September 20, 2004

Fifield Realty Corp.
2010 Main Street, Suite 610
Irvine, CA 96214
Attention: Mr. David K. Robbins,
Executive Vice President & Principal

Subject: **TECHNICAL TRANSMITTAL**
Support of Environmental Impact Report
Proposed High-Rise Condominium Building Development
10250 Wilshire Boulevard
Los Angeles, California
MACTEC PROJECT 4951-04-1071

Gentlemen:

Pursuant to our Scope of Work (SOW) dated May 26, 2004, in connection with the Environmental Impact Report (EIR) for the property located at 10250 Wilshire Boulevard, Los Angeles, California (Site), MACTEC Engineering and Consulting, Inc. (MACTEC) is pleased to submit this Technical Transmittal describing our findings.

OBJECTIVES

The general objective of this phase of work is to provide analysis of the following issues:

1. Determine the settlement, if any, resulting from groundwater withdrawal after drawdown around the Site has been estimated based on the pump test results,

2. Assess liquefaction potential, and,

3. Define how groundwater flows across the Site.

Additionally, MACTEC has been requested to:

4. Define the distribution of benzene, if any, at the Site.

RESULTS

The results of this phase of the investigation are described below. This section describes settlement at the Site as it relates to the geologic and groundwater conditions, historic groundwater levels, and local offsite and onsite investigations.

1 SETTLEMENT

Based on the information provided below, the groundwater levels at and in the vicinity of the Site have fluctuated, during the recent past, *more* than the planned depth of the Site excavation and the associated dewatering. This demonstrates that the Site has experienced more natural groundwater-level fluctuation than will occur when the Site is dewatered for construction. This further demonstrates that the settlement that could be caused by the planned dewatering at the Site has already occurred, and only minor settlement could occur at the Site and adjacent sites as a result of the dewatering operations at the Site.

We have computed a settlement of approximately 0.2 to 0.3 inch at the Site and about 0.1 to 0.2 inch at areas adjacent to the Site. Based on prior settlement monitoring, the actual settlement that could occur at the Site and at adjacent areas will be much less than the computed settlement. In addition, the computed and actual settlement at the adjacent sites decrease with increased distance from the Site.

Settlement ranging between 0.1 and 0.2 inch does not cause distress, such as surface or wall cracks in adjacent buildings, homes or streets. Consequently, the construction of the project, including dewatering, will not produce settlement or subsidence that will adversely impact the adjacent properties or streets.

GEOLOGIC AND GROUNDWATER CONDITIONS

The Site is located in the northwestern portion of the Coastal Plain of the Los Angeles Groundwater Basin within the physiographic feature known as the Sawtelle Plain. On a regional scale, the Sawtelle Plain is a Pleistocene-age alluvial fan that developed by alluvial dissection of the surrounding Santa Monica Plain, the surface of which now lies below the general elevation of the surrounding Santa Monica and Ocean Park Plains. The Sawtelle Plain slopes gently toward the southeast and is generally mantled by a thin veneer of younger alluvium (Dibblee, 1991 and Department of Water Resources, 1961).

In the immediate Site vicinity, the exposed geologic materials consist of Pleistocene-age alluvial deposits (older alluvial-fan deposits). Locally, the Pleistocene alluvial surface has been incised by stream activity from drainage of the fan surface, resulting in low-lying drainages located along the southern edge of the older alluvial deposits. Specifically, the Site is located at the western edge of a broad low-lying drainage, one of the several drainages that have been incised into the southern edge of the upper surface of the Pleistocene fan. The drainage is in-filled with a thin veneer of Holocene-age (younger) alluvial deposits.

Based on our explorations, the geologic materials underlying the Site consist of an interbedded sequence of well-graded sand, silty-sand, silt, silty-clay, and sandy-clay with some gravel to a depth of 100 feet beneath the existing ground surface (bgs). These soils form a complex multiple-aquifer system at the Site which is comprised of at least seven distinct soil layers (see Figure 6). The coarse-grained soils form aquifers and the fine-grained deposits form aquitards. The upper 45 to 50 feet of materials encountered in our explorations are likely to be of Holocene age and the lower portion of the sequence is considered to be Pleistocene age. The majority of the planned excavation will be within the upper 45 to 50 feet of soils. Three small sumps will be located within the excavation at a depth of approximately 58 feet bgs. The size of the excavation is small with respect to the aerial extent of the regional complex-multiple aquifer system.

The Site lies within Township 1 South, Range 15 West, Section 23, as shown on the U. S. Geological Survey, Beverly Hills, 7½-minute Quadrangle (Figure 1). The Site aquifer system is part of the much larger complex multiple-aquifer system called the Santa Monica Groundwater Sub-basin that is contiguous throughout the Los Angeles County Groundwater Basin. Groundwater flow in the Holocene deposits is southeast (see *3 Groundwater Beneath The Site*, below).

Data obtained from 29 wells installed by MACTEC during June 7 through 10, 2004 indicate that the depth to groundwater ranged between 24 and 30 feet below top of casing (TOC) on July 2, 2004 (see *3 Groundwater Beneath The Site*, below). These depths correspond to groundwater elevations between 299 and 301 feet relative to the National Geodetic Vertical Datum of 1929 (NGVD). The elevation of the base of the planned excavation at the center of the building and the lowest sump will be approximately 281 and 269 feet NGVD, respectively. Thus for the most part, the groundwater level in the excavation will be lowered by 18 to 20 feet. At the sumps, the groundwater level will be lowered by 30 to 32 feet.

HISTORIC GROUNDWATER LEVELS

Groundwater levels in the past have fluctuated significantly, and hence the soils at the Site and in the immediate vicinity have already experienced multiple cycles of loading and unloading based on variations of groundwater levels. A groundwater-level fluctuation is a rise or fall in the groundwater level as measured in a well or boring. When groundwater levels fall, loading occurs and when levels rise, unloading occurs. Data supporting the evidence that groundwater levels in the past have fluctuated significantly are presented below.

Table 1 is a list of groundwater levels obtained from surrounding wells and borings advanced at MACTEC project locations. Groundwater data for wells were obtained from the State of California Department of Water Resources (DWR), the California Geological Survey (CGS, formerly known as the California Division of Mines and Geology), the U. S. Geological Survey (USGS), other agencies including the County of Los Angeles, and from MACTEC Projects.

Figure 1 shows the Site location, MACTEC projects, wells, and the maximum amount of observed groundwater-level fluctuation in wells. These data are summarized on Table 1. Maximum groundwater-level fluctuation for wells in the immediate vicinity of the Site (wells 15, 16, 17, 20, and 21) ranged from 19 to 139 feet. Maximum fluctuations for wells south of the Site near Culver City ranged from 12 to 47 feet. It is not known how deep the wells are and what parts of the regional complex multiple aquifer system the wells are completed in. Thus it is not known if these wells are in the same aquifer system as the Site wells.

Groundwater in the shallow Holocene deposits flows under unconfined, confined, and semi-confined conditions. Groundwater levels in the vicinity of the Site are influenced by local precipitation, irrigation, and local groundwater pumping. Each of these stresses can produce groundwater-level fluctuations.

The historic, since the turn of the century, high groundwater level in the *area* of the Site is approximately 23 feet bgs (California Division of Mines and Geology, 1998). This is based on information and published data obtained from the CGS, the USGS, the DWR, monitoring data from nearby groundwater wells, and available subsurface information from nearby MACTEC geotechnical investigations. The information from California Division of Mines and Geology (1998) indicates however, that less than 1 mile south of the Site, the historic depth to groundwater was less than 10 feet. Data from a well previously constructed at the Site indicate that the high groundwater level at the Site was 17.7 feet TOC on August 6, 2003. The well with the greatest historic water-level fluctuation of 139 feet is Well ID 20 (see Table 1), State well 1S/15W-12L01 near Roxbury Park. The depth of this well is not known.

The nearest groundwater monitoring well is Well ID 14, County of Los Angeles Well No. 2583B, located approximately 2,000 feet east of the Site. This well is not currently being monitored. However, groundwater-level information for the monitoring period of 1963 to 1978 is on file in the County of Los Angeles offices. Based on the available information, the groundwater level in this artesian well fluctuated during the monitoring period from 12.8 feet TOC to flowing at the ground surface during 1976.

The next closest monitoring well is Well ID 17, California State Well No. 1S/R15W-24M01, located approximately 3,200 feet east of the Site. Monitoring information for this well was available for the period from 1934 to 1946. During this time period, the groundwater level fluctuated approximately 19 feet. The highest groundwater level recorded was at a depth of 45 feet in 1938 and the deepest water level was measured at a depth of 64 feet in 1936. Groundwater levels at the Site have similarly risen and fallen during this time period because the Site most likely experienced the same meteorological stresses that caused the fluctuations at well 17. This 19-foot groundwater-level fluctuation during the two-year period in well 17 represents approximately the same magnitude of decline that will be encountered at the Site when the Site is dewatered. Prior to, and since recording of these data began, there have been similar periods of groundwater-level fluctuation.

As described above, another source of groundwater-level data is from previously conducted MACTEC geotechnical investigations at adjacent sites (Table 1, Site IDs 1 through 13). These sites are plotted as “blue” symbols on Figure 1. The borehole information from each MACTEC Project is summarized on Table 1. As a result of variation in existing grades, these data indicate that the depth and occurrence of groundwater is variable from site to site.

The embedded table below contains a brief summary of groundwater levels encountered in borings that were advanced at nearby properties along Wilshire Boulevard between Comstock and Beverly Glen. These properties are shown on the aerial photograph on Figure 2. Based on our review of the geotechnical reports of these properties, the excavations at these properties did not extend below the water table. Thus, groundwater was not encountered at these sites.

Geotechnical Investigations Conducted Along Wilshire Boulevard Between Beverly Glen and Comstock (*see* Figure 2)

MACTEC Project Site ID /Address/Year of Investigation	Depth to Groundwater (Feet bgs[1])/ Approximate Groundwater Elevation (Feet NGVD[2])
2 /865 and 875 Comstock/ 1960	30-45 / 299-306.5[b]
3 /865 and 875 Comstock/ 1960	Groundwater not encountered to 35 feet max depth drilled / soils dry above 340-343[b]
12 /10350 Wilshire Blvd/ 1980	45 – 66.5 /303.5 to 308 [b]
9a /10351 Wilshire Blvd/ 1987	Groundwater not encountered to 42 feet max depth drilled / soils dry above 321.5-338.1[b]
Site /10250 Wilshire Blvd/ 1972	25 / 300 [b]
Site /10250 Wilshire Blvd/ 2003	18 / 300 [w3]

Notes: **bgs[1]** - below ground surface
NGVD[2] - National Geodetic Vertical Datum of 1929
[3] - Top of well casing not surveyed
[b] - Borehole Data
[w] - Well Data

GROUND SETTLEMENT AND GROUNDWATER DEWATERING

MACTEC has been involved in many projects where dewatering was used to lower the groundwater level below the bottom of planned subterranean excavation(s). Settlement monuments were used at several of these projects to measure the vertical settlement at and within the vicinity of the sites.

Table 2 summarizes information from several prominent projects where the subterranean excavation extended below the groundwater table. At each of theses sites listed on Table 2, dewatering was used to lower the groundwater level below the bottom of the planned excavations to allow for construction of each building under dry conditions. A sub-drain system was constructed beneath the lower basement floor slab of each of the buildings to prevent the groundwater from entering the building basements. Each project listed on Table 2 occurred without any reported settlement problems. To our knowledge, no distress evidences caused by dewatering-related settlement were observed at any of the existing structures adjacent to these projects.

The first four projects listed on Table 2 are located on Figure 1. The approximate planned excavation and groundwater depths for the Site are shown to enable a comparison to sites in the immediate vicinity where dewatering has occurred or is occurring and to demonstrate that the Site conditions are typical for this area.

The projects listed in Table 2 are 4 to 20 years old. Since MACTEC is one of the firms that would be first informed of any associated settlement problems related to the project, we have not been informed that any settlement distress caused by dewatering or the continuous groundwater withdrawal by the sub-drain systems has occurred at the project sites or at adjacent project sites.

One Westwood Building: One of these sites, located within the Site vicinity, is the One Westwood Building located at 10990 Wilshire Boulevard in Westwood, California (Figure 1). One Westwood Building is 17 stories high and underlain by a 7-level subterranean base structure that encompasses the entire site. A parking structure was constructed prior to the construction of One Westwood Building near the east property line of Westwood One. The parking structure is an on-grade, 13-level structure. The lower level of the One Westwood Building base structure was established at an elevation of 230 feet NGVD and the depth of the excavation extended about 80 feet below grade adjacent to the parking structure. Groundwater levels were measured at depths ranging between 42 and 46 feet below existing grade. These depths correspond to elevations ranging between 256 and 259 feet NGVD. The dewatering system placed at the One Westwood Building site lowered the water level at the site more than 30 feet, approximately 50% more than is planned at the subject Site.

To measure the settlement of the adjacent parking structure during the dewatering operation of the One Westwood site, settlement monuments were placed inside the parking structure. The monuments were placed on the 18 columns of the two rows of columns adjacent to Westwood One. The settlement measurements included the vertical and horizontal movements of the columns, and the measurements were made between October 31, 1984 and September 30, 1986.

Based on the settlement measurement data available to us, the vertical settlements of the columns ranged between 0.1 and 0.2 inches. Such settlements did not have any adverse effects, such as cracking, on the existing parking structure. Neither did we receive any subsidence complaints from adjacent landowners.

Weyerhaeuser Atriums: This project consists of four areas where dewatering was necessary to lower the groundwater levels to below the bottom of each excavation. Prior to discharging the collected groundwater from the sub-drain systems into the nearest storm drain, the groundwater is periodically tested for a list of contaminants pursuant to the requirements of the Los Angeles Regional Water Quality Control Board (LARWQCB). MACTEC has been collecting groundwater samples at the Weyerhaeuser Atrium for the last 10 to 12 years as required by the LARWQCB.

407 North Maple Drive: This is an active site where dewatering is presently occurring via a sub-drain system. The sub-drain dewatering system is currently removing approximately 420 gallons per minute (gpm) or approximately 0.6 million gallons of groundwater per day (MGD). During the initial phase of this project, the network of production wells was removing approximately 1 MGD or approximately 700 gpm. There are also two adjacent sites that are being dewatered. We are not aware of any reported adverse settlement effects due to dewatering at or adjacent to these sites.

ONSITE INVESTIGATION

To compute the settlements at the Site, and adjacent sites, resulting from the proposed dewatering operation, MACTEC established the lines of equal drawdown, based on the results of the pumping tests performed at the Site.

As groundwater levels are drawn down, the submerged soils lose their buoyancy (loss of weight carried by water) and the weight of the previously submerged soils increases. This increase in the weight increases the vertical soil pressure on the underlying soils and causes the soils to settle.

MACTEC computed the settlements at the Site and adjacent sites which would result from dewatering at the Site to lower the groundwater level below the bottom of the planned excavation, approximately 46 feet bgs. The results demonstrate that the settlement at the Site will range between 0.2 and 0.3 inches and the adjacent settlement (up to a few hundred feet from the site) will range between 0.1 and 0.2 inches at the existing grade. This settlement decreases with distance from the Site. As previously stated, such settlements will not have any adverse impacts at the Site, or the adjacent sites. However, as noted above, the vast majority of the settlement has occurred as the result of past groundwater-level fluctuations.

MACTEC plans to monitor and measure the settlements at the Site and adjacent sites during the dewatering operation. Based on our experience, we fully expect that the measured settlements at the monitoring points will be less than the computed settlements.

CONCLUSION

Based on the information provided in the section on historic groundwater levels, the groundwater levels at and in the vicinity of the Site have fluctuated during the recent past *more* than the planned depth of the Site excavation and the associated groundwater levels that will be encountered during dewatering. This demonstrates that:

- the Site has experienced at least as much groundwater-level fluctuation than will occur when the Site is dewatered,
- the majority of the settlement that could be caused by the planned dewatering at the Site has already occurred, and,
- only very minor settlement could occur at the Site and adjacent sites as a result of natural water-level fluctuation and the dewatering operations at the Site.

The data obtained for offsite and onsite investigations indicate that dewatering of the Site may cause minor settlement that will not adversely impact the surrounding sites. It is our opinion that dewatering for excavation purposes and settlement between 0.1 and 0.2 inches does not cause distress, such as surface or wall cracks in adjacent buildings, homes or streets. Consequently, the proposed construction of the project, including dewatering, will not produce settlement or subsidence that will adversely impact the adjacent properties or streets.

2 LIQUEFACTION

Liquefaction is a process by which water-saturated unconsolidated sediments lose their strength due to increased pore pressure during or after an earthquake. Liquefaction potential increases as the depth to groundwater decreases.

The Californian Geological Survey (CGS) has been performing a comprehensive and thorough evaluation of the liquefaction potential of the soils throughout California, including the area where the Site is located. Except for some remote areas that are not developed, the liquefaction evaluation has been completed. The evaluation of the liquefaction potential is based on the geologic and soils conditions within each geologic quadrangle and on the depth to groundwater. The geologic and soils conditions are determined by exploration borings performed by CGS and by geotechnical engineering firms and the depth to groundwater is determined by water monitoring wells that were installed throughout California.

Based on the information provided above, CGS determines the zones underlain by liquefiable soils and the zones underlain by non-liquefiable soils for each geologic quadrangle in California. The zones are identified by a light color for the zones underlain by unliquefiable soils and by a dark color for zones underlain by liquefiable soils.

The Site is part of the Beverly Hills Quadrangle and its location is identified on Figure 3. Based on the liquefaction map presented in Figure 3, the Site is not underlain by liquefiable soils. There is no liquefaction present at the Site, and its build-out will not increase any liquefaction potential.

3 GROUNDWATER BENEATH THE SITE

Groundwater at the Site moves through a complex multiple-aquifer system comprised of at least seven distinct soil layers. The aquifer system consists primarily of an interbedded sequence of well-graded sand, silty-sand, silt, silty-clay, and sandy-clay, with some gravel. The following soil properties, hydraulic conductivity (*K*), specific storage (S_s), and effective porosity (n_e), determine for the most part, how groundwater moves (magnitude of the rate of flow and direction of flow) and is stored in this aquifer system. Because the properties of the soils at the Site vary in direction and magnitude, this complex aquifer system is considered an anisotropic porous media.

Our investigation is focused on describing how groundwater moves within the upper 55 to 70 feet of subsurface soils. Groundwater at the Site and throughout the Los Angeles region moves not as relatively fast-flowing surface water, as in a river, but as a relatively slow-moving body of subsurface water through a complex, non-homogenous and anisotropic porous media. Groundwater moves both horizontally and vertically at the Site.

