ALL ITEMS FOR CONSIDERATION BY THE CITY COUNCIL/ COMMISSION ARE AVAILABLE FOR PUBLIC VIEWING IN THE OFFICE OF THE CITY CLERK AND THE CENTRAL LIBRARY

AGENDA FOR THE CONCURRENT ADJOURNED REGULAR MEETINGS OF THE CITY COUNCIL OF THE CITY OF COMMERCE AND THE COMMERCE COMMUNITY DEVELOPMENT COMMISSION CITY HALL EMERGENCY OPERATIONS CENTER 2535 COMMERCE WAY, COMMERCE, CALIFORNIA

TUESDAY, NOVEMBER 14, 2006 – 6:30 P.M.

CALL TO ORDER — Mayor/Chairperson Ramos

ROLL CALL — City Clerk/Acting Secretary Olivieri

PUBLIC COMMENT (TIME LIMITATION - 5 MINUTES)

Citizens wishing to address the City Council/Commission on any item on the agenda or on any matter not on the agenda may do so at this time. However, State law (Government Code Section 54950 et seq.) prohibits the City Council/Commission from acting upon any item not contained on the agenda posted 72 hours before a regular meeting and 24 hours before a special meeting. Upon request, the City Council/Commission may, in their discretion, allow citizen participation on a specific item on the agenda at the time the item is considered by the City Council/ Commission. Request to address City Council/Commission cards are provided by the City Clerk/Acting Secretary. If you wish to address the City Council/ Commission at this time, please complete a speaker's card and give it to the City Clerk/Acting Secretary prior to commencement of the City Council/Commission meeting. Please use the microphone provided, clearly stating your name and address for the official record and courteously limiting your remarks to five (5) minutes so others may have the opportunity to speak as well.

To increase the effectiveness of the Public Comment Period, the following rules shall be followed:

No person shall make any remarks which result in disrupting, disturbing or otherwise impeding the meeting.

SCHEDULED MATTERS

1. Presentation on Redevelopment Under California Law

   The **City Council and Commission** will receive a report on, and take the appropriate action with respect to, the basic purposes for redevelopment as outlined under the laws of the State of California, including an update on recent legislation affecting redevelopment signed into law by the Governor.

2. Presentation on Bond Financing for Redevelopment and City Projects

   The **City Council and Commission** will receive a report on, and take the appropriate action with respect to, the history and future uses of bond financing for redevelopment and City projects.

3. Review of Capital Improvement Projects Budget for Fiscal Year 2006-07

   Final on Consent

   The **City Council and Commission** will consider, and take the appropriate action with respect to, the Acting City Administrator/Executive Director's proposed Capital Improvement Projects budget for fiscal year 2006-07.

4. Veterans Memorial Park Reconstruction Project

   The **City Council and Commission** will consider entering into an arrangement to use redevelopment funds for the reconstruction of public grounds and facilities at Veterans Memorial Park. The entering into this arrangement first requires the City to make a finding that there are no other reasonable means of financing the building, facilities, structures or other improvements and provided that the Commission finds it necessary to carry out the redevelopment plan for Project Area 1 and makes certain specific findings under California Health and Safety Code §33445(a).

5. Agreement with Associated Soils Engineering, Inc. for Purpose of Performing Geotechnical Engineering Services at Veterans Memorial Park Recreation Facility

   The **City Council** will consider entering into an Agreement with Associated Soils Engineering, Inc., of Signal Hill, California, in the amount of $10,510.00, for geotechnical services at Veterans Memorial Park determined necessary due to severe damage to the roof framing and supports and authorizing the Mayor and City Clerk to execute the Agreement.

6. Acceptance of Proposal to Supply Power to Pole-Mounted Holiday Lights on Light Poles Located on Eastern Avenue

   The **City Council** will consider accepting the proposal from Merchants Electric Co., of Commerce, California, in an amount not-to-exceed $38,500.00, to supply power to pole-mounted holiday lights on light poles located on Eastern Avenue.

ADJOURNMENT

LARGE PRINTS OF THIS AGENDA ARE AVAILABLE UPON REQUEST FROM THE CITY CLERK'S OFFICE, MONDAY-FRIDAY, 8:00 A.M.-6:00 P.M.

ASSOCIATED SOILS ENGINEERING, INC.
Consulting Geotechnical Engineers

REPORT OF FOUNDATION DISTRESS INVESTIGATION AND REPAIR RECOMMENDATIONS

For

Veteran's Memorial Park Recreation Building
6364 Zindell Avenue
City of Commerce, California

Prepared For:
City of Commerce
2535 Commerce Way
Commerce, CA 90040

Project No. 07-5972
March 27, 2007

2860 WALNUT AVE - SIGNAL HILL, CALIF 90755 - PHONE 562/426-7990 - FAX 562/426-1842

ASSOCIATED SOILS ENGINEERING, INC.
Consulting Geotechnical Engineers

March 27, 2007
Project No. 07-5972

CITY OF COMMERCE
2535 Commerce Way
Commerce, California 90040

Attention: Mr. Larry Garcia, Public Services Manager

Subject: Report of Foundation Distress Investigation and Repair Recommendations
Veteran's Memorial Park Recreation Building
6364 Zindell Avenue, City of Commerce, California

Ladies and Gentlemen,

Presented herewith is the Report of Foundation Distress Investigation and Repair Recommendations ("Soils Report") prepared by Associated Soils Engineering Inc. (ASE) for the proposed distressed foundation repair at the subject site. This work was conducted in accordance with ASE's Proposal No. P06-149, dated September 21, 2006, which was subsequently authorized by the City of Commerce (i.e. City of Commerce Services Agreement dated November 14, 2006).

The subject geotechnical investigation was planned and performed based on a site reconnaissance conducted by ASE's engineering geologist prior to the submission of proposal, as well as relevant project plans (i.e. Architectural and Structural plan sheets for the Recreation Building, prepared by Anthony & Langford Architects, A.I.A. and Russ Conners Associates, Structural Engineers, with majority of plans dated May 29, 1969, with other plan sheets dated subsequently) provided by the City of Commerce.

The purpose of this study was to 1) delineate site subsurface soils conditions within the areas of Veteran's Memorial Park Recreation Building (the "Building") affected by the observed settlements and cracking, 2) identify potential factors that might have contributed to the foundation distress experienced on site, and 3) evaluate and formulate feasible mitigative measures in stopping the ongoing foundation distress and restoring the Building to its original elevations and functionality.

Based on the findings and results of ASE's field investigation and geotechnical analyses, the pattern of settlement within the affected areas of the Building has been established, the subsurface soils conditions underlying the Building have been delineated, and the likely geotechnical factors contributing to the foundation distress experienced on site have been identified. Repair/restoration measures/recommendations deemed feasible by ASE for the planned foundation repair have been formulated and, together with the summary of findings of the geotechnical field investigation, the study of site seismicity, and the results of laboratory tests performed, are presented in the Soils Report.

We at ASE appreciate the opportunity to provide our professional services on this important project, and look forward to assisting you during site grading and building construction.

If you have any questions or require additional information, please contact the undersigned.

Respectfully submitted,
ASSOCIATED SOILS ENGINEERING, INC.

Gary L. Martin
Project Engineer

Lawrence J.D. Chang. PE
Civil Engineer, RCE 67987

REGISTERED PROFESSIONAL ENGINEER
LAWRENCE J.D. CHANG
No. 67987
Exp. 6/30/07
CIVIL
STATE OF CALIFORNIA

Edward C. (Ted) Riddell, CEG-1775
Engineering Geologist

REGISTERED GEOLOGIST
E.C. RIDDELL
No. 1775
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

GM/LC/ECR:lc/cmc

Distribution: (6) Addressee

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1.0 INTRODUCTION

This Soils Report presents the results of ASE's geotechnical investigation for the planned distress foundation repair at the Veteran's Memorial Park Recreation Building, located at 6364 Zindell Avenue, in the City of Commerce, California (i.e. the "Site"). The approximate location of the Site is shown on the Site Location Map (Figure 1). The purpose of this investigation was to 1) delineate site subsurface soils conditions within the areas of Veteran's Memorial Park Recreation Building (the "Building") affected by the observed settlements and cracking, 2) identify potential factors that might have contributed to the foundation distress experienced on site, and 3) evaluate and formulate feasible mitigative measures in stopping the ongoing foundation distress and restoring the Building to its original elevations and functionality. This Soils Report presents the summary of the data collected, and the results of ASE's engineering evaluations/analyses, which provide the basis for the formulation of relevant geotechnical conclusions and recommendations.

1.1 PROJECT AND SITE DESCRIPTION

The existing building is located at the subject address within the Veterans Memorial Park, just to the west of the southeastern end of Zindell Drive in the City of Commerce, California. The site is relatively flat and the building is bounded by lawn and/or concrete walkways on all sides. The building consists of four floors, two of which are subterranean and they are of differing size and configuration and neither are the same configuration as the building envelope. The most apparent settlement distress is at the northeast corner of the building where the exterior column appears to have settled roughly four-inches in relation to the building walls. Significant cracking and apparent settlement in the concrete slab of the upper basement floor adjacent to the basement walls in the northeast portion of the basement was also noted.

2.0 SCOPE OF INVESTIGATION

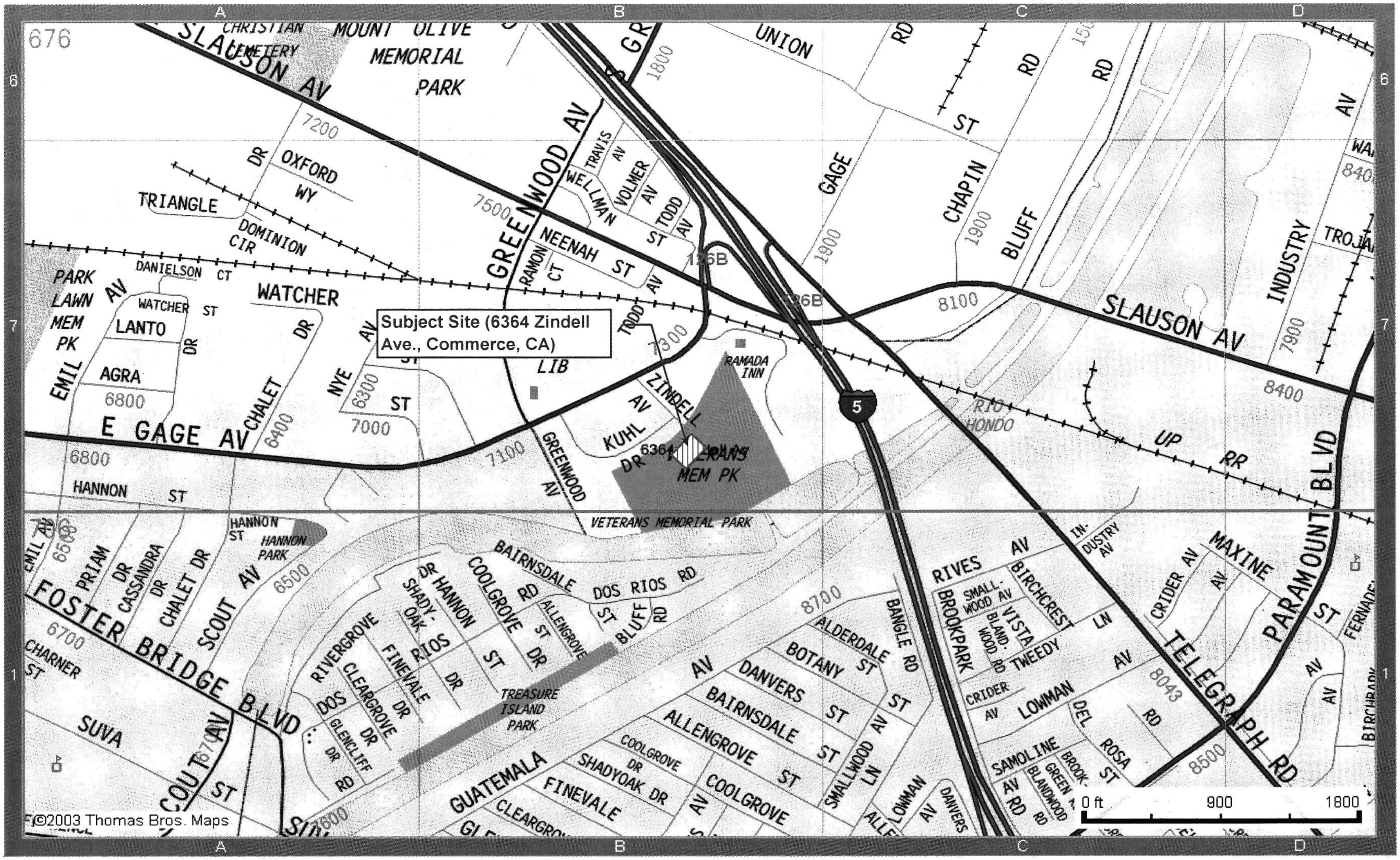
As per stipulated in the City of Commerce Services Agreement dated November 14, 2006, the scope of the subject investigation has included the performance of the following geotechnical tasks:

a. Review of all available geotechnical and structural engineering data on and adjacent to the site. Based upon our meeting in the field, building plans are to be provided by you.

b. Geologic reconnaissance and mapping of alluvial and existing fill areas and to establish location and accessibility of planned borings. The boring locations were pre-marked by ASE for subsequent checking/identification by Underground Service Alert to insure that no utilities would be damaged during field investigation.

c. Perform a "Manometer" floor level survey of the exposed portions of the upper basement floor to determine the extent of the distress.

d. Excavation, sampling and logging of three (3) exploratory borings ranging in depth from 25 feet to 50 feet 6 inches utilizing a truck mounted hollow stem auger rig to determine



Associated Soils Engineering, Inc. 2860 Walnut Avenue Signal Hill, CA 90755 Tel (562) 426-7990 Fax (562) 426-1842	Site Location Map	**Proj. Name:**	*Foundation Distress Inv. & Repair Recomm., Veteran's Memorial Park Recreation Bldg., 6364 Zindell Ave.,Commerce, CA*
		Figure 1	**Proj. No.:** 07-5972
			Date: March, 2007

ASSOCIATED SOILS ENGINEERING, INC. Consulting Geotechnical Engineers general subsurface conditions, liquefaction potential and to delineate any other soil and geologic parameters that may affect the proposed foundation repair. All exploratory borings were backfilled with cuttings on the same day upon completion of investigation.

e. Appropriate laboratory testing for determination of classification, consolidation, shear strength, maximum density/optimum moisture content, expansion characteristics and soil corrosivity of soil materials as necessary to supplement any existing data.

f. Prepare a single Report of Foundation Distress Investigation and Repair Recommendations for the parcel addressing the geotechnical parameters outlined above and including recommendations that would typically be used for earthwork factors, settlement, seismicity evaluation and repair recommendations.

Please be reminded that this geotechnical exploration did not include any evaluation or assessment of hazardous or toxic materials that may or may not exist on or beneath the site. ASE does not consult in the field of potential site contamination/mitigation.

3.0 SITE EXPLORATION

ASE's on-site subsurface geotechnical exploration was performed on February 26, 2007, consisting of advancing three (3) exploratory borings at the approximate locations shown on the attached Boring Location Map, Plate 1. The exploratory drillings were excavated by Choice Drilling, Inc. utilizing a truck-mounted drilling rig equipped with 8-inch-diameter hollow-stem augers, with sampling by both Standard Penetration Test (SPT) sampler and Modified California barrel sampler. The borings extended to depths ranging from twenty-five (25) feet to fifty (50) feet six (6) inches from the corresponding existing grades.

Continuous observations of the materials encountered in the borings were recorded in the field. The soils were classified in the field by visual and textural examination, and were further examined/re-classified in ASE's laboratory with relatively undisturbed ring samples and disturbed SPT and bulk soil samples obtained from the field. Relatively undisturbed samples of soils were extracted in 2.375-inch I.D. thin walled brass rings. All samples were secured timely in moisture-resistant bags to minimize the loss of field moisture, followed by prompt delivery to the laboratory for ensuing testing.

Detailed descriptions of the soils encountered and conditions observed during the subsurface exploration are shown in the attached Field Logs of Boring, Plates B-1 through B-3, appended in Appendix A. The boring logs also present the USCS classifications of soils encountered, depths and types of soil samples, field dry densities and moisture contents, as well as the corresponding laboratory tests performed.

Upon completion of exploration, all exploratory borings were backfilled with drill cuttings and tamped manually.

4.0 SUBSURFACE CONDITIONS

4.1 SUBSURFACE SOIL CONDITIONS

On-site subsurface soils encountered in the 3 exploratory borings can be generally categorized into 3 different soil strata, namely upper artificial fill, intermediate debris fill and underlying native alluvial soils.

The upper artificial fill layer was seen to vary in thickness from about 1 foot in Boring B-1 to 4.5 feet in Boring B-2, consisting mainly of dark brown to dark grayish brown very fine to fine-grained silty clay with trace fine sand to sandy silt with clay. Soils in this unit were generally moist and medium dense, with organic odor sensed in Boring B-3.

