	9E9
31.500	105.596	1.284	1.149	8	27	19	9E9
32.500	106.197	1.336	1.187	8	27	19	9E9
33.500	132.218	2.197	1.606	8	33	23	9E9
34.500	108.265	2.546	2.193	7	37	25	9E9
35.500	116.455	2.282	1.829	7	40	27	9E9
36.500	125.468	2.384	1.766	7	43	28	9E9
37.500	119.345	2.383	1.862	7	41	27	9E9
38.500	117.027	1.925	1.521	8	30	19	9E9
39.500	119.052	1.835	1.422	8	31	20	9E9
40.500	127.177	2.974	2.180	7	44	27	9E9
41.499	139.993	2.594	1.728	8	36	22	9E9
42.499	130.103	2.015	1.423	8	34	21	9E9
43.499	133.965	2.518	1.736	8	35	21	9E9
44.499	121.550	2.097	1.581	8	32	19	9E9
45.499	120.948	2.336	1.804	7	41	24	9E9
46.499	113.613	2.082	1.686	7	39	23	9E9
47.499	135.552	2.726	1.873	7	46	26	9E9
48.499	148.948	2.215	1.373	8	39	22	9E9
49.499	152.283	2.541	1.562	8	39	22	9E9

INPUT FILE: C:\temp\C-3.CSV

Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	58.905	1.141	1.929	7	19	29	9E9
1.500	87.228	2.540	2.904	6	34	51	5.824
2.500	102.663	3.728	3.623	6	39	59	6.849
3.500	95.383	3.503	3.654	6	37	56	6.377
4.500	80.855	2.218	2.661	6	32	48	5.539
5.500	110.175	3.644	3.223	6	43	65	7.514
6.500	77.438	1.732	2.116	7	26	39	9E9
7.500	91.823	1.699	1.756	7	31	47	9E9
8.500	94.223	2.009	2.060	7	31	47	9E9
9.500	95.175	2.036	2.049	7	32	48	9E9
10.500	108.267	2.467	2.207	7	36	51	9E9
11.500	97.145	1.697	1.674	7	32	42	9E9
12.500	119.880	2.405	1.935	7	40	49	9E9
13.500	109.303	2.731	2.391	7	36	42	9E9
14.500	85.888	1.640	1.772	7	30	33	9E9
15.500	123.890	2.272	1.736	7	42	44	9E9
16.500	169.908	4.335	2.513	7	55	55	9E9
17.500	98.427	1.640	1.585	7	33	31	9E9
18.500	91.342	1.404	1.454	8	23	21	9E9
19.500	109.673	1.503	1.308	8	28	25	9E9
20.500	120.849	1.770	1.400	8	30	27	9E9
21.500	199.858	6.424	3.150	7	65	56	9E9
22.500	166.717	5.564	3.277	6	65	55	11.228
23.500	88.983	1.822	1.897	7	31	26	9E9
24.500	127.902	3.951	3.004	6	50	41	8.667
25.500	96.403	2.431	2.378	7	33	27	9E9
26.500	128.212	3.025	2.223	7	43	34	9E9
27.500	236.578	7.237	3.029	12	114	89	9E9
28.500	120.870	2.390	1.872	7	41	31	9E9
29.500	126.473	2.927	2.229	7	42	32	9E9
30.500	458.122	5.934	1.291	9	88	65	9E9
31.500	286.881	7.540	2.592	7	93	68	9E9
32.500	191.067	5.847	2.999	7	62	45	9E9
33.500	200.033	7.114	3.472	12	98	69	9E9
34.500	205.390	4.655	2.201	7	68	47	9E9
35.500	388.433	4.651	1.186	9	75	52	9E9

211 W. Temple St
Los Angeles, CA

CPT Shear Wave Measurements

Location	Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
C-2	5.48	7.42	5.14	1443	
	10.31	11.46	7.49	1530	1719
	15.23	16.03	10.39	1543	1576
	20.16	20.77	13.68	1518	1441
	25.18	25.67	17.94	1431	1150
	30.11	30.52	21.85	1397	1241
	35.08	35.43	25.91	1368	1210
	40.75	41.06	29.37	1398	1625
	45.23	45.51	32.57	1397	1391
	50.09	50.34	35.79	1407	1501
C-3	5.48	7.42	5.52	1344	
	10.41	11.55	8.19	1410	1547
	15.42	16.21	10.93	1483	1701
	20.16	20.77	13.68	1518	1658
	25.15	25.64	16.61	1544	1663
	30.14	30.55	19.67	1553	1604
	35.09	35.44	22.82	1553	1553

Shear Wave Source Offset = 5 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)

APPENDIX C

LABORATORY TESTING

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory excavations in Appendix A.