Static groundwater velocity rates currently vary between 0.03 and 4 feet/day across the Site. Velocity rates will increase as the excavation is constructed; early, as the excavation is deepened, groundwater velocity discharge rates will be large near the pumping wells and the excavation seepage faces. As the construction proceeds and the water table is lowered, the discharge rates and corresponding groundwater velocities will decrease. When pumping ceases, the water table will rise to its pre-pumping static position and groundwater velocity discharge rates will approach their pre-pumping rates. As discussed below, groundwater flow direction is generally south-southeast, but this direction also is variable.

For discussion purposes, two relatively permeable zones have been identified; a shallow zone that occurs up to a depth of approximately 30 feet below ground surface (bgs), and a deep zone that occurs between approximately 40 and 55 feet bgs.

It has been previously misstated that groundwater at the Site is a "…groundwater river flowing underground...". To more clearly define how groundwater moves at the Site, MACTEC used water levels measured in the 29 wells to construct groundwater elevation contour maps in plan and in section. These wells were constructed as a series of nine nested wells. At each location, three wells were constructed forming a "nest": a shallow well was screened across the water table, a deeper well was screened across a fine-grained zone, and the deepest well was screened across the lower sand. At nest 2 (N2) one deeper well was completed. MACTEC will continue to monitor groundwater levels at the Site during dewatering.

HORIZONTAL COMPONENT OF GROUNDWATER FLOW

SHALLOW ZONE

The depth to the water table in wells completed in the shallow zone, ranged from 24.63 to 30.31 feet from top of casing (TOC), elevation 301.20 to 299.53 feet NGVD.

Figure 4 shows the horizontal component of groundwater flow in the phreatic zone, the shallowest portion of the aquifer system. This figure shows the elevation of the water table and the horizontal component of groundwater flow. The direction of groundwater movement is indicated by the red and blue arrows. Groundwater generally moves south-southeast, along a flowpath roughly parallel with the eastern boundary of the Site. The horizontal component of groundwater velocity ranges from approximately 1.5 feet/day between nests N7 and N8, to 4 feet/day between nests N1 and N7.

Figure 4 shows a southwest component of groundwater flow. This is partially due to the nature of the well completions that were used to construct this potentiometric surface. The nested "N" wells were completed using relatively short screens and gravel packs, typically less than 10 feet. Groundwater levels in these wells reflect relatively *small discrete* soil intervals. The pumping well, Q1, which was also included on this figure, has a longer twenty-foot screen in a gravel pack that is fully penetrating. Groundwater levels measured in Q1 represent an *average* of the heads in all aquifer/aquitard units tapped by this 20-foot long interval. The southwest component of groundwater flow is thus due in part to the difference in lengths of the screen/gravel packs intervals in the "N" and "Q" wells

DEEP ZONE

The depth to the groundwater in wells completed in the deep zone ranged from 24.58 to 31.62 feet from TOC, elevation 300.97 to 298.84 feet NGVD.

Figure 5 shows the horizontal component of groundwater flow in the deep zone of the multiple aquifer system. Groundwater generally moves from north to south. A minor westerly component of groundwater flow is more prevalent in the deeper zone than in the shallow zone.

VERTICAL COMPONENT OF GROUNDWATER FLOW

The nested wells (N1 through N9) were constructed to monitor groundwater levels within the vertical soil column at the Site and as such were used to determine the vertical components of groundwater flow throughout the complex multiple-aquifer system. Figure 6 is a fence diagram of the Site showing lithology and groundwater elevations. It was constructed roughly parallel with the principal direction of horizontal groundwater flow.

Using the July 2, 2004 water levels, this figure shows that groundwater moves predominantly horizontally within the sands and vertically in the clays. Groundwater flow in the silts is at an oblique angle. A small portion of the groundwater in the sands moves vertically downward into the underlying silts and clays. A portion of the groundwater in the silts moves vertically upward near nested well N9 and a small portion moves vertically downward. Groundwater in the middle sand unit near nested well N9 moves predominantly horizontally. The vertical component of groundwater flow ranges from approximately 0.03 to 0.30 feet/day.

CONCLUSION

Groundwater at the Site moves, not as "…a groundwater river flowing underground..." or as a relatively fast-flowing river, but as a relatively slow-moving body of subsurface water through a complex, non-homogenous and anisotropic porous media. Groundwater generally moves southeast, along a flowpath roughly parallel with the eastern boundary of the Site. A minor westerly component of groundwater flow occurs in the shallow zone and is more prevalent in the deeper zone. The groundwater velocity ranges from approximately 1.5 feet/day to 4 feet/day. The vertical component of groundwater flow ranges from approximately 0.03 to 0.30 feet/day.

4 BENZENE IN GROUNDWATER

Groundwater samples were collected from all nine (9) shallow wells and the pumping well (Q1). Benzene and other volatile organic compounds (VOCs), including Ethylbenzene, Toluene, and

Total Xylenes (BTEX) were not detected above the method detection limits. The method detection limits for all VOCs were 0.5 ug/L, except for Total Xylenes, which was 1 ug/L.

CONCLUSION

Benzene was not detected in groundwater samples collected at the Site. MACTEC will continue to monitor benzene in groundwater at the Site during dewatering.

References:

California Department of Water Resources, 1961, “Planned Utilization of Groundwater Basins of the Coastal Plain of Los Angeles County,” Bulletin 104, Appendix A.

California Division of Mines and Geology, 1999, “State of California Seismic Hazard Zones, Beverly Hills Quadrangle, Official Map” released March 25, 1999.

California Division of Mines and Geology, 1998, "Seismic Hazard Evaluation of the Beverly Hills 7.5 Minute Quadrangle, Los Angeles County, California" Open-File Report 98-14.

Dibblee, T. W., Jr., 1991, "Geologic Map of the Beverly Hills and Van Nuys (South ½) Quadrangles, California" *Dibblee Geological Foundation Map DF-31.*

Hill, R. L., Sprotte, E. C., Chapman, R. H., Chase, G. W., Bennett, J. H., Real, C. R., Borchardt, G., and Weber, F. H., Jr., 1979, “Earthquake Hazards Associated with Faults in the Greater Los Angeles Metropolitan Area, Los Angeles County, California, Including Faults in the Santa Monica–Raymond, Verdugo–Eagle Rock and Benedict Canyon Fault Zones,” *California Division of Mines and Geology, Open File Report 79-16LA.*

Los Angeles, City of, 1996, “Safety Element of the Los Angeles City General Plan.”

Thank you for the opportunity to submit this Technical Transmittal, and to continue working with you on this project. If you have any questions, please contact us.

Sincerely,

MACTEC ENGINEERING AND CONSULTING, INC.

Morley J. Weitzman, R.G. AZ	William J. O’Braitis, R.G.
Senior Hydrogeologist	Principal Geologist

Jake Kharraz
Principal Engineer

(2 copies submitted)

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REPORT OF GEOTECHNICAL INVESTIGATION PROPOSED HIGH-RISE CONDOMINIUM BUILDING DEVELOPMENT

10250 WILSHIRE BOULEVARD
LOS ANGELES, CALIFORNIA

Prepared for:

FIFIELD COMPANIES

Chicago, Illinois

August 26, 2003

MACTEC Project 4953-03-2451

MACTEC

MACTEC

August 26, 2003

Mr. Mike Pepper
Fifield Companies
20 North Wacker Drive
Chicago, IL 60606

Subject: **Report of Geotechnical Investigation**
Proposed High-Rise Condominium Building Development
10250 Wilshire Boulevard
Los Angeles, California
MACTEC Project 4953-03-2451

Dear Mr. Pepper:

We are pleased to submit this report presenting the results of our geotechnical investigation for the site of the proposed high-rise condominium building development at 10250 Wilshire Boulevard in Los Angeles, California. Our investigation was conducted in general accordance with our proposal dated July 2, 2003, as authorized by you on July 7, 2003.

The scope of our services was planned with Mr. Dale Yonkin of Nadel Architects Inc. Mr. Yonkin advised us of the general structural features of the proposed condominium development.

The results of our investigation and design recommendations are presented in this report. Please note that you or your representative should submit copies of this report to the appropriate governmental agencies for their review and approval prior to obtaining a building permit.

MACTEC Engineering and Consulting
200 Citadel Drive • Los Angeles, CA 90040
323-889-5300 • Fax: 323-721-6700

REPORT OF GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE CONDOMINIUM BUILDING DEVELOPMENT

10250 WILSHIRE BOULEVARD
LOS ANGELES, CALIFORNIA

Prepared for:

FIFIELD COMPANIES

Chicago, Illinois

MACTEC Engineering and Consulting, Inc.
F/k/a/ Law/Crandall, A Division of Law Engineering and Environmental Services, Inc.

Los Angeles, California

August 26, 2003

MACTEC Project 4953-03-2451

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APPENDIX: FIELD EXPLORATIONS AND LABORATORY TEST RESULTS

SUMMARY

We have performed a geotechnical investigation for the proposed high-rise condominium building development at 10250 Wilshire Boulevard in Los Angeles, California. The final details of the structural features of the proposed condominium development have not been established at this time and foundation load information is not available. However, based on information provided to us, the proposed high-rise condominium building will consist of a 21-story residential tower underlain by three subterranean parking levels.

We previously performed two preliminary geotechnical investigations of the subject site for similar high-rise condominium developments that were not constructed. Our preliminary geotechnical investigations for the previous projects were submitted in reports dated August 25, 1967 and November 7, 1972 (our Job Nos. A-65278 and A-72242, respectively). We drilled three borings for our previous preliminary geotechnical investigations of the subject site. The results of these borings (including their laboratory test results) are applicable to the currently proposed condominium building development. These borings, including their laboratory test results are incorporated in this report. To supplement our previous exploration borings, we recently drilled two additional borings within the subject site at locations that were not previously explored. To determine the fluctuation of ground water level and to perform chemical testing on water samples obtained at the site, we converted one of the borings into a monitoring well.

Based on current and prior exploration borings drilled to depths of about 100 feet below the existing grade, the natural soils underlying the site consist primarily of silt, clay, silty sand, and sands with considerable amounts of gravel and some cobbles in the sandy deposits. The upper natural soils are medium stiff or medium dense and become stiffer and denser with an increase in depth. Water was encountered at depths as shallow as 23 feet below the existing grade in the previous borings and at a depth of 18 feet in the recent borings.

The natural soils at and below the planned level of excavation are generally stiff or dense, and the proposed condominium building development may be supported on spread footings established in the stiff and dense undisturbed natural soils. Individual footings or a combination of individual and combined or continuous footings may be used.

As an alternative to spread footings, the proposed building may be supported on a mat-type foundation at the excavation level. Recommendations for design of spread footings and mat foundation are presented in the following sections.

The floor slab of a building supported on spread footings may be supported on grade at the excavation level. The mat foundation of a mat supported building may be used as the on-grade floor slab or a floor slab may be placed over the mat foundation. However, because of the ground water level, a permanent subdrain system would be necessary beneath the floor slab or beneath the mat foundations.

No significant difficulties due to the soil conditions are anticipated in excavating at the site; conventional earthmoving equipment may be used. When the necessary space is available for sloped excavation, temporary unsurcharged embankments may be sloped back without shoring. Shoring should be used where sloped excavations are not possible. The planned excavation will extend below the groundwater level and dewatering prior to excavation will be necessary to permit excavation for the basement level and for the mat-foundation or spread footings.

1.0 SCOPE

This report presents the result of our geotechnical investigation performed for the proposed high-rise condominium building development at 10250 Wilshire Boulevard in Los Angeles, California.

We previously performed two preliminary geotechnical investigations of the subject site for similar high-rise condominium developments that were not constructed. Our preliminary geotechnical investigations for the previous projects were submitted in reports dated August 25, 1965 and November 7, 1972 (our Job Nos. A-65278 and A-72242, respectively). In addition to our prior geotechnical investigation at the subject site, we were recently provided with a report of a foundation investigation dated July 20, 1988 and prepared by R.T. Franklin & Associates. The results of this investigation were used in the extent possible in developing the geotechnical recommendations presented in this report.

We drilled three borings for our previous preliminary geotechnical investigations of the subject site. The results of these borings (including their laboratory test results) are applicable to the currently proposed condominium building development. These borings, including their laboratory test results are incorporated in this report.

To supplement our previous exploration borings, we recently drilled two additional borings within the subject site at locations that were not previously explored. To determine the fluctuation of ground water level and to perform chemical testing on water samples obtained at the site, we converted one of the borings into a monitoring well.

It should be noted that the prior explorations at the subject site were performed by LeRoy Crandall and Associates (currently MACTEC). We acknowledge that we have reviewed the field data and the results of the laboratory tests performed by LeRoy Crandall and Associates and concur with the data findings.

The location of the proposed condominium development relative to the adjacent streets, and the locations of the current and prior borings are shown in Figure 1, Plot Plan.

The scope of the investigation included the following:

- Evaluate the current and previous subsurface explorations to determine the nature and stratigraphy of the subsurface soils within the new construction site.
- Evaluate the liquefaction potential of the soils underlying the site.
- Provide recommendations for appropriate foundations together with the necessary design parameters.
- Provide the results of a ground motion study.
- Determine the applicable site coefficient and seismic zonation based on the current Uniform Building Code.
- Provide recommendations for floor slab support.
- Provide recommendations for dewatering, excavation, and shoring.
- Provide recommendations for design of walls below grade and retaining walls.
- Provide recommendations for a subdrain system.
- Provide recommendations for design of asphaltic and concrete paving.
- Provide recommendations relating to earthwork and grading.
- Provide the results of a soil corrosivity study.

Except for the ground motion study, the evaluation of the liquefaction potential of the soils underlying the site, determination of the site coefficient, seismic zonation, and near-source factors, the scope of this investigation did not include geologic or seismic studies for the site. However, this does not imply that there are geologic or seismic hazards affecting the site.

Our recommendations are based on the results of our current and prior field explorations, laboratory tests, and appropriate engineering analyses. The results of the current and prior field explorations and the laboratory tests, which form the basis of our recommendations, are presented in the Appendix.

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has been prepared for Fifield Companies and their design consultants to be used solely in the design of the proposed condominium building development. This report has not been prepared for use by other parties, and may not contain sufficient information for purposes of other parties or other uses.

2.0 STRUCTURAL CONSIDERATIONS

The final details of the structural features of the proposed condominium development have not been established at this time and foundation load information is not available. However, based on our discussions with Mr. Yonkin, the project architect, and on plans provided to us, the proposed high-rise condominium building to be constructed at the subject site will consist of a 21-story residential tower underlain by three subterranean parking levels. The subterranean parking levels will extend beyond the tower in plan. Excavation up to about 35 to 40 feet deep will be required.

3.0 SITE CONDITIONS

The site of the proposed building was previously occupied by buildings that have been demolished. The site is currently a vacant lot. Spot elevations describing the existing topography of the site are shown in Figure 1. Underground utility lines and old obstructions may still exist beneath the site.

4.0 FIELD EXPLORATION AND LABORATORY TESTS

4.1 FIELD EXPLORATIONS

The site of the proposed development was recently explored by drilling two additional borings (Borings 3 and 4) at locations that were not previously explored. In addition, the three borings (Borings 1, 2 and 5) drilled previously at the subject site, are applicable to the new project. These borings, including their laboratory test results are incorporated into this report. Furthermore, for sampling and testing of the ground water and for future monitoring of the ground water level beneath the site, Boring 3, drilled recently, was converted into a water monitoring well. Details of the explorations and logs of the borings are presented in the attached Appendix.

4.2 LABORATORY TESTS

Laboratory tests were recently and previously performed on selected samples obtained from the borings to aid in the classification of the soils and to determine the pertinent engineering properties of the foundation soils. The following tests were performed:

- Moisture content and dry density determinations.
- Direct shear.
- Consolidation.
- Compaction.
- Stabilometer (R-value).

All testing was performed in general accordance with applicable ASTM specifications. Details of the laboratory testing program and test results are also presented in the Appendix. The results of the soil corrosivity study are presented at the end of the Appendix.

5.0 SOIL CONDITIONS

Three of the five exploration borings at the subject site were drilled in 1965 and 1972 and the other two borings were drilled recently. The soil conditions beneath the subject site, and beneath any other site, do not change significantly with time. Therefore, as previously stated, the results of the prior exploration borings drilled at the site are still applicable to the currently proposed condominium development to be constructed at the site.

Existing fill soils, about 2½ feet thick, were encountered in one of the two recent borings. The existing fill soils, which consist of clay are not uniformly well compacted and contain some debris. Deeper and/or poorer quality fill could occur between the boring locations and elsewhere at the site, particularly at the location of the existing buildings that were demolished. However, the existing fill soils will be removed automatically by the planned excavation.

Based on current and prior exploration borings drilled to depths of about 100 feet below the existing grade, the soils underlying the site consist primarily of silt, clay, silty sand, and sands with considerable amounts of gravel and some cobbles in the sandy deposits. The upper natural soils are medium stiff or medium dense and become stiffer and denser with an increase in depth. The soils at and below the excavation level are generally stiff or dense.

Water was encountered at depths as shallow as 23 feet below the existing grade in the previous borings and at a depth of 18 feet in the recent borings. However, based on the official data from the California Geological Survey (CGS), the historical ground-water level at the site is at a depth of about 23 feet below the existing grade.

Based on the soil corrosivity study performed by M. J. Schiff & Associates, Inc., Consulting Corrosion Engineers on soil samples obtained at the site, the site is classified as corrosive to ferrous metals, aggressive to copper, and not corrosive to portland cement concrete. The Schiff report, presented at the end of this report, should be referred to for a discussion of the corrosive characteristics of the soils at the site.

6.0 LIQUEFACTION POTETNIAL

Liquefaction potential is greatest where the ground water level is shallow, and loose, fine sands occur within 50 feet of the ground surface. Liquefaction potential decreases with increasing grain size and clay and gravel content, but increases as the ground acceleration and duration of shaking during an earthquake increase.

Based on the current official map of the State of California Seismic Hazard Zones (Beverly Hills Quadrangle, March 25, 1999) the site is not within a liquefiable zone. In addition, based on our past experience at the site and at nearby sites, the soils underlying the site are not subject to liquefaction. Accordingly, the potential for liquefaction of the soils underlying the site is considered to be very low.

7.0 RECOMMENDATIONS

7.1 GENERAL

As previously stated, the detailed structural features, including foundation load information, of the proposed condominium building are not available at this time. Therefore, the foundation design recommendations presented in this report should be reviewed and modified by us, if necessary, when the foundation loads are available.

The natural soils at and below the planned level of excavation are generally stiff or dense, and the proposed condominium building development may be supported on spread footings established in the stiff and dense undisturbed natural soils. Individual footings or a combination of individual and combined or continuous footings may be used.