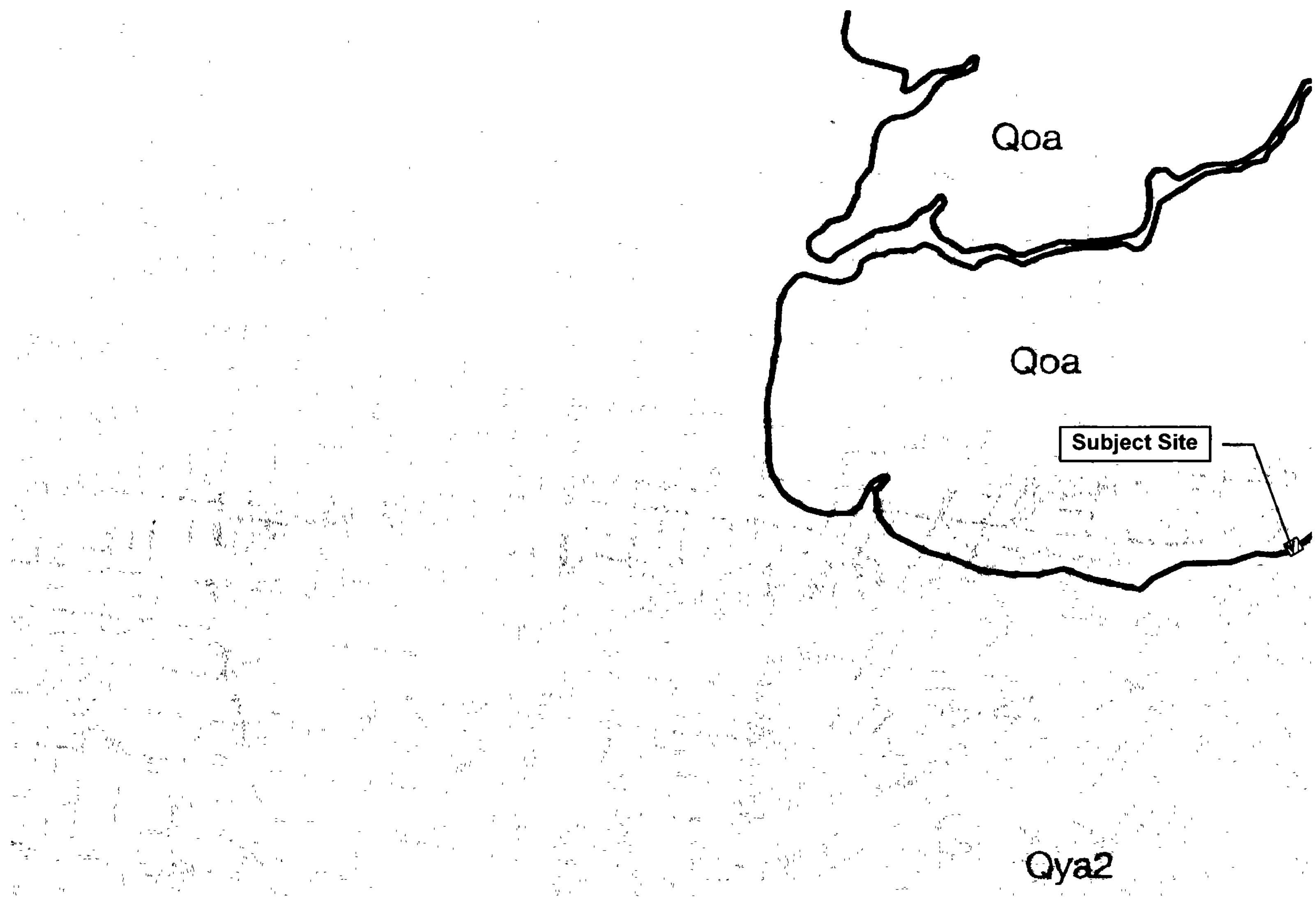
Immediately beneath the upper artificial fill layer in each exploratory boring was artificial fill of sand, silt and clay containing significant amount of trash and debris ranging in thickness from about 9 feet in Boring B-3 to 25 feet in Boring B-1.Debris fill consisting mostly of trash in a soil matrix was present below depths of 9.5 feet in Boring B-1 and 15 feet in Boring B-2. In addition to noticeable organic odor, the debris fill layer was found to be highly heterogeneous, porous and moist, exhibiting a loose to medium dense density in its present state. Photos of samples containing debris retrieved from the exploratory borings are shown in Appendix D, Photo Evidence of Debris Fill.

The native alluvial soil underlying the debris fill in all 3 exploratory borings is part of the Quaternary-age older alluvium (CDMG, 1998) that is characterized with alternating beds of medium dense to very dense sand and silt, with some clay. Figure 2, Local Geologic Map, depicts the distribution of different geologic materials in the vicinity of the Site. In specific, site native alluvial soils were found to be damp to moist and medium dense to very dense, consisting predominantly of gray/light gray to light olive/yellowish brown fine to medium-grained clean sands, silty sand with gravel, sandy silt with clay, and clayey silt.

4.2 GROUNDWATER AND CAVING

No groundwater was encountered in ASE's field exploration to the maximum explored depth of 50 feet 6 inches in Boring B-2.

Published data in Seismic Hazard Zone Report 034 by the CGS (i.e. Seismic Hazard Zone Report for the South Gate 7.5-Minute Quadrangle, Los Angles County, California, 1998) indicates that the historic high groundwater level in the subject area is between 10 to 20 feet below grade. Maps reviewed indicate that the subject site is approximately 118 feet above Mean Sea Level ("MSL").



(Excerpt of the Quaternary Geologic Map of the South Gate 7.5- Minute Quadrangles, California Division of Mines and Geology, Seismic Hazard Zone Report 034, 1998)

LEGEND

▥	Approximate Site Location
Q_{oa}	Quaternary-age older alluvium consisting of alternating beds of medium dense to very dense sand, clay and silt

SOILS ENGINEERING, INC.
Consulting Geotechnical Engineers

Associated Soils Engineering, Inc.

2860 Walnut Avenue

Signal Hill, CA 90755

Tel (562) 426-7990 Fax (562) 426-1842

Project:	***Foundation Distress Inv. & Repair Recomm., Veteran's Memorial Park Recreation Bldg., 6364 Zindell Ave., Commerce, CA***		
Figure 2	Local Geologic Map		
Proj. No.:	**07-5972**	**Date:**	**March, 2007**

A review of the Los Angeles County Public Works Department Hydraulic/Water Conservation Records Division website (www.ladpw.org) indicates that the historic high groundwater level in Well No. 1572S, closest to the site on the northwest side of the Rio Hondo Channel between Telegraph Road and the Santa Ana Freeway (Interstate 5), was 62.0 feet below ground surface elevation on April 29, 1994. The ground surface elevation of the well is 147.5 feet above MSL. The depth to groundwater for the most recent reading in this well (taken May 15, 2002) was 80.0 feet below the ground surface.

Generally, seasonal and long-term fluctuations in the groundwater may occur as a result of variations in subsurface conditions, rainfall, run-off conditions and other factors. Therefore, variations from our observations may occur.

The use of hollow-stem augers during drilling precluded observation of potential caving conditions which may have otherwise occurred in an uncased hole. Caving and/or sloughing was not measured within the borings during the extraction of auger stem at the completion of boring operations. However, caving and/or soil sloughing may be likely in excavations greater in dimension than our test borings.

4.3 UTILITIES

A standing water pipe with tap was noticed on the north end of the site just off the private access road. Irrigation lines are present in turf and planter areas. No other overhead or underground utilities were encountered within the site during the course of our field work for this project. Overhead lines were present along the northerly limit of the private access road serving the neighboring single family residence. Other utilities, though not known at the time of this report preparation, may be present on site, and should be located and incorporated into site development plans accordingly.

5.0 FAULTING AND SEISMICITY

Commerce, like the rest of Southern California, is located within a seismically active region as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional faults such as the San Andreas, San Jacinto, Newport-Inglewood and Whittier-Elsinore fault zones.

By the definition of the CGS, an active fault is one which has had surface displacement within the Holocene Epoch (roughly the last 11,000 years). The CGS has defined a potentially active fault as any fault which has been active during the Quaternary Period (approximately the last 1,600,000 years). These definitions are used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazard Zones Act of 1972 and as subsequently revised in 1997 as the Alquist-Priolo Earthquake Fault Zoning Act and Earthquake Fault

Zones. The intent of the act is to require fault investigations on sites located within Special Studies Zones to preclude new construction of certain inhabited structures across the trace of active faults. The subject site is not located within the Alquist-Priolo Earthquake Fault Zone. No evidence of active or potentially active faulting was observed during our investigation.

Several sources were consulted for information pertaining to site seismicity. The majority of data was obtained from the publication by Sadigh, Chang et.al. (1997) which has been incorporated into a digital program by Blake (2000) that allows for an estimation of peak horizontal acceleration using a data file of approximately 150 digitized California faults. This program compiles various information including the dominant type of faulting within a particular region, the maximum earthquake magnitude each fault is capable of generating, the estimated slip-rate for each fault, and the approximate location of the fault trace. Printouts of the results of the fault search for the subject site are shown in Appendix C.

The subject site is likely to be subject to strong seismic ground shaking during the life of the project. The Puente Hills Blind Thrust Fault is closest to the site and is located approximately 3.4 miles (5.5 km) away. Other nearby active faults include the Whittier Fault, the Upper Elysian Park Blind Thrust Fault and the Newport-Inglewood (L.A. Basin) Fault, located approximately 6.3 miles (10.2 km), 6.4 miles (10.3 km) and 9.6 miles (15.5 km) away, respectively.

Based on the referenced literature and deterministic analysis, the Puente Hills Blind Thrust Fault, approximately 3.4 miles (5.5 km) from the site, would probably generate the most severe site ground motions. A Maximum Probable Earthquake, i.e. the maximum earthquake that is considered likely to occur during a 100-year time interval, of 7.1 Mw (moment magnitude as per USGS) has been assessed along the Puente Hills Blind Thrust Fault. As shown in Appendix C, estimated peak horizontal ground acceleration ("PGA") resulting from the above-stated maximum earthquake on the Puente Hills Blind Thrust Fault is on the order of 0.535g should this event occur at the fault's closest approach to the site. In addition, approximately 42 active or potentially active faults have been found within 62 miles (100 km) of site.

The seismicity of the site was also evaluated utilizing probabilistic analysis available from CGS. As described in Cao et al (2005) and Peterson et al (1996), the CGS analytical framework considers two earthquake sources, i.e. fault sources and area sources, together with geologic/soil characteristics and tectonic movements, for the quantification of PGA of bedrock that carries a 10% exceedance probability in 50 years. As site-specific ground conditions, e.g. soft rock and alluvium, might attenuate or amplify bedrock-based PGA's, CGS further incorporates recommendations proposed by NEHRP (1994 & 1997) that modify bedrock-based PGA's for both soft rock sites and alluvium sites. For structural design with a typical damping ratio of 5%, two spectral acceleration ("S_a") values representing structural periods of 0.2 second (typical of low-rise buildings) and 1.0 second (typical of multi-story buildings) have also been analyzed. As shown in the Appendix C, CGS's probabilistic analysis with a soil classification of S_D, as the site is underlain predominantly by undocumented debris fill and native alluvial soils, indicates that the site is subject to a PGA of 0.446g, a Sa (0.2 sec) of 1.084g, and a S_a (1.0 sec) of 0.524g.

As the PGA assessed from the deterministic analysis using "EQFAULT", i.e. 0.535g, appears to be more conservative, it is recommended herein to be incorporated in project structural repair design and planning, if dynamic structural analysis is adopted. It should be noted that the earthquake design requirements listed in the 2001 CBC and other governing standards apply only for faults classified as "active" in accordance with the most recent fault listing as per the United States Geological Survey (USGS) or the CGS. Based on the faulting information evaluated above, the following seismic design parameters have been derived:

CBC Seismic Design Parameters

2001 CBC Chapter 16 Table #	**Seismic Parameter**	**Indicated Value or Classification**
16-I	Seismic Zone Factor	0.40
16-J	Soil Profile Type	S_D
16-Q	Seismic Coefficient C_a	0.44 N_a
16-R	Seismic Coefficient C_v	0.64 N_v
16-S	Near-Source Factor N_a	1.00
16-T	Near-Source Factor N_v	1.18
16-U	Seismic Source Type	B

The Structural Consultant should review the above parameters and the 2001 CBC to evaluate the seismic design. Final selection of design coefficients should be made by the Structural Consultant based on the local laws and ordinances, expected structure response, and the desired level of conservatism. If site-dependent earthquake response spectra or other specific design parameters are needed by the Structural Consultant, or are required by the local government agency with jurisdiction over the project, ASE should be promptly contacted for further evaluation.

6.0 MANOMETER SURVEY

A manometer survey aiming at mapping the settlement pattern of the affected areas within the Building was performed by ASE's engineer on February 23, 2007, with results shown on the appended Plate 2, Results of Manometer Survey.

As evidenced in Plate 2, a relative differential settlement up to 4 inches has been mapped between the control point and the northeast corner of the Building. While clearly suggesting both a northerly descending trend and an easterly descending trend from the control point, the area where the maximum differential settlement was mapped happens to coincide with the deepest debris fill (i.e. up to 28 feet deep from existing surface grade) encountered in

Borings B-1 and B-2. As less debris fill was encountered in Boring B-3 located to the east of the Building footprint (i.e. up to 12.5 feet deep from existing surface grade), the magnitude of differential settlement was also found to reduce to around 1 inch, signaling that 1) the significant settlement experienced at the Building has been adversely impacted by the presence of highly heterogeneous and loose debris fill, and 2) the scale of differential settlement recorded at different areas within the Building appears to be in direct proportion to the local thickness of the underlying debris fill. As such, any mitigative measures planned for the distressed foundation repair should either modify/replace the undesirable debris fill thoroughly and replaced with engineered, compacted fill, or relieve/separate the foundation bearing from the settling debris fill, or both.

7.0 GEOTECHNICAL EVALUATION AND REPAIR RECOMMENDATIONS

In formulating pertinent recommendations for the planned foundation distress repair, the following factors have been considered by ASE:

1. The highly heterogeneous, incompetent debris fill encountered from roughly 1 foot below existing site grade to as deep as 28 feet practically envelopes and partially underlies the 2-level subterranean structure where the subject foundation distress has been observed. ASE anticipates that the debris fill in its present state is prone to further settlement, highly uneven in part, likely caused by continuing decomposition of its organic content, as well as by additional volumetric compression of its loose, porous structure.
2. The site native soils underlying the undesirable debris fill were found to be consisting mainly of medium dense to very dense, damp to very moist sand/silty sand and sandy/clayey silt, deemed capable of providing sufficient bearing support for the future, repaired structural foundation.
3. The proposed repair measures should incur minimum disruption to the ongoing operation of the Building, and should not cause extensive structural removal and alteration.
4. As part of the foundation repair, the displaced/settled portion of the Building should be restored back to its pre-distress leveling and grade.
5. While the intended target of stopping ground movement and restoring the Building should be accomplished upon completion of the planned foundation repair, the proposed repair measures should be cost and time effective.

The following foundation repair and mitigative measures and criteria have taken the above factors into consideration, and should be considered in the project design, plans and specifications and implemented during foundation repair. Please be reminded that, as foundation distress repair typically involves specially-designed and, often times, patented construction techniques and equipment, contractors specialized in similar foundation repair works should be engaged in proposing, planning and designing site-specific, performance-based repair program, which in turn should be reviewed and approved by the Geotechnical Consultant prior to contract awarding.

As actual site subgrade conditions may deviate from the findings/conclusions gathered from ASE's limited-scale site exploration, it is advisable that a rigorous testing and inspection program should be formulated by the Geotechnical Consultant and implemented during site foundation repair. This is to ensure the conformance of project performance specifications and, if necessary, to timely review/evaluate alternative measures should vastly different subgrade conditions arise that could otherwise render the proposed repair works inefficient or ineffective.

7.1 SITE PREPARATION:

Prior to the inception of repair work on site, it will be necessary to remove designated existing improvements, if any, including any remaining exposed and buried obstructions, which may be in the areas of proposed foundation repair. All debris generated from site demolition operations should be disposed of off-site.

Any underground utilities to be present within the zone of proposed foundation repair should be cut off or re-routed a minimum of 5 feet from the area of foundation repair. The ends of cut off lines should be plugged a minimum of 5 feet with concrete exhibiting minimum shrinkage characteristics to prevent water migration to or from hollow lines. Capping of lines may also be required should the plug be subject to any line pressure. Encountering of other underground utilities on site that might undermine the stability of the repaired foundation should be brought to the immediate attention of the Geotechnical Consultant for evaluation and remedial recommendations, as appropriate.

Local ordinances relative to abandonment of underground utilities, if more restrictive, will supersede the above minimum requirements.

7.2 FOUNDATION REPAIR ALTERNATIVES:

In view of stopping the ongoing subgrade movement on site that has resulted in the subject foundation distress, and of restoring the affected areas of the Building back to their respective pre-distress grades and functionalities, the following foundation repair alternatives have been evaluated by ASE to be geotechnically feasible. The City should review and select the suitable alternative based on the targeted totality and end result of repair and budget and time constraints.

7.2.1 Alternative 1: Total Removal and Restoration:

A total removal of all debris fill surrounding and underlying the affected areas of the Building followed by replacement with engineered, compacted fill to grade and re-leveling of displaced building foundation, is deemed possible. However, this alternative is likely to be associated with the following disadvantageous factors:

a. The extent of the debris fill is unknown and may need to be chased and removed entirely to fully eliminate any future settlement potential.

b. Given the anticipated tight space, area below the existing club and meeting room footings and slabs at lower subterranean level may need to be backfilled with control density fill (i.e. slurry fill) after removal of debris fill.

c. Very extensive underpinning design and installation, some of it sacrificial in nature, will be required to support existing structural and non-structural features that are to be exposed during debris removal and fill re-placement.

d. The extent of removal and fill placement would inevitably impact the ongoing operation of the Building.

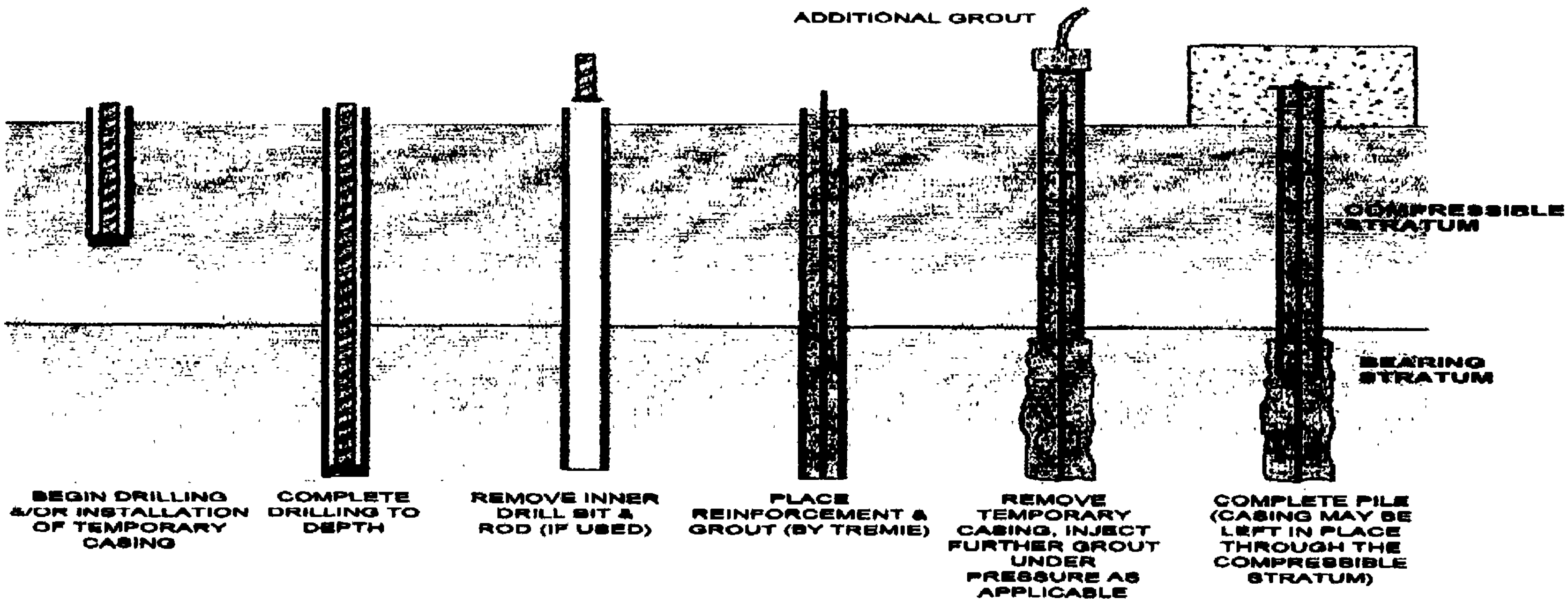
This alternative is therefore not considered by ASE to be suitable for the planned foundation repair.