In-Place Moisture and Density Tests

The moisture content and dry density of relatively undisturbed samples obtained from the exploratory excavations were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory excavations in Appendix A.

Atterberg Limits

Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System (USCS). The test results and classifications are shown on Figure C-1.

Direct Shear Tests

Direct shear tests were performed on relatively undisturbed samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figures C-2 through C-3.

Expansion Index Tests

The expansion index of a selected material was evaluated in general accordance with Uniform Building Code (UBC) Standard No. 18-2 (ASTM D 4829). Specimen was molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. Results of this test are presented on Figure C-4.

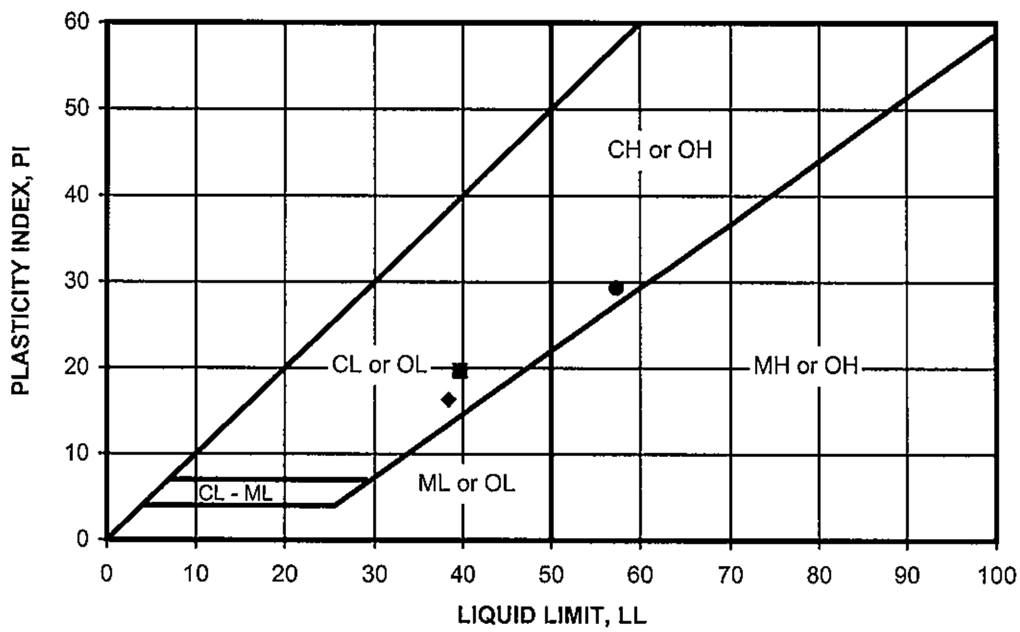
Soil Corrosivity Tests

Soil pH, and resistivity tests were performed on representative samples in general accordance with California Test (CT) 643. The soluble sulfate and chloride content of selected samples were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure C-5.