As an alternative to spread footings, the proposed building may be supported on a mat-type foundation at the excavation level. Recommendations for design of spread footings and mat foundation are presented in the following sections.

No significant difficulties due to the soil conditions are anticipated in excavating at the site; conventional earthmoving equipment may be used. Where the necessary space is available for sloped excavation, temporary unsurcharged embankments may be sloped back without shoring. Shoring should be used where sloped excavations are not possible.

The planned excavation will extend below the groundwater level and dewatering prior to excavation will be necessary to permit excavation for the basement level, for the mat-foundation or spread footings, and for the elevator pits. It should be realized that a permit from the State of California Regional Water Quality Board will have to be obtained to discharge the water from the dewatering system into the storm drain.

The floor slab of a building supported on spread footings may be supported on grade at the excavation level. The mat foundation of a mat supported building may be used as the on-grade floor slab or a floor slab may be placed over the mat foundation. However, because of the ground water level, a permanent subdrain system would be necessary to prevent the development of hydrostatic pressures on the mat foundation or the floor slab of the lower level and on the lower portions of the subterranean walls, unless the building is designed to resist the hydrostatic pressures. The present and future ground water quality at the site may impact water disposal requirements.

Minor structures will be part of the proposed development. These minor structures may be supported on shallow spread footings established in either properly compacted fill and/or the undisturbed natural soils.

Where stepped footings are to be used, such footings should be stepped at 1½:1 (horizontal to vertical) or flatter. To avoid surcharging the existing and new underground structures and basement walls, footings adjacent to these structures should extend below planes drawn upward at 45 degrees from the bases of the walls or from the inverts of the utility lines. However, the subterranean walls could be designed to support the lateral surcharge pressures from the footings if the footings did not extend below the 45-degree plane.

7.2 FOUNDATIONS

Bearing Values

Spread footings, if used for support of the proposed building, carried at least 1 foot into the undisturbed stiff or dense natural soils at the planned excavation level, and at least 3 feet below the lowest adjacent grade or floor level can be designed to impose a net dead-plus-live load pressure of up to 8,000 pounds per square foot. The footing excavations should be deepened as necessary to extend into undisturbed natural soils.

The mat-type foundation supporting the entire building established in the undisturbed natural soils may be designed to impose a net dead-plus-live load soil pressure of up to 5,000 pounds per square foot. The mat should be sufficiently reinforced and thickened to distribute the imposed loads uniformly across the mat.

Footings for minor structures (including auxiliary retaining walls, free-standing walls, and elevator pit walls) that are structurally separate from the proposed building may be designed to impose a net dead-plus-live load pressure of 1,500 pounds per square foot, at a depth of 2 feet below the adjacent grade. Such footings may be established in either properly compacted fill and/or undisturbed natural soils.

The recommended bearing values are net values, and the weight of concrete in the footings and mat may be taken as 50 pounds per cubic foot; the weight of soil backfill may be neglected when determining the downward loads from the structure. A one-third increase in the above bearing values may be used when considering wind or seismic loads.

While the actual bearing value of the compacted fill will depend on the materials used and the compaction methods employed, the quoted bearing value will be applicable if acceptable soils are used and are compacted as recommended. The bearing value of the compacted fill should be confirmed during grading.

Settlement

The settlement of the proposed condominium building supported on spread footings or mat-type foundation in the manners recommended, will depend on the foundation loads imposed. Therefore, settlement analyses should be performed when the foundation load information is available. The results of the settlement analyses will be used to confirm or modify the foundation design recommendations presented in this report.

Modulus of Subgrade Reaction

To assist in the structural analyses of the mat foundation, a modulus of subgrade reaction (k) of 50 pounds per cubic inch may be used for the soils underlying the mat foundation. This value was estimated from available data and published empirical relationships.

Lateral Loads

Lateral loads may be resisted by soil friction and passive resistance against the footings or the mat foundation. A coefficient of friction of 0.5 may be used between spread footings the floor slab and the mat and the supporting soils. The passive resistance of the undisturbed natural soils against footings or the mat may be assumed to be 300 pounds per cubic foot. A one-third increase in the passive value may be used for wind or seismic loads.

The passive resistance of the soils and the frictional resistance between the floor slab, footings or the mat and the supporting soils may be combined without reduction in determining the total lateral resistance.

Ultimate Values

The recommended bearing and lateral load design values for the proposed building are for use with loadings determined by a conventional working stress design. When considering an ultimate design approach, the recommended design values may be multiplied by the following factors:

Design Item	Ultimate Design Factor
Bearing Value	3.0
Passive Pressure	1.75
Coefficient of Friction	1.25

In no event, however, should foundation sizes be less than those required for dead-plus-live loads when using the working stress design values.

Foundation Observation

To verify the presence of satisfactory soils at design elevations, all footing or mat excavations should be observed by personnel of our firm. Footings or mat should be deepened as necessary to reach satisfactory supporting soils. Where foundation excavations are deeper than 4 feet, the sides of the excavations should be sloped back at 1:1 or shored for safety.

Backfill around and over foundations and utility trench backfill within the building area should be mechanically compacted; flooding should not be permitted.

Inspection of the foundation excavations may also be required by the appropriate reviewing governmental agencies. The contractor should be familiar with the inspection requirements of the reviewing agencies.

7.3 GROUND MOTION STUDY

Ground motions were postulated corresponding to earthquake levels having a 10% probability of exceedence during a 50-year time period (designated the Design Basis Earthquake DBE) and a 10% probability of exceedence during in a 100-year time period (designated the Maximum Capable

Earthquake, MCE). The probabilistic response spectra developed for this study are referred to as the site-specific response spectra.

The site-specific response spectra for the DBE and MCE levels of shaking specified were determined by a Probabilistic Seismic Hazard Analysis (PSHA) using the computer program EZFRISK, version 5.72. The response spectra were developed using the average ground motion from the attenuation relations discussed Abrahamson & Silva (1997), Sadigh et al. (1997), and Boore et al. (1997). For the Boore et al. attenuation, we have used a shear wave velocity of 360 meters per second (1181 feet per second) for the upper 30 meters (100 feet) below the planned foundation level. For the Abrahmson & Silva and the Sadigh et al. attenuation equations, we have used the attenuation coefficients for deep soil sites. Dispersion in the ground motion attenuation relationships was considered by inclusion of the standard deviation of the ground motion data. The response spectra for the horizontal component of shaking for the DBE and MCE ground motions are presented in Figures 2 and 3 for structural damping values of 2%, 5%, 7%, and 10%. The response spectra in digitized form are shown in Tables 1 and 2.

7.4 SITE COEFFICIENT AND SEISMIC ZONATION

The site coefficient, S, can be determined as established in the Earthquake Regulations under Section 1969 of the Uniform Building Code (UBC), of the 1997 edition, for seismic design of the proposed development. Based on a review of the local soil and geologic conditions, the site may be classified as Soil Profile Type S_D, as specified in the 1997 code. The site is located within UBC Seismic Zone 4.

The site is near the Santa Monica fault, which has been determined to be a Type B seismic source by the California Division of Mines and Geology. The development is located at a distance of less than 2 kilometers from the Santa Monica fault according to Map M-32 (ICBO, 1998). At this distance for a seismic source type B, the near source factors, N_a and N_v, are to be taken as 1.3 and 1.6, respectively, according to Tables 16-S and 16-T of the 1997 UBC.

7.5 FLOOR SLAB OR MAT SUPPORT

The required filter material for the subdrain system will offer adequate support for the floor slab or the mat foundation of the lower subterranean parking level. The at-grade concrete slabs and walks adjacent to the proposed building may be also supported on grade.

The lower floor slab or the mat of the building will be used for parking and should not be sensitive to capillary moisture, however, where vinyl or other moisture-sensitive floor covering is planned for portions of the lower floor slab or the mat, we recommend that the floor slab or the mat foundation be underlain by a capillary break consisting of a vapor-retarding membrane at least 10 mils thick. A 2-inch-thick layer of sand should be placed beneath the membrane to decrease the possibility of damage to the membrane.

If a membrane is used, a low-slump concrete should be used to minimize possible curling of the slab or the mat. A 2-inch-thick layer of coarse sand should be placed over the membrane to reduce slab curling. Care should be taken during the placement of the concrete to prevent displacement of the sand. The concrete slab should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.

Where vinyl or other moisture-sensitive floor covering is not planned, the floor slab or the mat foundation may be supported directly on the subdrain materials.

7.6 EXCAVATION SLOPES AND DEWATERING

Excavation up to about 35 to 40 feet deep will be required for the lower subterranean parking level of the proposed development. Where the necessary space is available, temporary unsurcharged embankments may be sloped back at 1:1 without shoring. Where space is not available, shoring will be required. Data for design of shoring are presented in a following section.

Inspection of the foundation excavations may also be required by the appropriate reviewing governmental agencies. The contractor should be familiar with the inspection requirements of the reviewing agencies.

Where sloped embankments are used, the tops of the slopes should be barricaded to prevent vehicles and storage loads within 10 feet of the tops of the slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes; we should be advised of such heavy vehicles so that specific setback requirements can be established. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes, where necessary, to prevent runoff water form entering the excavation and eroding the slope faces.

The soils at the excavated level will be wet and spongy. To provide support for foundations and a working base for men and equipment, a layer of 1½-inch crushed rock at least 1-foot-thick will be necessary over the excavated surface.

The excavation should be observed by personnel of our firm so that any necessary modifications based on variations in the soil conditions encountered can be made. All applicable safety requirements, including OSHA requirements, should be met.

The excavation for the spread footings or the mat foundation will extend below the ground water level, and dewatering of the excavation will be required.

The dewatering could be done by means of dewatering wells located around the perimeter of the site with supplementary wells located within the limits of the excavation. The dewatering system should be placed several weeks prior to the start of excavation. In addition, a few monitoring wells should be placed at the site to monitor the water level. The excavation at the site should not start until the water level is withdrawn a few feet below the bottom of the excavation. In addition, drainage trenches excavated at the bottom of the excavation and backfilled with crushed rock may be used to supplement the wells. The trenches could be placed in areas between the foundation locations and should drain, together with the wells, into sumps equipped with pumps. The trenching should be coordinated with the construction sequencing.

The dewatering system should be designed by a competent dewatering contractor. The contractor should determine the size, spacing, and depths of the dewatering wells. In addition, the contractor should determine the locations and sizes of any necessary trenches within the excavation, and the volume of water inflow from the dewatering system.

It should be realized that a permit from the State of California Regional Water Quality Control Board would have to be obtained to discharge the water into a storm drain. To obtain such a permit, additional chemical tests may have to be performed on ground water samples obtained at the site to verify that chemicals or pollutants within the water do not exceed the allowable limits for discharging into the storm drain.

7.7 SHORING

General

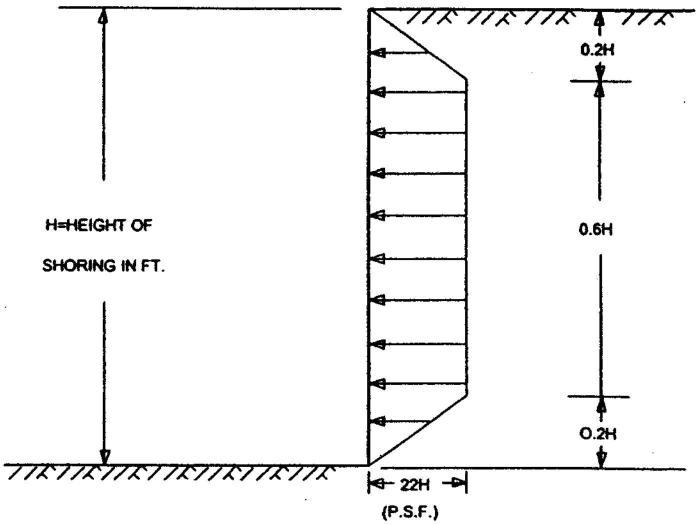
Where there is not sufficient space for sloped embankments, shoring will be required. One method of shoring would consist of steel soldier piles placed in drilled holes, backfilled with concrete, and tied back with earth anchors. Some difficulty may be encountered in the drilling of the soldier piles and the anchors because of shallow ground water and caving in the sandy and gravelly deposits. Special techniques and measures will be necessary in some areas to permit the proper installation of the soldier piles and the tie-back anchors. In addition, if there is not sufficient space to install the tie-back anchors to the desired lengths on any side of the excavation, the soldier piles of the shoring system may be internally braced.

The following information on the design and installation of the shoring is as complete as possible at this time. We can furnish any additional required data as the design progresses. Also, we suggest that our firm review the final shoring plans and specifications prior to bidding or negotiating with a shoring contractor.

Lateral Pressures

For design of cantilevered shoring, a triangular distribution of lateral earth pressure may be used. It may be assumed that the retained soils with a level surface behind the cantilevered shoring will exert a lateral pressure equal to that developed by a fluid with a density of 30 pounds per cubic foot. Where retained soils are partially sloped at 1:1 above the shoring, it may be assumed that the soils will exert lateral pressures equal to 60 pounds per cubic foot.

For the design of tied-back or braced shoring, we recommend the use of a trapezoidal distribution of earth pressure. The recommended pressure distribution, for the case where the grade is level behind the shoring, is illustrated in the following diagram with the maximum pressure equal to 22H in pounds per square foot, where H is the height of the shoring in feet. Where a combination of sloped embankment and shoring is used, the pressure would be greater and must be determined for each combination. However, where the required soils are sloped at 1:1 above the shoring, it may be assumed that the soils will exert lateral pressure equal to 44H pounds per cubic foot.



In addition to the recommended earth pressure, the upper 10 feet of shoring adjacent to the streets and vehicular traffic areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected. Furthermore, the shoring system should be designed to support the lateral surcharge pressures imposed by concrete trucks, cranes, and other heavy construction equipment placed near the shoring system.

Design of Soldier Piles

For the design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 500 pounds per square foot per foot of depth at the excavated surface, up to a maximum of 5,000 pounds per square foot. To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavations may be a lean-mix concrete. However, the concrete used in that portion of the soldier pile, which is below the planned excavated level, should be of sufficient strength to adequately transfer the imposed loads to the surrounding soils.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the anchor load. The coefficient of friction between the soldier piles and the retained earth may be taken as 0.3. This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean-mix concrete and the retained earth. In addition, provided that the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. For resisting the downward loads, the frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken equal to 400 pounds per square foot.

Lagging

Continuous lagging will be required between the soldier piles. The soldier piles and anchors should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. We recommend that the lagging be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot.

Anchor Design

Tie-back friction anchors may be used to resist lateral loads. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees with the vertical through the bottom of the excavation. The anchors should extend at least 30 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities.

The capacities of anchors should be determined by testing of the initial anchors as outlined in a following section. For design purposes, we estimate that drilled friction anchors will develop and average friction value of 500 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 6 feet on centers, no reduction in the capacity of the anchors needs to be considered due to group action.

Anchor Installation

The anchors may be installed at angles of 15 to 40 degrees below the horizontal. Caving of the anchor holes should be anticipated and provisions made to minimize such caving. The anchors should be filled with concrete placed by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. To minimize chances of caving, we suggest that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill may contain a small amount of cement to allow the sand to be placed by pumping.

Anchor Testing

Our representative should select at least two of the initial anchors for 24-hour 200% test, and five additional anchors for quick 200% tests. The purpose of the 200% test is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

The total deflection during the 24-hour 200% test should not exceed 12 inches during loading; the anchor deflection should not exceed 0.75 inch during the 24-hour period, measured after the 200% test load is applied. If the anchor movement after the 200% load has been applied for 12 hours is less than 0.5 inch, and the movement over the previous 4 hours has been less than 0.1 inch, the test may be terminated.

For the quick 200% tests, the 200% test load should be maintained for 30 minutes. The total deflection of the anchor during the 200% quick test should not exceed 12 inches; the deflection after the 200% test load has been applied should not exceed 0.25 inch during the 30-minute period. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

All of the production anchors should be pretested to at least 150% of the design load; the total deflection during the tests should not exceed 12 inches. The rate of creep under the 150% test should not exceed 0.1 inch over a 15-minute period for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked-off at the design load. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than 10% from the design load, the load should be reset until the anchor is locked-off within 10% of the design load.

The installation of the anchors and the testing of the completed anchors should be observed by our firm.

Internal Bracing

Raker bracing may be used to internally brace the soldier piles. If used, raker bracing could be supported laterally by temporary concrete footing (deadmen) or by the permanent interior footings. For design of such temporary footings, poured with the bearing surface normal to the rakers inclined at 45 to 60 degrees with the vertical, a bearing value of 4,500 pounds per square foot may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. To reduce the movement of the shoring, the rakers should be tightly wedged against the footings and/or shoring system.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized, however, that some deflection will occur. We estimate that this deflection could be on the order of 1 inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of the utilities in the adjacent streets. If it is desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design.

Monitoring

Some means of monitoring the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles. We will be pleased to discuss this further with the design consultants and the contractor when the design of the shoring system has been finalized.

We recommend that any adjacent existing structure be surveyed for horizontal and vertical locations. Also, a careful survey of existing cracks and offsets in any adjacent structure should be performed and recorded and photographic records made.

7.8 WALLS BELOW GRADE

Lateral Pressures

For design of cantilevered retaining walls, where the surface of the backfill is level, it may be assumed that the soils will exert a lateral pressure equal to that developed by a fluid with a density of 35 pounds per cubic foot.

The subterranean walls should be designed to resist a trapezoidal distribution of lateral earth pressure. The lateral earth pressure on the permanent subterranean walls will be similar to that recommended for design of temporary shoring except that the maximum lateral pressure will be 24H in pounds per square foot, where H is the height of the walls in feet. The recommended earth pressure assumes that a subdrain system will be installed below the floor slab of the lower subterranean level and behind the subterranean walls, so that external hydrostatic pressure will not be developed, if the ground water rises to the historical depth of 25 feet below the existing grade.

In addition, to the recommended earth pressure, the upper 10 feet of walls adjacent to streets and vehicular traffic areas should be designed to resist a uniform lateral pressure of 120 pounds per square foot, acting as a result of an assumed 350 pounds per square foot surcharge behind the walls due to normal traffic. If the traffic is kept back at least 10 feet from the walls, the traffic surcharge may be neglected.

Waterproofing

As discussed in a following section, a subdrain system is recommended so that external water pressure will not be developed against the basement walls. However, walls below grade should be waterproofed.

Backfill

Required soil backfill should be mechanically compacted, in layers not more than 8 inches thick, to at least 90% of the maximum density obtainable by the ASTM Designation D1557-91 method of compaction. The backfill should be sufficiently impermeable when compacted to restrict the inflow of surface water. The placement of the upper on-site clay soils as a backfill behind walls below grade should be avoided. Some settlement of the deep backfill should be allowed for in planning sidewalks and utility connections.