7.2.2 Alternative 2: Micropiling and Re-Leveling:

7.2.2.1 Micropiling System:

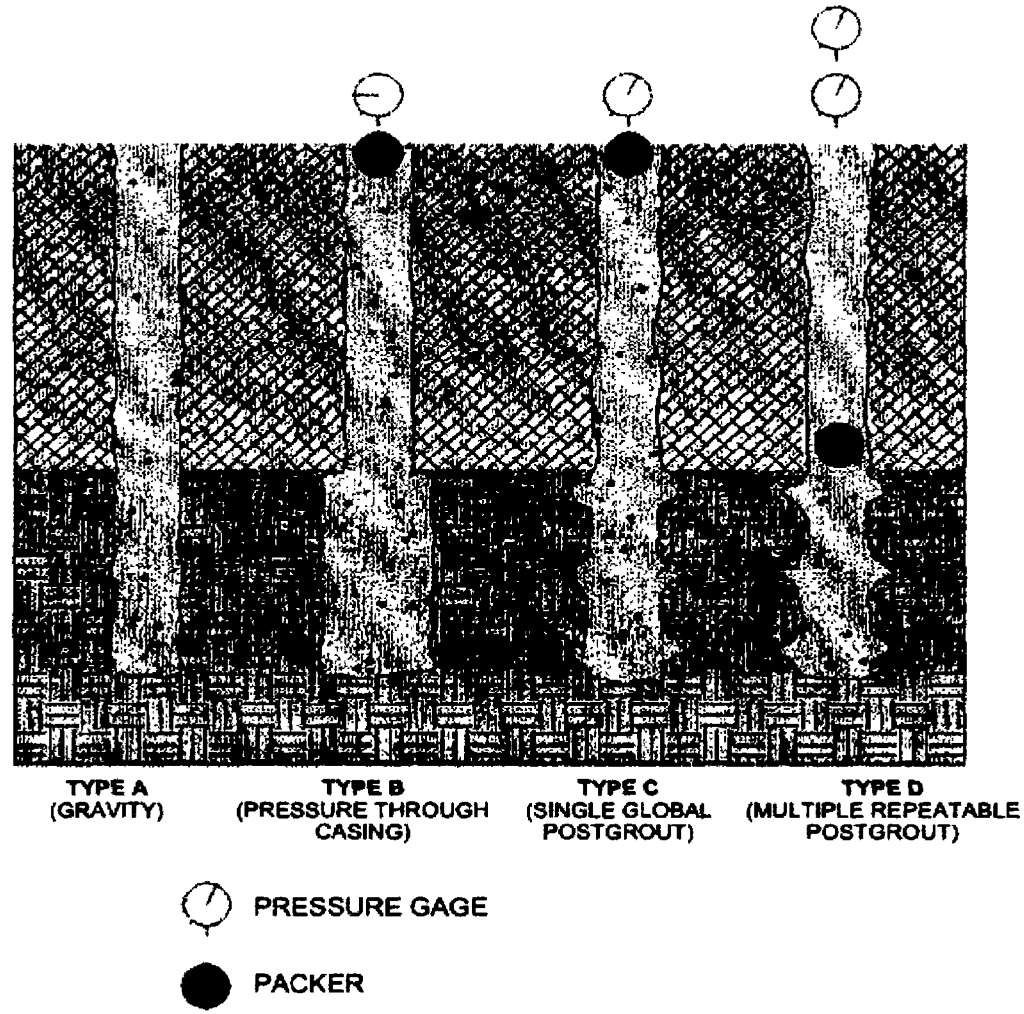
ASE's field investigation revealed that medium dense to very dense native alluvial soils underlie the undesirable debris fill at varying depths in all of the 3 borings explored. The settlement pattern measured by manometer survey, detailed in Section 6.0 above, indicates that areas within the Building experiencing most settlement coincide with exploratory borings, i.e. Borings B-1 and B-2, in which thicker debris fill layer was encountered, signaling that the magnitude of settlement may be in direct proportion to the presence and thickness of debris fill on site. As such, without thorough removal of debris fill and replacement with engineered fill as discussed in Section 7.2.1 above, gaining bearing support from and re-leveling the Building upon the debris fill in its present state is deemed unfeasible. More over, due to the highly heterogeneous nature of and the potentially high organic content within the debris fill, any remedial measure aiming at modifying or solidifying the debris fill, such as chemical grouting, pressure grouting or deep soil mixing, is not anticipated to be effective, both performance-wise and cost-wise.

Micropiling, a relative small-diameter (typically less than 12 inches), often reinforced, drilled or grouted replacement pile system, is deemed by ASE to be suitable for the intended foundation distress repair and re-leveling within the Building. Typical construction sequencing of micropile installation and configuration of 4 different types of micropiling system as per defined by FHWA (2000) are depicted in Figure 3, Construction Sequence of Micropiling with Casing, and Figure 4, Categorization of Micropiling Based on Grouting Procedure, respectively. While installation of micropiles has constantly been performed in areas with limited access and headroom, such as the Building, it



* Diagram duplicated from Figure 1-1, Micropile Construction Sequence using Casing, as appeared on *Micropile Design and Construction Guidelines*, 2000, issued by FHWA.

ASSOCIATED SOILS ENGINEERING, INC Consulting Geotechnical Engineers **Associated Soils Engineering, Inc.** **2860 Walnut Avenue** **Signal Hill, CA 90755** **Tel (562) 426-7990 Fax (562) 426-1842**	Construction Sequence of Micropiling with Casing	**Proj. Name:**	***Foundation Distress Inv. & Repair Recomm., Veteran's Memorial Park Recreation Bldg., 6364 Zindell Ave.,Commerce, CA***
		Figure 3	**Proj. No.:** 07-5972
			Date: March, 2007



* Diagram duplicated from Figure 2-5, Micropile Calssification based on Type of Grouting, as appeared on *Micropile Design and Construction Guidelines*, 2000, issued by FHWA.

Associated Soils Engineering, Inc. **2860 Walnut Avenue** **Signal Hill, CA 90755** **Tel (562) 426-7990 Fax (562) 426-1842**	Categorization of Micropiling Based on Grouting Procedure	**Proj. Name:**	***Foundation Distress Inv. & Repair Recomm., Veteran's Memorial Park Recreation Bldg., 6364 Zindell Ave.,Commerce, CA***
		Figure 4	**Proj. No.:** 07-5972
			Date: March, 2007

is the ability to carry and transfer the loading of the distressed foundation through the debris fill into the underlying native, firm soil layer that is considered the most essential.

For preliminary design purpose, all micropiles installed on site should be cased with galvanized steel pipes through the debris fill layer to protect from potential decomposition and corrosion, and should gain embedment into the underlying firm native soils. The embedment (or grout bulb) depth into the underlying firm, native soils should be determined by the specialty contractor based on his preferred configuration and layout of micropiles, and by the superimposed foundation loading. As a general guideline, the nominal grout-to-ground bond strength values as per recommended in FHWA (2000) for native soils encountered on site on a boring-by-boring basis are tabulated as follows:

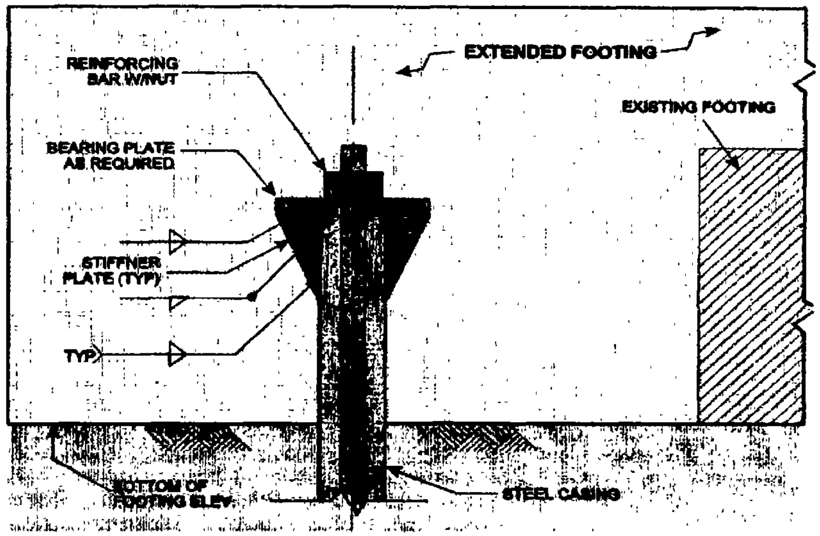
	Location [2]		
	B-1 (below 25' deep)	B-2 (below 28' deep)	B-3 (below 12.5' deep)
Nominal Grout-to-Ground Bond Strength (kips/ft^2)[1]	5.0	2.7	2.7

1. Bond strength is for per square foot of contact area between grout bulb and the surrounding native soils, and is taken as the median value of the range stipulated in FHWA (2000). A factor of safety of **2.5** should be applied in deriving the allowable bond strength value. Final design value should be adjusted based on results of field load testing. The bond strength is deemed equally applicable to the support of vertical compressive loading or uplift force.

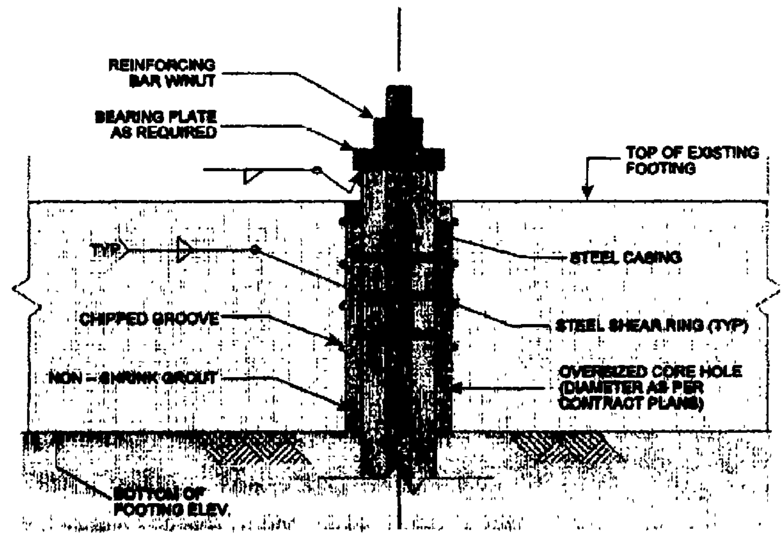
2. Due to the variation of soil types and properties encountered on site, different values should be applied to different soils anticipated in different areas. The demarcation of different "soil areas" should be defined based on the results of field load testing. For preliminary design purpose, the lowest value among all boring locations may be used for quantifying micropile dimension, depth and layout.

There is no group reduction or stiffening factor applicable to individual micropiles provided a minimum pile center-to-center spacing equivalent to 4 pile diameters is maintained. Should closer pile spacing be needed, the Geotechnical Consultant should be consulted for further evaluation and recommendation.

Figure 5, Typical Micropile – Footing Connection Arrangements, illustrates the general connection detail between installed micropiles and the existing footings. It should be noted that, depending on area accessibility and constraints and magnitude and complexity of structural loading, different micropile – footing connection arrangements may be selected for different applications. In general, footings that are easily accessible from ground surface may be supported by micropiles installed outside of footing perimeter together with an newly extended/enlarged footing structurally connected to the



(For micropiles installed outside of the perimeter of existing footing.)



(For micropiles installed within the footprint of existing footing.)

* Diagram duplicated from Figures 5-6 and 5-7, Pile to Footing Connection Detail, as appeared on *Micropile Design and Construction Guidelines*, 2000, issued by FHWA.

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Consulting Geotechnical Engineers

Associated Soils Engineering, Inc.
2860 Walnut Avenue
Signal Hill, CA 90755
Tel (562) 426-7990 Fax (562) 426-1842

Typical Micropile - Footing Connection Arrangements	**Proj. Name:**	*Foundation Distress Inv. & Repair Recomm., Veteran's Memorial Park Recreation Bldg., 6364 Zindell Ave.,Commerce, CA*	
	Figure 5	**Proj. No.:**	07-5972
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existing footing to limit impact/alteration to the existing footing. For footings that are buried underneath subterranean structure where access is limited, micropiles may need to be installed through holes cored through the existing footings. Other micropile-footing connection arrangements may be utilized, provided the details are reviewed and accepted by both the Structural Consultant and Geotechnical Consultant.

For foundations subject to lateral loading, it might be beneficial to install micropiles in a battered manner. However, the layout as to which micropiles needed to be battered at what angle should be planned by the specialty contractor based on the structural loading information provided by the Structural Consultant, and should be subject to review and acceptance by the Geotechnical Consultant prior to installation. There is no reduction to the grout-to-ground bond strength values tabulated above for battered micropiles.

As the debris fill on site is not anticipated to be removed should the micropiling alternative be adopted, the settlement taking place in the debris fill layer is anticipated to continue as a result of decomposition of organic content and volumetric compression of porous, loose fill structure. This situation implies that downdrag may develop against the micropiles due to differential movement between surrounding debris fill and micropile casing. However, considering that 1) galvanized steel pipe possessing relatively small surface frictional resistance will likely be used as micropile casing, 2) the debris fill is porous and loose in nature that may not be in constant, intimate contact with the steel casing, and 3) lateral force exerted by debris fill onto the steel casing is anticipated to be relatively small due to the heterogeneous and loose fill structure, it is ASE's assessment that downdrag should not be a critical factor affecting the long-term performance of installed micropiles. Nonetheless, this assessment should be re-visited upon the availability of results of field load testing prior to finalization of production micropiling program.

7.2.2.2 Pile Load Testing:

The performance and capacity of micropiles may vary significantly from the preliminary assumptions made based strictly on limited information/findings gathered during field investigation phase. It is therefore prudent to carry out pile load testing program to 1) verify/revise pile design load capacity, 2) monitor pile-soil displacement pattern upon loading, 3) evaluate adequacy of the design and installation procedures proposed by the specialty contractor, and 4) optimize final production layout and configuration.

Per the recommendations of FHWA (2000), as a minimum, the following tabulated pile load testing criteria should be considered and incorporated in the micropile design and construction.

Item	Quantity / Criterion	Remarks
Verification Test Pile	1 ~ 2	1. Sacrificial piles tested to **2.5** times of design load, or until the reach of failure, whichever comes first. 2. The failure is defined as the gradient of the tangent to the load-displacement curve exceeds 0.15mm/kN.
Proof Test Pile	2 ~ 4	1. Sacrificial piles or part of the production piles (if failure does not occur during pile testing) tested to **1.67** times of design load, or until the reach of failure, whichever comes first. 2. Same failure criterion as that of verification test pile applies.
Pile Creep Test	< 2mm per log cycle of time	1. Mainly for micropiles installed into cohesive, clayey soils. (Should be decided at the discretion of the Geotechnical Consultant at the time of pile load testing on a pile-per-pile basis.)

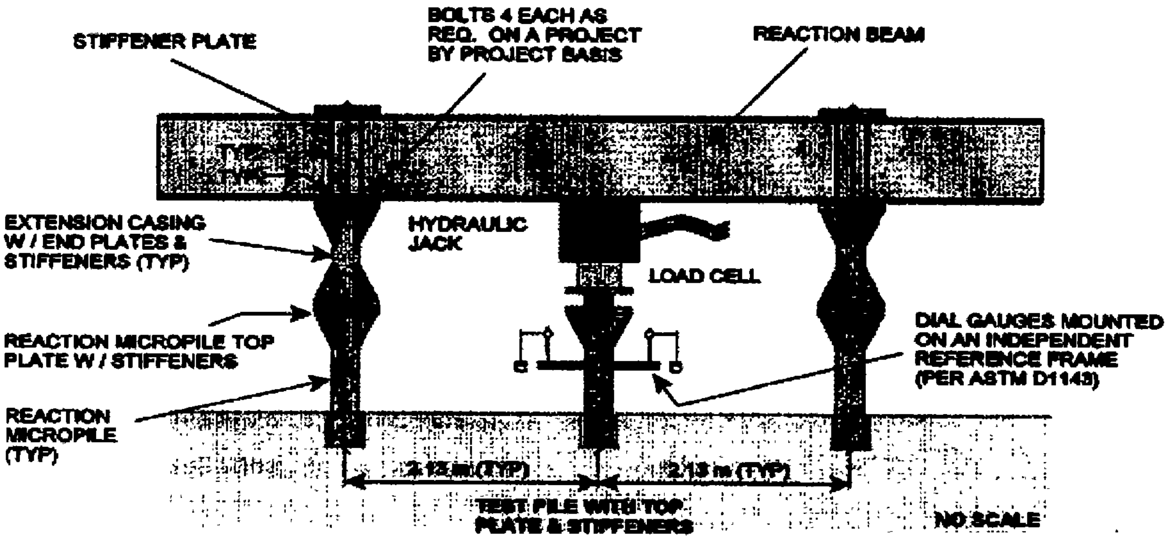
Pile load testing should be performed based on procedures and criteria as per stipulated in the latest edition of ASTM D-1143 and D-3689 test methods. Figure 6, Typical Micropile Load Test Set-up, illustrates the typical configuration of pile load test arrangements.

While verification test piles should be sacrificial in nature, the proof test piles may be deemed as part of the production piling program, provided they are not classified as "failure" upon completion of proof load testing. For each failed production pile, unless otherwise reviewed and approved by the Geotechnical Consultant, a minimum of two (2) replacement piles should be installed as remedy.