R-Value

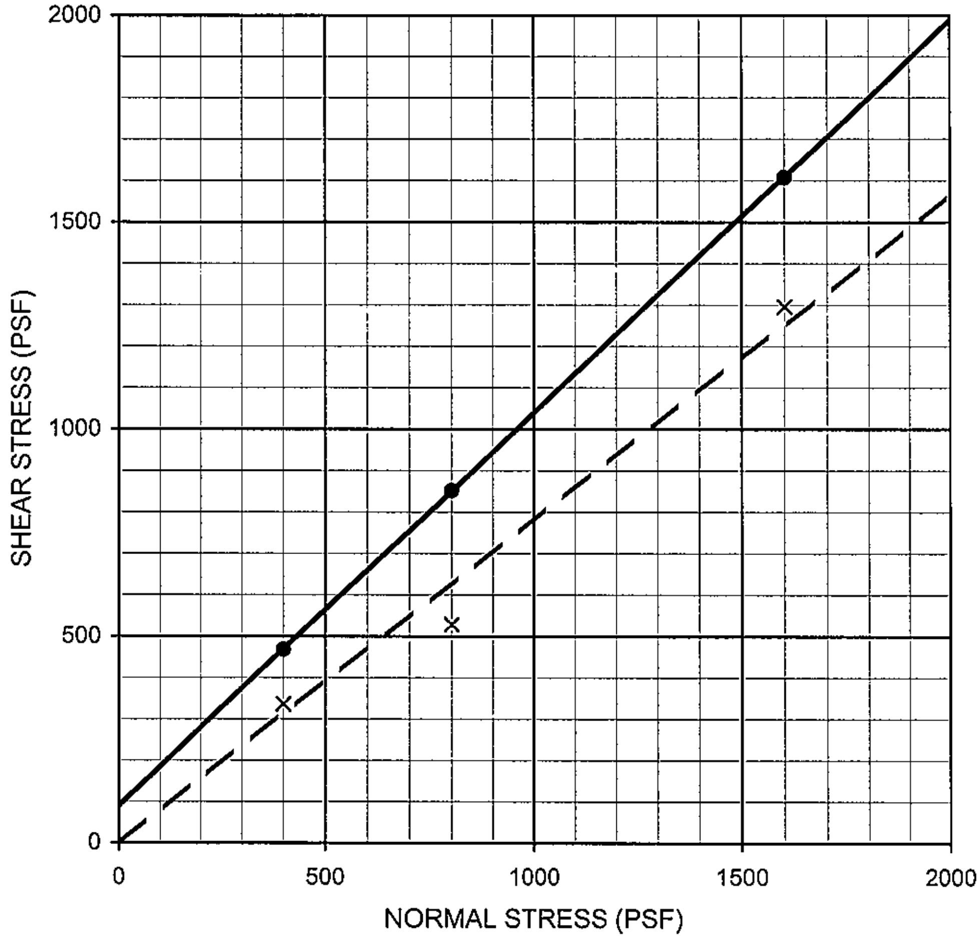
The resistance value, or R-value, for site soils was evaluated in general accordance with California Test (CT) 301. A sample was prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure C-6.

SYMBOL	LOCATION	DEPTH (FT)	LIQUID LIMIT, LL	PLASTIC LIMIT, PL	PLASTICITY INDEX, PI	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS (Entire Sample)
●	B-1	10.0-11.5	57	28	29	CH	---
■	B-2	5.0-6.5	40	20	20	CL	---
◆	B-3	20.0-21.5	38	22	16	CL	---