7.9 SUBDRAIN SYSTEM

Ground water was encountered above the planned lower subterranean parking level and provisions must be taken to protect the building from hydrostatic pressure. The following recommendations for subdrain system are given beneath the floor slab (if spread footings are used) and beneath the mat foundation (if the mat is used) to support the building.

There are two alternative procedures that might be followed. A permanent subdrain system could be installed beneath the lower floor or mat of the building to maintain the water level below the lower subterranean level, or the lower subterranean floor slab or mat and the lower portions of the subterranean walls could be waterproofed and designed for the possible hydrostatic pressure. To compute the hydrostatic pressure, it may be assumed that the water level will be at a depth of 15 feet below the existing grade. The design of the lower floor slab or mat to resist the possible hydrostatic pressure would require a thorough waterproofing installation and relatively thick floor slab or mat. Installation of a completely watertight waterproofing system will be difficult. If such a system is desired, we suggest consulting with a contractor experienced in the installation of such a system.

If it is decided to install a subdrain system, it should be realized that a permit from the State of California Regional Water Quality Control board will have to be obtained to discharge the subdrain water into the storm drain. To obtain such a permit, chemical tests will have to be performed on ground water samples obtained at the site to verify that chemicals or pollutants within the water do not exceed the allowable limits for discharging into the storm drain or for using irrigation or other purposes. A water treatment system will be required if the chemicals or pollutants within the water exceeds the allowable limits.

For a subdrain system, we recommend that the lower floor or mat of the building be underlain by a layer of filter material approximately 1 foot thick. The filter material should be drained by subdrain pipes leading to sump areas equipped with automatic pumping units. We suggest that the filter material meet the requirements of Class 2 Permeable Material as defined in Section 68 of the latest edition of the State of California, Department of Transportation, Standard Specifications. If Class 2 material is not available, ¾-inch crushed rock separated from the adjacent soils by a filter fabric may be used. The crushed rock should have less than 5% passing a No. 200 sieve. The drain lines should consist of perforated pipe placed, with the perforations down, in trenches extending at least 1 foot below the filter material. The trenches should be backfilled with material meeting the requirements of the Class 2 Permeable Material or lined with filter fabric and filled with ¾-inch crushed rock. The drain lines should extend around the perimeter of the building and should be spaced approximately 40 feet apart within the interior of the building. A slope of at least 2 inches per 100 feet should be used for the drain lines. Based on the results of a field pumping test, we suggest that the pumps and sumps be sized for a total inflow into the system of 450 gallons per minute. The actual inflow into the subdrain system is expected to be less.

In addition to the above drainage system, some means of draining the soils outside the exterior walls will be required. The means of accomplishing drainage outside the walls will depend primarily on the selected method of shoring and the method of constructing the exterior building walls. A drainage system behind the basement walls may be provided by strips of Miradrain 6000 (or equivalent). In our opinion, Miradrain 6000 (or equivalent), attached to the lagging and protected from the concrete placement of the walls, would provide satisfactory drainage. Continuous Miradrain may be placed at a depth starting at about 3 feet below the existing grade.

The Miradrain should be connected to weep holes at the bottom of the excavation. The weep holes should consist of solid pipes spaced at 8 feet on centers. At the connection of the weep holes and the Miradrain, the weep holes should be embedded in 1 cubic foot of free-drainage aggregate surrounded by a filter fabric. The weep holes should drain into the subdrain system placed beneath the slab of the lower subterranean level or into a solid pipe placed beneath the edge of the lower floor slab. The solid pipe should discharge into the sump.

The installed drainage system should be observed by personnel from our firm prior to being backfilled. Inspection of the drainage system may also be required by the reviewing governmental agencies.

We can provide additional data for design of the subdrain system as the features of the system are developed. In addition, we suggest that the design be reviewed after the excavation has been completed. If necessary, the system could be modified as indicated by the observed conditions, including the quantity of water pumped during construction dewatering.

7.10 PAVING

General

To provide support for paving, the subgrade soils should be prepared as recommended in Section 7.11. Compaction of the subgrade to at least 90%, including trench backfills, will be important for paving support. The preparation of the parking area subgrade should be done immediately prior to the placement of the base course. Proper drainage of the paved areas should be provided since this will reduce moisture infiltration into the subgrade and increase the life of the paving.

To provide information for paving design, stabilometer (R-value) test was performed on a sample of the upper soils. The results of the test, which indicated an R-value of 34, are presented in Figures A-5.1 through A-5.3.

Asphaltic Paving

The required asphaltic paving and base course thickness will depend on the anticipated wheel loads and volume of traffic (Traffic Index). The recommended paving sections for a range of Traffic Indices are presented below. The paving sections were determined using the Caltrans design method.

Assumed Traffic Index	Asphaltic Paving (Inches)	Base Course (Inches)
4½ (Automobile Parking)	3	4
5½ (Driveways with Light Truck Traffic)	3	7
6½ (Roadways with Heavy Truck Traffic)	4	7

Careful inspection is recommended to verify that the recommended thicknesses or greater are achieved and that proper construction procedures are used. We could provide paving sections for other Traffic Index values if desired.

Portland Cement Concrete Paving

Portland cement concrete paving sections were determined in accordance with procedures developed by the Portland Cement Association. Concrete paving sections for a range of Traffic Indices are presented below. We have assumed that the portland cement concrete will have a compressive strength of at least 3,000 pounds per square inch.

Assumed Traffic Index	Paving Section (Inches)	Base Course (Inches)
4½ (Automobile Parking)	6	2
5½ (Driveways with Light Truck Traffic)	6½	2
6½ (Driveways with Heavy Truck Traffic)	7	2

We recommend that the concrete paving be properly reinforced. In addition, dowels are recommended at joints in the paving to reduce possible offsets.

Base Course

The base course for both asphaltic and concrete paving should meet the specifications for Class 2 Aggregate Base as defined in Section 26 of the latest edition of the State of California, Department of Transportation, Standard Specifications. Alternatively, the base course could meet the specifications for untreated base as defined in Section 200-2 of the latest edition of the Standard Specifications for Public Works Construction. The base course should be compacted to at least 95%.

7.11 GRADING

General

To provide support for the footings of minor structures, and for at-grade concrete walks and slabs adjacent to the building and for paving, all existing fill and disturbed natural soils should be excavated and replaced with properly compacted fill, and all required additional fill should be properly compacted. The footings of the minor structures may be established in properly compacted fill and/or undisturbed natural soils.

Where excavations for minor footings are deeper than 4 feet, the sides of the excavations should be sloped back at 1:1 (horizontal to vertical) or shored for safety. All footing and utility trench backfills should be mechanically compacted; flooding should not be permitted.

Site Preparation and Moisture Conditioning

After excavating as recommended, the exposed soils in areas to receive additional fill should be inspected and any disturbed deposits should be excavated. The moisture content of the soils should be determined, and the soils should be slowly and uniformly moistened (or dried) as necessary to bring the soils to a uniformly moist condition. The moisture content of the cohesive soils and compacted fill should be brought to between 2% and 4% over optimum moisture content to a depth of 6 inches. The moisture content of any non-expansive materials should be brought to within 2% of optimum moisture content. The moisture content of the subgrade should be checked and approved prior to placing the required fill.

Subgrade Preparation

After moistening as required, the exposed soils should be compacted to at least 90% of the maximum dry density obtainable by the ASTM Designation D1557-91 method of compaction. The upper cohesive soils have a high moisture content, and it will be necessary to allow the surface to dry out prior to compacting. As an alternative, a layer of coarse crushed rock about 12 inches thick could be placed over the exposed soils to provide a base for the compaction of the required fill. A geotextile fabric could be placed to help stabilize the subgrade soils and reduce the amount of gravel required. Where grading is interrupted by rain, fill operations should not be resumed until the moisture content and dry density of the placed fill are satisfactory.

Compaction

After compacting the exposed soils, or after placing the gravel layer, the required fill should be placed in horizontal lifts not more than 8 inches thick and compacted to at least 90%. Relatively non-expansive soils shall be compacted at a moisture content varying no more than 2% below or above optimum moisture content. It is recommended that the moisture content of the on-site cohesive soils at the time of compaction be brought to between 2% and 4% over optimum moisture content.

Material for Fill

The on-site soils, less debris or organic materials within any existing fill soils, may be used in the required fills. Any on-site clay soils should not be used as backfill behind any walls below grade. All required imported fill should consist of relatively non-expansive soils. The Expansion Index of the selected relatively non-expansive material should be less than 35. Any import material should contain sufficient fines (binder material) so as to provide a compacted fill that will be relatively impermeable and will be stable in shallow trenches.

Field Observation

The reworking of the upper soils and the compaction of all required fill should be observed and tested by a representative of our firm. This representative should have at least the following duties:

- Observe the clearing and grubbing operations to assure that all unsuitable materials have been properly removed.
- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade, observe subgrade scarification, and delineate areas requiring overexcavation.
- Perform visual observation to evaluate the suitability of on-site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary.

- Perform field density and compaction testing to determine the percentage of compaction achieved during fill placement.
- Observe and probe foundation bearing materials to confirm that suitable bearing materials are present at the design grades.

The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits may be obtained and arrangements may be made for the required inspection(s).

8.0 BASIS FOR RECOMMENDATIONS

The recommendations provided in this report are based on our understanding of the described project information and on our interpretation of the data collected during the subsurface exploration. We have made our recommendations based on experience with similar subsurface conditions under similar loading conditions. The recommendations apply to the specific project discussed in this report; therefore, any change in the proposed condominium development configuration, location, or site grades should be provided to us so we may review our conclusions and recommendations and make any necessary modifications.

The recommendations provided in this report are also based on the assumption that the necessary geotechnical observations and testing during construction will be performed by representatives of our firm. The field observation services are considered a continuation of the geotechnical investigation and essential to determine that the actual soil conditions are as anticipated. This also provides for a procedure whereby the client can be advised of unanticipated or changed conditions that would require modifications of our original recommendations. In addition, the presence of our representative at the site provides the client with an independent professional opinion regarding the geotechnical-related construction procedures. If another firm is retained for the geotechnical observation services, our professional responsibility and liability would be limited to the extent that we would not be the geotechnical engineer of record.

TABLES

Table 1: Horizontal Ground Motion Pseudospectral Velocity in Inches/Second

	2% damping		5% damping		7% damping		10% damping	
Period in Seconds	DBE 10% in 50 years	MCE 10% in 100 years	DBE 10% in 50 years	MCE 10% in 100 years	DBE 10% in 50 years	MCE 10% in 100 years	DBE 10% in 50 years	MCE 10% in 100 years
0.01	0.40	0.50	0.40	0.50	0.40	0.50	0.40	0.50
0.10	9.21	11.65	7.70	9.74	7.14	9.04	6.56	8.29
0.20	25.09	31.91	19.38	24.65	17.29	21.98	15.07	19.16
0.30	34.59	44.10	28.17	35.92	25.82	32.92	23.32	29.73
0.40	41.71	53.45	33.97	43.54	31.13	39.90	28.12	36.04
0.50	46.66	60.17	38.01	49.01	34.83	44.92	31.46	40.57
0.60	50.42	65.28	41.07	53.17	37.63	48.73	34.00	44.01
0.75	53.96	70.10	43.96	57.10	40.28	52.33	36.39	47.27
1.00	56.99	73.95	46.42	60.24	42.54	55.20	38.42	49.86
1.50	59.26	76.05	48.27	61.94	44.24	56.76	39.96	51.28
2.00	57.73	73.20	47.02	59.63	43.09	54.64	38.92	49.36

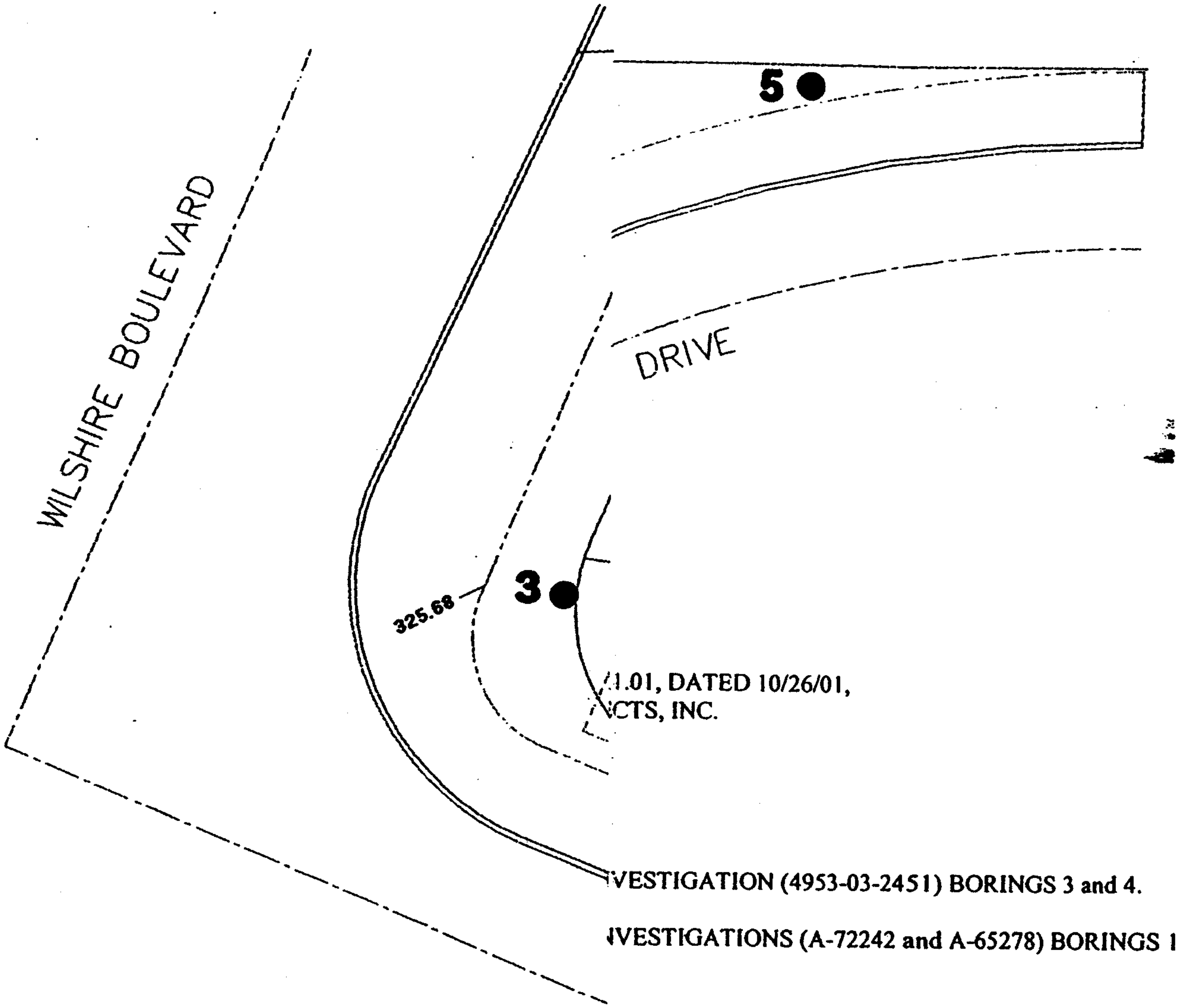
By: MM 8/4/03
Chkd:

Table 2: Horizontal Ground Motion Pseudospectral Acceleration in g's

	2% damping		5% damping		7% damping		10% damping	
Period in Seconds	DBE 10% in 50 years	MCE 10% in 100 years	DBE 10% in 50 years	MCE 10% in 100 years	DBE 10% in 50 years	MCE 10% in 100 years	DBE 10% in 50 years	MCE 10% in 100 years
0.01	0.65	0.82	0.65	0.82	0.65	0.82	0.65	0.82
0.10	1.50	1.90	1.25	1.58	1.16	1.47	1.07	1.35
0.20	2.04	2.59	1.58	2.00	1.41	1.79	1.22	1.56
0.30	1.87	2.39	1.53	1.95	1.40	1.78	1.26	1.61
0.40	1.70	2.17	1.38	1.77	1.27	1.62	1.14	1.47
0.50	1.52	1.96	1.24	1.59	1.13	1.46	1.02	1.32
0.60	1.37	1.77	1.11	1.44	1.02	1.32	0.92	1.19
0.75	1.17	1.52	0.95	1.24	0.87	1.13	0.79	1.02
1.00	0.93	1.20	0.75	0.98	0.69	0.90	0.62	0.81
1.50	0.64	0.82	0.52	0.67	0.48	0.62	0.43	0.56
2.00	0.47	0.60	0.38	0.48	0.35	0.44	0.32	0.40

By: MM 8/4/03
Chkd: ______

FIGURES



1.01, DATED 10/26/01,
CTS, INC.

VESTIGATION (4953-03-2451) BORINGS 3 and 4.

VESTIGATIONS (A-72242 and A-65278) BORINGS 1, 2 and 5.