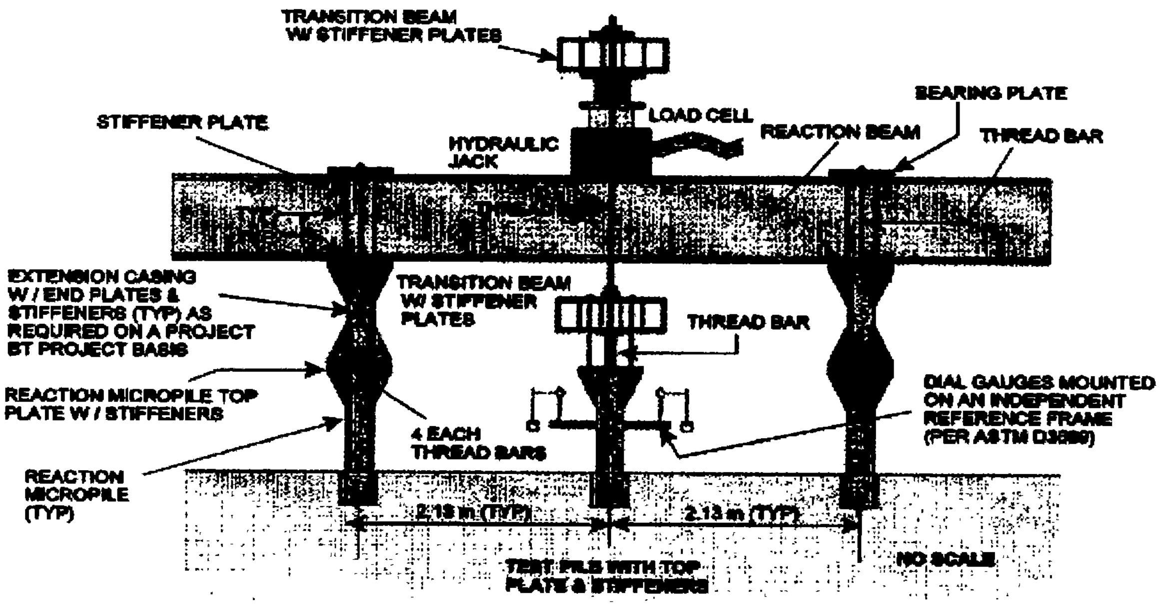
Due to the presence of highly heterogeneous debris fill with varying thickness around the affected areas of the Building, actual production pile lengths, layout and configuration should not be finalized until the availability of the pile load test data.

7.2.2.3 Re-leveling with Underpinning:

The utilization of micropiling as discussed in the previous section is anticipated to be effective in transferring the structural loading of the Building to the underlying firm, native soils while eliminating the bearing on the settling debris fill. This measure, however, does not provide the means of re-leveling the Building. A separate procedure aiming at underpinning and re-leveling the Building should be implemented.



TYPICAL COMPRESSION LOAD TEST ARRANGEMENT (ASTM D1143)



TYPICAL TENSION LOAD TEST ARRANGEMENT (ASTM D3689)

* Diagrams duplicated from Figure 7-1, Compression Load Test Arrangement, and Figure 7-2, Tension Load Test Arrangement, as appeared on *Micropile Design and Construction Guidelines*, 2000, issued by FHWA.

Associated Soils Engineering, Inc.
2860 Walnut Avenue
Signal Hill, CA 90755
Tel (562) 426-7990 Fax (562) 426-1842

Project:	*Foundation Distress Inv. & Repair Recomm., Veteran's Memorial Park Recreation Bldg., 6364 Zindell Ave., Commerce, CA*
Figure 6	**Typical Micropile Load Test Set-up**
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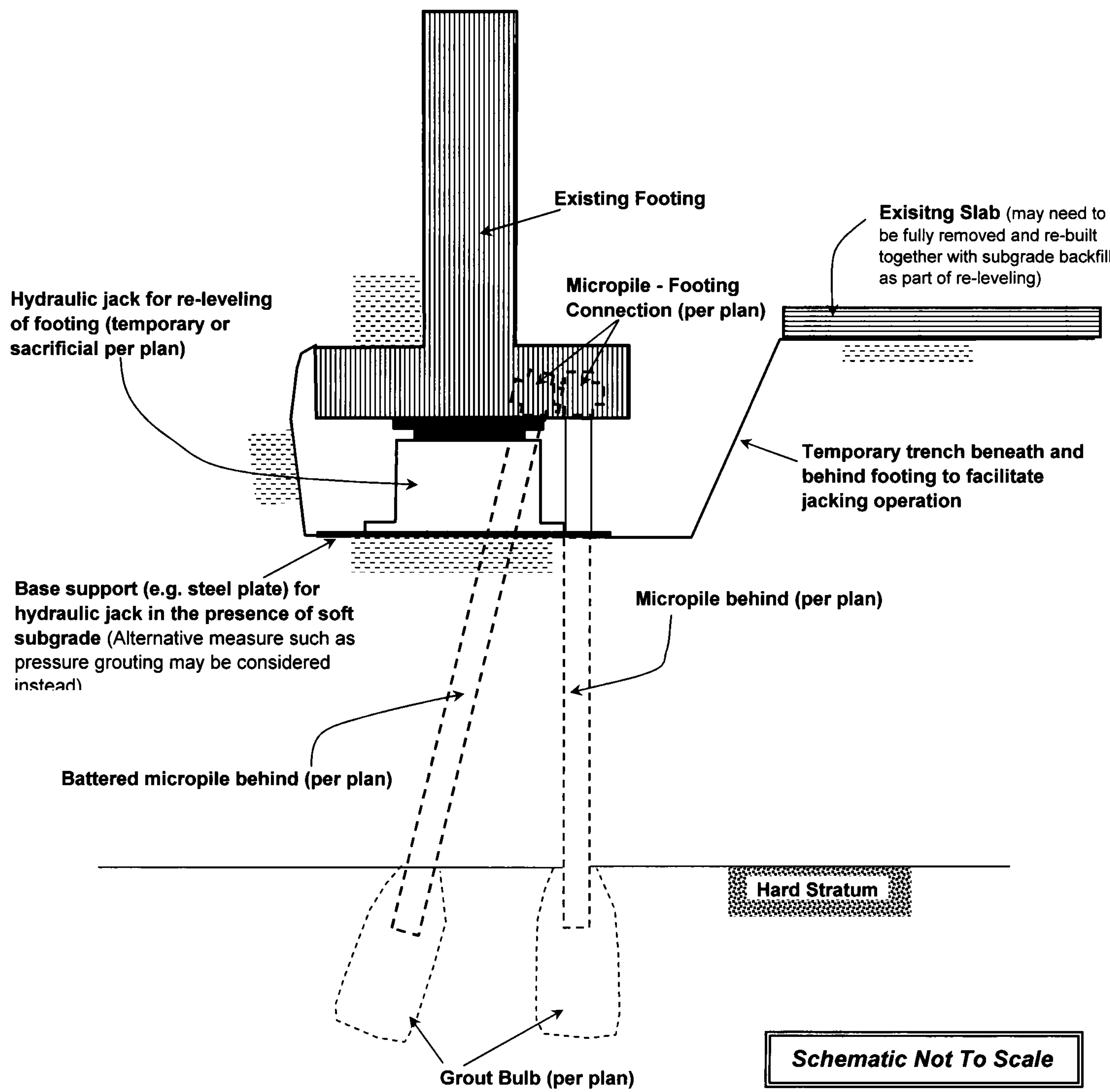
Considering the extent and limited access of the affected area of the Building, it is ASE's opinion that temporary underpinning and permanent re-leveling of existing footings with hydraulic jacks is deemed feasible. Depending on the structural loading and wall footing configuration, hydraulic jacks with sufficient lifting capacity should be placed at pre-determined center-to-center distances in pre-dug holes where sufficiently large space immediately below and beyond the existing footings is opened, as shown in Figure 7, Typical Arrangement of Footing Underpinning. Due to the expected presence of heterogeneous, loose debris fill below the footings, it might be necessary to support the hydraulic jacks with oversized steel plates to provide better bearing and reduce settlement upon jacking. If deemed necessary by the specialty contractor to control excessive subgrade settlement beneath the hydraulic jacks, subject to the review and approval of the Geotechnical Consultant, it might be prudent to apply localized pressure grouting to improve bearing capacity in subgrade layers anticipated to support the hydraulic jacks prior to the excavation for underpinning.

Once the hydraulic jacks are in place, the re-leveling of the Building should follow by jacking slowly in small increments in an orderly manner such that no undesirable excessive stress will be experienced by the Building that may otherwise lead to the development of cracks or distortion. The jacking operation may be terminated with hydraulic jacks locked in positions when the leveling survey indicates that the Building has been re-leveled to its original, or pre-determined, elevations. A routine, preferably daily, survey of the conditions of the hydraulic jacks and the leveling of the re-leveled Building should be conducted. Timely adjustment to the hydraulic jacks should be carried out if movement or settlement is detected.

Upon completion of installation of micropiling per plan, when the structural loading of the Building is being carried by the micropiles, as schematically shown in Figure 7, the hydraulic jacks can be sacrificed and left in place, or can be removed, followed by backfilling the openings with lean concrete or 2-sack control density fill.

For both interior and exterior isolated pad footings carrying heavy concentrated loading, it may not be structurally feasible to excavate openings directly underneath the footings as this could undermine the structural stability of the Building. Alternative measures such as installing hydraulic jacks in pre-dug holes on both sides of the footing and lifting the footing by a steel H-beam spanning intimately underneath the footing, supported by the hydraulic jacks, should be developed and implemented by the specialty contractor. The isolated pad footing should then be supported by micropiles installed through corings



Schematic Not To Scale

Associated Soils Engineering, Inc.	**Project:**	*Foundation Distress Inv. & Repair Recomm., Veteran's Memorial Park Recreation Bldg., 6364 Zindell Ave.,Commerce, CA*		
2860 Walnut Avenue **Signal Hill, CA 90755**	**Figure 7**	**Typical Arrangement of Footing Underpinning**		
Tel (562) 426-7990 Fax (562) 426-1842	**Proj. No.:**	**07-5972**	**Date:**	**March, 2007**

within the footing, or by a footing enlargement/pile cap supported by micropiles installed outside of the footing, as depicted in Figure 5.

7.2.2.4 Repair to Slabs:

For affected areas within the Building that are constructed with structural slabs, the underpinning and re-leveling of footings as discussed in the previous section is anticipated to restore the elevations of structural slabs to their pre-distress conditions as the slabs and the footings are structurally integrated. Any gaps or voids that might be present underneath the structural slabs subsequent to the re-leveling are not geotechnically anticipated to affect the performance of the structural slabs since no bearing of the structural slabs is derived from subgrade soils.

For affected areas within the Building that are constructed with slabs-on-grade, the following two mitigative measures may be considered for pertinent slab repair:

a. The affected slabs-on-grade may be cosmetically repaired by thickening the slabs with additional concrete overlay to pre-distress elevations, or by removing the distressed slabs and replaced with newly cast slabs. This measure, however, does not alleviate the slabs from future settlement and separation from the surrounding footings as the subgrade debris fill remains in place.

b. Replacing the distressed slabs-on-grade with newly designed structural slabs tied to the stabilized/restored footings. This measure is anticipated to eliminate the bearing of slabs on the settling debris fill and, therefore, to minimize future slab displacement. However, this measure is more costly and may require re-design or strengthening of existing footings, or installation of new footings and grade beams.

7.2.3 Other Alternatives:

As previously mentioned in Section 7.0, the subject foundation repair work should be performance-orientated. As such, other foundation distress repair alternatives, such as helical piers or ground modifications, may also be considered. Specialty contractors should be allowed to propose alternative repair measures by submitting complete design-build package, in addition to the main bid package initiated by the City. Information such as past successful records, valid references and follow-up monitored performance data regarding the proposed alternative repair measures should be submitted by the specialty contractor for the review and consideration of the City and the Geotechnical Consultant.

7.3 BACKFILLING AND COMPACTION REQUIREMENTS:

All debris fills excavated during foundation and slab repair should be disposed of off site. Any imported soil required for backfill should consist of predominantly granular material which exhibits an "E.I." less than 20 when tested in accordance with 2001 CBC 18-2 Expansion Test Procedures, and should be free of debris, particles greater than 4 inches in maximum dimension, organic matter or other deleterious materials. All potential import material must be approved by the Geotechnical Consultant or his representative, prior to its arrival on site.

Fill soils should be placed in horizontal lifts not exceeding 8 inches in loose thickness, moisture conditioned to within 3 percentage points above optimum moisture content and, unless otherwise specified, compacted to at least 90 percent relative compaction as per determined by ASTM D1557-02 test procedures. Existing site soils disturbed as a result of the foundation repair works should be scarified to a minimum depth of 12 inches, and moisture conditioned and re-compacted the same as the fill soils.

7.4 SOILS CORROSIVITY:

Soils corrosivity tests were performed by Quartech Consultants, Inc. (QCI Job No. 07-064-03a, dated March 2, 2007) on representative samples of site soil. These tests were meant to determine the corrosive potential of on-site soils against proposed concrete foundations and underground metal conduits. No corrosivity testing was performed on samples representing debris fill encountered on site, mainly due to the highly heterogeneous nature of the materials. However, specific sampling and testing may need to be implemented at locations where the presence of potentially corrosive debris fill is deemed imposing significant impact on the long-term performance of the installed foundation repair features.

7.4.1 Concrete Corrosion:

Disintegration of concrete may be attributed to the chemical reaction of soil sulfates and hydrated lime and calcium aluminate within the cement. The severity of the reaction resulting in expansion and disruption of the cement is primarily a function of the concentration of the soluble sulfates and the water-cement ratio of the concrete.

Soluble sulfate content around 0.026% by weight was obtained in on-site fill soils within 5 feet from existing surface grade. For site native soils encountered at greater depth, soluble sulfate contents ranging from 0.0065 to 0.013% by weight have been recorded. Per Table 19-A-4 of 2001 CBC, soils exhibiting soluble sulfate content less than 0.1% by weight are classified as having "Negligible" sulfate exposure. As such, for structural features to be in direct contact with on-site surficial fill soils and deep, native soils, the tested negligible soluble sulfate content indicates that there should be no special geotechnical restriction on the type of Portland cement or water-cement ratio to be used.

7.4.2 Metal Corrosion:

In the evaluation of soil corrosivity to metal, the hydrogen ion concentrates (pH) and the electrical resistivity of the site and backfill soils are the principal variables in determining the service life of ferrous metal conduit. The pH of soil and water is a measure of acidity or alkalinity, while the resistivity is a measure of the soils resistance to the flow of electrical current.

Currently available design charts indicate that corrosion rates decrease with increasing resistivities and increasing alkalinities. It can also be noted that for alkaline soils, the corrosion rate is more influenced by resistivity than by pH.

A resistivity value of 1165 ohm-cm as well as a corresponding pH-value of 7.71, classifies the on-site surficial fill soils tested to be severely corrosive to buried ferrous metals. Based on California Test 643, the year to perforation for 18-gauge steel in contact with on-site surficial fill soils of similar resistivity and pH-value is approximately <u>28</u> years for the corrosive soils. For on-site, native soils at greater depth, the tested resistivity values between 1260 and 1500 ohm-cm, together with the corresponding pH-values between 8.46 and 8.7, indicated that the native soils are also severely corrosive to buried ferrous metals. Based on California Test 643, the year to perforation for 18-gauge steel in contact with on-site native soils at greater depth is approximately <u>28</u> years. In lieu of additional testing, alternative piping materials, i.e. plastic piping, may be used instead of metal if longer service life is desired or required. This low resistivity value of on-site soils may also have implications to other building materials and depths of embedment for steel reinforcement etc. It is recommended that a qualified corrosion consultant be engaged to review the building plans.

Soluble chloride contents ranging from 38 to 61 ppm recorded in ASE's limited laboratory tests on both on-site surficial fill soils and native soils at greater depth are considered low to the threshold values of 100 and 200 ppm per Federal Highway Administration Standards (FHWA), 2002 and Caltrans Standards, 1999, respectively. Therefore, no special measure in terms of rebar protection against chloride corrosion is recommended herein as a result of the low soluble chloride content tested.

7.5 UTILITY TRENCHES:

If constructed or re-routed as part of the foundation distress repair, all utility trenches should be backfilled with approved fill material compacted to relative compaction of not less than 90 percent. Care should be taken during backfilling to prevent utility line damage. Should more stringent relative compaction be required by the grading code of the local authority, the later should govern.

The on-site surficial fill soils may be used for backfilling utility trenches from one foot above the top of pipe to the surface, provided the material is free of organic matter and deleterious substances. No on-site debris fill should be allowed for utility trench backfill. Any soft and/or loose materials or fill encountered at pipe invert should be removed and replaced with properly compacted fill or adequate bedding material.

Bedding materials should consist of sand with a Sand Equivalent ("SE", per California Test Method 217) value not less than 30. On-site soils are not deemed suitable for bedding or shading of utilities. Imported soils for pipe bedding should consist of non-expansive granular soils.

The walls of temporary construction trenches may not be stable when excavated nearly vertical due to potential for caving. Shoring of excavation walls or flattening of slopes will be required for temporary excavations deeper than 4 feet.

Trenches should be located so as not to impair the bearing capacity of soils or cause settlement under foundations. As a guide, trenches parallel to foundations should be clear of a 45-degree plane extending outward and downward from the edge of the foundations.

All work associated with trenches, excavations and shoring must conform to the State of California Safety Code.

8.0 PLAN REVIEW, OBSERVATIONS AND TESTING

All foundation excavations and repairs should be observed by a representative of the Geotechnical Consultant to verify compliance with approved repair plans and specifications, to record the actual production quality delivered by the Contractor, to evaluate the pertinency of exposed subgrade soils conditions in relation to the information presented in the Soils Report, and to timely capture soils or foundation conditions that should be brought to the immediate attention of the Geotechnical Consultant. In addition, grading and fill compaction should be performed under the observation of and testing by a Geotechnical Consultant or his representative.

Upon the completion of foundation repair plans and specifications, they should be forwarded to the Geotechnical Consultant for review of conformance with the intent of the data and recommendations presented.

9.0 CLOSURE

This report has been prepared for the exclusive use of the City of Commerce and its authorized consultants/contractors. No portion of this report may be used by other parties or for other purposes.

The Owner or his representatives are responsible for ensuring the information and recommendations contained in this report are brought to the attention of the project engineers and architects, incorporated into the project plans, and implemented by project contractors. This report should be named on project plans as a part of the project specifications.

The findings contained in this report are based upon our evaluation and interpretation of the information obtained from the limited number of test borings and the results of laboratory testing and engineering analysis. As part of the engineering analysis it has been assumed, and is expected, that the geotechnical conditions existing across the area of study are similar to those encountered in the test excavations. However, no warranty is expressed or implied as to the conditions at locations or depths other than those excavated. Should conditions encountered during construction differ significantly from those described in this report, this office should be contacted immediately for recommendations prior to continuation of work.

Our findings and recommendations were obtained in accordance with generally accepted current professional principles and local practice in geotechnical engineering and reflect our best professional judgment. We make no other warranty, either express or implied.