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

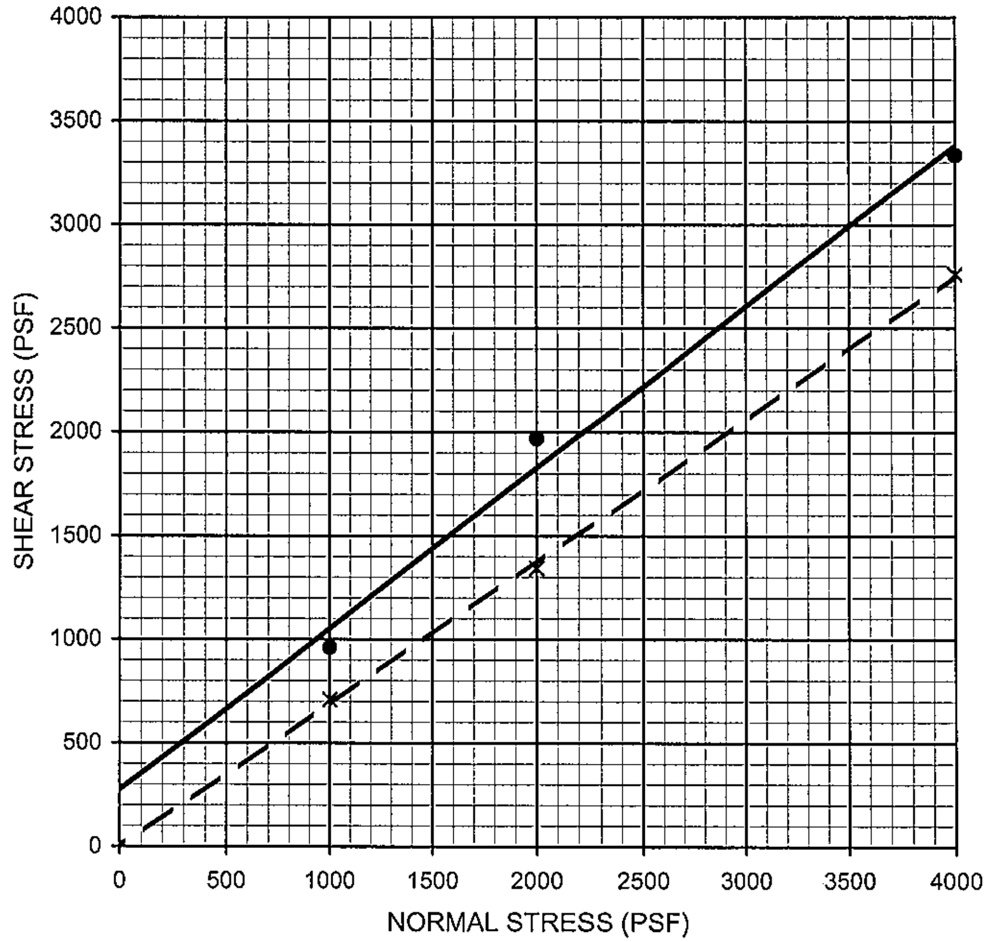
Ninyo & Moore		ATTERBERG LIMITS TEST RESULTS	FIGURE
PROJECT NO.	DATE	HALL OF JUSTICE, 211 WEST TEMPLE STREET LOS ANGELES, CALIFORNIA	C-1
207247038	9/10		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Equivalent Soil Type
Claystone	—●—	B-3	5.0-6.5	Peak	90	44	CL
Claystone	- - X - -	B-3	5.0-6.5	Ultimate	0	38	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	HALL OF JUSTICE, 211 WEST TEMPLE STREET LOS ANGELES, CALIFORNIA	C-2
207247038	9/10		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Equivalent Soil Type
Claystone	—●—	B-3	25.0-26.5	Peak	275	38	CL
Claystone	- - X - -	B-3	25.0-26.5	Ultimate	0	35	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	HALL OF JUSTICE, 211 WEST TEMPLE STREET LOS ANGELES, CALIFORNIA	C-3
207247038	9/10		

EET

SAMPLE LOCATION	SAMPLE DEPTH (FT)	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (PCF)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (IN)	EXPANSION INDEX	POTENTIAL EXPANSION
B-3	0.5-4.0	13.0	98.9	27.4	0.077	77	Medium

PERFORMED IN GENERAL ACCORDANCE WITH ☐ UBC STANDARD 18-2 ☑ ASTM D 4829

Ninyo & Moore		EXPANSION INDEX TEST RESULTS	FIGURE
PROJECT NO.	DATE	HALL OF JUSTICE, 211 WEST TEMPLE STREET LOS ANGELES, CALIFORNIA	C-4
207247038	9/10		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2] (ppm)	SULFATE CONTENT [2] (%)	CHLORIDE CONTENT [3] (ppm)
B-1	5.0-6.5	7.2	350	5700	0.570	100

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643

[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417

[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

Ninyo & Moore

CORROSIVITY TEST RESULTS

PROJECT NO.	DATE
207247038	9/10

HALL OF JUSTICE, 211 WEST TEMPLE STREET
LOS ANGELES, CALIFORNIA

FIGURE

C-5

SAMPLE LOCATION	SAMPLE DEPTH (FT)	SOIL TYPE	R-VALUE
B-2	1.5-5.0	CLAYSTONE	42

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

Ninyo & Moore		R-VALUE TEST RESULTS	FIGURE
PROJECT NO.	DATE	HALL OF JUSTICE, 211 WEST TEMPLE STREET LOS ANGELES, CALIFORNIA	C-6
207247038	9/10		