PROPOSED
CONDOMINIUM BUILDING
21 STORIES + 3 SUBTERRANEAN
PARKING LEVELS

PLOT PLAN
SCALE 1" = 30'

MACTEC

JOB: 4953-03-2451

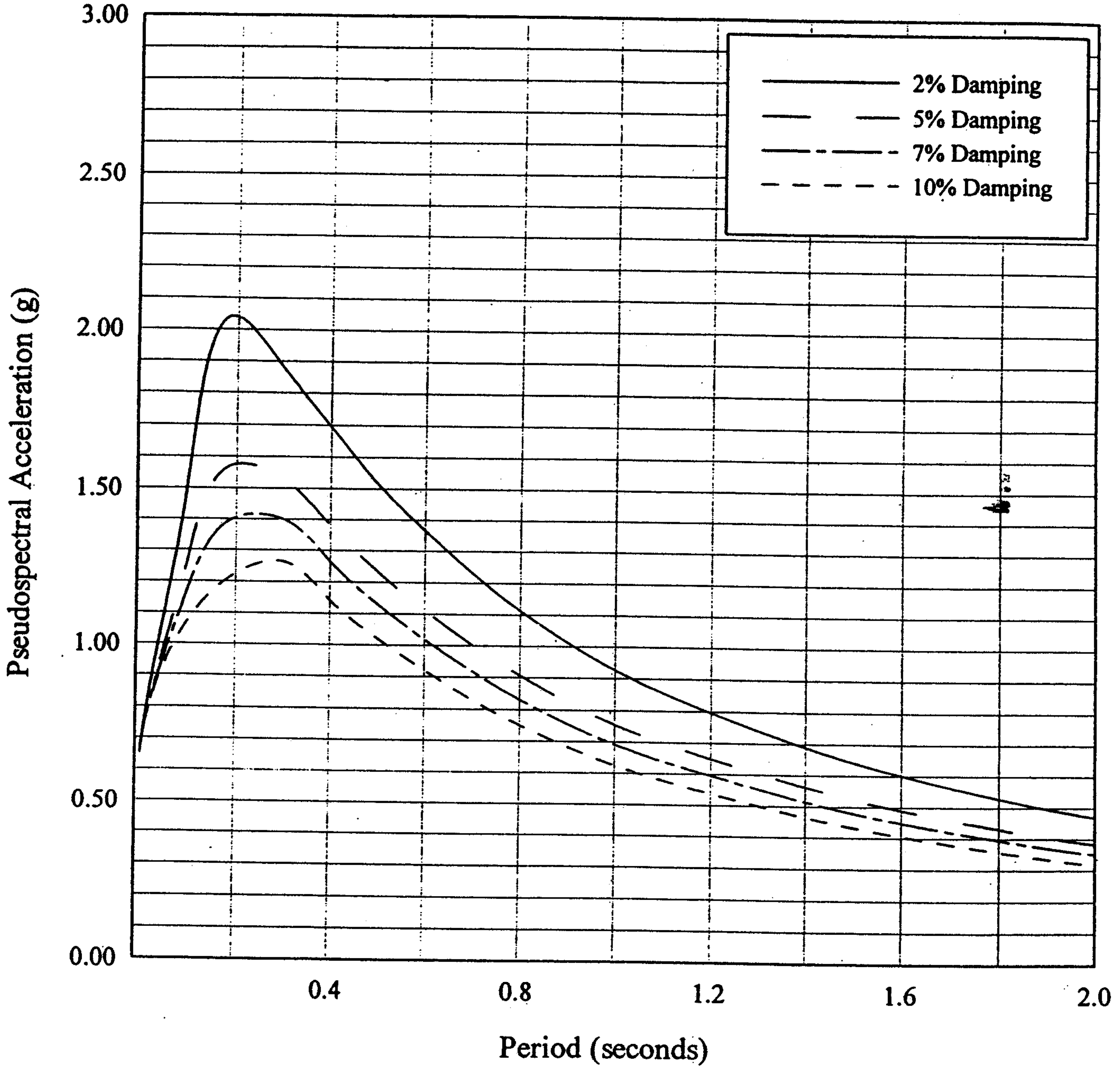
DATE: 8/1/03

F.T.: n/a

DR.: MM

O.E.: MM

CHKD:



HORIZONTAL RESPONSE SPECTRA

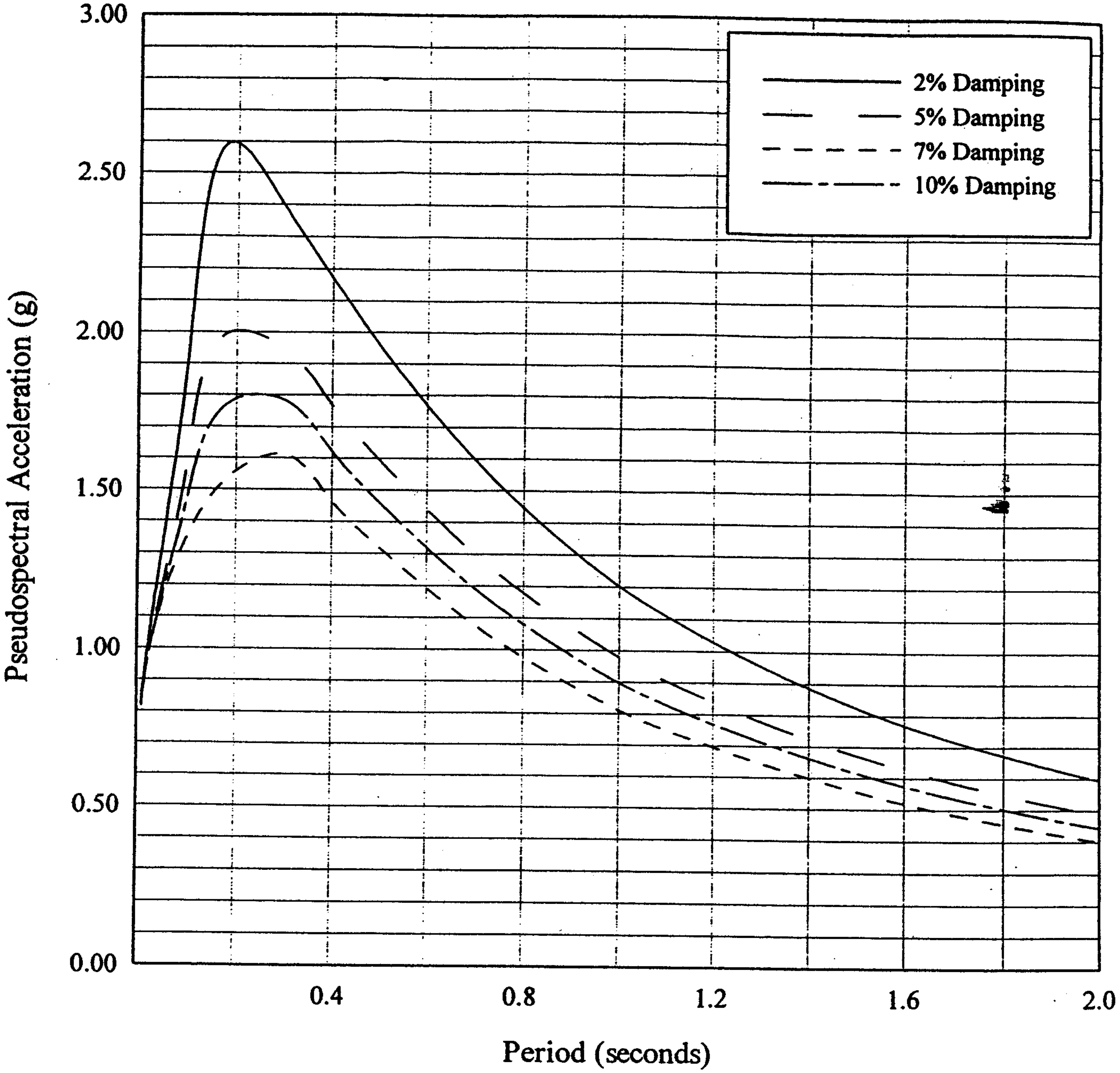
Design Basis Earthquake (DBE)
10% Probability of Exceedence in 50 Years

MACTEC

32451\calculations\ground motion\SpectraDBEfinal.grf

Figure 2

JOB: 4953-03-2451 DATE: 8/1/03 F.T.: n/a DR.: MM O.E.: MM CHKD: Ms



HORIZONTAL RESPONSE SPECTRA

Maximum Capable Earthquake (MCE)
10% Probability of Exceedence in 100 Years

MACTEC

32451\calculations\ground motion\SpectraMCEfinal.grf

Figure 3

APPENDIX

FIELD EXPLORATIONS AND LABORATORY TEST RESULTS

APPENDIX

FIELD EXPLORATIONS AND LABORATORY TEST RESULTS

FIELD EXPLORATIONS

The soil and groundwater conditions beneath the site of the proposed condominium building were recently explored by drilling two borings (Boring 3 and 4, Figure A-1.3 and A-1.4) to a depth of 75 feet below the existing grade using 5-inch-diameter rotary wash type drilling equipment. After completion of drilling, Boring 3 was converted into a ground water monitoring well. A detailed description of the water monitoring well installation is presented in the section below.

In addition to the recent boring, the three previous borings (Borings 1, 2 and 5) which were drilled for a similar project that was not constructed, are applicable to the new development, and these borings are presented in this Appendix.

The soils encountered were logged by our field technician, and undisturbed and bulk samples were obtained for laboratory inspection and testing. The logs of the borings are presented in Figures A-1.1 through A-1.5; the depths at which undisturbed samples were obtained are indicated to the left of the boring logs. The number of blows or energy required to drive the Crandall sampler 12 inches is indicated on the logs. The soils are classified in accordance with the Unified Soil Classification System described in Figure A-2.

GROUND WATER MONITORING WELL INSTALLATION

After completion of drilling of Boring 3, a 2-inch-diameter PVC pipe was installed to a depth of 75 feet. The pipe was perforated between 10 and 66 feet. The space between the pipe and the boring wall was backfilled with sand to within 8 feet of ground surface. A bentonite seal was placed between depths of 2 and 8 feet, and a cement/grout was placed above 2 feet. A cap was placed over the monitoring well.

LABORATORY TEST RESULTS

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to determine their engineering properties.

The field moisture content and dry density of the soils encountered were determined by performing tests on the undisturbed samples. The results of the tests are shown to the left of the boring logs.

Direct shear tests were performed on selected undisturbed samples to determine the strength of the soils. The tests were performed at field moisture content and after soaking to near-saturated moisture content and at various surcharge pressures. The yield-point values determined from the direct shear tests are presented in Figure A-3, Direct Shear Test Data.

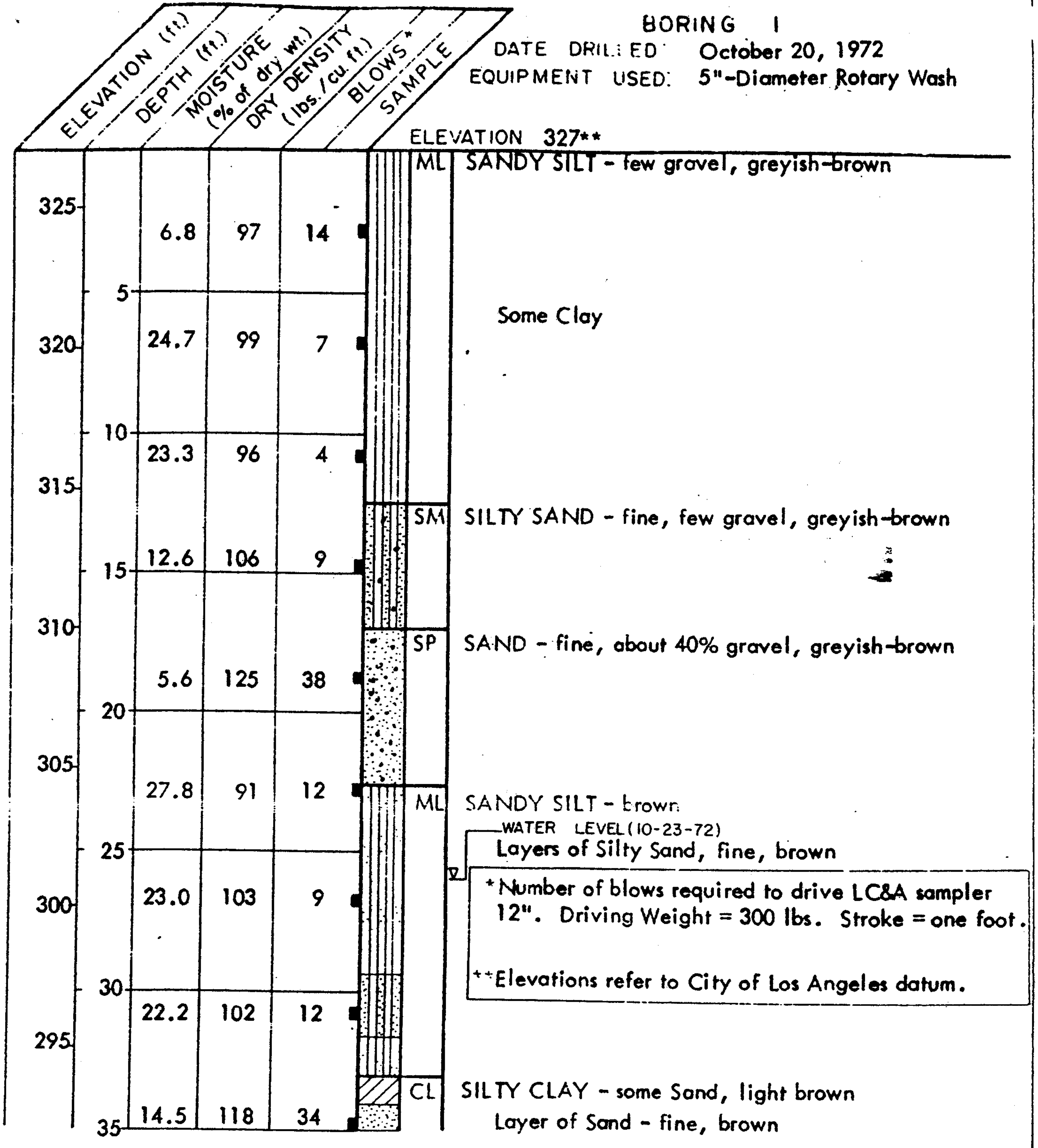
Confined consolidation tests were performed on seven undisturbed samples to determine the compressibility of the soils. The samples were tested at field moisture content. To simulate the effect of planned excavation, the samples were loaded, unloaded, and subsequently reloaded. The results of the tests are presented in Figures A-4.1 through A-4.4, Consolidation Test Data.

The optimum moisture content and maximum dry density of the upper soils were determined by performing a compaction test on a sample obtained from Boring 3. The test was performed in accordance with the ASTM Designation D1557-B method of compaction. After completion of the compaction test, a stabilometer (R-value) test was performed on the sample. The results of the tests are presented in Figures A-5.1 through A-5.3, Compaction and R-value Test Data. The tests were performed for us by LaBelle•Marvin, Inc., Professional Pavement Engineering.

BORING 1

DATE DRILLED October 20, 1972
EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 327**



MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	BLOWS*
6.8	97	14
24.7	99	7
23.3	96	4
12.6	106	9
5.6	125	38
27.8	91	12
23.0	103	9
22.2	102	12
14.5	118	34

ML SANDY SILT - few gravel, greyish-brown

Some Clay

SM SILTY SAND - fine, few gravel, greyish-brown

SP SAND - fine, about 40% gravel, greyish-brown

ML SANDY SILT - brown

WATER LEVEL (10-23-72)

Layers of Silty Sand, fine, brown

*Number of blows required to drive LC&A sampler 12". Driving Weight = 300 lbs. Stroke = one foot.

**Elevations refer to City of Los Angeles datum.

CL SILTY CLAY - some Sand, light brown

Layer of Sand - fine, brown

(CONTINUED ON FOLLOWING PLATE)

NOTE THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

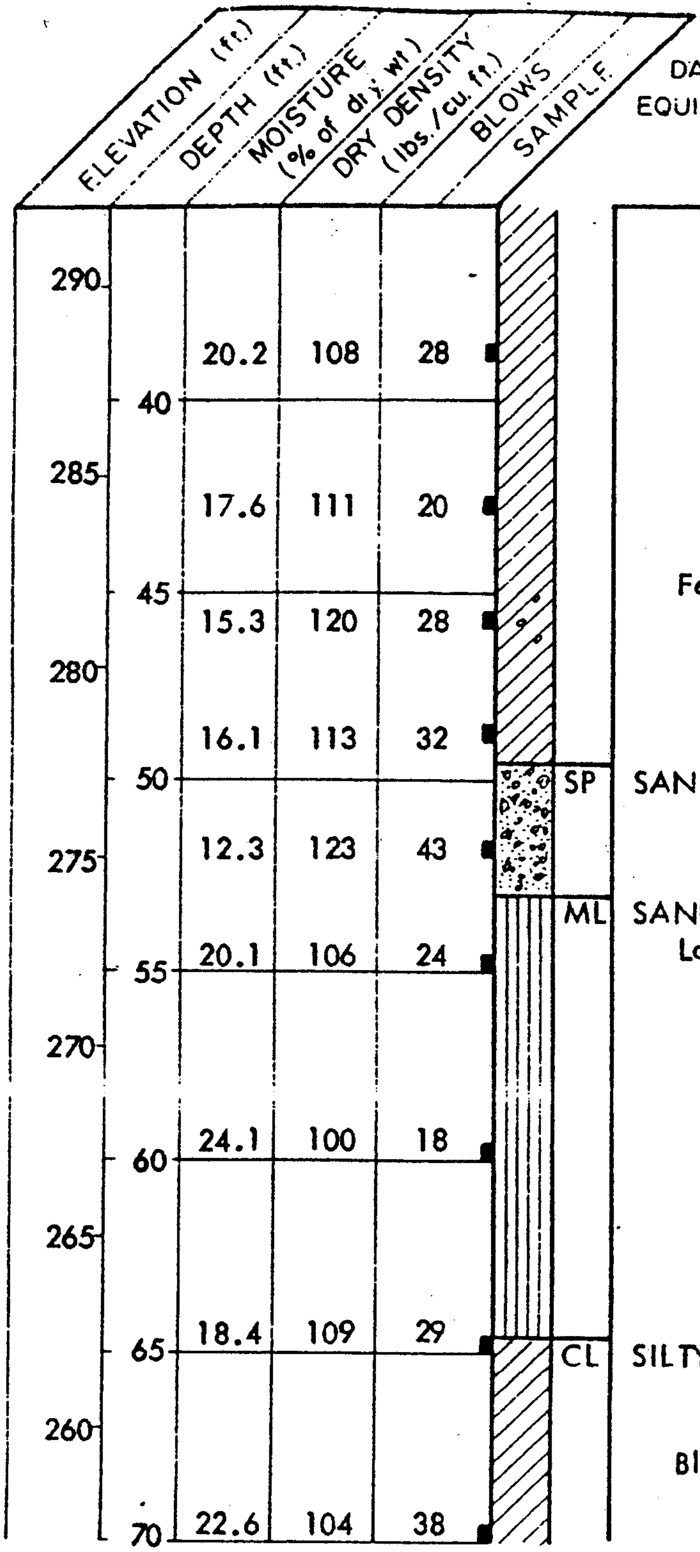
LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREIN APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING 1 (CONTINUED)

DATE DRILLED: October 20, 1972
EQUIPMENT USED: 5"-Diameter Rotary Wash



MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	BLOWS
20.2	108	28
17.6	111	20
15.3	120	28
16.1	113	32
12.3	123	43
20.1	106	24
24.1	100	18
18.4	109	29
22.6	104	38