Geotechnical observations and testing should be provided on a continuous basis during foundation distress repair at the site to confirm preliminary design assumptions and to verify conformance with the intent of our recommendations. If parties other than Associated Soils Engineering, Inc. are engaged to provide geotechnical services during construction, Associated Soils Engineering, Inc. will assume without reservation that the newly engaged party is to fully assume complete responsibility for the geotechnical phase of the project by either concurring with the recommendations in this report or providing alternative recommendations.

This concludes our scope of services as indicated in our proposal dated September 21, 2006, however, our report is subject to review by the controlling authorities for the project. Any further geotechnical services that may be required of our office to respond to questions/comments of the controlling authorities after their review of the report will be performed on a time-and-expense basis as per our current fee schedule. We would not proceed with any response to report review comments/questions without authorization from your office.

APPENDIX A SITE EXPLORATION

Field exploration was performed by drilling three (3) exploratory borings at the approximate locations indicted on the attached Boring Location Map (Plate 1). Exploratory borings were drilled by Choice Drilling, Inc., utilizing a truck mounted, rotary drilling rig equipped with 8-inch diameter continuous flight, hollow-stem rotary augers, and a soil bit. The borings extended to depths ranging from 25 feet to 50 feet 6 inches below ground surface (bgs).

Continuous observations of the materials encountered in the borings were recorded in the field. The soils were classified in the field by visual and textural examination and these classifications were supplemented by obtaining bulk soil samples for future examination in the laboratory. Relatively undisturbed samples of soils were extracted in a Modified California cutting shoe. Additional samples were obtained in a Standard Penetration Test (SPT) sampler and in moisture-resistant bags to minimize the loss of field moisture, followed by transporting to ASE's laboratory for ensuing testing.

Upon completion of exploration, the borings were loosely backfilled with the excavated materials.

Description of the soils encountered, depth of samples, field density and field moisture content of tested samples, SPT and Modified California sampler blow counts, as well as the respective laboratory tests performed are presented in the following Field Logs of Boring, Plates A-1 through A-7.

FIELD LOG OF BORING B-1

ASSOCIATED SOILS ENGINEERING, INC

Project:	*Distress Evaluation-Veterans Memorial Park Rec. Building*
Location:	*6364 Zindell Ave.,Commerce*
Project No.	*07-5972*

Dates(s) Drilled:	**2/26/07**	Logged By:	**Gary Martin**
Drilled By:	**Choice Drilling,Inc.**	Total Depth:	**35 Feet**
Rig Make/Model:	**Mobile B-61**	Hammer Type:	**Wireline Downhole**
Drilling Method:	**Hollow-stem Auger**	Hammer Weight/Drop:	**140 Lb./±30 In.**
Hole Diameter:	**8 Inches**	Surface Elevation:	**Unknown**

Comments: Groundwater not encountered. Backfill not determined.

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: BULK	SAMPLE INTERVALS: DRIVE	TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0					CL	SILTY CLAY: FILL-Dark brown,very moist,trace fine sand				MAX,EXPANSIO CORROSIVITY TESTS
						CL	SILTY CLAY: FILL-Very dark grayish brown with dark grayish brown,moist,trace fine sand and gravel,with paper and rubber pieces				
			■	17(Ring)				104.1	16.3		
							with pastic pipe piece				
5	5		■	14(Ring)		CL	same as above,with dark gray lens,with asphalt piece,with lens Gray Fine to Medium Sand (SP)	103.0	18.3		
							with brick pieces				
						Tr	DEBRIS FILL: FILL-Very dark gray to black,very moist,organic trash including wood pieces,with some Clayey Fine to Coarse Sand (SC) matrix				
10	10		■	28(Ring)			#unable to retrieve relatively undisturbed sample	#	29.0		
15	15		■	33(Ring)		Tr	same as above,becoming black,with mostly wood pieces	#	44.9		
							#unable to retrieve relatively undisturbed sample				
							with wire piece				

SOILS ENGINEERING, INC

FIELD LOG OF BORING B - 1

Project: ***Distress Evaluation-Veterans Memorial Park Rec. Building***

Location: ***6364 Zindell Ave.,Commerce*** | Project No. ***07-5972***

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: BULK	DRIVE	TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
						Tr	DEBRIS FILL: FILL-Black,Very moist,organic trash including mostly wood pieces,with some wire pieces,with Dark Gray with Olive Gray Silty Clay with trace Fine Sand (CL) matrix				
20	20			6(Ring)		Tr	mostly wood pieces #unable to retrieve relatively undisturbed sample	#	25.0		
25	25			50/6" (Ring)		ML SM	SANDY SILT WITH CLAY: NATIVE-Gray,very moist to wet,fine-grained sand,with small root	96.7 117.1	27.0 10.8		
							SILTY SAND WITH GRAVEL: Gray,moist,fine to coarse-grained sand,trace clay				
30	30			52/6" (Ring)		SP	SAND: Gray to light gray,damp,fine to coarse-grained sand +no recovery	+			
				20/6" (Ring)		SP	same as above *insufficient sample for density	*	3.5		
				21/6" (Ring)		SP	SAND: Light gray,damp,fine-grained sand	95.3	6.4		
35	35					ML	SILT WITH CLAY: Gray,very moist	96.0	27.0		

FIELD LOG OF BORING B-2

ASSOCIATED SOILS ENGINEERING, INC

Project:	*Distress Evaluation-Veterans Memorial Park Rec. Building*
Location:	*6364 Zindell Ave., Commerce*
Project No.	*07-5972*

Dates(s) Drilled:	**2/26/07**	Logged By:	**Gary Martin**
Drilled By:	**Choice Drilling, Inc.**	Total Depth:	**50 feet 6 Inches**
Rig Make/Model:	**Mobile B-61**	Hammer Type:	**Automatic**
Drilling Method:	**Hollow-stem Auger**	Hammer Weight/Drop:	**140 Lb./±30 In.**
Hole Diameter:	**8 Inches**	Surface Elevation:	**Unknown**

Comments: Groundwater not encountered. Backfill not determined.

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0		CL	SILTY CLAY: FILL-Very dark grayish brown, very moist, trace fine sand				
		22(Ring)	CL	SILTY CLAY WITH SAND: FILL-Dark brown with gray, moist to very moist, fine-grained sand, with lens Olive Gray Clayey Sand (SC)	104.5	19.8		
5	5	38(Ring)	CL	SANDY CLAY: FILL-Very dark grayish brown with olive, moist, fine-grained sand, trace gravel, with wood pieces, organic odor	113.5	16.3		
		7(SPT)	CL	SILTY CLAY WITH SAND: FILL-Dark gray, very moist, fine-grained sand, with brick, wood and glass pieces, organic odor		41.8 41.0		
			ML	CLAYEY SILT WITH SAND: FILL-Black with dark brown and white, very moist to wet, with rubber piece and organics, with 4-inch thick layer white chalk-like material				
10	10	17/6" (Ring) 20/6" (Ring)	CL	SILTY CLAY: FILL-Very dark gray with brown and white, very moist, trace fine sand, with rubber pieces, and clumps of white chalk-like material #unable to retrieve relatively undisturbed sample	# #	30.7 13.0		
		28(Ring)	ML	CLAYEY SILT: FILL-Olive brown, moist, with lots of wood and rubber-like pieces #unable to retrieve relatively undisturbed sample	#	35.1		
15	15		ML	CLAYEY SILT WITH SAND: FILL-Very dark grayish brown, wet, fine-grained sand, with organic trash, plastic and rubber pieces #unable to retrieve relatively undisturbed sample				

FIELD LOG OF BORING B - 2

ASSOCIATED SOILS ENGINEERING, INC

Project: ***Distress Evaluation-Veterans Memorial Park Rec. Building***

Location: ***6364 Zindell Ave.,Commerce*** | Project No. ***07-5972***

DEPTH (Ft.)	ELEVATION (MSL)	TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
15	15	13(SPT)	Tr	DEBRIS FILL: FILL-Black,wet,organic trash including wood,wire,metaland rubber pieces +no recovery		+		
		50/6" (Ring)	Tr	#unable to retrieve relatively undisturbed sample	#	58.0		
20	20	14(SPT)	Tr	@no moisture obtained,too much trash		@		
		50/3" (Ring)	Tr	#unable to retrieve relatively undisturbed sample	#	32.8		
25	25	16(SPT)	Tr	@no moisture obtained,sample mostly wood		@		
		21/6" (Ring)	ML	SANDY SILT WITH CLAY: NATIVE-Gray,very moist,fine-grained sand	95.1	28.2		
		50/4" (Ring)	SM	SILTY SAND WITH GRAVEL: Gray,moist,fine to coarse-grained sand,with lens Clayey Silt *insufficient sample for density	*	13.0		
30	30	8/6"(SPT)	ML	SANDY SILT WITH CLAY: Dark gray,very moist to wet,fine-grained sand		31.6		
		12/6" (SPT)	SP	SAND: Light olive gray,damp,fine to medium-grained sand		3.1		
			ML	CLAYEY SILT: Olive,moist,trace very fine sand				
		50/6" (Ring)	SP	SAND: Light gray,damp,fine-grained sand	97.0	5.8		SHEAR
35	35							

SOILS ENGINEERING, INC

FIELD LOG OF BORING B - 2

Project: ***Distress Evaluation-Veterans Memorial Park Rec. Building***

Location: ***6364 Zindell Ave.,Commerce*** | Project No. ***07-5972***

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
35	35	$	SP	SAND: Gray,damp,fine to medium-grained sand $moisture sample from upper 6" of SPT		4.2		
		7/6"(SPT)	CL	SILTY CLAY: Olive gray,wet		37.0		
		10/6" (SPT)	ML	CLAYEY SILT: Light olive brown with gray,very moist		26.6		
		49(Ring)	ML	SILT WITH CLAY: Olive with light olive brown,moist,trace very fine sand	101.5	24.2		CONSOL,SHEAR
40	40	19(SPT)	ML	SANDY SILT: Olive gray with light olive brown,moist to very moist,fine-grained sand		22.7		
		21/6" (Ring)	ML	same as above	98.4	17.8		CONSOL CORROSIVITY TESTS
		36/6" (Ring)	ML	SANDY SILT WITH CLAY: Light olive brown,very moist,fine-grained sand	102.4	25.0		
45	45	37(SPT)	SP	SAND: Light yellowish brown,damp,fine to medium-grained sand		4.2		CORROSIVITY TESTS
		75/6" (Ring)	SP	same as above	100.8	4.2		CONSOL,SHEAR
50	50	54(SPT)	SP	same as above		5.3		CORROSIVITY TESTS

ASSOCIATED SOILS ENGINEERING, INC

FIELD LOG OF BORING B-3

Project:	***Distress Evaluation-Veterans Memorial Park Rec. Building***
Location:	***6364 Zindell Ave.,Commerce***
Project No.	***07-5972***

Dates(s) Drilled:	**2/26/07**	Logged By:	**Gary Martin**
Drilled By:	**Choice Drilling,Inc.**	Total Depth:	**25 Feet**
Rig Make/Model:	**Mobile B-61**	Hammer Type:	**Wireline Downhole**
Drilling Method:	**Hollow-stem Auger**	Hammer Weight/Drop:	**140 Lb./±30 In.**
Hole Diameter:	**8 Inches**	Surface Elevation:	**Unknown**

Comments: Groundwater not encountered. Backfill not determined.

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0		ML	SANDY SILT WITH CLAY: FILL-Dark gray,moist,fine-grained sand,with organic odor				
		15/6" (Ring)	ML	becoming very dark grayish brown,clay content decreases	95.9	14.6		
		18/4" (Ring)	ML	CLAYEY SILT WITH SAND: FILL-Very dark gray,trace black,moist,fine-grained sand,with rubber pieces,organic odor	106.8	15.8		
				with gravel layer at 4 feet				
5	5	33/5" (Ring)	ML	SANDY SILT: FILL-Black to very dark gray,moist,fine-grained sand,trace clay,with organic odor,with brick and porcelain pieces	102.8	17.0		
				with abundant gravel below 5.5 feet				
				with wood pieces				
10	10	45(Ring)	SM	SILTY SAND WITH GRAVEL: FILL-Dark gra to gray,moist,fine to medium-grained sand,with rock particles,with lens Very Dark Grayish Brown Silty Clay (CL)	98.1	9.3		
				no gravel/rock below 11 feet				
			ML	SANDY SILT: NATIVE-Gray,moist to very moist,fine-grained sand,with lens Fine Sand (SP)				
15	15	23(Ring)			89.5	22.7		

FIELD LOG OF BORING B - 3

ASSOCIATED SOILS ENGINEERING, INC

Project: ***Distress Evaluation-Veterans Memorial Park Rec. Building***

Location: ***6364 Zindell Ave.,Commerce*** | Project No. ***07-5972***

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: BULK	DRIVE	TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
						ML	SANDY SILT: NATIVE-Gray,moist to very moist,fine-grained sand,with lens Fine Sand (SP)				
20	20			11/6" (Ring)		SP	SAND: Light gray,damp,fine to medium-grained sand	96.6	3.1		
				20/6" (Ring)		SP	SAND: Light gray,damp,fine-grained sand	96.2	4.5		
				34(Ring)		SP	same as above,becoming gray	96.2	7.0		
25	25										

APPENDIX B LABORATORY TESTING

After samples were visually classified in the laboratory, a testing program aiming at providing sufficient data for the ensuing engineering evaluation and analysis was established, which consisted of:

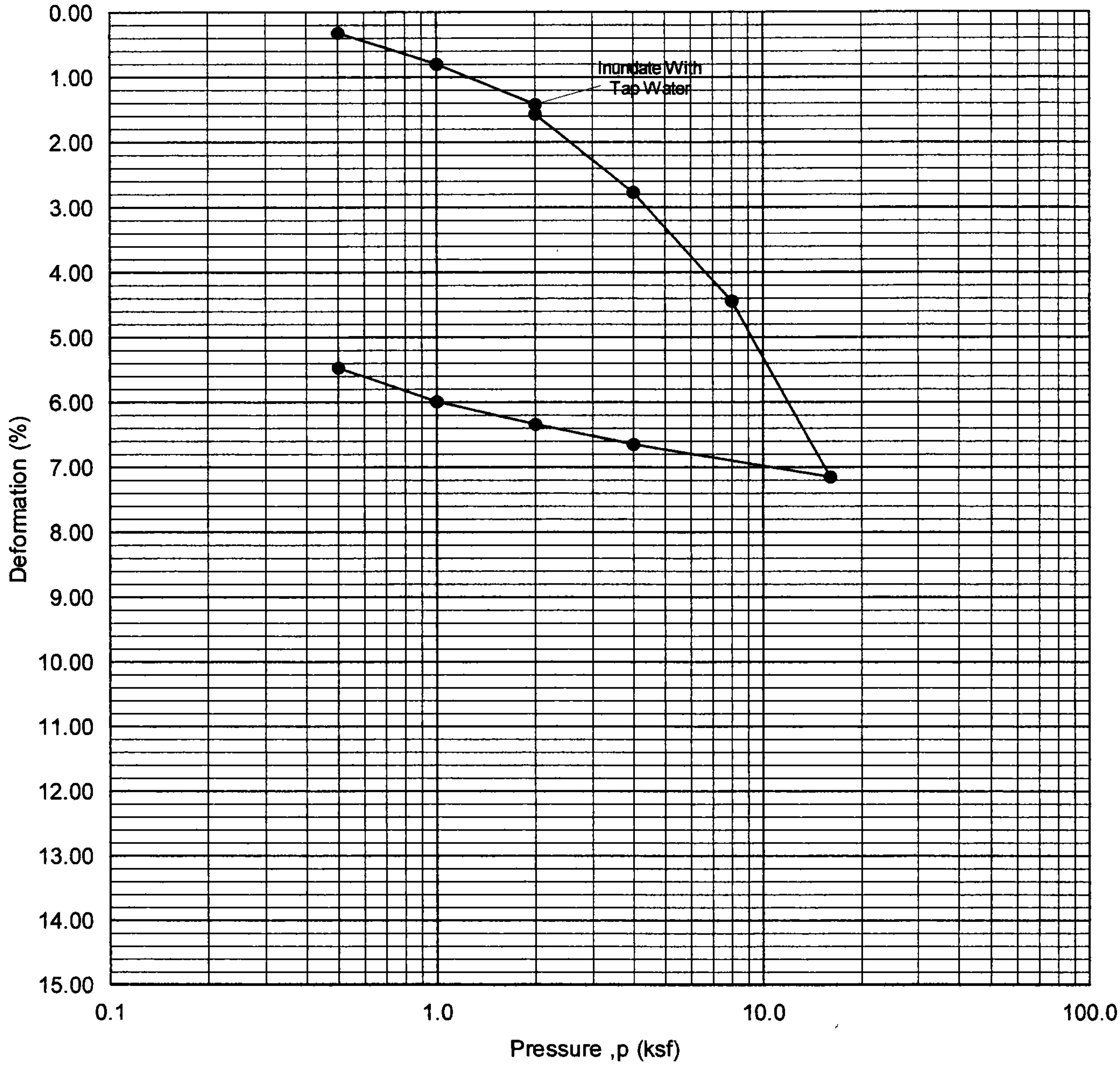
- **Moisture Content and Density Tests:**
  Relatively undisturbed soil samples retained within the Modified California barrel sampler were tested in the laboratory to determine their respective in-place dry densities and moisture contents. The results are presented on the respective Field Logs of Boring, Plates B-1 through B-3.

- **Uni-axial Consolidation and Swelling Tests:**
  Consolidation tests were performed on selected relatively undisturbed and remolded soil samples in general accordance with the latest version of ASTM D2435. Two of the samples were inundated during testing to represent adverse field conditions. The percent consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are presented on Plates C-1 through C-3.

- **Direct Shear Tests:**
  In order to determine shear strength parameters of representative soil samples, direct shear tests were performed on both relatively undisturbed and remolded ring samples in accordance with ASTM D 3080. The test results are presented on Plates D-1 through D-3.