Few gravel

SP SAND - fine, about 40% gravel, light brown and grey

ML SANDY SILT - brown
Layers of Silty Clay - brown

CL SILTY CLAY - reddish-brown

Bluish-grey

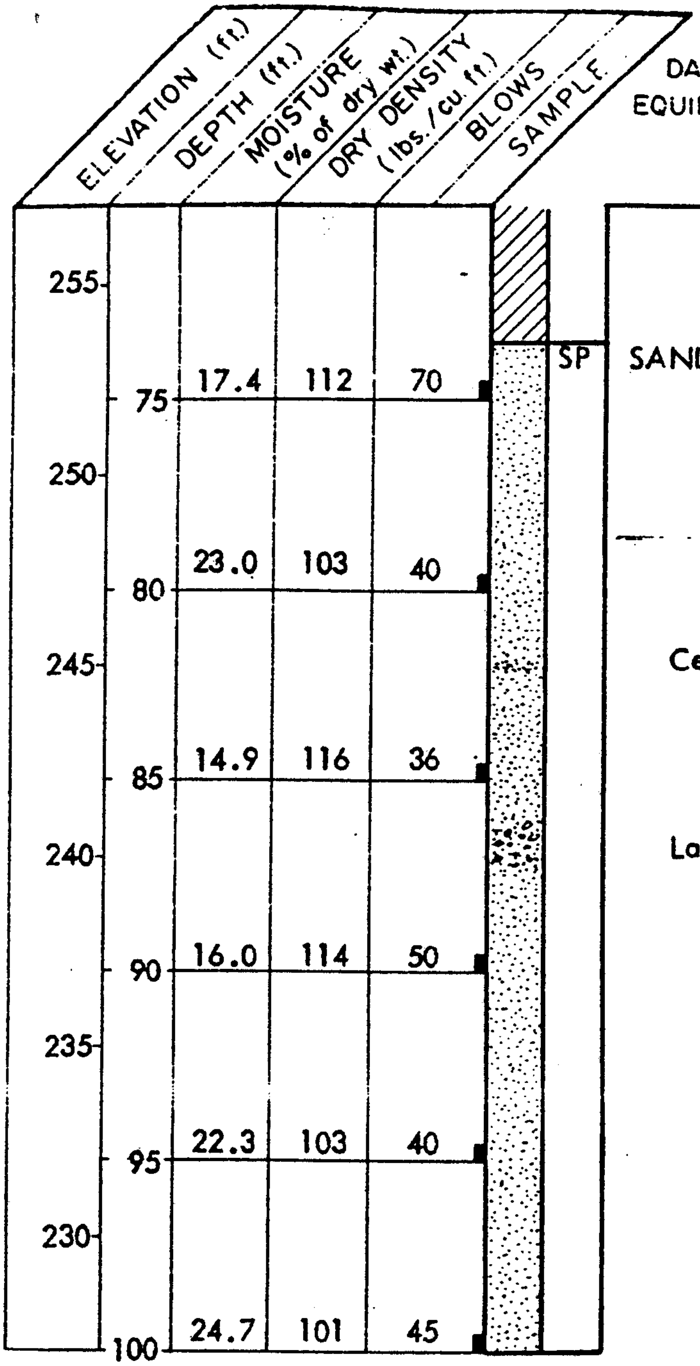
(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

FIGURE A-1.1b

BORING 1 (CONTINUED)
DATE DRILLED: October 20, 1972
EQUIPMENT USED: 5"-Diameter Rotary Wash



DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	BLOWS
75	17.4	112	70
80	23.0	103	40
85	14.9	116	36
90	16.0	114	50
95	22.3	103	40
100	24.7	101	45

SP SAND - fine, grey

Cemented layers

Layer containing large amount of gravel

NOTE: Drilling mud used in drilling process. Mud removed after drilling completed; water level measured at a depth of 26' 2 days after removing mud.

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

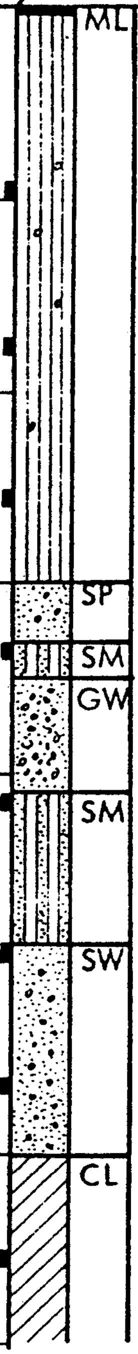
LOG OF BORING

BORING 2

DATE DRILLED: October 21, 1972
EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 325

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	BLOWS	SAMPLE
		11.1	116	14	■
320	5				
		10.1	107	9	■
315	10				
		11.5	118	10	■
310	15				
		9.8	107	12	■
305	20				
		10.9	125	13	■
		13.3	111	12	■
300	25				
		16.0	116	22	■
295	30				
		20.6	106	28	■
290	35				



ML 2½" Asphaltic Paving
SANDY SILT - some Clay, few gravel, dark brown

SP SAND - fine, about 10% gravel, light brown

SM SILTY SAND - fine, very silty, brown

GW SANDY GRAVEL - well graded, light brown and grey

SM SILTY SAND - fine, brown

∇ WATER LEVEL (10-23-72)

SW SAND - well graded, about 30% gravel, greyish-brown

CL SILTY CLAY - very silty, brown

(CONTINUED ON FOLLOWING PLATE)

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

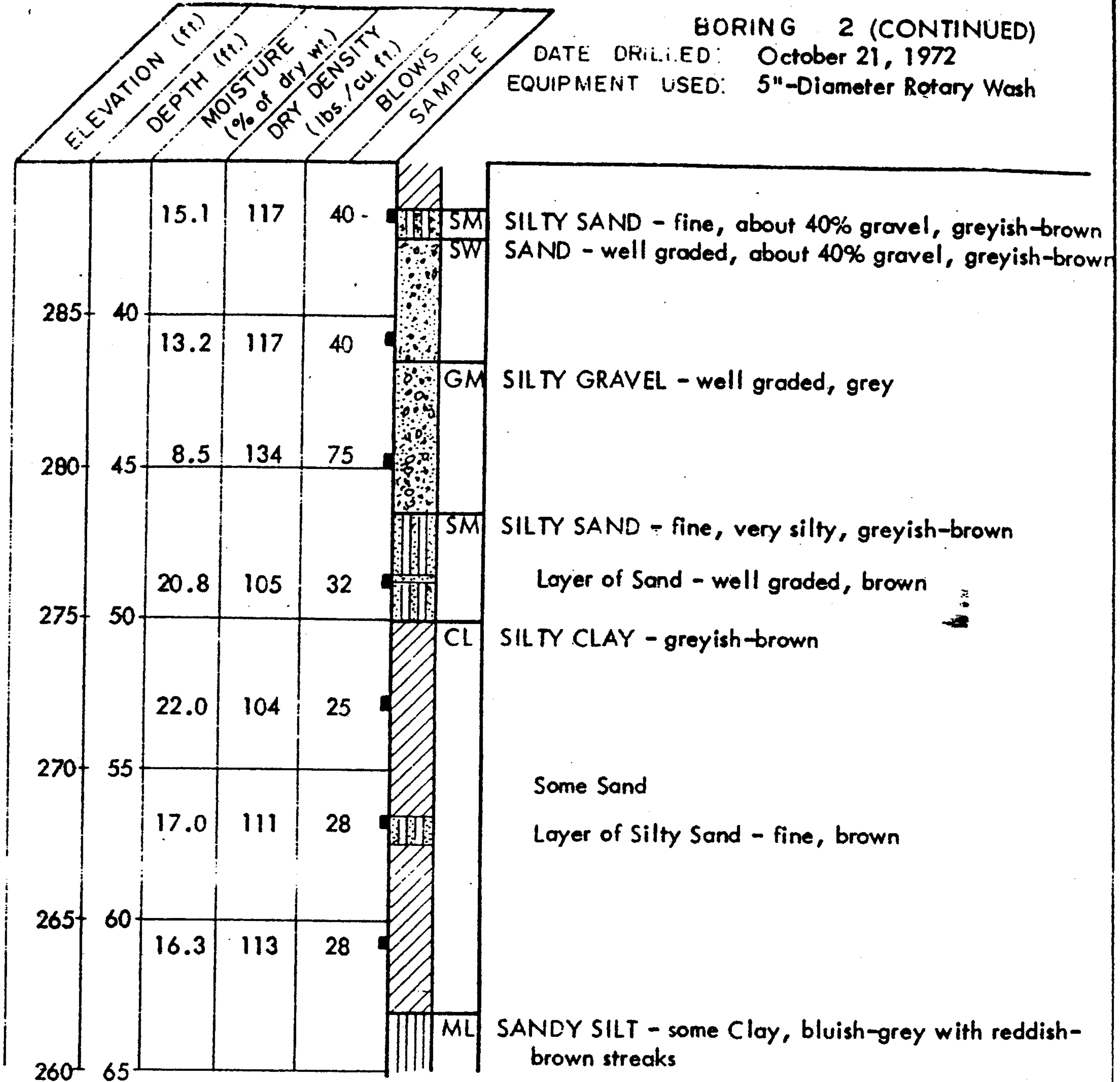
LOG OF BORING

LEROY CRANDALL AND ASSOCIATES
FIGURE A-1.2a

BORING 2 (CONTINUED)

DATE DRILLED: October 21, 1972

EQUIPMENT USED: 5"-Diameter Rotary Wash



MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	BLOWS
15.1	117	40 -
13.2	117	40
8.5	134	75
20.8	105	32
22.0	104	25
17.0	111	28
16.3	113	28

SM	SILTY SAND - fine, about 40% gravel, greyish-brown
SW	SAND - well graded, about 40% gravel, greyish-brown
GM	SILTY GRAVEL - well graded, grey
SM	SILTY SAND - fine, very silty, greyish-brown
	Layer of Sand - well graded, brown
CL	SILTY CLAY - greyish-brown
	Some Sand
	Layer of Silty Sand - fine, brown
ML	SANDY SILT - some Clay, bluish-grey with reddish-brown streaks

(CONTINUED ON FOLLOWING PLATE)

NOTE THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

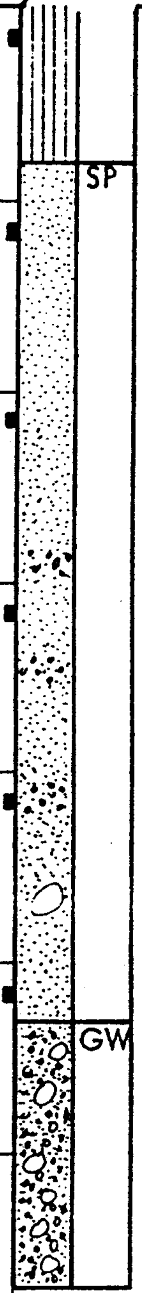
LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

FIGURE A-1.2b

BORING 2 (CONTINUED)
DATE DRILLED: October 21, 1972
EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	BLOWS
		19.1	107	35
255	70	18.9	108	40
250	75	19.5	109	58
245	80	22.8	104	55
240	85	23.6	101	50
235	90	22.1	103	55
230	95			
225	100			



SAND - fine, bluish-grey

Layers containing large amount of gravel

Large cobble

SANDY GRAVEL - well graded, about 20% cobbles, grey

(LARGE COBBLE. BORING TERMINATED DUE TO DIFFICULTY IN DRILLING.)

NOTE: Drilling mud used in drilling process. Mud removed after drilling completed; water level measured at a depth of 23' 2 days after removing mud.

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOG OF BORING

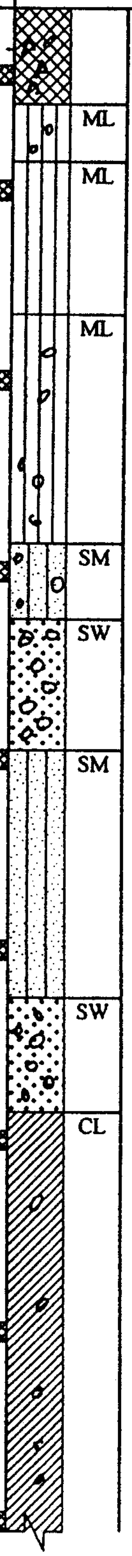
LEROY CRANDALL AND ASSOCIATES

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL

BORING 3***

DATE DRILLED: July 16, 2003
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 326.0

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
325		12.6	121	7	▩
320	5	16.6	105	3	▩
315	10	16.6	107	3	▩
310	15	13.4	108	5	▩
305	20	25.4	102	7	▩
300	25	20.2	106	15	▩
295	30	20.2	106	14	▩
290	35	23.1	107	18	▩
	40	21.4	106	22	▩



FILL - SILTY CLAY - pieces of paving and asphaltic Concrete, brown

ML — CLAYEY SILT - thin layer of Silty Clay, some Sand, few Gravel, light brown

ML — SANDY SILT - brown

ML — SANDY SILT - some Gravel, light brown

* Number of blows required to drive the Crandall sampler 12 inches using a 140 pound hammer falling 30 inches.

*** BORINGS 3 and 4 WERE DRILLED RECENTLY FOR CURRENT INVESTIGATION.

SM — SILTY SAND - fine, some Gravel, light brown

SW — SAND - well graded, about 20% Gravel, light brown

SM — SILTY SAND - fine, light brown

SW — SAND - well graded, about 20% Gravel, light brown

CL — SILTY CLAY - some Sand and Gravel, brown

Field Tech: AR
Prepared By: DBS
Checked By: MS

(CONTINUED ON FOLLOWING FIGURE)

Proposed High-Rise Condominium
10250 Wilshire Blvd.
Los Angeles, California

MACTEC

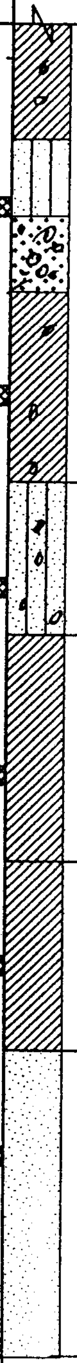
LOG OF BORING

Project: 4953-03-2451 FIGURE A-1.3a

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

BORING 3*** (Continued)

DATE DRILLED: July 16, 2003
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 326.0 **



ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
285							
							Layer of Silty Sand
	45	18.0	111	45	☒		
							Layer of well graded Sand, about 30% Gravel
280							
	50	27.6	97	20	☒		
							Light brown
275							
						SM	SILTY SAND - fine, thin layers of Sandy Silt, some Gravel, greyish brown
	55	19.7	106	24	☒		
270							
						CL	SILTY CLAY - light brown
	60	20.2	109	28	☒		
265							
						CL	Sandy Clay - gray
	65	21.9	104	19	☒		
260							
						SP	SAND - fine, light grey
	70	22.6	105	40	☒		
255							
							Grey
	75	16.7	116	45	☒		
250							
	80						

END OF BORING AT 75 FEET

NOTE: Drilling mud used in drilling process. Mud removed after completion of drilling. Boring was converted into a ground water monitoring well.

Field Tech: AR
Prepared By: DBS
Checked By: MS

Proposed High-Rise Condominium
10250 Wilshire Blvd.
Los Angeles, California

MACTEC

LOG OF BORING

Project: 4953-03-2451

FIGURE A-1.3b

BORING 4***

DATE DRILLED: Jul 16, 2003
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 324.0

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
							8" of Gravel
						ML	CLAYEY SILT - few Gravel, some Clay lenses, brown
320							
	5	15.8	103	5	⊠	ML	SANDY SILT - some Clay, brown
						CL	SILTY CLAY - some Sand, few Gravel, light brown
315							
	10	17.8	106	7	⊠		
310						ML	SANDY SILT - some Gravel, brown
	15	27.5	91	5	⊠		
							Layer of Sand and Gravel
						CL	SILTY CLAY - few Gravel, brown
305							
	20	21.0	107	7	⊠		
						SM	SILTY SAND - fine to course, about 20% Gravel, greyish brown
300							
	25	27.5	97	6	⊠		
						SW	SAND - well graded, about 30% Gravel, few Cobbles, greyish brown
295							
	30	22.3	103	15	⊠	SM	SILTY SAND - fine to medium, about 10% Gravel, greyish brown
						CL	SILTY CLAY - few Gravel, light brown
290							
	35	22.1	106	15	⊠		
285							
	40						

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

H11SOIL_CRANDALL 32451.GPJ LAW CRAN.GDT 7/21/03

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: AR
Prepared By: DBS
Checked By: MS

Proposed High-Rise Condominium
10250 Wilshire Blvd.
Los Angeles, California

MACTEC

LOG OF BORING

Project: 4953-03-2451

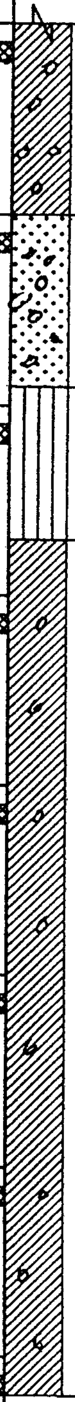
FIGURE A-1.4a

BORING 4*** (Continued)

DATE DRILLED: Jul 16, 2003
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 324.0

ELEVATION (ft): 280, 275, 270, 265, 260, 255, 250, 245

DEPTH (ft)	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
	17.4	114	22	☒
45	13.8	122	50/9"	☒
50	30.6	95	25	☒
55	26.3	100	18	☒
60	23.0	105	23	☒
65	24.9	101	24	☒
70	20.5	106	55	☒
75	21.4	107	55	☒
80				



More Gravel

SW SAND - well graded, about 20% Gravel, greyish brown

ML SANDY SILT - light brown

CL SILTY CLAY - few Gravel, light brown

Bluish grey

END OF BORING AT 76 FEET

NOTE: Drilling mud used in drilling process. Mud removed after completion of drilling. Water level measured at a depth of 18', 20 minutes after removal of mud.

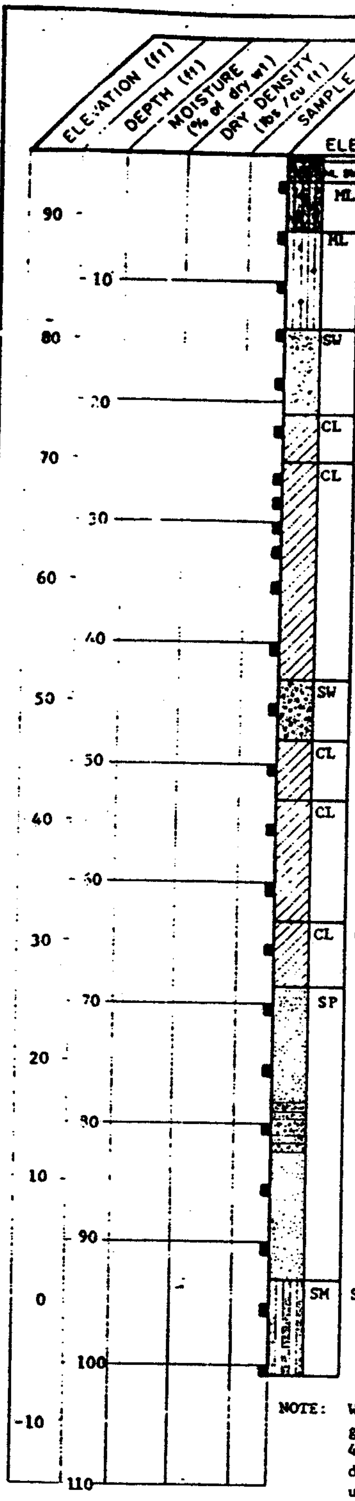
THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

Field Tech: AR
Prepared By: DBS
Checked By: MS

BORING 5 (PREVIOUS INVESTIGATION A-65278)

DATE DRILLED August 12 & 13, 1965
EQUIPMENT USED. 18"-Diameter Bucket to 49'
5"-Diameter Rotary Wash below 49'

ELEVATION 94.9 (Assumed)



4" ASPHALTIC PAVING

FILL - SILTY SAND and SANDY SILT - 40 to 50% concrete pieces, light brown

FILL - CLAYEY SILT - few gravel, brown
10 to 15% debris (pieces of brick and metal)
(Exposed 4" sewer pipe)

SANDY SILT - few gravel, greyish-brown

SAND - well graded, 10 to 15% gravel, mottled brown

SILTY CLAY - 5 to 10% gravel, brown

SANDY CLAY - mottled grey and brown

▽ WATER LEVEL (8-13-65)

GRAVELLY SAND - well graded, 35 to 45% gravel, brown

SANDY CLAY - mottled grey and brown

SILTY CLAY - brown

Dark brown

CLAY - slightly silty, bluish-grey

SAND - fine, dark bluish-grey

Layers of GRAVELLY SAND - 25 to 30% gravel

SILTY SAND - fine, very silty, bluish-grey

NOTE: Water encountered at a depth of 40 feet below ground surface. Heavy caving occurred between 43 and 48 feet (to 4 feet in diameter) during the drilling with bucket equipment. Drilling mud used in drilling process with rotary wash equipment.

JOB A-65278 DATE [illegible] OF [illegible] CHKD [illegible]

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than the No.200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No.4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GRAVELS WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 50% of coarse fraction is SMALLER than the No.4 sieve size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS (More than 50% of material is SMALLER than the No.200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS (Liquid limit GREATER than 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 silts
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

No. 200 No. 40 No. 10 No. 4 3/4 in. 3 in. (12 in.)

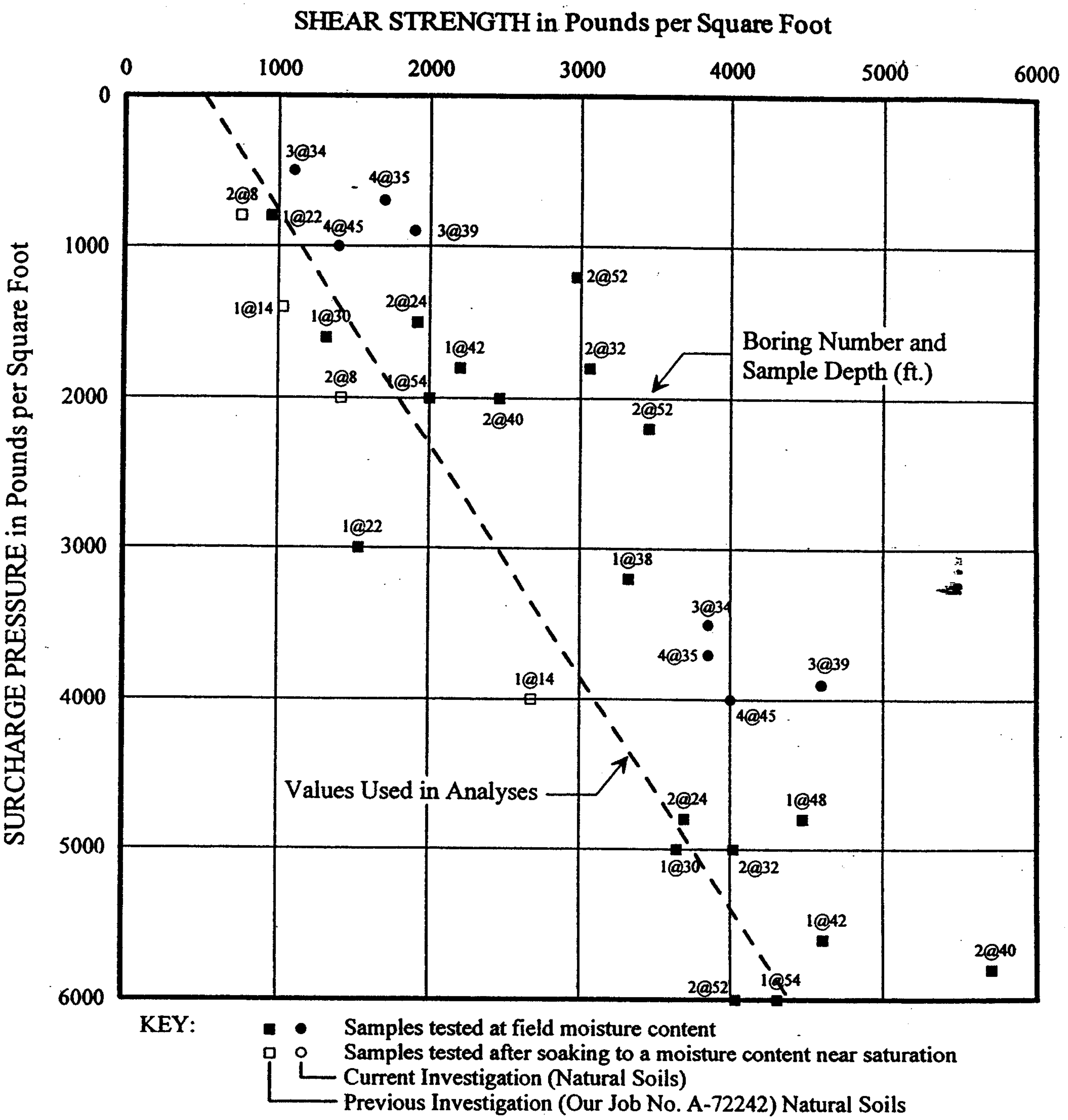
U. S. STANDARD SIEVE SIZE

UNIFIED SOIL CLASSIFICATION SYSTEM

REFERENCE:
The Unified Soil Classification System, Corps of Engineers, U.S. Army
Technical Memorandum No. 3-357, Vol. 1, March, 1953. (Revised April, 1960).