- **Maximum Dry Density and Optimum Moisture Content Test:**
  The following maximum density test was conducted in accordance with ASTM D1557-01, Method A, using 5 equal layers, 25 blows each layer, 10-pound hammer, 18 inch drop in a 1/30 cubic foot mold. The results are as follows:

Boring No.	Depth (ft)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Material Classification
B-1	0 ~ 5	125.5	10.5	SM

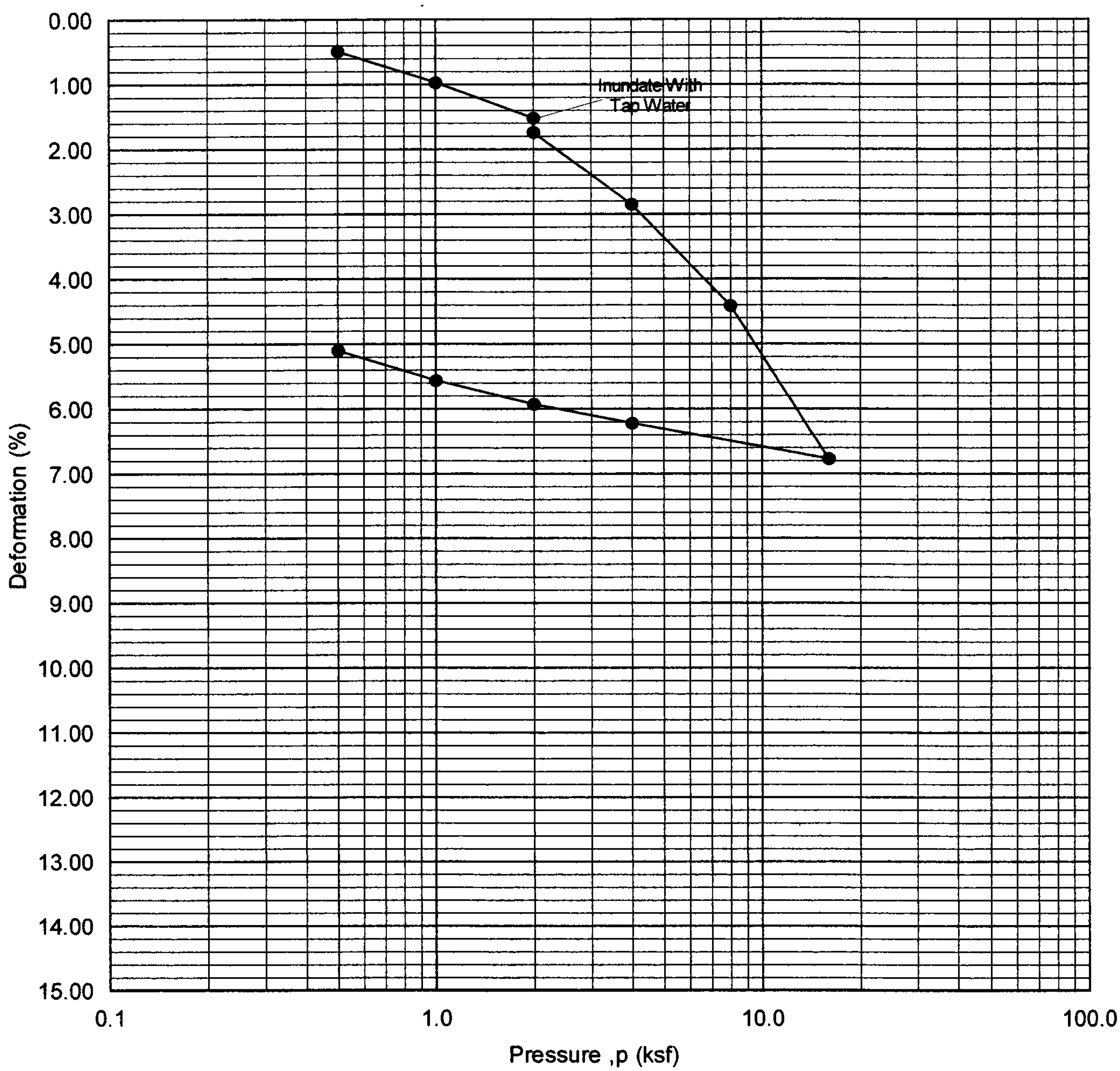


Boring No. : B-2
Depth (ft.) : 38.0
Sample Type: Silt with Clay

Dry Density (pcf) = 101.5
Moisture (%) = 24.2

Project Name: Distress Evaluation-Veterans Memorial Park Rec. Bldg.	Project No.: 07-5972
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-1

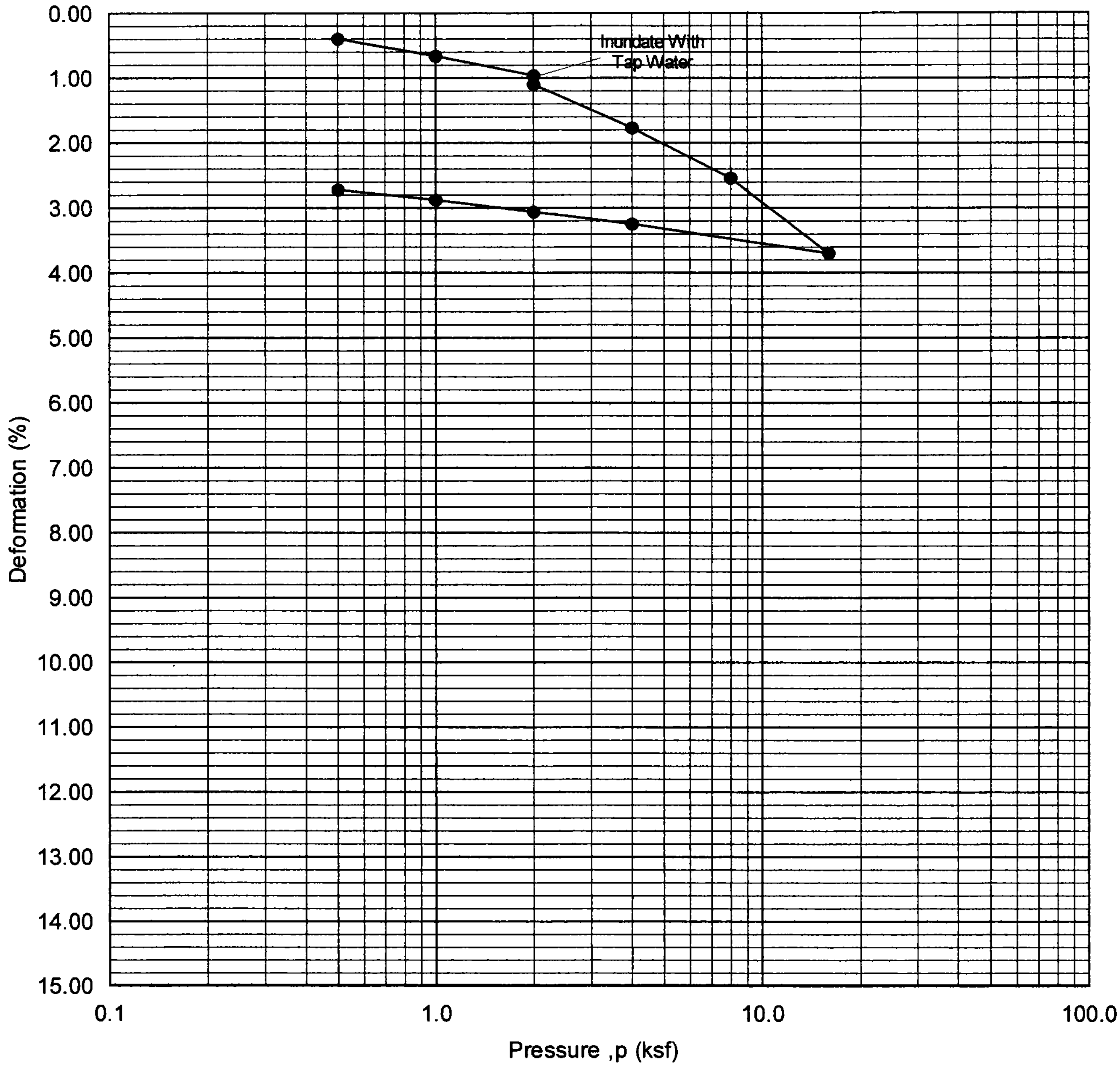


Boring No. : B-2
Depth (ft.) : 43.0
Sample Type: Fine Sandy Silt

Dry Density (pcf) = 98.4
Moisture (%) = 17.8

Project Name: Distress Evaluation-Veterans Memorial Park Rec. Bldg.	Project No.: 07-5972
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-2

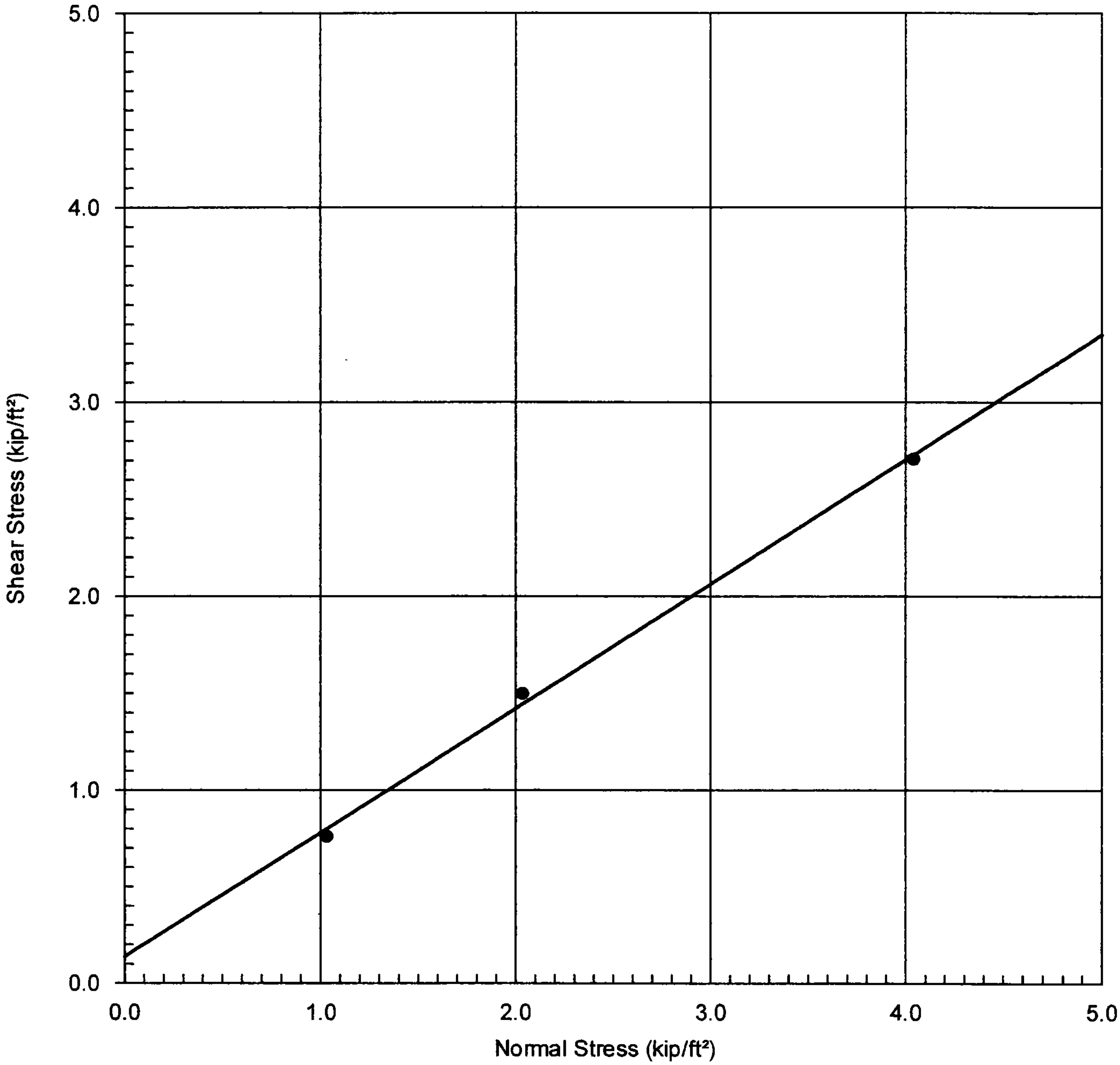


Boring No. : B-2
Depth (ft.) : 48.0
Sample Type: Fine to Medium Sand

Dry Density (pcf) = 100.8
Moisture (%) = 4.2

Project Name:Distress Evaluation-Veterans Memorial Park Rec. Bldg.	Project No.: 07-5972
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-3

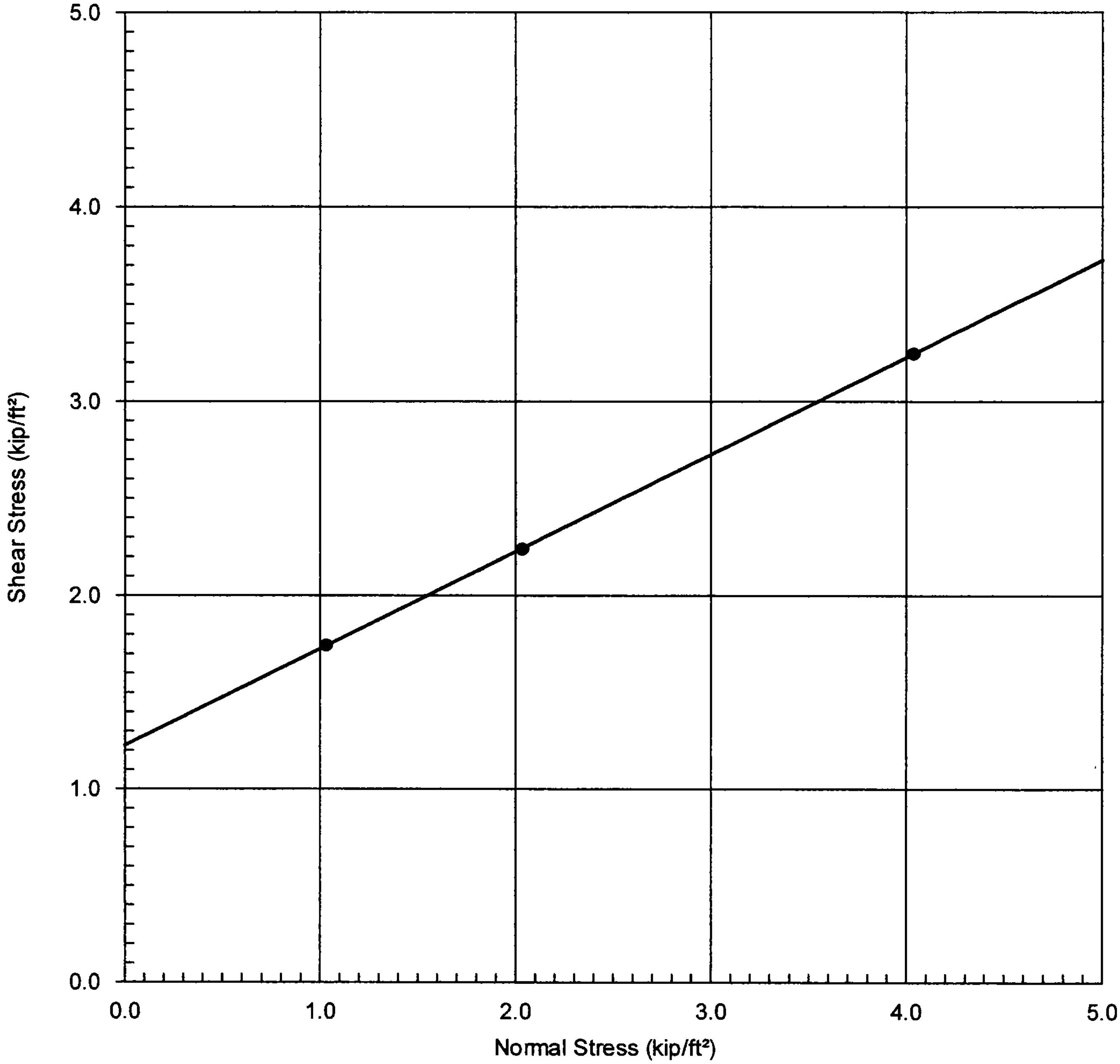


Boring No. : B-2
Depth (ft.) : 33.0
Sample : Relatively Undisturbed
Sample Type : Fine Sand

Cohesion(C) = 135 psf
Friction (ϕ) = 32.5°
Dry Density (pcf) = 97.0
Moisture (%) = 5.8

Project Name: Distress Evaluation-Veterans Memorial Park Rec. Bldg.	Project No.: 07-5972
ASSOCIATED SOILS ENGINEERING, INC.	DIRECT SHEAR TEST RESULTS (ASTM D 3080)

PLATE D-1

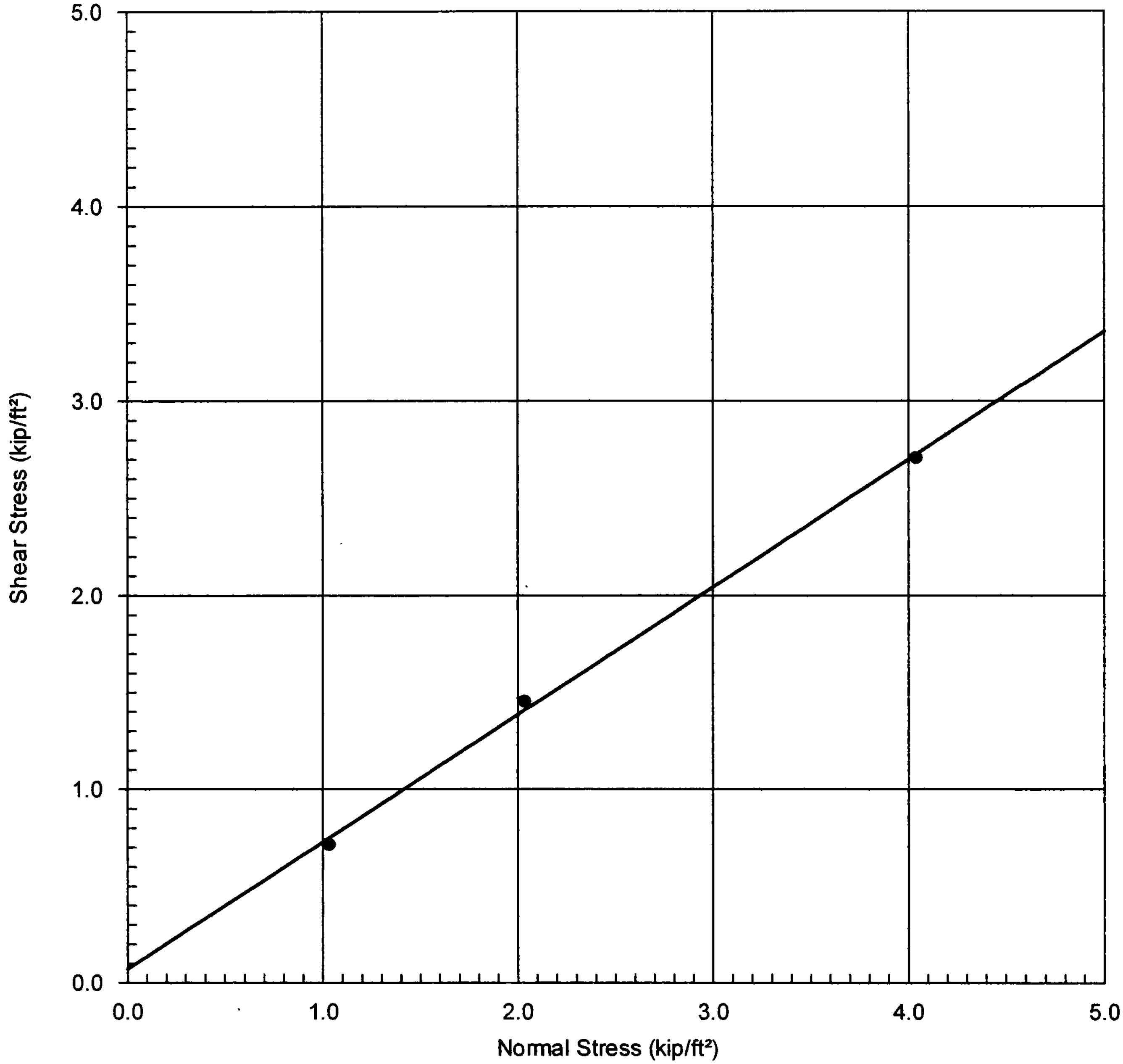


Boring No. : B-2
Depth (ft.) : 38.0
Sample : Relatively Undisturbed
Sample Type : Silt with Clay

Cohesion(C) = 1225 psf
Friction (ϕ) = 26.5°
Dry Density (pcf) = 101.5
Moisture (%) = 24.2

Project Name: Distress Evaluation-Veterans Memorial Park Rec. Bldg.	Project No.: 07-5972
ASSOCIATED SOILS ENGINEERING, INC.	DIRECT SHEAR TEST RESULTS (ASTM D 3080)

PLATE D-2



Boring No. : B-2
Depth (ft.) : 48.0
Sample : Relatively Undisturbed
Sample Type : Fine to Medium Sand

Cohesion(C) = 70 psf
Friction (ϕ) = 33°
Dry Density (pcf) = 100.8
Moisture (%) = 4.2

Project Name: Distress Evaluation-Veterans Memorial Park Rec. Bldg.	Project No.: 07-5972
ASSOCIATED SOILS ENGINEERING, INC.	DIRECT SHEAR TEST RESULTS (ASTM D 3080)

PLATE D-3

Cal Land Engineering, Inc.
dba Quartech Consultants
Geotechnical, Environmental, and Civil Engineering

Client Name: Associated Soils Engineering, Inc.
Project Name: Distress Investigation
Project No.: ASE 07-5972
For: The City of Commerce
Address: Veteran's Memorial Park Rec. Bldg., 6364 Zindell Ave., City of Commerce, CA

QCI Project No.:07-064-03a
Date: March 2, 2007
Summarized by: ABK

Sample ID	Sample Depth	pH CT-532 (643)	Chloride CT-422 (ppm)	Sulfate CT-417 (% By Weight)	Resistivity CT-532 (643) (ohm-cm)
B-1	0-5'	7.71	43	0.0260	1,165
B-2	43'	8.70	61	0.0130	1,260
B-2	45' &50' Combined	8.46	38	0.0065	1,500

576 East Lambert Road, Brea, California 92821; Tel: 714-671-1050; Fax: 714-671-1090

APPENDIX B LABORATORY TESTING - continued

- **Expansion Test:**
  An expansion test was performed on a soil sample to determine the swell characteristics. The expansion test was conducted in accordance with a modification of the California Building Code (2001 Edition) Standard No. 18-2, Expansion Index Test. The expansion sample was remolded to approximately 90 percent relative compaction at near optimum moisture content, subjected to 144 pounds per square foot surcharge load and saturated. The test results are tabulated below:

Sample Location	Max. Dry Density (pcf)	Optimum Moisture Content (%)	Molded Dry Density (pcf)	Molded Moisture Content (%)	Percent Saturation (%)
B-1 @ 0 ~ 5'	125.5	10.5	113.9	10.1	57.4
	Expansion Index			**Expansion Classification**	
	34			***Low***	

- **Soil Corrosivity Tests:**
  Tests of soluble sulfate and chloride contents were performed in accordance with California Test Methods 417 and 422, respectively, by Cal Land Engineering, Inc. (QCI Job No.: 07-064-03a, dated 3/2/07) to assess the degree of corrosivity of the subgrade soils with regard to concrete and normal grade steel. Resistivity and pH-value tests were performed in accordance with California Test Method 643 to assess the degree of corrosivity of the subgrade soils with regard to ferrous metal piping. The test results are shown below.

Sample Location	Sulfate Content[1] Degree of Exposure	Chloride Content[2] Degree of Corrosivity	Resistivity[3] Degree of Corrosivity	pH-Value[3]
B-1 @ 0 ~ 5'	0.026% negligible	43 ppm non-corrosive	1165 ohm-cm corrosive	7.71
B-2 @ 43'	0.013% negligible	61 ppm non-corrosive	1260 ohm-cm corrosive	8.70
B-2 @ 45' & 50' combined	0.0065% negligible	38 ppm non-corrosive	1500 ohm-cm corrosive	8.46

1 California Test Method 417
2 California Test Method 422
3 California Test Method 643

APPENDIX C

SEISMICITY DATA

PROBABILISTIC SEISMIC ASSESSMENT UTILIZING CGS's ANALYSIS

Project Site Coordinates: *Longitude* - **W 118.1273^0**

Latitude - **N 33.9722^0**

Project Site Soil Classification: **Alluvium**

TABLE OF DESIGN GROUND MOTIONS

Soil Type / Design Acceleration (G)	*Firm Rock* (1)	*Soft Rock* (2)	*Alluvium* (2)
PGA(3)	0.409	0.409	0.446
S_a *(0.2 second)*(4)	0.981	0.986	1.084
S_a *(1.0 second)*(4)	0.351	0.434	0.524

(1) Classified by NEHRP (FEMA, 1997) as rocks having a shear wave velocity no less than 760 meters per second.

(2) Modification factors from PGA reflecting local site soils conditions are per NEHRP (FEMA, 1997), which are ground acceleration-dependent.

(3) Per Cao et al. (2003), it is defined as the peak ground acceleration for the subject site that carries a 10% probability of being exceeded in 50 years.

(4) Spectra acceleration derived from respective PGA with a 5% damping ratio incorporated.

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*                     *
*   E Q F A U L T     *
*                     *
*   Version 3.00      *
*                     *
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 07-5972

DATE: 03-08-2007

JOB NAME: Distress Evaluation-Veterans Memorial Park Recreation Building
6364 Zindell Avenue,City of Commerce
CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: C:\Program Files\EQFAULT1\Cgsflte.dat

SITE COORDINATES:
SITE LATITUDE: 33.9722
SITE LONGITUDE: 118.1273

SEARCH RADIUS: 62 mi

ATTENUATION RELATION: 20) Sadigh et al. (1997) Horiz. - Soil
UNCERTAINTY (M=Median, S=Sigma): M   Number of Sigmas: 0.0
DISTANCE MEASURE: clodis
SCOND: 0
Basement Depth: 5.00 km   Campbell SSR: Campbell SHR:
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\Program Files\EQFAULT1\Cgsflte.dat

MINIMUM DEPTH VALUE (km): 0.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
PUENTE HILLS BLIND THRUST	3.4(5.5)	7.1	0.535	X
WHITTIER	6.3(10.2)	6.8	0.298	IX
UPPER ELYSIAN PARK BLIND THRUST	6.4(10.3)	6.4	0.320	IX
NEWPORT-INGLEWOOD (L.A.Basin)	9.6(15.5)	7.1	0.257	IX
RAYMOND	10.1(16.2)	6.5	0.247	IX
VERDUGO	11.1(17.9)	6.9	0.274	IX
HOLLYWOOD	11.7(18.9)	6.4	0.201	VIII
SIERRA MADRE	14.5(23.3)	7.2	0.256	IX
SAN JOSE	14.9(24.0)	6.4	0.160	VIII
CLAMSHELL-SAWPIT	16.0(25.7)	6.5	0.162	VIII
PALOS VERDES	17.0(27.4)	7.3	0.184	VIII
SANTA MONICA	17.8(28.7)	6.6	0.153	VIII
CHINO-CENTRAL AVE. (Elsinore)	20.6(33.1)	6.7	0.140	VIII
SAN JOAQUIN HILLS	22.2(35.7)	6.6	0.121	VII
SIERRA MADRE (San Fernando)	23.2(37.3)	6.7	0.123	VII
MALIBU COAST	23.6(38.0)	6.7	0.121	VII
CUCAMONGA	25.2(40.5)	6.9	0.128	VIII
SAN GABRIEL	25.4(40.9)	7.2	0.119	VII
NORTHRIDGE (E. Oak Ridge)	26.0(41.8)	7.0	0.132	VIII
NEWPORT-INGLEWOOD (Offshore)	29.0(46.7)	7.1	0.097	VII
ELSINORE (GLEN IVY)	29.3(47.1)	6.8	0.079	VII
SANTA SUSANA	32.2(51.8)	6.7	0.084	VII
ANACAPA-DUME	32.5(52.3)	7.5	0.142	VIII
SAN ANDREAS - 1857 Rupture M-2a	36.3(58.4)	7.8	0.119	VII
SAN ANDREAS - Cho-Moj M-1b-1	36.3(58.4)	7.8	0.119	VII
SAN ANDREAS - Mojave M-1c-3	36.3(58.4)	7.4	0.092	VII
SAN ANDREAS - Whole M-1a	36.3(58.4)	8.0	0.134	VIII
HOLSER	38.2(61.5)	6.5	0.058	VI
SIMI-SANTA ROSA	39.5(63.5)	7.0	0.081	VII
SAN JACINTO-SAN BERNARDINO	40.2(64.7)	6.7	0.049	VI
SAN ANDREAS - SB-Coach. M-2b	42.2(67.9)	7.7	0.095	VII
SAN ANDREAS - San Bernardino M-1	42.2(67.9)	7.5	0.083	VII
SAN ANDREAS - SB-Coach. M-1b-2	42.2(67.9)	7.7	0.095	VII
OAK RIDGE (Onshore)	43.8(70.5)	7.0	0.071	VI
CLEGHORN	44.6(71.8)	6.5	0.037	V
SAN CAYETANO	48.5(78.0)	7.0	0.062	VI
CORONADO BANK	50.0(80.4)	7.6	0.073	VII
ELSINORE (TEMECULA)	50.1(80.6)	6.8	0.040	V
SAN JACINTO-SAN JACINTO VALLEY	51.3(82.6)	6.9	0.042	VI
NORTH FRONTAL FAULT ZONE (West)	54.6(87.9)	7.2	0.062	VI

DETERMINISTIC SITE PARAMETERS

Page 2

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
SAN ANDREAS - Carrizo M-1c-2	54.7(88.1)	7.4	0.056	VI
SANTA YNEZ (East)	60.9(98.0)	7.1	0.039	V

****

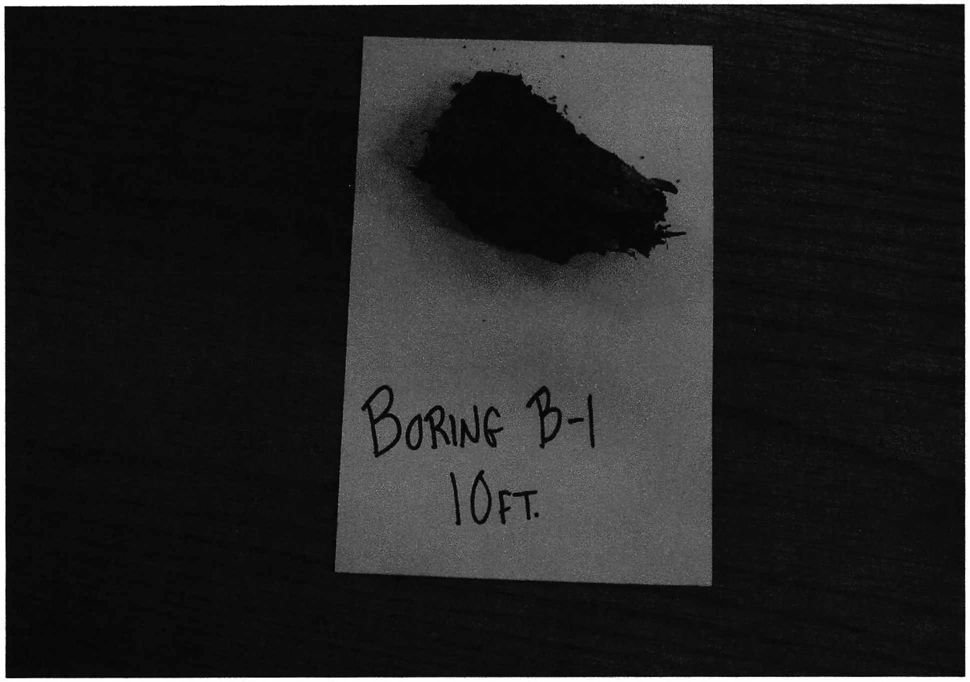
-END OF SEARCH- 42 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

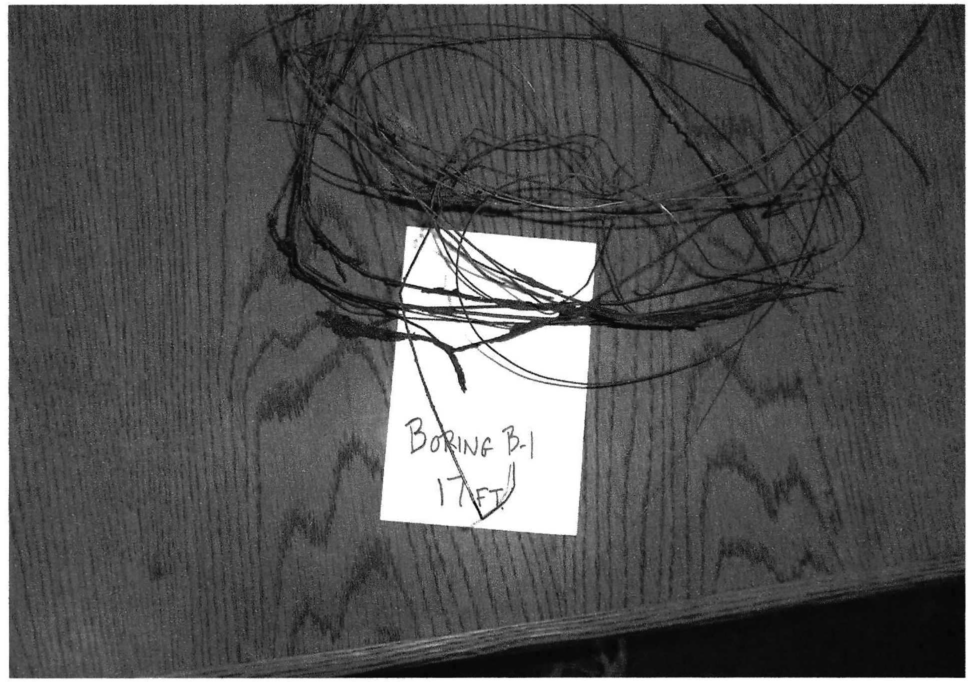
THE PUENTE HILLS BLIND THRUST FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 3.4 MILES (5.5 km) AWAY.

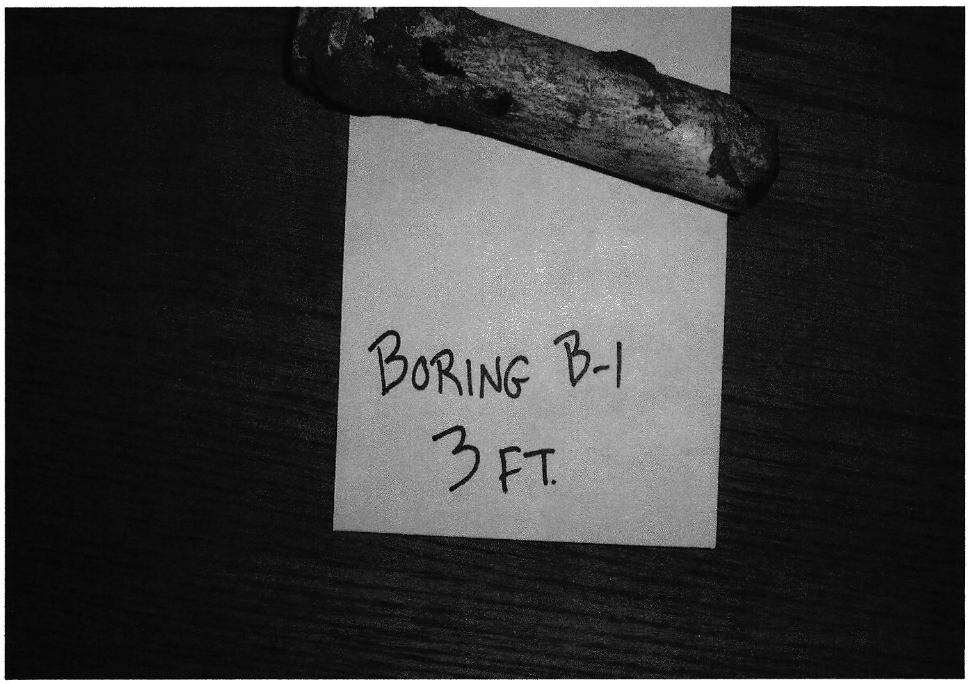
LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.5353 g

APPENDIX D

PHOTO EVIDENCE OF DEBRIS FILL


BORING B-1
10 FT.


BORING B-1
17 FT.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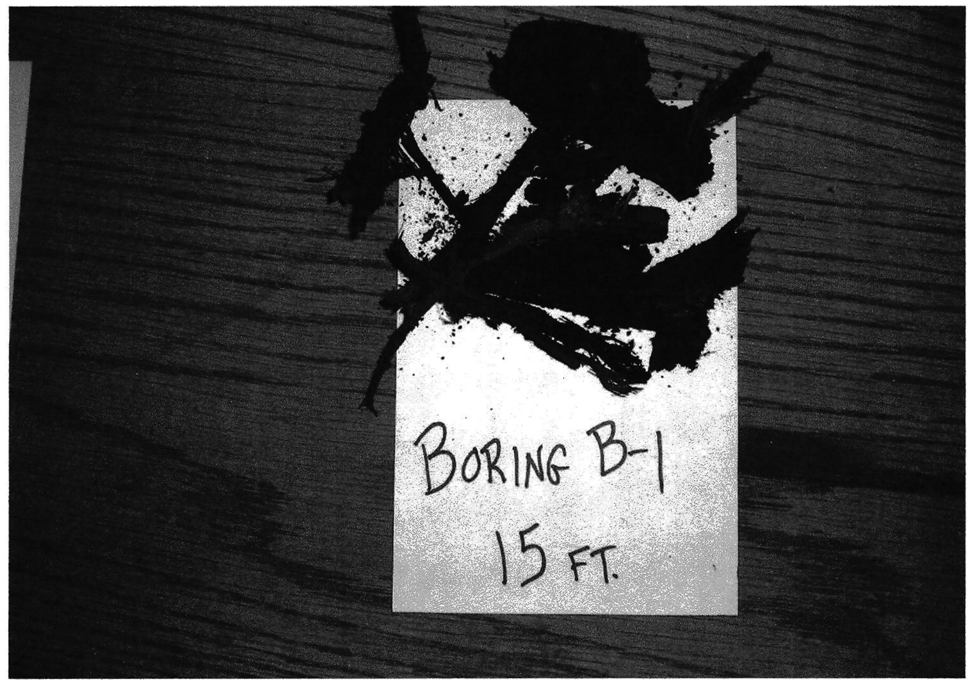
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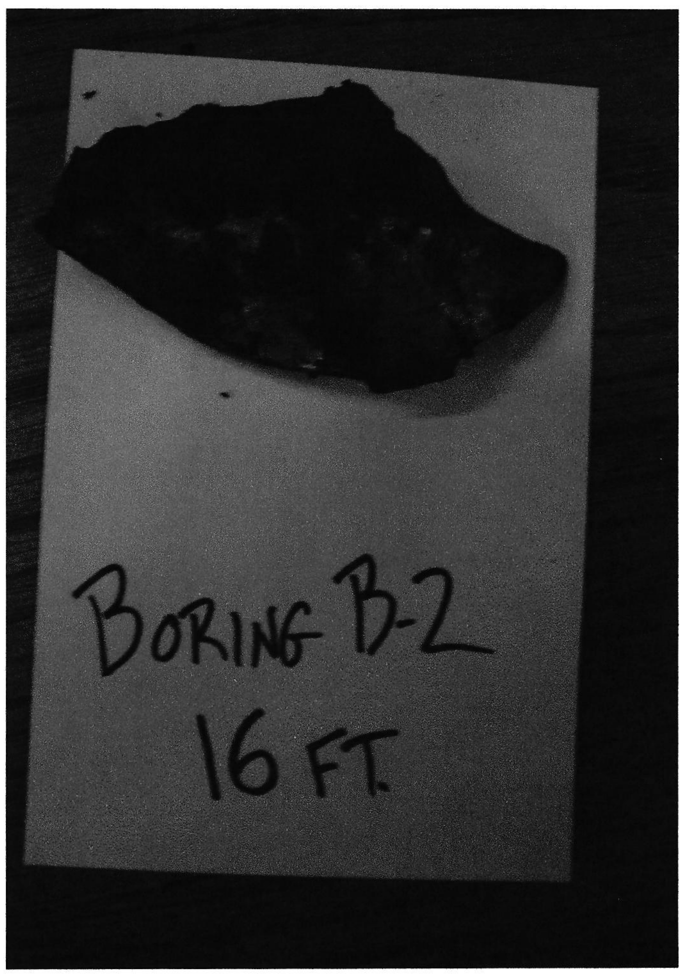
15 FT.


BORING B-1
20 FT.


BORING B-2
10 FT.


BORING B-2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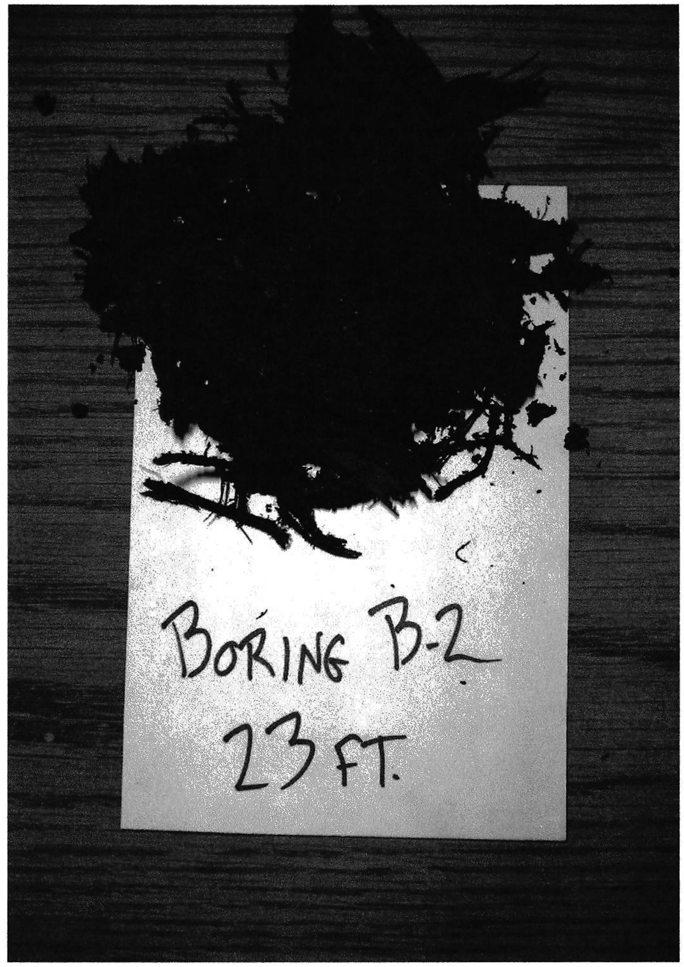
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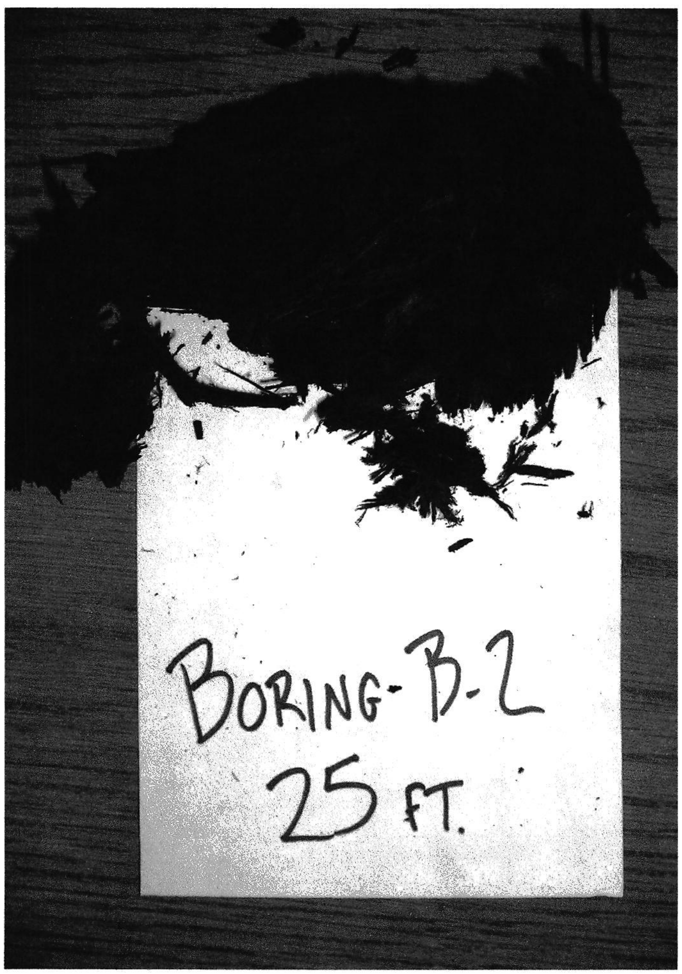
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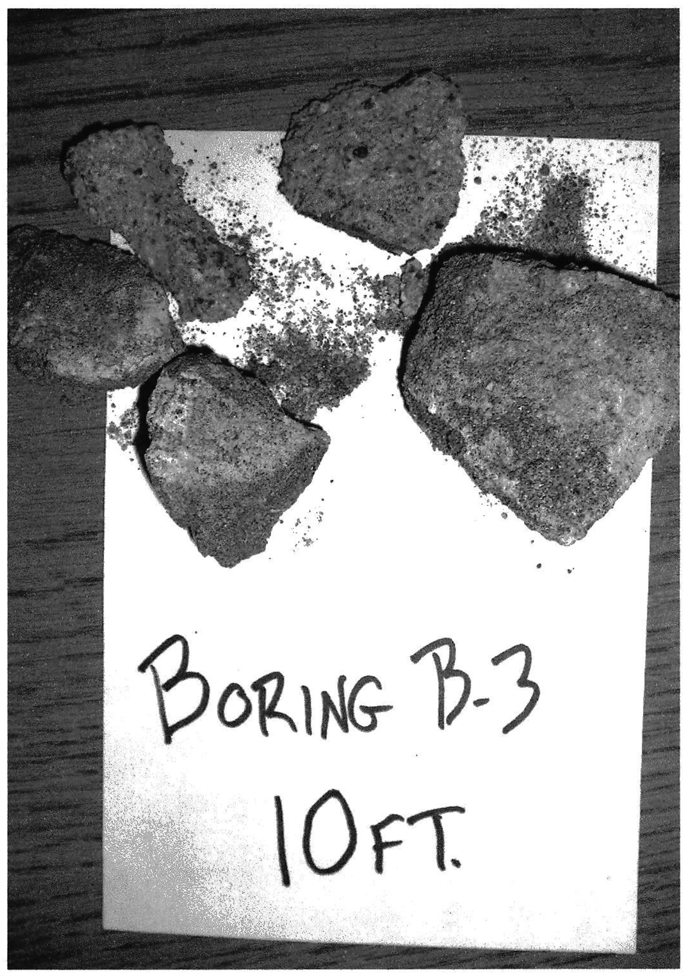
BORING B-3
10 FT.

APPENDIX E

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APPENDIX F REFERENCES

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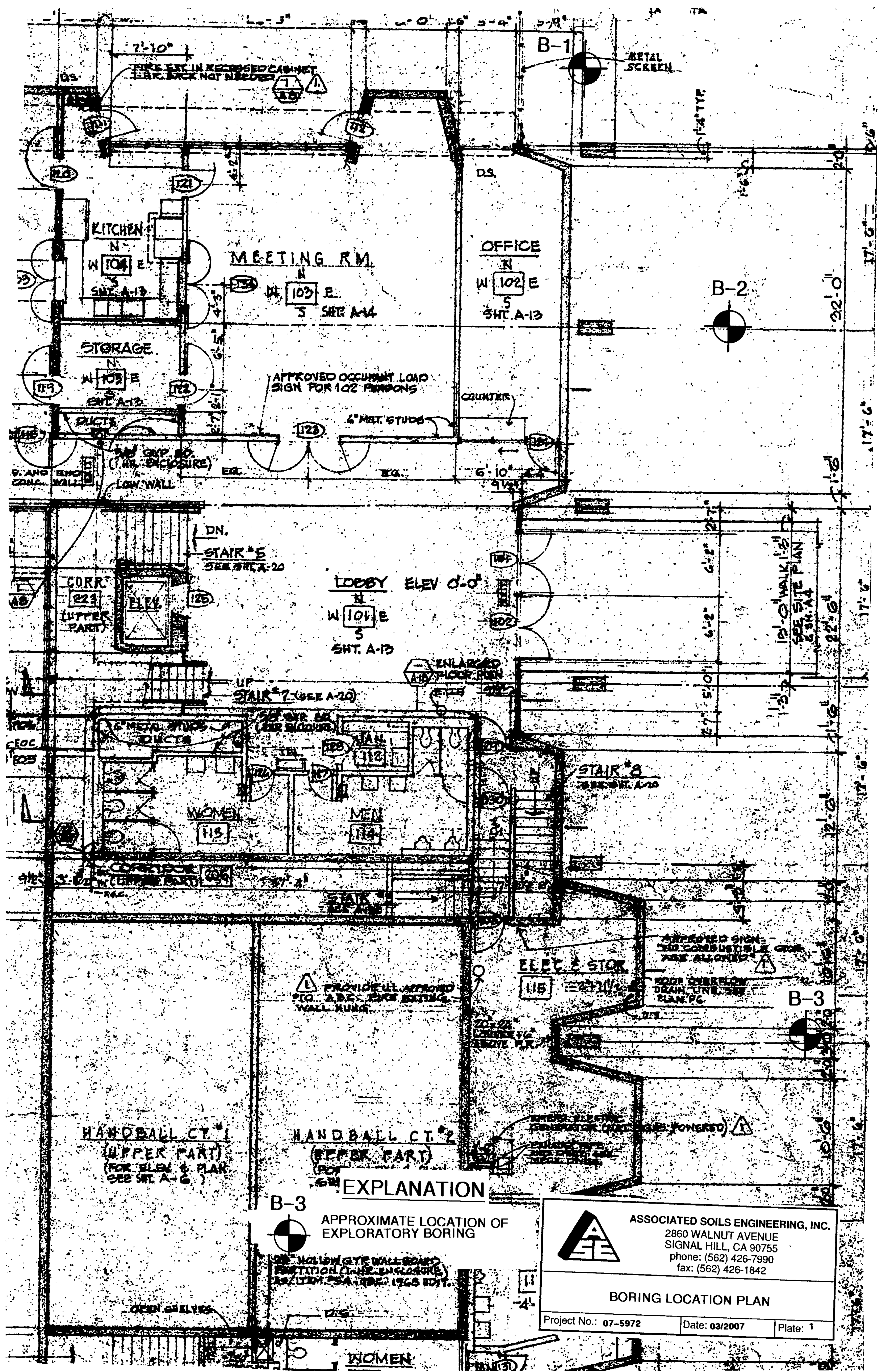
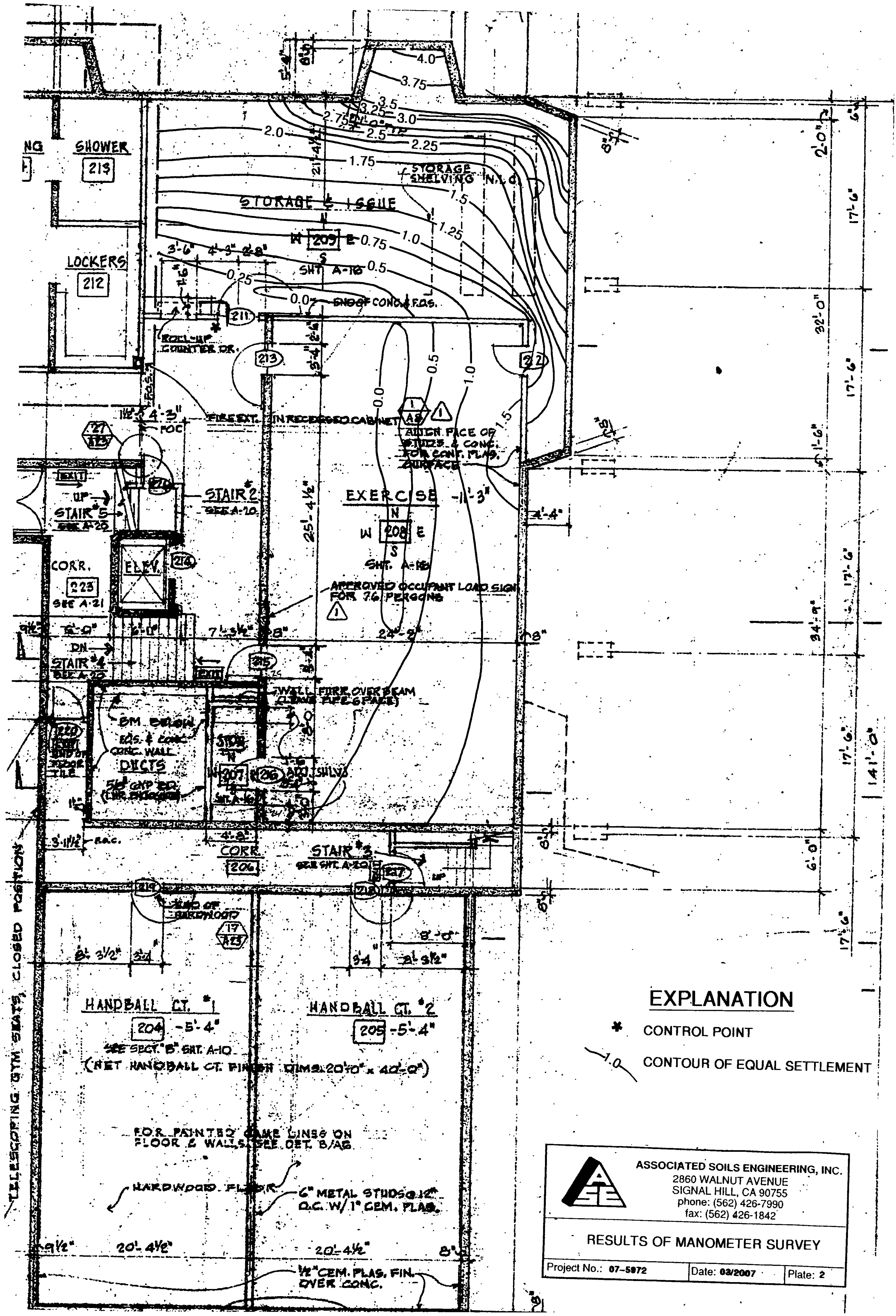

B-1
METAL SCREEN
B-2
B-3
KITCHEN
STORAGE
MEETING RM.
OFFICE
LOBBY ELEV 0'-0"
CORR.
STAIR #5
STAIR #7 (SEE A-20)
STAIR #8
WOMEN
MEN
JAN.
ELEC. & STOR.
HANDBALL CT. #1 (UPPER PART)
HANDBALL CT. #2 (UPPER PART)
APPROVED OCCUPANT LOAD SIGN FOR 102 PERSONS
COUNTER
EXPLANATION
APPROXIMATE LOCATION OF EXPLORATORY BORING
ASSOCIATED SOILS ENGINEERING, INC.
2860 WALNUT AVENUE
SIGNAL HILL, CA 90755
phone: (562) 426-7990
fax: (562) 426-1842
BORING LOCATION PLAN
Project No.: 07-5972
Date: 03/2007
Plate: 1



EXPLANATION

* CONTROL POINT

1.0 CONTOUR OF EQUAL SETTLEMENT

ASSOCIATED SOILS ENGINEERING, INC.
2860 WALNUT AVENUE
SIGNAL HILL, CA 90755
phone: (562) 426-7990
fax: (562) 426-1842

RESULTS OF MANOMETER SURVEY

Project No.: 07-5972	Date: 03/2007	Plate: 2