LAW/CRANDALL

FIGURE A-2

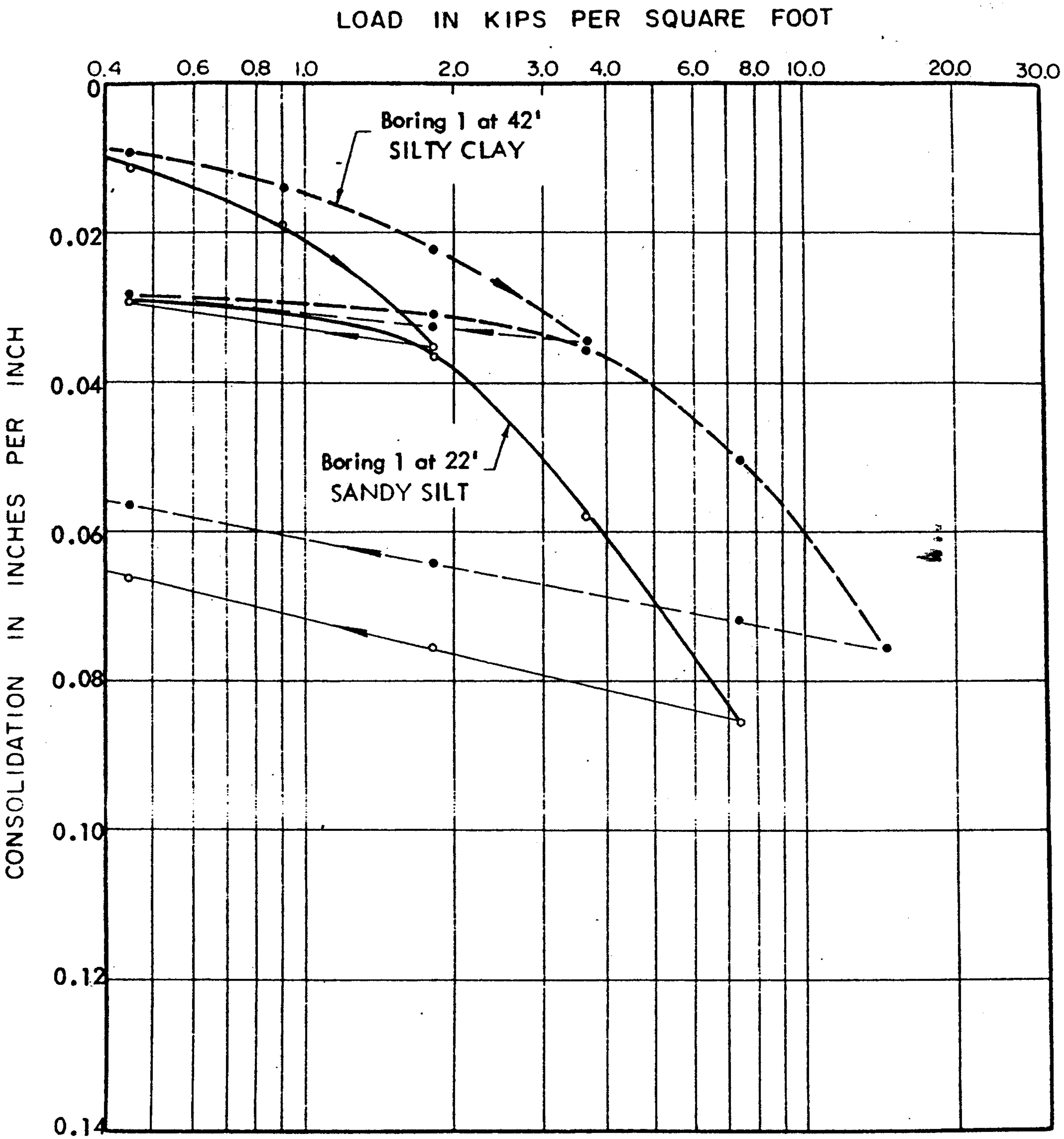


DIRECT SHEAR TEST DATA

JOB 4953-03-2451 DATE August 4, 2003 F.T. RT DR. MM O.E. MS CHKD MS

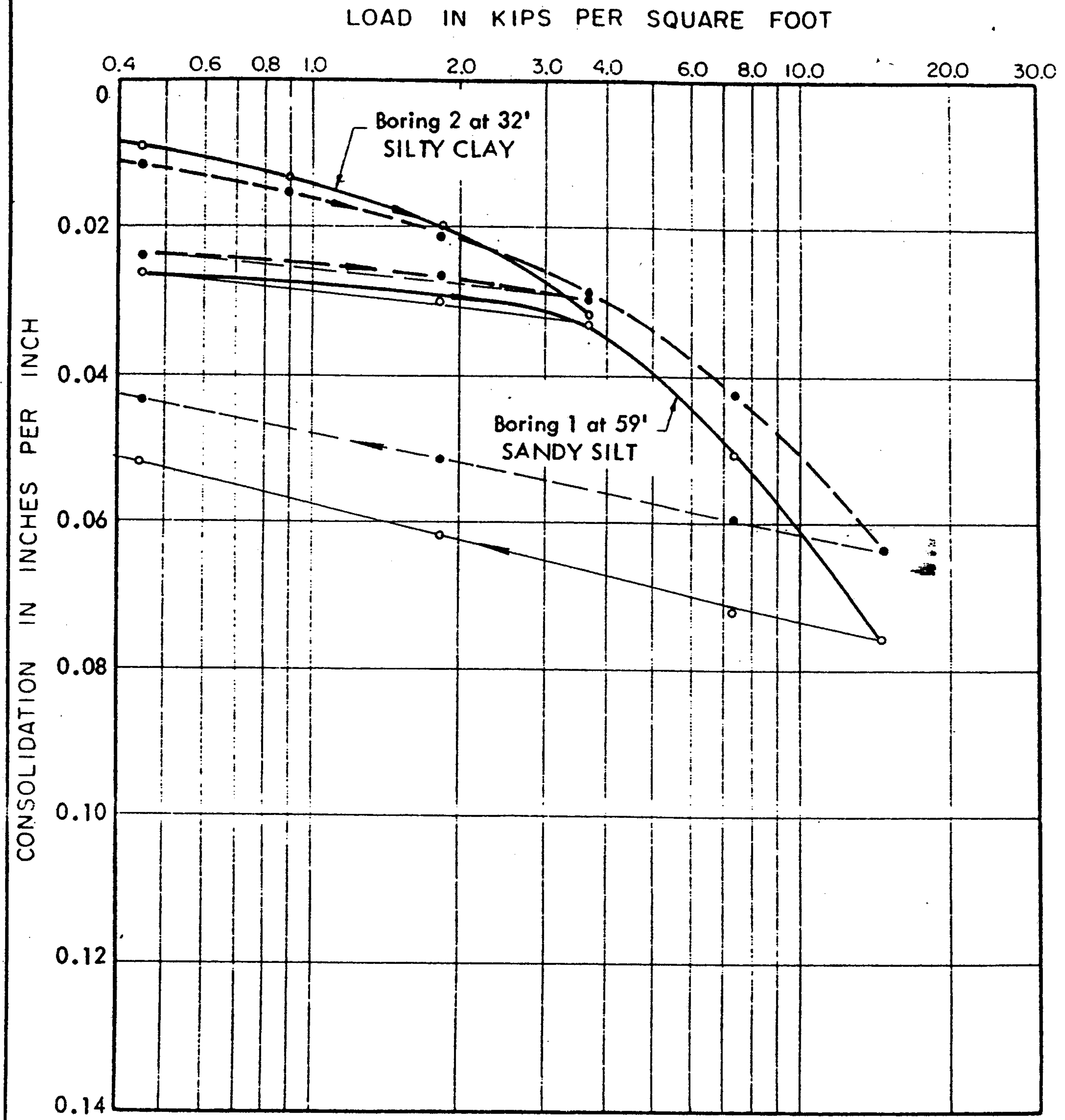
MACTEC

FIGURE A - 3



NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

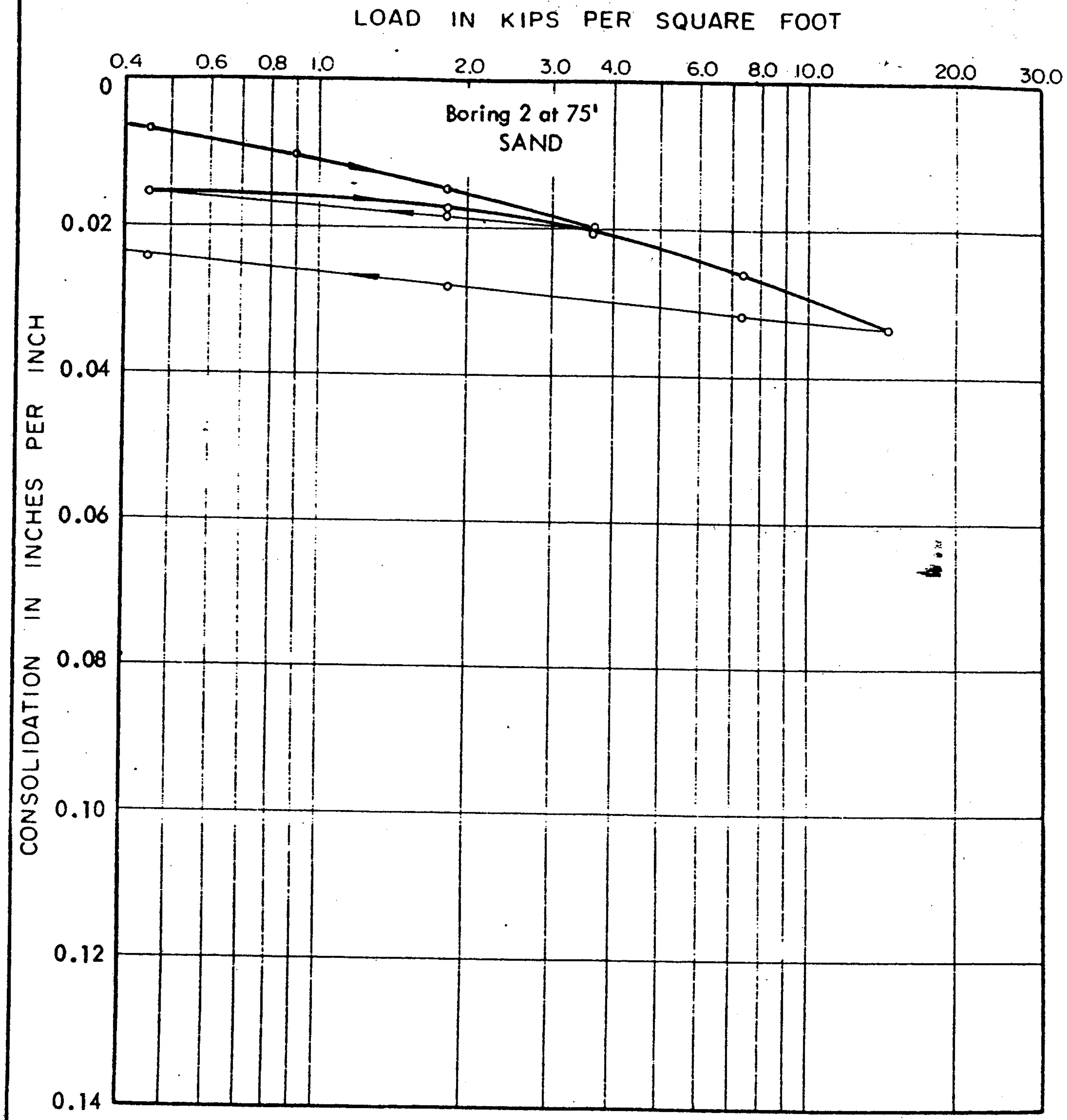


NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

FIGURE A-4.2

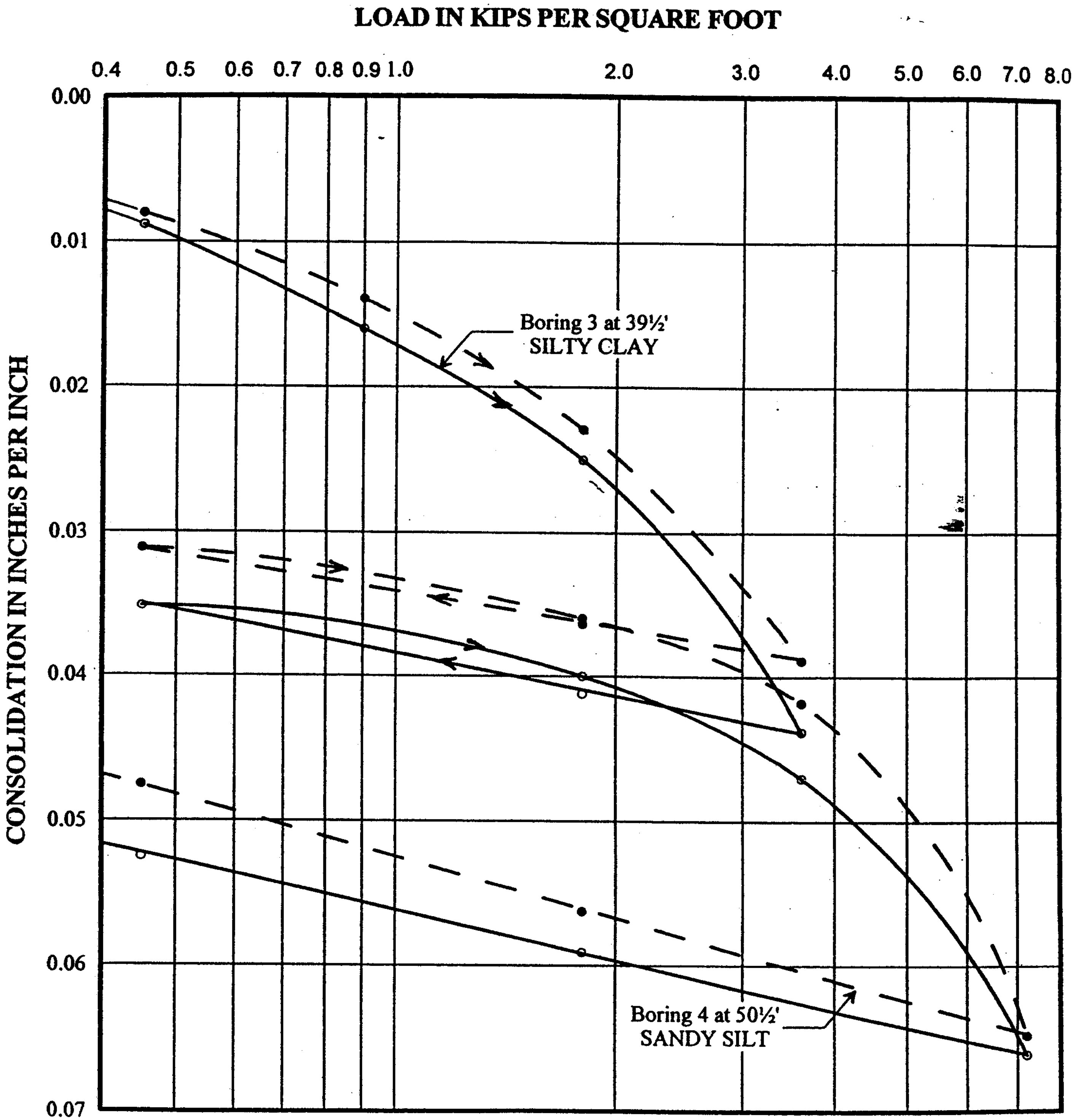


NOTE: Sample tested at field moisture content.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

FIGURE A-4.3



NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

MACTEC

FIGURE A - 4.4

JOB 4953-03-2451 DATE August 25, 2003 F.T. BE — DR. MM O.E. MM CHKD MS

• ANALYSIS
• DESIGN

LaBelle • Marvin

• SOILS, ASPHALT TECHNOLOGY

PROFESSIONAL PAVEMENT ENGINEERING
A CALIFORNIA CORPORATION

July 22, 2003

Mr. Mike Shahabi
Mactec
200 Citadel Drive
Los Angeles, California 90040

Project No. 30075

Dear Mr. Shahabi:

Testing of the soil sample delivered to our laboratory has been completed with the following results:

Reference: **03-2451**

MD Curve

Sample ID	**Maximum Dry Density** (ASTMD 1557 B)	**Optimum Moisture**	**Soil Description**
03-2451	127.3 pcf	10.2 %	Brown Sandy Silt

R-Value data sheets are attached for your use and file. The opportunity to be of service is sincerely appreciated Please do not hesitate to call should you have any questions.

Respectfully Submitted,

Steven R. Marvin
RCE 30659

SRM:ks
Attachments

FIGURE A-5.1

R - VALUE DATA SHEET

PROJECT NUMBER 30075

BORING NUMBER: p.n. 03-2451 B-3 @ 1.5'-4'

SAMPLE DESCRIPTION: Brown Silty Sand

Item	SPECIMEN		
	a	b	c
Mold Number	7	8	9
Water added, grams	18	41	28
Initial Test Water, %	9.9	12.0	10.8
Compact Gage Pressure,psi	350	120	240
Exudation Pressure, psi	455	149	348
Height Sample, Inches	2.39	2.43	2.43
Gross Weight Mold, grams	3219	3197	3220
Tare Weight Mold, grams	2125	2116	2113
Sample Wet Weight, grams	1094	1081	1107
Expansion, Inches x 10exp-4	62	9	33
Stability 2,000 lbs (160psi)	20 / 45	33 / 78	26 / 57
Turns Displacement	3.82	4.90	4.00
R-Value Uncorrected	63	35	53
R-Value Corrected	60	33	51
Dry Density, pcf	126.2	120.3	124.5
	DESIGN CALCULATION DATA		
Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.41	0.69	0.50
G. E. by Expansion	2.07	0.30	1.10

Equilibrium R-Value 41 by EXPANSION

Examined & Checked: 7 /22/ 03

STEVEN R. MARVIN
30659
Exp. 3/31/04
CIVIL

Steven R. Marvin, RCE 30659

REMARKS:
Gf = 1.25
0.0% Retained on the 3/4" sieve.

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

FIGURE A-5.2

R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 30075 PN. 03-2451

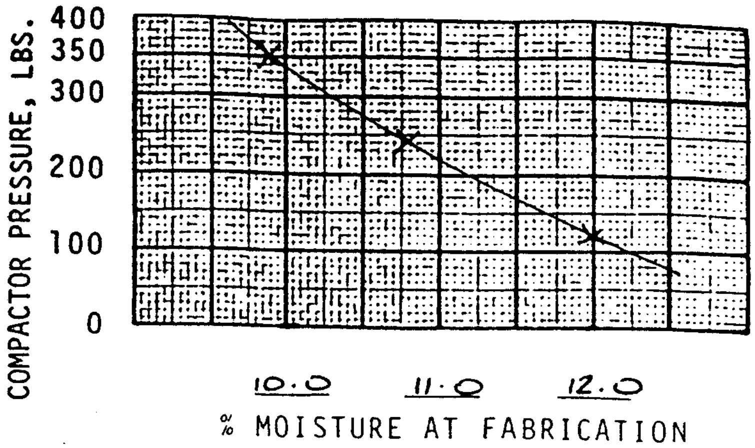
BORING NO. B-3 @ 1.5'-4'

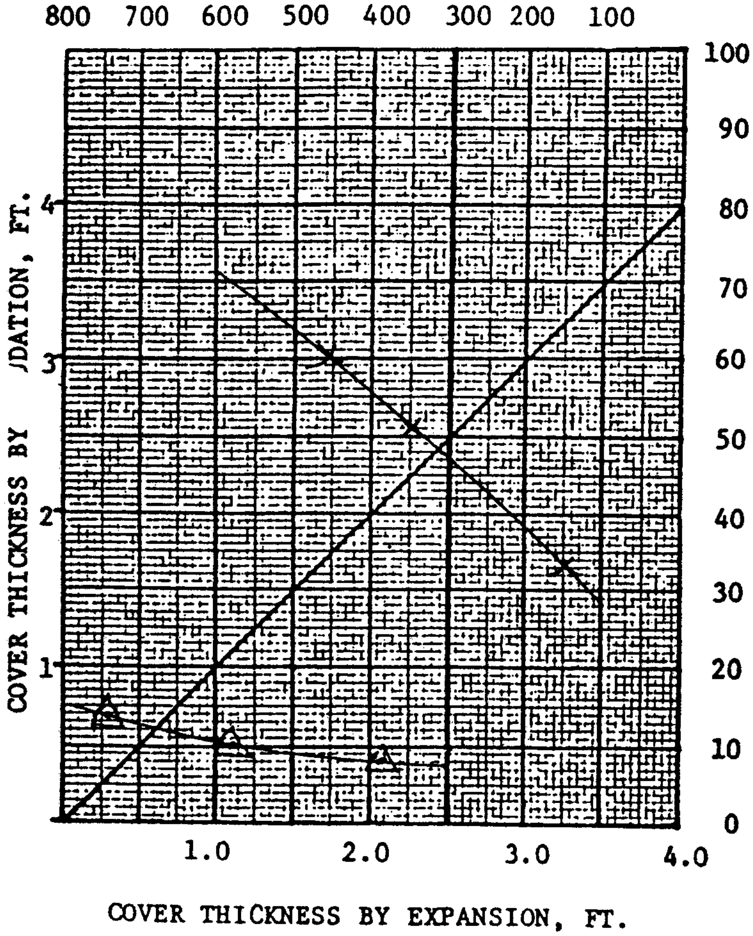
DATE 7-22-03

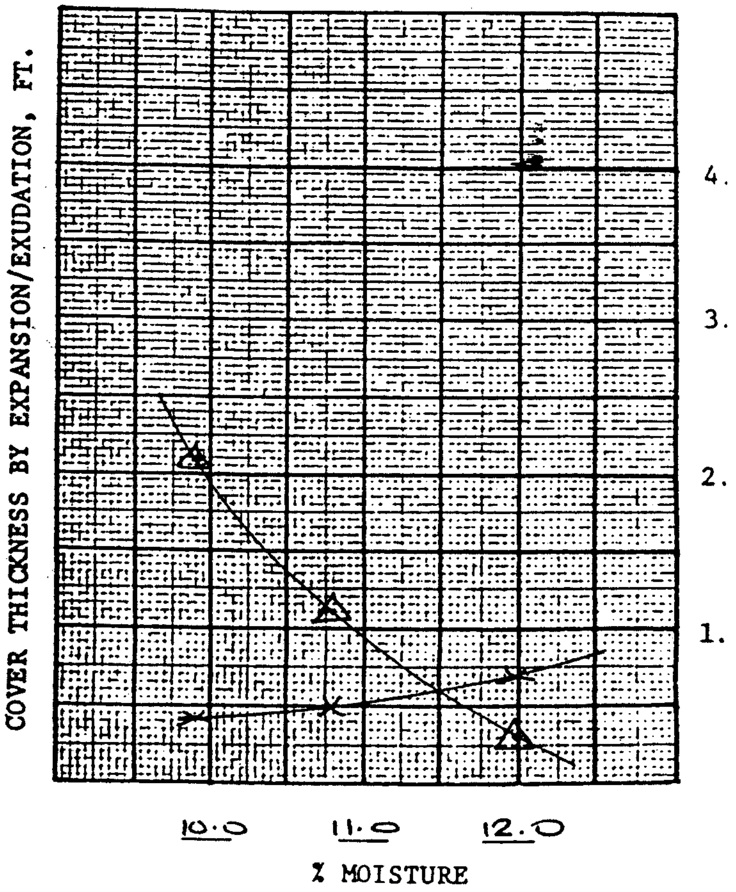
TRAFFIC INDEX Assume 4.0

R-VALUE BY EXUDATION 47

R-VALUE BY EXPANSION 41







✱ ✱ ✱ R-VALUE vs. EXUD. PRES.

× × × T by EXUDATION

△ △ △ EXUD. T vs. EXPAN. T

△ △ △ T by EXPANSION

REMARKS

Gf=1.25

FIGURE A-5.3

LaBelle • Marvin

SOIL CORROSIVITY TEST

M.J. SCHIFF & ASSOCIATES, INC.

Consulting Corrosion Engineers - Since 1959
431 W. Baseline Road
Claremont, CA 91711

Phone: (909) 626-0967 / Fax: (909) 626-3316
E-mail: mjsa@mjschiff.com
http://www.mjschiff.com

August 4, 2003

MACTEC
200 Citadel Drive
Los Angeles, CA 90400

Attention: Mr. Mike Shahabi

Re: Soil Corrosivity Study
Proposed High Rise Condominium Building Development
10250 Wilshire Boulevard
Los Angeles, California
Your # 4953-03-2451, MJS&A #03-0863HQ

INTRODUCTION

Laboratory tests have been completed on three soil samples you provided logs for the referenced housing project. The purpose of these tests was to determine if the soils may have deleterious effects on underground utility piping, hydraulic elevator cylinders, and concrete structures. We assume that the samples provided are representative of the most corrosive soils at the site.

The proposed project is construction of a high-rise condominium building. The new building will be 21-story residential tower with three subterranean parking structures. The subterranean parking levels will extend beyond the tower in plan. The site of proposed development is currently a vacant lot. The water table is 20 feet deep.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. Our recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, we will be happy to work with them as a separate phase of this project.

TEST PROCEDURES

The electrical resistivity of each sample was measured in a soil box per ASTM G57 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soils and for ammonium and nitrate. Sulfide and oxidation-reduction (redox) potential were determined on sample S-3. Test results are shown on Table 1.

SOIL CORROSIVITY

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and chemical contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:

Soil Resistivity in ohm-centimeters			Corrosivity Category
over		10,000	mildly corrosive
2,000	to	10,000	moderately corrosive
1,000	to	2,000	corrosive
below		1,000	severely corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, chemical content, soil types, aeration, anaerobic conditions, and site drainage.

Electrical resistivities were in the moderately corrosive category with as-received moisture. When saturated, the resistivities were in moderately corrosive and corrosive categories.

Soil pH values varied from 5.5 to 6.7. This range is strongly acidic to neutral.

The soluble salt content of the samples was low and moderate.

The ammonium and nitrate concentrations were high enough to be deleterious to copper.

Tests were not made for sulfide and negative oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as corrosive to ferrous metals and aggressive to copper.

CORROSION CONTROL RECOMMENDATIONS

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

Steel Pipe

Abrasive blast underground steel piping and apply a dielectric coating such as polyurethane, extruded polyethylene, a tape coating system, hot applied coal tar enamel, or fusion bonded epoxy intended for underground use.

Bond underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

Electrically insulate each buried steel pipeline from dissimilar metals and metals with dissimilar coatings (cement-mortar vs. dielectric), and above ground steel pipe to prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection.

Apply cathodic protection to steel piping as per NACE International RP-0169-02.

As an alternative to dielectric coating and cathodic protection, apply a ¾ inch cement mortar coating or encase in cement-slurry or concrete 3 inches thick, using any type of cement.

Hydraulic Elevator

Coat hydraulic elevator cylinders as described above. Electrically insulate each cylinder from building metals by installing dielectric material between the piston platen and car, insulating the bolts, and installing an insulated joint in the oil line. Apply cathodic protection to hydraulic cylinders as per NACE International RP-0169-02. As an alternative to electrical insulation and cathodic protection, place each cylinder in a plastic casing with a plastic watertight seal at the bottom.

The elevator oil line should be placed above ground if possible but, if underground, should be protected as described above for steel utilities, or should be placed in a PVC casing pipe to prevent contact with soil and soil moisture.

Iron Pipe

Encase cast and ductile iron piping per AWWA Standard C105 or coat with epoxy or polyurethane intended for underground use. Note, the thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating. Electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints.

For ductile iron water bond all nonconductive type joints for electrical continuity and apply cathodic protection as per NACE International RP-0169-02.

Copper Tube

Buried copper tubing shall be protected by: 1) Encasing the copper in two layers of 10-mil thick polyethylene sleeves taking care not to damage the polyethylene. Protect wrapped copper tubing by applying cathodic protection as per NACE International RP-0169-02. Any damaged polyethylene shall be repaired by wrapping it in 20-mil thick pipe wrapping tape. The amount of cathodic protection current needed can be minimized by coating the tubing. 2) Preventing soil contact. Soil contact may be prevented by placing the tubing above ground. 3) Install a factory coated copper pipe with a minimum of 100-mill thickness such as "Aqua Shield" or similar products. Polyethylene coating protects against elements that corrode copper and prevents contamination between copper and sleeving. However, it must be continuous with no cuts or defects if installed underground.

Plastic and Vitrified Clay Pipe

No special precautions are required for plastic and vitrified clay piping placed underground from a corrosion viewpoint. Protect all fittings and valves with wax tape per AWWA C217-99 or epoxy.

All Pipe

On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217-99 after assembly.

Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete

Any type of cement may be used for concrete structures and pipe because the sulfate concentration is negligible, 0 to 0.1 percent, per 1997 Uniform Building Code (UBC) Table 19-A-4 and American Concrete Institute (ACI-318) Table 4.3.1.

Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils.

Post Tensioning Strands and Anchors.

Protect post-tensioning strands and anchors against corrosion in an aggressive environment per the Post-Tensioning Institute Guide Specification for unbonded Single Strand Tendons. This should include the use of polyethylene encased anchors.

CLOSURE

Our services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
M.J. SCHIFF & ASSOCIATES, INC.

Adrineh Avedisian

Reviewed by,

REGISTERED PROFESSIONAL ENGINEER
STEVEN R. FOX
No. 1137
EXP. 3-31-07
CORROSION
STATE OF CALIFORNIA

Steven R. Fox, P.E.

Enc: Table 1

M. J. Schiff & Associates, Inc.
Consulting Corrosion Engineers - Since 1959
431 W. Baseline Road
Claremont, CA 91711

Phone: (909) 626-0967 Fax: (909) 626-3316
E-mail lab@mjschiff.com
website: mjschiff.com

Table 1 - Laboratory Tests on Soil Samples

Proposed High-Rise Condominium Building Development
Your #4953-03-2451, MJS&A #03-0863HQ
25-Jul-03

Sample ID			3 @ 4.5'	4 @ 5.5'	4 @ 35.5'
Resistivity		**Units**			
as-received		ohm-cm	3,100	3,600	3,900
saturated		ohm-cm	2,600	1,400	1,400
pH			6.4	5.5	6.7
Electrical Conductivity		mS/cm	0.23	0.29	0.09
Chemical Analyses					
Cations					
calcium	Ca^{2+}	mg/kg	100	116	28
magnesium	Mg^{2+}	mg/kg	32	34	22
sodium	Na^{1+}	mg/kg	ND	ND	5
Anions					
carbonate	CO_3^{2-}	mg/kg	24	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	82	171	88
chloride	Cl^{1-}	mg/kg	45	60	35
sulfate	SO_4^{2-}	mg/kg	192	190	47
Other Tests					
ammonium	NH_4^{1+}	mg/kg	8.2	11.4	0.6
nitrate	NO_3^{1-}	mg/kg	52.2	64.3	ND
sulfide	S^{2-}	qual	na	na	na
Redox		mv	na	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed