location, setting and association. A property nominated solely under Criterion C (design) would usually rely primarily upon integrity of design, materials and workmanship. The California Register regulations include similar language with regard to integrity, but also state that "it is possible that historical resources may not retain sufficient integrity to meet the criteria for listing in the National Register, but they may still be eligible for listing in the California Register." Further, according to the NRHP guidelines, the integrity of a property must be evaluated at the time the evaluation of eligibility is conducted. Integrity assessments cannot be based on speculation with respect to historic fabric and architectural elements which may exist but are not visible to the evaluator, or on restorations which are theoretically possible but which have not occurred. (CCR §4852 (c))

The minimum age criterion for the National Register of Historic Places (NRHP) and the California Register of Historical Resources (CRHR) is 50 years. Properties less than 50 years old may be eligible for listing on the NRHP if they can be regarded as "exceptional," as defined by the NRHP procedures, or in terms of the CRHR, "if it can be demonstrated that sufficient time has passed to understand its historical importance" (Chapter 11, Title 14, §4842(d)(2))

Historic resources as defined by CEQA also includes properties listed in "local registers" of historic properties. A "local register of historic resources" is broadly defined in §5020.1 (k) of the Public Resources Code, as "a list of properties officially designated or recognized as historically significant by a local government pursuant to a local ordinance or resolution." Local registers of historic properties come essentially in two forms: (1) surveys of historic resources conducted by a local agency in accordance with Office of Historic Preservation procedures and standards, adopted by the local agency and maintained as current, and (2) landmarks designated under local ordinances or resolutions. These properties are "presumed to be historically or culturally significant... unless the preponderance of the evidence demonstrates that the resource is not historically or culturally significant." (PRC §§ 5024.1, 21804.1, 15064.5)

Long Beach Landmark Criteria

According to §2.63.050 of the Long Beach Municipal Code (Criteria for designation of landmarks and landmark districts), a cultural resource may be recommended for designation as a landmark or landmark district if it manifests one of the following criteria:

A. It possesses a significant character, interest or value attributable to the development, heritage or cultural characteristics of the city, the southern California region, the state or the nation; or
B. It is the site of a historic event with a significant place in history; or
C. It is associated with the life of a person or persons significant to the community, city, region or nation; or
D. It portrays the environment in an era of history characterized by a distinctive architectural style; or
E. It embodies those distinguishing characteristics of an architectural type or engineering specimen; or
F. It is the work of a person or persons whose work has significantly influenced the development of the city or the southern California region; or
G. It contains elements of design, detail, materials, or craftsmanship which represent a significant innovation or
H. It is a part of or related to a distinctive area and should be developed or preserved according to a specific historical, cultural or architectural motif; or
I. It represents an established and familiar visual feature of a neighborhood or community due to its unique location or specific distinguishing characteristic; or

J. It is, or has been, a valuable information source important to the prehistory or history of the city, the southern California region or the state; or

K. It is one of the few remaining examples in the city, region, state or nation possessing distinguishing characteristics of an architectural or historical type; or

L. In the case of the designation of a tree(s) based on historic significance, that the tree(s) is (are) associated with individuals, places and/or events that are deemed significant based on their importance to national, state and community history; or

M. In the case of the designation of a tree(s) based on cultural contribution, that the tree(s) is (are) associated with a particular event or adds (add) significant aesthetic or cultural contribution to the community. (Ord. ORD-05-0026 § 1, 2005; Ord. C-6961 § 1 (part), 1992).

2. Impact Thresholds and Mitigation

According to the Public Resources Code, "a project that may cause a substantial change in the significance of an historical resource is a project that may have a significant effect on the environment." The Public Resources Code broadly defines a threshold for determining if the impacts of a project on an historic property will be significant and adverse. By definition, a substantial adverse change means, "demolition, destruction, relocation, or alterations," such that the significance of an historical resource would be impaired. For purposes of NRHP eligibility, reductions in a property's integrity (the ability of the property to convey its significance) should be regarded as potentially adverse impacts. (PRC §21084.1, §5020.1(6))

Further, according to the CEQA Guidelines, "an historical resource is materially impaired when a project... [d]emolishes or materially alters in an adverse manner those physical characteristics of an historical resource that convey its historical significance and that justify its inclusion in, or eligibility for, inclusion in the California Register of Historical Resources [or] that account for its inclusion in a local register of historical resources pursuant to section 5020.1(k) of the Public Resources Code or its identification in an historical resources survey meeting the requirements of section 5024.1(g) of the Public Resources Code, unless the public agency reviewing the effects of the project establishes by a preponderance of evidence that the resource is not historically or culturally significant."

The lead agency is responsible for the identification of "potentially feasible measures to mitigate significant adverse changes in the significance of an historical resource." The specified methodology for determining if impacts are mitigated to less than significant levels are the *Secretary of the Interior's Standards for the Treatment of Historic Properties with Guidelines for Preserving, Rehabilitating, Restoring, and Reconstructing Historic Buildings and the Secretary of the Interior's Standards for Rehabilitation and Guidelines for Rehabilitating Historic Buildings* (1995), publications of the National Park Service. (PRC §15064.5(b)(3-4))

3. Historical Setting

General Historical Context

The present city of Long Beach is located on a portion of the 300,000 acres of land granted to Manuel Nieto by the Spanish colonial government in 1784. This tract would subsequently be divided into five smaller land grants, including Rancho Los Alamitos and Rancho Cerritos, on which Long Beach would later be established. The former was purchased in 1840 by real estate speculator and cattleman Abel Stearns, who was in the process of amassing one of the largest land-holdings in Southern California, known collectively as Stearn's Ranchos. Rancho Los Alamitos was purchased in 1843 by Los Angeles merchant John Temple. Both Stearns and

Temple became victims of the prolonged droughts of the early 1860s, eventually selling the two ranchos to Jotham Bixby.

The first effort to develop the ranchos was attempted by William E. Wilmor, in 1880, on a portion of the Bixby landholdings. He called his townsite the “American Colony” or “Willmore City.” Willmore was a few years too early to benefit from the enormous railroad-inspired Southern California land boom of the late 1880s, and was undercapitalized. His efforts failed, but Willmore’s 1882 subdivision formed the precursor to modern Long Beach. The townsite was purchased in 1884 by the Long Beach Land and Water Company, which began making significant improvements, including the construction of a wharf and hotel, and connecting the town to the Southern Pacific Railroad’s Wilmington branch. The elements for growth now in place, the expansion was explosive, especially after the opening of a Pacific Electric line to the city in 1902. Long Beach, which had become one of the region’s premier seaside resorts, was incorporated as a city in 1908.

The city began to take on a more commercial and industrial character with the construction of harbor facilities, beginning with the relocation of the Craig Shipbuilding Company to Long Beach in 1907. The Port of Long Beach continued to expand as oceanfront lands were reclaimed, particularly after the discovery of major oil fields at nearby Signal Hill in 1921. The 1920s would be a defining decade for Long Beach, as it expanded rapidly on the twin pillars of tourism and commerce, emerging as a city rivaling Los Angeles for regional stature and importance.

The devastating 1933 Long Beach earthquake was a major setback for Long Beach, particularly coming as it did at the nadir of the Great Depression. The city’s fortunes would return fairly quickly, however, with the continued development of local oil resources during the 1930s, and the establishment of the Long Beach Navy Base and Shipyard in 1940.

Site-Specific Context

The project site is located within a portion of the city generally known today as North Long Beach. The area is located within the northern reaches of Rancho Los Cerritos, which was subdivided in 1887 as the California Cooperative Colony Tract. South Street marked the southern boundary of the subdivision, which remained predominately agricultural until after 1915, when the Los Angeles County Flood Control District was created in response to heavy flooding occurring during the winter of 1914-5. Over the next 20 years, the District addressed flooding on the Dominquez Creek, Rio Hondo and Los Angeles River which had plagued the North Long Beach area, vastly improving its development potential.

The massive oil strike on nearby Signal Hill in 1921 led to the rapid growth of population in unincorporated North Long Beach, which was then commonly known as Virginia City, after the nearby Virginia Country Club, and its subdivision for modest working-class residences. When the Virginia Country Club was relocated from Rancho Los Alamitos to Rancho Los Cerritos in 1921, this section of North Long Beach began to attract more prosperous residents. In response to residential growth, related commercial districts sprung up along Atlantic Avenue and American Avenue (now, Long Beach Boulevard), a process which continued into the 1940s. The area was annexed to the City of Long Beach in 1923. Construction in the area waned during the 1930s, along with the rest of the city, but rebounded rapidly after 1940, when the opening of the Long Beach Navy Base and Shipyard replenished the demand for labor.

4. Potential Historic Resources

Previously Identified Historic Resources

This project area has been the subject of previous recent efforts to identify historic properties. As a result of these efforts, three properties on the project site were determined to be potentially eligible for the NRHP, CRHR or for listing as Long Beach City Landmarks. Numerous other properties on the project site which were determined to be ineligible were subsequently demolished. [Figure 2]

5870-74 Atlantic Avenue. This property consists of a one and two-story reinforced concrete commercial building and theater with a rectangular plan located on a corner parcel. The primary (western) elevation on Atlantic Avenue consists of a one-story commercial storefront wrapping the corner of 59th Street. The storefront features a band of display windows above a low bulkhead with a shallow projecting canopy above. The upper facade consists of a featureless parapet. The parapet steps upwards to the south to meeting the two-story western elevation of the theater. The street elevation of the theater is characterized by curved walls forming a deeply inset semicircular forecourt featuring a terrazzo floor. The upper facade features narrow vertical reveals and a projecting marquee. The primary architectural feature of the theater is the tower, which is characterized by stepped vertical fins terminating in an open metal-frame diamond-shaped lighting element set on a hexagonal base. The architectural style of the building is Moderne.

A previous investigator found this property to have been constructed for Ivan C. Hanson in 1940 as the Atlantic Theatre Building. However additional research indicates that ground was broken for construction in April 1941 and the building was completed in May 1942. (*Long Beach Independent,* 4-18-1941, 5-19-1942)

The building was designed by the Los Angeles and Kansas City architectural firm of Carl Heinrich Boller. Shortly after its founding in Missouri circa 1902, the firm was expanded to include Robert Otto Boller, and came to be known as Boller Brothers, recognized for their expertise in theater design. Carl Boller relocated to Southern California and opened an office in Los Angeles during the early 1920s. In Southern California, in addition to this property, his firm is credited with designs for the Inglewood Theatre (Inglewood, 1922), Ritz Theatre (Long Beach, 1924), Largo Theatre (Watts, 1924), Corona Theater (Corona, 1929), and Stadium Theatre (Los Angeles, 1931). The firm also designed theaters in Montrose, Santa Ana, and Covina. (Jones & Stokes, 2005)

The design integrity of the property is generally good. Apparent alterations include the replacement or alteration of the original theater marquee in 1977 with a marquee which is evidently smaller than the original. A fountain located at the front of the forecourt replaced a freestanding ticket both. The commercial storefront is somewhat altered, including the removal or enclosure of transom windows over the display windows and the removal of stepped fluted pylons which projected above the top of the parapet. This latter alteration is more significant, as this design element was originally intended to reinforce the verticality of the tower. The integrity of setting for the property is substantially diminished due to the loss of much of the historically related commercial and residential area. [Figure 3, Photos 1-2]

This property was determined to be potentially eligible for the NRHP as an individual property in 2005 for its architectural significance.

5832-34 Atlantic Avenue. This property consists of a one-story commercial building constructed in 1948. The primary western elevation features a deep, cantilevered stucco-clad canopy with canted edges. The off-centered double-door entrance is deeply recessed beneath the canopy and flanked by low concrete planters.

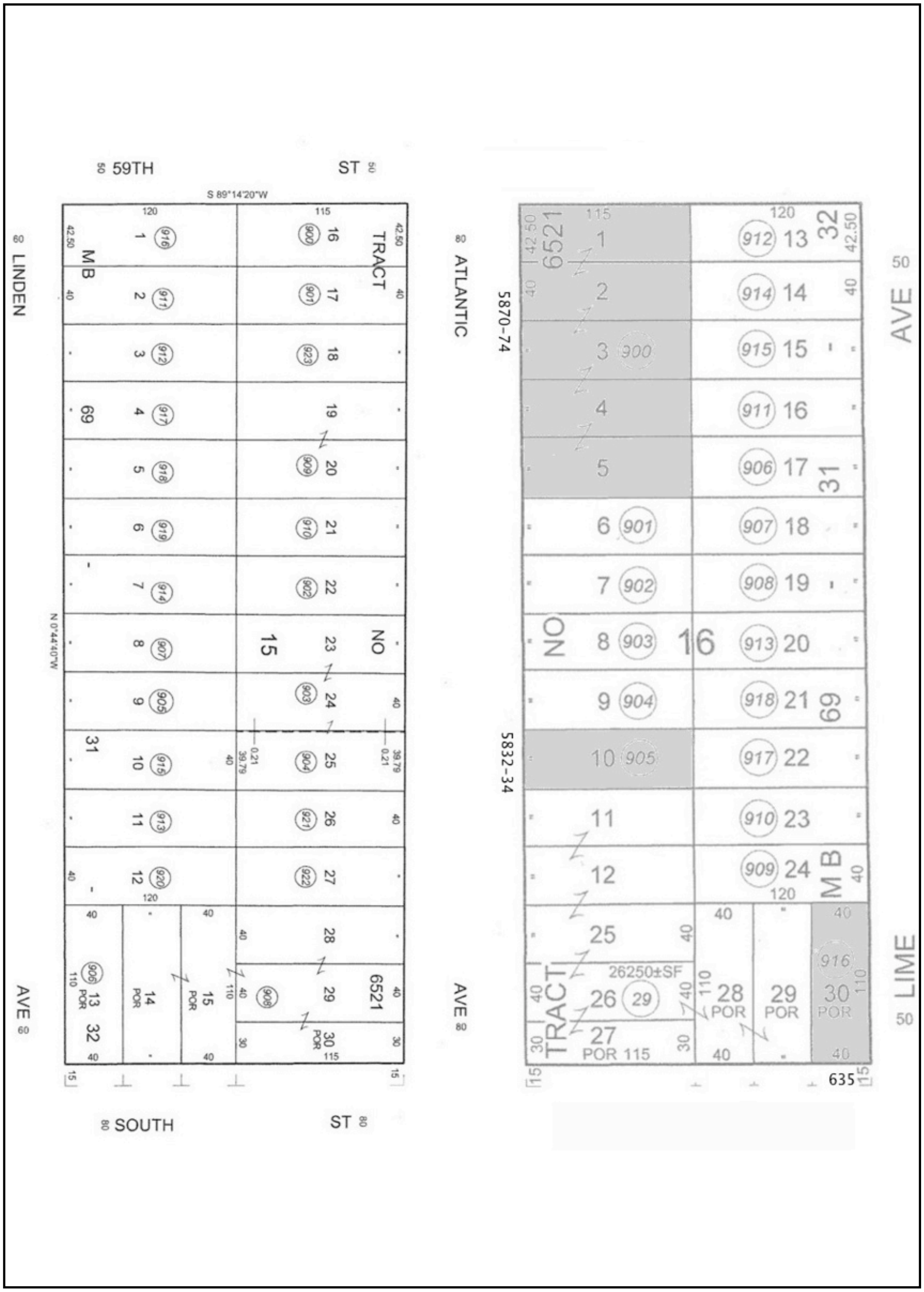

Figure 2. Location of Previously Identified Historic Resources [Source: Los Angeles County Assessor, Map Book 7124, Page 32; Map Book 4125, Page 33]

Three storefront bays feature plate glass windows organized in a sawtooth pattern facing southwest. A large trapezoidal pylon sign is attached to the northwestern corner of the building supported by angled wood struts. A substantial addition was made to the rear of the building and the sign added in 1949. The main elevation was altered in 1951. The building was originally constructed as a restaurant and later used as a tavern. The original building and the 1949 addition were designed by Long Beach architect Thomas J. Russell. No architect was apparently utilized for the 1951 alterations. The architectural style is Late Moderne. The integrity of the property appears to be good, although the display windows are currently covered with plywood and the extent of the 1951 alterations are unclear. The integrity of setting for the property is substantially diminished due to the loss of the much of the historically related commercial and residential area. [Photo 3]

This property was identified in a 2007 historic resources report as as a representative example of a post-war North Long Beach commercial building and as a scarce example of Late Moderne architecture in the area, and determined to be individually eligible for designation as a Long Beach City Landmark on this basis. (Moruzzi, 2007)

635 South Street. This property consists of a one story commercial building constructed in 1948. The storefront facade of the principal southern elevation consists of three bays with a centered entrance flanked by plate glass display windows located above a moderately low bulkhead. A single bay display window wraps to the eastern elevation. Very narrow blind transoms are located above the display windows. The upper facade consists of a large, featureless soffit projecting over the sidewalk. Four small wood multi-paned windows are located on the eastern elevation. The windows and doors are defined by pilasters clad in pale artificial stone laid in a Roman brick pattern. This materials also clads the bulkhead on the eastern elevation. The balance of the building is clad in rough stucco. The roof behind the upper facade is apparently flat. The buildings architectural style is mildly Modern. This property's integrity is generally good, although the current stucco cladding does not appear to be original, and the artificial stone cladding on the bulkhead below the east-facing display window suggests that this material was also originally found below the south-facing display windows. The integrity of setting for the property is substantially diminished due to the loss of the much of the historically related commercial and residential area. [Photo 4]

This property was identified in a 2005 survey as a representative example of a post-war North Long Beach commercial building and determined to be individually eligible for designation as a Long Beach City Landmark on this basis. (Jones & Stokes, 2005)

Non-historic Properties on Project Site

Previous recent efforts to identify historic properties have resulted in the determination that two properties on the project site were ineligible for the NRHP, CRHR or for listing as Long Beach City Landmarks. (Jones & Stokes, 2005)

5869 Atlantic Avenue. This one story commercial building was constructed in 1951 and heavily altered in 1972. This property was determined to be ineligible for any listing or designation in 2005. [Photo 5]

5800 Atlantic Avenue. This one story commercial building was constructed in 1996 and determined to be ineligible for any listing or designation in 2005. [Photo 6]

Identified Historic Resources in Project Vicinity

Previous recent efforts to identify historic properties have resulted in the determination that several properties in the immediate vicinity of the proposed project are potentially eligible for the CRHR or for listing as Long Beach City Landmarks. (Jones & Stokes, 2005)

620-630 E. South Street. This one-story commercial building constructed in 1934 in the Streamline Moderne style was determined to be eligible for individual listing on the CRHR. [Photo 7]

5800 Block, Atlantic Avenue. The majority of the one-story commercial buildings located along the west side of Atlantic Avenue between E. South and 56th streets constructed between the late 1920s and the 1940s were determined to be eligible as Long Beach City Landmarks. [Photo 8]

5. Project Impacts

A. The project will result in the demolition of the property at 5870-74 Atlantic Avenue, which has been determined to be eligible for individual listing on the NRHP and is therefore regarded as an historic resource for the purposes of CEQA. The demolition or destruction of an historic resource is a significant impact which cannot be mitigated to a less than significant and adverse level. (Class 1)

B. The project will result in the demolition of the property at 5832-34 Atlantic Avenue, which has been determined to be eligible for designation as a Long Beach City Landmark, and is therefore regarded as an historic resource for the purposes of CEQA. The demolition or destruction of an historic resource is a significant impact which cannot be mitigated to a less than significant and adverse level. (Class 1)

C. The project will result in the demolition of the property at 635 E. South Street, which has been determined to be eligible for designation as a Long Beach City Landmark, and is therefore regarded as an historic resource for the purposes of CEQA. The demolition or destruction of an historic resource is a significant impact which cannot be mitigated to a less than significant and adverse level. (Class 1)

D. The project will introduce new construction into the setting of the property at 620-630 E. South Street, which has been determined to be eligible for individual listing on the CRHR and is therefore regarded as an historic resource for the purposes of CEQA. However, the integrity of the historic setting for this property has already been substantially diminished due to the loss of some of the historically related commercial and residential area and new construction within its immediate vicinity. Further, the scale, massing and character of the proposed project is generally consistent with the historic scale of commercial development along E. South Street and Atlantic Boulevard. This impact is less than significant. (Class 3)

E. The project will introduce new construction into the setting of properties located on the west side of the 5800 block of Atlantic Avenue, several of which have been determined to be eligible for designation as Long Beach City Landmarks and therefore are regarded as historic resources for the purposes of CEQA. However, the integrity of the historic setting for these properties has already been somewhat diminished due to the loss of some of the historically related commercial and residential area and new construction within their immediate vicinity. Further, the scale, massing and character of the proposed project is generally consistent with the historic scale of commercial development along E. South Street and Atlantic Boulevard. This impact is less than significant. (Class 3)

6. Project Alternatives

Project alternatives which may serve to reduce or avoid the significant impacts of a proposed project should be evaluated within an EIR. According to the CEQA Guidelines, an EIR should consider "... a range of reasonable alternatives to the project, or to the location of the project, which could feasibly attain the basic objectives of the project and evaluate the comparative merits of the alternatives." The alternatives discussion "... shall focus on alternatives capable of eliminating any significant adverse environmental impacts or reducing them to a level of insignificance, even if these alternatives would impede to some degree the attainment of the project objectives, or would be more costly." (PRC §15126(d))

The current project description does not include project alternatives which would serve to reduce or eliminate the significant adverse impacts on historic resources described in Section 5, above. If such an alternative were to be considered within the project EIR, it should evaluate the feasibility of adaptive reuse of the three eligible properties proposed for demolition.

7. Mitigation Measures and Residual Impacts

Background

A principle of environmental impact mitigation is that some measure or combination of measures may, if incorporated into a project, serve to avoid or reduce significant and adverse impacts to a historic resource. In reference to mitigating impacts on historic resources, the CEQA Guidelines state:

> Where maintenance, repair, stabilization, rehabilitation, restoration, preservation, conservation or reconstruction of the historical resource will be conducted in a manner consistent with the *Secretary of the Interior's Standards for the Treatment of Historic Properties with Guidelines for Preserving, Rehabilitating, Restoring, and Reconstructing Historic Buildings* (1995), Weeks and Grimmer, the project's impact on the historical resource shall generally be considered mitigated below a level of significance and thus is not significant. (PRC §15126.4 (b)(1))

These standards, developed by the National Park Service, represent design guidelines for carrying out historic preservation, restoration and rehabilitation projects. The Secretary's Standards and the supporting literature describe historic preservation principles and techniques, and offers recommended means for carrying them out. Adhering to the Standards is the only method described within CEQA for reducing project impacts on historic resources to less than significant and adverse levels.

The demolition of an historic property cannot be seen as conforming with the *Secretary of the Interior's Standards*. Therefore, the absolute loss of an historic property should generally be regarded as an adverse environmental impact which cannot be mitigated to a less than significant and adverse level. Further, the usefulness of documentation of an historic resource, through photographs and measured drawings, as mitigation for its demolition, is limited by the CEQA Guidelines, which state:

> In some circumstances, documentation of an historical resource, by way of historic narrative, photographs or architectural drawings, as mitigation for the effects of demolition of the resource will not mitigate the effects to a point where clearly no significant effect on the environment would occur. (CEQA Guidelines §15126.4 (b)(2))

Implied by this language is the existence of circumstances whereby documentation may mitigate the impact of demolition to a less than significant level. However, the conditions under which this might be said to have occurred are not described in the Guidelines. It is also noteworthy that the existing CEQA case law does not appear to support the concept that the loss of an historic resource can be mitigated to less than adverse impact levels by means of documentation or commemoration. (*League for Protection of Oakland's Architectural and Historic Resources v. City of Oakland* [1997] 52 Cal. App. 4th 896; *Architectural Heritage Association v. County of Monterey* [2004] 19 Cal. Rptr. 3d 469)

Taken in their totality, the CEQA Guidelines require a project which will have potentially adverse impacts on historic resources to conform to the *Secretary of the Interior's Standard*s, in order for the impacts to be mitigated to below significant and adverse levels. However, CEQA also mandates the adoption of feasible mitigation measures which will reduce adverse impacts, even if the residual impacts after mitigation remain significant. Means other than the application of the Standards would necessarily be required to achieve this level of mitigation. In determining what type of additional mitigation measures would reduce impacts to the greatest extent feasible, best professional practice dictates considering the level of eligibility of the property, as well as by what means it derives its significance.

Mitigation programs for impacts on historic resources tend to fall into three broad categories: documentation, design and interpretation. Documentation techniques involve the recordation of the site according to accepted professional standards, such that the data will be available to future researchers, or for future restoration efforts. Design measures could potentially include direct or indirect architectural references to a lost historic property, e.g., the incorporation of historic artifacts, into the new development, or the relocation of the historic property to another suitable site. Interpretative measures could include commemorating a significant historic event or the property's connection to historically significant themes.

Project Mitigation

A. In consultation with the Long Beach Historic Preservation Officer, an historic preservation professional qualified in accordance with the *Secretary of the Interior's Standards* shall be selected to complete Documentation Reports on the eligible properties to be demolished. Properties determined eligible for listing on the NRHP shall be documented at HABS/HAER Level 2 standards. Properties determined to be eligible for City Landmark listing shall be documented with archival quality photographs of a type and format approved by the Long Beach Historic Preservation Officer. This documentation, along with historical background of the properties prepared for this property, shall be submitted to an appropriate repository approved by the Long Beach Historic Preservation Officer. The documentation reports shall be completed and approved by the Long Beach Historic Preservation Officer prior to the issuance of demolition permits.

B. In consultation with the Long Beach Historic Preservation Officer, an historic preservation professional qualified in accordance with the *Secretary of the Interior's Standards* shall be selected by the city to prepare an on-site interpretive plan, focusing on the significant historic themes associated with the properties to be demolished and the historical development of North Long Beach. The plan may consist of a public display or other suitable interpretive approaches, as approved by the Long Beach Historic Preservation Officer, and be installed in an appropriate public location within the proposed Library-Community Center building. The interpretive plan shall be completed and approved prior to the issuance of building permits for the proposed Library-Community Center building, and shall be installed within one year of occupancy of the proposed Library-Community Center building. If the proposed Library-Community Center building is not occupied within two years after the issuance of demolition permits, another suitable temporary or permanent location for the interpretive display shall be determined, subject to the approval of the Long Beach Historic Preservation Officer. The interpretive display shall remain in public view for a minimum of five years, and if removed, appropriately archived.

Impacts After Mitigation

Significant and adverse.

8. Selected Sources

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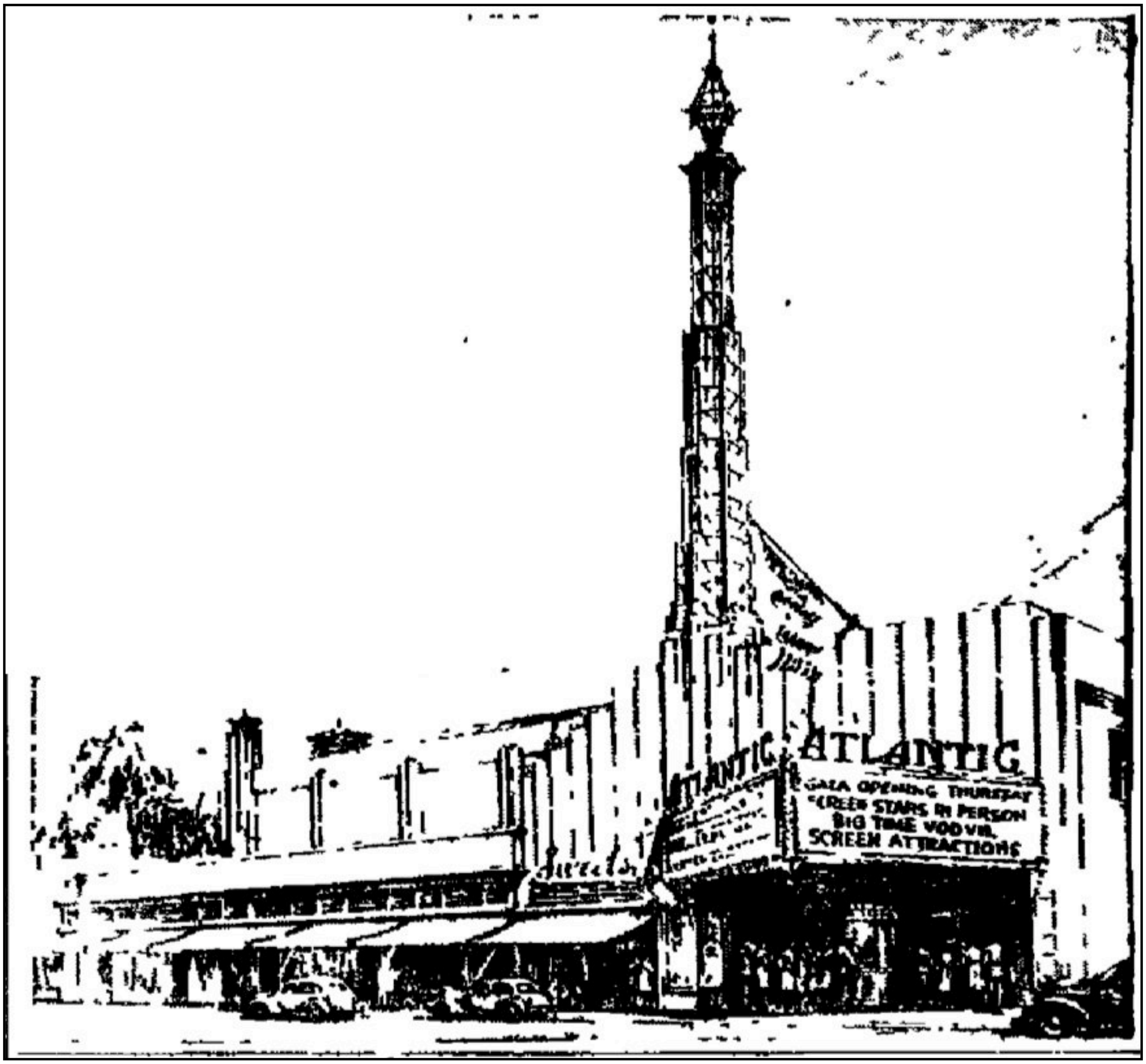

Figure 3. Atlantic Theater building, architectural rendering. [Source: *Long Beach Independent*, 5-19-1942]

Photo 1. 5870 S. Atlantic Avenue, western and southern elevations. [27 Aug 2008]

Photo 2. 5870 S. Atlantic Avenue, western and northern elevations. [27 Aug 2008]

Photo 3. 5834 S. Atlantic Avenue, western and northern elevations. [27 Aug 2008]

Photo 4. 635 E. South Street, southern and eastern elevations. [27 Aug 2008]

Photo 5. 5869 S. Atlantic Avenue, eastern and northern elevations. [27 Aug 2008]

Photo 6. 5800 S. Atlantic Avenue, southern and eastern elevations. [27 Aug 2008]

Photo 7. 620-630 E. South Street, northern and eastern elevations. [27 August 2008]


Photo 8. 5800 block, Atlantic Avenue, west side. [27 August 2008]

Appendix D

Geotechnical Report

Geotechnologies, Inc.
Consulting Geotechnical Engineers

439 Western Avenue
Glendale, California 91201-2837
818.240.9600 • Fax 818.240.9675

March 5, 2008
File No. 19593

AMCAL Multi-Housing
30141 Agoura Road, Suite 100
Agoura Hills, California 91301

Attention: Dayna Ranger

Subject: Preliminary Geotechnical Engineering Investigation
Proposed Mixed Use Development
Northwest Corner South Street and Atlantic Boulevard, "West Block"
Long Beach, California

Ladies and Gentlemen:

This letter transmits the Preliminary Geotechnical Engineering Investigation for the subject property prepared by Geotechnologies, Inc. This report provides geotechnical recommendations for the development of the site, including earthwork, seismic design, retaining walls, excavations, shoring and foundation design. Engineering for the proposed project should not begin until approval of the geotechnical investigation is granted by the local building official. Significant changes in the geotechnical recommendations may result due to the building department review process. This report is preliminary in nature due to the lack of a well defined development plan. This report is therefore not intended for submission for building permit purposes. A comprehensive report should be prepared when the development plan achieves refinement.

The validity of the recommendations presented herein is dependant upon review of the geotechnical aspects of the project during construction by this firm. The subsurface conditions described herein have been projected from limited subsurface exploration and laboratory testing. The exploration and testing presented in this report should in no way be construed to reflect any variations which may occur between the exploration locations or which may result from changes in subsurface conditions.

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
EDWARD F. HILL
No. 2126
Exp. 6/30/08
GEOTECHNICAL
STATE OF CALIFORNIA

EDWARD F. HILL
G.E. 2126/E.G. 1403

EFH:km

Distribution: (7) Addressee

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PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED MIXED USE DEVELOPMENT

NORTHWEST CORNER SOUTH STREET AND ATLANTIC BOULEVARD

"WEST BLOCK"

LONG BEACH, CALIFORNIA

INTRODUCTION

This report presents the results of the preliminary geotechnical engineering investigation performed on the subject property. The purpose of this investigation was to identify the distribution and engineering properties of the earth materials underlying the site, and to provide geotechnical recommendations for the design of the proposed development.

This report is preliminary in nature due to the lack of a well defined development plan. This report is therefore not intended for submission for building permit purposes. A comprehensive report should be prepared when development plan achieves refinement.

This investigation included 15 exploratory excavations, collection of representative samples, laboratory testing, engineering analysis, review of published geologic data, review of available geotechnical engineering information and the preparation of this report. The exploratory excavation locations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. The entire project comprises two city blocks and consists of a mixed use development. This report addresses the portion of the project known as the "West Block" The structures are proposed to be between 2 and 6 stories over a possible subterranean parking level. If the partial subterranean level is not selected, the structures will be built at- or near existing grade. Column loads are estimated to be between 400 and 800 kips. Wall loads are estimated to be between 3 and 6 kips per lineal foot. Grading will consist of excavations up to 12 feet for the possible subterranean level. If the structures are to be at grade, grading will consist of removal and recompaction for existing unsuitable soils.

Any changes in the design of the project or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained in this report should not be considered valid until reviewed and modified or reaffirmed, in writing, subsequent to such review.

SITE CONDITIONS

The property is located at the northwest corner of South Street and Atlantic Boulevard in the City of Long Beach, California. At the time this report was prepared this portion of the project was known as the "West Block". The site is relatively level with little elevation change. Drainage across the site is by sheetflow to the improved streets.

The site is currently developed with several residential and commercial structures. The vegetation on the site consists of mature trees, shrubs and grasses. The neighboring development consists of one to three story residential and commercial structures.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on January 25, 28 and February 1, 2008 by excavating 15 exploratory excavations. The exploratory excavations varied in depth from 30 to 60 feet. The borings were excavated with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers. The exploration locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-15.

The location of exploratory excavations was determined by information furnished by the client. Elevations of the exploratory excavations were determined by hand level or interpolation from data provided. The location and elevation of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill materials were encountered in each of the exploratory excavations. The fill was found to vary from 1 to 2-1/2 feet in depth and consists of silty sands which are yellowish brown, moist, medium dense, medium grained and contains asphalt fragments and gravel. The native soils underlying the site consist of silty sands, sandy to clayey silts and occasional clays and sands. The native soils are dark to yellowish brown, moist to wet, medium dense and fine grained.

The geologic materials consist of detrital sediments deposited by river and stream action typical to this area of Los Angeles County. More detailed descriptions of the earth materials encountered may be obtained from individual logs of the subsurface excavations.

Groundwater and Caving

Groundwater was encountered between the depth of 29 and 37 feet below ambient site grade. Caving could not be directly observed during exploration due to the type of drilling equipment utilized.

The historic high groundwater level was established by review of California Geological Survey Seismic Hazard Evaluation Report 028 Plate 1.2 entitled "Historically Highest Ground Water Contours". Review of this plate indicates that the historically highest groundwater level is on the order of 30 feet below grade.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject property is located in the northern portion of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are characterized by northwest-trending blocks of mountain ridges and sediment-floored valleys. The dominant geologic structural features are northwest trending fault zones that either die out to the northwest or terminate at east-trending reverse faults that form the southern margin of the Transverse Ranges.

The Los Angeles Basin is located at the northern end of the Peninsular Ranges Geomorphic Province. The basin is bounded by the east and southeast by the Santa Ana Mountains and San Joaquin Hills, to the northwest by the Santa Monica Mountains. Over 22 million years ago the Los Angeles basin was a deep marine basin formed by tectonic forces between the North American and Pacific plates. Since that time, over 5 miles of marine and non-marine sedimentary rock as well as intrusive and extrusive igneous rocks have filled the basin. During the last 2 million years, defined by the Pleistocene and Holocene epochs, the Los Angeles basin and surrounding mountain ranges have been uplifted to form the present day landscape. Erosion of the surrounding mountains, has resulted in deposition of unconsolidated sediments in low-lying areas by rivers such as the Los Angeles River. Areas that have experienced subtle uplift have been eroded with gullies.

The site is underlain by unconsolidated alluvial sediments deposited by river and stream action, that are deeper than 200 feet.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey(CGS), faults may be categorized as active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last 11,000 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.

Buried thrust faults are faults without a surface expression but are a significant source of seismic activity. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large earthquakes in the southern California area. Due to the buried nature of these thrust faults, their existence is usually not known until they produce an earthquake. The risk for surface rupture potential of these buried thrust faults is inferred to be low (Leighton, 1990). However, the seismic risk of these buried structures in terms of recurrence and maximum potential magnitude, is not well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be precluded.

Using the computer program EQFAULT significant faults within a 60 mile radius of the site and their distance to the site is presented in Table I in the Appendix. The program EQFAULT, measures the shortest distance to faults in a three dimensional system. Some of the attenuation relationships utilized in the program returns a distance of 0.0 miles where the depth to a dipping fault plane is less than 10 km. For depths greater than 10 km, these attenuation relationships cause the program to return the inferred depth to the fault plane minus 10 km.

HISTORIC SEISMICITY

The epicenters of earthquakes with magnitudes of 5.0 or greater, and located within a radius of 60 miles of the site are listed on Table II, Historical Earthquake Epicenters, in the Appendix. The location of the earthquake epicenters is shown on Figure II, Earthquake Epicenters Map. Other pertinent information regarding these earthquakes is also provided on Table II.

SEISMIC HAZARDS

The primary geologic hazard at the site is moderate to strong ground motion (acceleration) caused by an earthquake on any of the local or regional faults. The potential for other earthquake-induced hazards was also evaluated including surface rupture, liquefaction, dynamic settlement, inundation and landsliding.

Ground Motion

The seismic exposure of the site may be investigated in two ways. The deterministic method calculates an estimated maximum earthquake magnitude for a fault based on formulas which correlate the fault trace to the theoretical maximum magnitude earthquake. The probabilistic method considers the probability of exceedance of various levels of ground motion (acceleration) and is calculated by consideration of risk contributions from all possible earthquake scenarios on all faults within a prescribed search radius. The CGS database of faults and historical earthquakes is used for both methods.

Seismic Hazard Zone Report

The CDMG has published Seismic Hazard Zone Report 028, Seismic Hazard Zone Report for the Long Beach 7.5-Minute Quadrangle, Los Angeles County, California (1997, revised 2005). Figure 3.3 (Alluvium Conditions) indicates the PGA_{DBE} for this area of Los Angeles to be 0.48g. Figure 3.4 (Predominant Earthquake) indicates an earthquake with a moment magnitude of 6.8 (M_W) as the Design-Basis Earthquake (DBE) ground motion for this area of Los Angeles.

Deterministic Method

The deterministic method is used to predict a unique outcome for a given earthquake scenario. All known faults within the defined search radius are assigned an estimated maximum earthquake magnitude based on their length. Then, the resulting ground acceleration that the earthquake is capable of producing is calculated based on an appropriate attenuation relationship. The selected ground motion is simply the highest attenuated ground motion.

Table I in the Appendix shows known faults within a 60-mile radius of the site based on the current understanding of regional seismo-tectonics. For this investigation, the attenuation relationship of Boore, et. al. (1997) was selected. The resulting peak site accelerations at the site from the maximum-earthquake for each fault are shown on Table I.

Using this methodology, the maximum earthquake resulting in the largest estimated maximum earthquake site acceleration at the site would be a magnitude 7.1 event on the Newport-Inglewood(L.A. Basin) Fault. Such an event would be expected to generate peak horizontal accelerations at the site of 0.990g.

Probabilistic Method

The probabilistic seismic hazard analysis (PSHA) determines the probability of exceedance of various levels of ground motion and is calculated by summing the risk contributions of all of the regional faults to obtain values for the sites. For this study 39 regional faults were used. These faults are located within a specified search radius of 60 miles from the site. Figure III in the Appendix indicates the return periods of various levels of mean peak horizontal acceleration. Typical earthquake ground motions used for seismic design of structures are often taken as those with a 2 percent and a 10 percent probability of exceedance in a 50-year structural life. The 10 percent probability earthquake has a corresponding return period of 475 years. The 2 percent probability earthquake has a return period of 2,475 years.

According to the 2001 California Building Code (2001CBC), Sections 1627A, 1629A.1, and 1631A.2, the Design-Basis Earthquake (DBE) ground motion is defined as the motion having a 10 percent probability of being exceeded in a 50-year period. The DBE ground motion has a statistical return period of approximately 475 years. The DBE ground motion is a probabilistic concept, expressed as Peak Ground Acceleration, PGA_{DBE}, and is used as a basis for structural design in the 2001 CBC and as a design basis ground motion for liquefaction hazard analyses in California.

The 2003 National Earthquake Hazards Reduction Program Provisions (2003NEHRP) provides the basis for the seismic design of structures section of both the 2006 International Building Code (2006IBC) and the 2007 California Building Code (2007CBC). The 2003NEHRP defines the Maximum Considered Earthquake (MCE) ground motion as the motion having a 2 percent probability of being exceeded in a 50-year period. The MCE ground motion has a statistical return period of approximately 2,475 years. The MCE ground motion is a probabilistic concept, expressed as Peak Ground Acceleration, PGA_{MCE}, and is used as a basis for structural design in the 2007CBC.

The enclosed probabilistic seismic hazard analysis was performed utilizing the computer program, FRISKSP V. 4.00, by Thomas F. Blake (2000). The attenuation relation of Boore *et al.* (1997) was utilized to determine the peak ground motions generated by regional earthquakes. The data used for performing the probabilistic seismic hazard analysis includes recorded and measured quantities such as slip-rate and fault rupture length. The analysis does not take into account the potential hazards from unknown buried thrust faults, many of which are still to be identified. Based on the attenuation relationship of Boore *et al.* (1997) the PGA_{DBE} is 0.44g and the PGA_{MCE} is 0.74g. The results of the probabilistic seismic hazard analysis is presented in Figure IV.

SECONDARY SEISMIC EFFECTS

The primary geologic hazard at the site is moderate to strong ground shaking caused by an earthquake on any of the local or regional faults. The potential for secondary geologic hazards was also evaluated including liquefaction, dynamic settlement, inundation and landsliding.

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. The Act defines "active" and "potentially active" faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,000 years. It is this recency of fault movement that the CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

CGS policy is to delineate a boundary from 200 to 500 feet wide on each side of the known fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

Ground rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature and results of site reconnaissance, no known active or potentially active faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

Liquefaction

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

The historic high groundwater level was obtained from review of California Geological Survey Seismic Hazard Evaluation Report 98-028. Review of this report indicates that the historically highest groundwater level is on the order of 30 feet below grade. A copy of this map is included in the Appendix.

Liquefaction analysis of the soils underlying the site was performed using the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. The liquefaction potential evaluation was performed by assuming a magnitude 7.1 earthquake and a peak horizontal acceleration of 0.48g. This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data. The enclosed liquefaction analysis included in the Appendix, indicates that site soils would not be prone to liquefaction during the ground motion expected during the design based earthquake.

Dynamic Dry Settlement

Seismically-induced settlement or compaction of dry or moist, cohesionless soils can be an effect related to earthquake ground motion. Such settlements are typically most damaging when the settlements are differential in nature across the length of structures.

Some seismically-induced settlement of the proposed structures should be expected as a result of strong ground-shaking, however, due to the uniform nature of the underlying earth materials, excessive differential settlements are not expected to occur.

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within the mapped tsunami inundation boundaries.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site lies within mapped inundation boundaries due to a seiche or a breached upgradient reservoir. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Landsliding

The probability of seismically-induced landslides occurring on the site is considered to be low due to the general lack of elevation difference slope geometry across or adjacent to the site.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the preliminary finding of this firm that construction of the proposed mixed use development is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

This report is preliminary in nature due to the lack of a well defined development plan. This report is therefore not intended for submission for building permit purposes. A comprehensive report should be prepared when the development plan achieves refinement.

Currently it is thought that the proposed development may be serviced by a subterranean level. Excavation of the proposed subterranean level will remove the unsuitable materials in the building area. All foundations may bear in native earth materials found at the level of the proposed excavation. Excavation of the proposed subterranean level will require shoring measures to provide a stable working area due to the proposed depth, the nature of the onsite soils, and the proximity of adjacent structures.

Where no subterranean level is proposed, existing fill materials and upper native soils should be removed to a minimum depth of four feet below proposed foundations and recompacted as controlled fill prior to foundation excavation. Conventional foundations bearing in newly placed controlled fill are recommended for foundation support.

Foundations for small outlying structures, such as property line walls, which will not be tied-in to the proposed mixed use development may be supported on conventional foundations bearing in native earth materials.

SEISMIC DESIGN CONSIDERATIONS

2007 California Building Code

According to Table 1613.5.2 of the 2007 California Building Code, based on the 2006 International Building Code, the subject site is classified as Site Class D , which corresponds to a "Stiff Soil" Profile. The following table outlines the Mapped Spectral Accelerations and Site Coefficients per the 2007CBC/2006IBC which may be used by the structural engineer for the seismic design and analysis of structures.

2007 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	1.664g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	1.664g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.109g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.627g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	0.940g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.627g

FILL SOILS

The maximum depth of fill encountered on the site was 2-1/2 feet. This material and any fill generated during demolition should be removed during the excavation of the subterranean level and wasted from the site. Where no subterranean level is proposed this material and any fill generated during demolition should be removed and recompacted as controlled fill prior to foundation excavation.

EXPANSIVE SOILS

The onsite earth materials are in the very low expansion range. The Expansion Index was found to be between 4 and 18 for bulk samples remolded to 90 percent of the laboratory maximum density. Special considerations are not required.

WATER-SOLUBLE SULFATES

The portland cement portion of concrete is subject to attack when exposed to water-soluble sulfates. Usually the two most common sources of exposure are from soil and marine environments.

The source of natural sulfate minerals in soils include the sulfates of calcium, magnesium, sodium, and potassium. When these minerals interact and dissolve in subsurface water, a sulfate concentration is created, which will react with exposed concrete. Over time sulfate attack will destroy improperly proportioned concrete well before the end of its intended service life.

The water-soluble sulfate content of the onsite earth materials was tested by California Test 417. The water-soluble sulfate content was determined to be less than 0.2% percentage by weight for the soils tested. Based on the 1997 Uniform Building Code, Table 19-A-4, the sulfate exposure is considered to be moderate for earth materials with less than 0.2% and Type II cement may be utilized for concrete foundations in contact with the site soils. In addition a water-cement ratio of 0.5 should be maintained in the poured concrete.

GRADING GUIDELINES

Site Preparation

All vegetation, existing fill, and soft or disturbed earth materials should be removed from the areas to receive controlled fill. The excavated areas shall be carefully observed by the geotechnical engineer prior to placing compacted fill.

Any vegetation or associated root system located within the footprint of the proposed structures should be removed during grading. Any existing or abandoned utilities located within the footprint of the proposed structures should be removed or relocated as appropriate. All existing fill materials and any disturbed earth materials resulting from grading operations should be removed and properly recompacted prior to foundation excavation.

Where no subterranean level is proposed, the proposed building areas shall be excavated to a minimum depth of four feet below the bottom of all foundations. The excavation shall extend at least five feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater. It is very important that the positions of the proposed structures are accurately located so that the limits of the graded area are accurate and the grading operation proceeds efficiently.

Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.

Compaction

All fill should be mechanically compacted in layers not more than 8 inches thick. All fill shall be compacted to at least 90 percent of the maximum laboratory density for the materials used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. using test method ASTM D 1557-02 or equivalent.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until a minimum of 90 percent compaction is obtained.

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed.

Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of relatively non-expansive soils with an expansion index of less than 20. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

Imported materials should be free from chemical or organic substances which could effect the proposed development. A competent professional should be retained in order to test imported materials and address environmental issues and organic substances which might effect the proposed development.

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with ASTM D-1557-02.

Wet Soils

At the time of exploration the soils which will be exposed at the bottom of the excavation were locally well above optimum moisture content. It is anticipated that the excavated material to be placed as compacted fill, and the materials exposed at the bottom of excavated plane will require significant drying and aeration prior to recompaction.

Pumping of the high-moisture content soils at the bottom of the excavation may occur during operation of heavy equipment. Where pumping is encountered, a layer of angular minimum ¾-inch gravel should be placed and vibrated to a dense state.

The gravel will function as a stabilization material upon which heavy equipment may operate. It is not recommended that rubber tire construction equipment attempt to operate directly on the subgrade soils prior to placing the gravel. Direct operation of rubber tire equipment on the soft subgrade soils will likely result in excessive disturbance to the soils, which in turn will result in a delay to the construction schedule since those disturbed soils would then have to be removed and properly recompacted. Extreme care should be utilized to place gravel as the subgrade becomes exposed.

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 5 and 15 percent should be anticipated when excavating and recompacting the existing fill and underlying native earth materials on the site to an average comparative compaction of 92 percent.

Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather. These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

Temporary drainage devices should be installed to collected and transfer excess water to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope.

Work may start again, after a period of rainfall, once the site has been reviewed by a representative of this office. Any soils saturated by the rain shall be removed and aerated so that the moisture content will fall within three percent of the optimum moisture content.

Surface materials previously compacted before the rain shall be scarified, brought to the proper moisture content and recompacted prior to placing additional fill, if considered necessary by a representative of this firm.

Abandoned Seepage Pits

No abandoned seepage pits were encountered during exploration and none are known to exist on the site. However, should such a structure be encountered during grading, options to permanently abandon seepage pits include complete removal and backfill of the excavation with compacted fill, or drilling out the loose materials and backfilling to within a few feet of grade with slurry, followed by a compacted fill cap.

If the subsurface structures are to be removed by grading, the entire structure should be demolished. The resulting void may be refilled with compacted soil. Concrete and brick generated during the seepage pit removal may be reused in the fill as long as all fragments are less than 6 inches in longest dimension and the debris comprises less than 15 percent of the fill by volume. All grading should comply with the recommendations of this referenced report.

Where the seepage pit structure is to be left in place, the seepage pits should cleaned of all soil and debris. This may be accomplished by drilling. The pits should be filled with minimum 1-1/2 sack concrete slurry to within 5 feet of the bottom of the proposed foundations. In order to provide a more uniform foundation condition, the remainder of the void should be filled with controlled fill.

Geotechnical Observations and Testing During Grading

Geotechncial observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by this firm during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

FOUNDATION DESIGN

Conventional

Conventional foundations may bear in newly placed controlled fill or native soils found 10 feet below site grades. All conventional foundations for a structure should bear in the same material.

Continuous foundations may be designed for a bearing capacity of 2,300 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

Column foundations may be designed for a bearing capacity of 2,600 pounds per square foot, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

The bearing capacity increase for each additional foot of width is 150 pounds per square foot. The bearing capacity increase for each additional foot of depth is 300 pounds per square foot. The maximum recommended bearing capacity is 5,000 pounds per square foot.

The bearing capacities indicated above are for the total of dead and frequently applied live loads, and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces.

Foundations bearing in controlled fill which are to be constructed adjacent to property lines and/or existing structures should be deepened, as appropriate, to bear below a 1:1 plane of foundation action projected up from the toe of the newly placed controlled fill. Foundations bearing in controlled fill which are to be constructed immediately adjacent to property lines and/or existing structures should be deepened to bear solely in native soils.

Miscellaneous Foundations

Conventional foundations for structures such as privacy walls or trash enclosures which will not be rigidly connected to the proposed the proposed mixed use structure may bear in native soils. Continuous footings may be designed for a bearing capacity of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing capacity increases are recommended.

Since the recommended bearing capacity is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

Foundation Reinforcement

All foundations should be reinforced with a minimum of two #4 steel bars. One should be placed near the top of the foundation, and one should be placed near the bottom.

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.34 may be used with the dead load forces.

Passive earth pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 300 pounds per cubic foot with a maximum earth pressure of 3,000 pounds per square foot.

When combining passive and friction for lateral resistance, the passive component should be reduced by one third. A one-third increase in the passive value may be used for wind or seismic loads.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is expected to be 1 inch and occur below the heaviest loaded columns. Differential settlement is not expected to exceed 1/4 inch.

Foundation Observations

It is critical that all foundation excavations are observed by a representative of this firm to verify penetration into the recommended bearing materials. The observation should be performed prior to the placement of reinforcement. Foundations should be deepened to extend into satisfactory earth materials, if necessary.

Foundation excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required foundation backfill should be mechanically compacted, flooding is not permitted.

RETAINING WALL DESIGN

Cantilever Retaining Walls

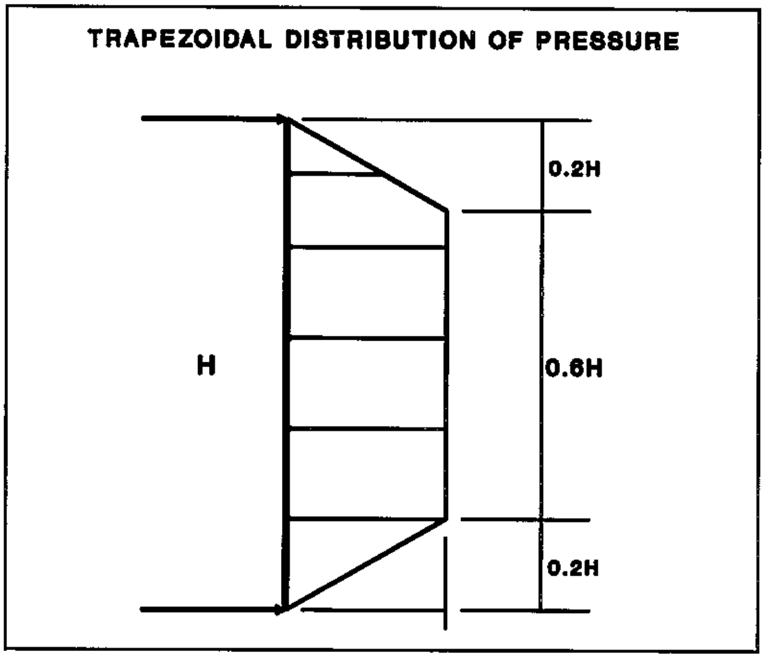
Retaining walls supporting a level backslope may be designed utilizing a triangular distribution of pressure. Retaining walls may be designed utilizing the following table:

HEIGHT OF WALL (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 12	40

For this equivalent fluid pressure to be valid, walls which are to be restrained at the top should be backfilled prior to the upper connection being made. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Restrained Retaining Walls

Restrained retaining walls may be designed to resist a trapezoidal pressure distribution of lateral earth pressure as indicated in the diagram below. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.



Design restrained walls as follows:

HEIGHT OF WALL "H" (feet)	DESIGN WALL FOR (Where H is the height of the wall)
Up to 12	26H

In addition to the recommended earth pressure, the upper ten feet of the retaining wall adjacent to streets, driveways or parking 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 walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected.

The lateral earth pressures recommended above for retaining walls assume that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. Also, where necessary, the retaining walls should be designed to accommodate any surcharge pressures that may be imposed by existing buildings on the adjacent property.

Waterproofing

Moisture effecting retaining walls is one of the most common post construction complaints. Poorly applied or omitted waterproofing can lead to efflorescense or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt. Efflorescence is common to retaining walls and does not effect their strength or integrity.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection to below grade walls.

Retaining Wall Drainage

Retaining walls should be provided with a subdrain covered with a minimum of 12 inches of gravel, and a compacted fill blanket or other seal at the surface. The onsite earth materials are acceptable for use as retaining wall backfill as long as they are compacted to a minimum of 90 percent of the maximum density as determined by ASTM D 1557-02 or equivalent.

Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location.

Where retaining walls are to be constructed adjacent to property lines there is usually not enough space for emplacement of a standard pipe and gravel drainage system. Under these circumstances, the use of a flat drainage produce is acceptable.

Some municipalities do not allow the use of flat-drainage products. The use of such a product should be researched with the building official. As an alternative, omission of one-half of a block at the back of the wall on eight foot centers is an acceptable method of draining the walls. The resulting void should be filled with gravel. A collector is placed within the gravel which directs collected waters through the wall to a sump or standard pipe and gravel system constructed under the slab. This method should be approved by the retaining wall designed prior to implementation.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent of the maximum density obtainable by the ASTM Designation D 1557-02 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Though groundwater was encountered during exploration at a depth of 29 feet below grade, the proposed subterranean level is to be serviced by the backdrainage system is only on the order of 12 feet below site grade. It is considered improbable that the ambient groundwater level would rise 17 feet during the design life of the structure to effect the retaining wall backdrainage system. Therefore the only water which could effect the proposed retaining walls would be irrigation waters and precipitation. Additionally the site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

Based on these considerations the retaining wall backdrainage system is not expected to experience an appreciable flow of water, and in particular, no groundwater will effect it. However, for the purposes of design, a flow of 5 gallons per minute may be assumed.

TEMPORARY EXCAVATIONS

Excavations on the order of 5 to 12 feet in vertical height will be required for the subterranean level or the recommended recompaction depending on the final disposition of the proposed development. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 slope gradient. A uniform sloped excavation does not have a vertical component.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads near the top of slope within a horizontal distance equal to the depth of the excavation. If the temporary construction embankments are to be maintained during the rainy season, berms are strongly recommended along the tops of the slopes to prevent runoff water from entering the excavation and eroding the slope faces. Water should not be allowed to pond on top of the excavation nor to flow towards it.

Excavation Observations

It is critical that the soils exposed in the cut slopes are observed by a representative of this office during excavation so that modifications of the slopes can be made if variations in the earth material conditions occur. Many building officials require that temporary excavations should be made during the continuous observations of the geotechnical engineer. All excavations should be stabilized within 30 days of initial excavation.

SHORING DESIGN

The following information on the design and installation of the shoring is as complete as possible at this time. It is suggested that this office review the final shoring plans and specifications prior to bidding or negotiating with a shoring contractor.

One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. The soldier piles may be designed as cantilevers or laterally braced utilizing drilled tied-back anchors or raker braces.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the earth materials. For design purposes, an allowable passive value for the earth materials below the bottom plane of excavation, may be assumed to be 600 pounds per square foot per foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed earth materials.

Groundwater was encountered during exploration at a depth of 29 feet below grade. Proposed piles are to be in excess of 29 feet in depth and will, therefore, encounter water. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 10 inches with a hopper at the top. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

A special concrete mix should be used for concrete to be placed below water. The design shall provide for concrete with a strength p.s.i. of 1,000 over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste shall be included. The slump shall be commensurate to any research report for the admixture, provided that it shall also be the minimum for a reasonable consistency for placing when water is present.

Casing may be required should caving be experienced in the granular earth materials. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn.

At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.34 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 500 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 7 feet below the bottom of excavated plane whichever is deeper.

Lagging

It is possible that lagging between soldier piles could be omitted within more cohesive earth materials where the clear spacing between soldier piles does not exceed four feet. In less cohesive earth materials, such as sands and gravels, lagging would be necessary. It is recommended that a representative of this firm observe the exposed earth materials to verify their nature and establish areas where lagging could be omitted, if any. At this time, it is expected that most of the excavation will require continuous lagging.

Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the earth materials, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot.

Tied-Back Anchors

Tied-back anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge.

Drilled friction anchors may be designed for a skin friction of 400 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. This skin friction is based on 15 foot high shoring, a tied back anchor elevation 6 feet below grade and a minimum twenty foot embedment beyond the potentially active wedge yielding an overburden of 12½ feet below ground surface. Where belled anchors are utilized, the capacity of belled anchors may be designed by applying the skin friction over the surface area of the bonded anchor shaft. The diameter of the bell may be utilized as the diameter of the bonded anchor shaft when determining the surface area. This implies that in order for the belled anchor to fail, the entire parallel soil column must also fail.

Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 2,500 pounds per square foot could be utilized for post-grouted anchors. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

Anchors should be placed at least 6 feet on center to be considered isolated. It is recommended that at least 3 of the initial anchors have their capacities tested to 200 percent of their design capacities for a 24-hour period to verify their design capacity.

The total deflection during this test should not exceed 12 inches. The anchor deflection should not exceed 0.75 inches during the 24 hour period, measured after the 200 percent load has been applied. All anchors should be tested to at least 150 percent of design load. The total deflection during this test should not exceed 12 inches.

The rate of creep under the 150 percent test load should not exceed 0.1 inch over a 15 minute period in order for the anchor to be approved for the design loading. After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. Where satisfactory tests are not attained, the anchor diameter and/or length should be increased or additional anchors installed until satisfactory test results are obtained. The installation and testing of the anchors should be observed by the geotechnical engineer. Minor caving during drilling of the anchors should be anticipated.

Anchor Installation

Tied-back anchors may be installed between 20 and 40 degrees below the horizontal. Caving of the anchor shafts, particularly within sand deposits, should be anticipated and the following provisions should be implemented in order to minimize such caving. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended 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 with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

Lateral Pressures

Cantilevered shoring supporting a level backslope may be designed utilizing a triangular distribution of pressure as indicated in the following table:

HEIGHT OF SHORING "H" (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 12	30

A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs, with the trapezoidal distribution as shown in the diagram in the 'Restrained Retaining Walls' section of this report. Restrained shoring supporting a level backslope may be designed utilizing a trapezoidal distribution of pressure as indicated in the following table:

HEIGHT OF SHORING "H" (feet)	DESIGN SHORING FOR (Where H is the height of the wall)
Up to 12	19H

Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be applied where the shoring will be surcharged by adjacent traffic or structures. Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of one inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in adjacent street and alleys. If desired to reduce the deflection, a greater active pressure could be used in the shoring design. Where internal bracing is used, the rakers should be tightly wedged to minimize deflection. The proper installation of the raker braces and the wedging will be critical to the performance of the shoring.

Monitoring

Because of the depth of the excavation, some mean of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles. Also, some means of periodically checking the load on selected anchors will be necessary, where applicable.

Some movement of the shored embankments should be anticipated as a result of the relatively deep excavation. It is recommended that photographs of the existing buildings on the adjacent properties be made during construction to record any movements for use in the event of a dispute.

Shoring Observations

It is critical that the installation of shoring is observed by a representative of this office. Many building officials require that shoring installation should be performed during continuous observation of a representative of the geotechnical engineer. The observations insure that the recommendations of the geotechnical report are implemented and so that modifications of the recommendations can be made if variations in the earth material or groundwater conditions warrant. The observations will allow for a report to be prepared on the installation of shoring for the use of the local building official, where necessary.

SLABS ON GRADE

Concrete Slabs-on Grade

Concrete floor slabs should be a minimum of 5 inches in thickness. Slabs-on-grade should be cast over undisturbed natural earth materials or properly controlled fill materials. Any earth materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent of the maximum dry density.

Outdoor concrete flatwork should be a minimum of 4 inches in thickness. Outdoor concrete flatwork should be cast over undisturbed natural earth materials or properly controlled fill materials. Any earth materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent of the maximum dry density.

Design Of Slabs That Receive Moisture-Sensitive Floor Coverings

In any areas where dampness would be objectionable, it is recommended that the floor slab should be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for concrete slabs-on-grade.

All concrete slabs-on-grade should be supported on vapor retarder. The design of the slab and the installation of the vapor retarder should comply with ASTM E 1643-98 and ASTM E 1745-97(Reapproved 2004). Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier should be covered with a thin layer of sand, to prevent punctures and aid in the concrete cure.

Concrete Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade due to settlement. However even where these recommendations have been implemented, foundations, stucco walls and concrete slabs-on-grade may display some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete cracking may be reduced and/or controlled by limiting the slump of the concrete used, proper concrete placement and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur.

For standard crack control maximum expansion joint spacing of 15 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 90 percent relative compaction.

Slab Reinforcing

Concrete slabs-on-grade should be reinforced with a minimum of #3 steel bars on 24-inch centers each way.

Outdoor flatwork should be reinforced with a minimum of #3 steel bars on 24-inch centers each way.

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 90 percent of the maximum density as determined by ASTM D 1557-02. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

Service	Asphalt Pavement Thickness Inches	Base Course Inches
Passenger Cars	3	4
Moderate Trucks	4	6

Aggregate base should be compacted to a minimum of 95 percent of the ASTM D 1557-02 laboratory maximum dry density. Base materials should conform with Sections 200-2.2 or 200-2.4 of the "Standard Specifications for Public Works Construction", (Green Book), 1991 Edition.

The performance of pavement is highly dependant upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress. If planter islands are planned, the perimeter curb should extend a minimum of 12 inches below the bottom of the aggregate base.

SITE DRAINAGE

Proper surface drainage is critical to the future performance of the project. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Proper site drainage should be maintained at all times.

All site drainage should be collected and transferred to the street in non-erosive drainage devices. The proposed structure should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within five feet of the building perimeter. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters which are located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill.

STORMWATER DISPOSAL

Recently regulatory agencies have been requiring the disposal of a certain amount of stormwater generated on a site by infiltration into the site soils. This requirement is not prudent engineering practice. Increasing the moisture content of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. This means that any overlying structure, including buildings, pavements and concrete flatwork, could sustain damage due to saturation of the subgrade soils. Structures serviced by subterranean levels could be adversely impacted by stormwater disposal by increasing the design fluid pressures on retaining walls and causing leaks in the walls. Proper site drainage is critical to the performance of any structure in the built environment.

The California Stormwater Quality Association recommends that infiltration devices should be greater than 6 meters (19-1/2 feet) from buildings, slopes and highway pavement. The Stormwater Managers Resource Center recommends that infiltration devices should be sited 25 feet downgradient of structures. This setback magnitude is not possible on most urban projects.

Where percolation of stormwater into the subgrade soils is not advisable, some Building Officials have allowed the stormwater to be filtered through soils in planter areas. Once the water has been filtered through a planter it may be released into the storm drain system. It is recommended that overflow pipes are incorporated into the design of the discharge system in the planters to prevent flooding. In addition, the planters shall be sealed and waterproofed to prevent leakage. Please be advised that adverse impact to landscaping and periodic maintenance may result due to excessive water and contaminates discharged into the planters.

It is recommended that the design team (including the structural engineer, waterproofing consultant, plumbing engineer, and landscape architect) be consulted in regards to the design and construction of filtration systems.

Please be advised that stormwater infiltration and treatment is a relatively new requirement by the various regulatory agencies and has been subject to change without notice.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

Geotechncial observations and testing during construction are considered to be a continuation of the geotechnical investigation. It is critical that this firm review the geotechnical aspects of the project during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. All foundations should be observed by a representative of this firm prior to placing concrete or steel. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

If conditions encountered during construction appear to differ from those disclosed herein, notify this office immediately so the need for modifications may be considered in a timely manner.

It is the responsibility of the contractor to ensure that all excavations and trenches are properly sloped or shored. All temporary excavations should be cut and maintained in accordance with applicable OSHA rules and regulations.

CLOSURE AND LIMITATIONS

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology and were prepared in accordance with generally accepted geotechnical engineering practice. Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

The scope of the geotechnical services provided did not include any environmental site assessment for the presence or absence of organic substances, hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere, or the presence of wetlands.

Proper compaction is necessary to reduce settlement of overlying improvements. Some settlement of compacted fill should be anticipated. Any utilities supported therein should be designed to accept differential settlement. Differential settlement should also be considered at the points of entry to the structure.

The City of Long Beach does not require corrosion testing. However, if corrosion sensitive improvements are planned, it is recommended that a comprehensive corrosion study should be commissioned. The study will develop recommendations to avoid premature corrosion of buried pipes and concrete structures in direct contact with the soils.

GEOTECHNICAL TESTING

Classification and Sampling

The soil is continuously logged by a representative of this firm and classified by visual examination in accordance with the Unified Soil Classification system. The field classification is verified in the laboratory, also in accordance with the Unified Soil Classification System. Laboratory classification may include visual examination, Atterberg Limit Tests and grain size distribution. The final classification is shown on the boring logs.

Samples of the earth materials encountered in the exploratory excavations were collected and transported to the laboratory. Undisturbed samples of soil are obtained at frequent intervals. Unless noted on the boring logs as an SPT sample, samples acquired while utilizing a hollow-stem auger drill rig are obtained by driving a thin-walled, California Modified Sampler with successive 30-inch drops of a 140-pound hammer. Samples from bucket-auger drilling are obtained utilizing a California Modified Sampler with successive 12-inch drops of a kelly bar, whose weight is noted on the boring logs. The soil is retained in brass rings of 2.50 inches inside diameter and 1.00 inches in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the boring logs as SPT samples are obtained in accordance with ASTM D 1586-99. Samples are retained for 30 days after the date of the geotechnical report.

Grain Size Distribution

These tests cover the quantitative determination of the distribution of particle sizes in soils. Sieve analysis is used to determine the grain size distribution of the soil larger than the Number 200 sieve.

ASTM D 422-63(Reapproved 2002) is used to determine particle sizes smaller than the Number 200 sieve. A hydrometer is used to determine the distribution of particle sizes by a sedimentation process.

The grain size distributions are plotted on the E-Plates presented in the Appendix of this report.

Moisture and Density Relationships

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples by ASTM D 4959-00 or ASTM D 4643-00. This information is useful in providing a gross picture of the soil consistency between exploration locations and any local variations. The dry unit weight is determined in pounds per cubic foot and shown on the "Boring Logs", A-Plates. The field moisture content is determined as a percentage of the dry unit weight.

Direct Shear Testing

Shear tests are performed by ASTM D 3080-04 with a strain controlled, direct shear machine manufactured by Soil Test, Inc. or a Direct Shear Apparatus manufactured by GeoMatic, Inc. The rate of deformation is approximately 0.025 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram," B-Plates.

Consolidation Testing

Settlement predictions of the soil's behavior under load are made on the basis of the consolidation tests ASTM D 2435-04. The consolidation apparatus is designed to receive a single one-inch high ring. Loads are applied in several increments in a geometric progression, and the resulting deformations are recorded at selected time intervals. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore fluid. Samples are generally tested at increased moisture content to determine the effects of water on the bearing soil. The normal pressure at which the water is added is noted on the drawing. Results are plotted on the "Consolidation Test," C-Plates.

Expansion Index Testing

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the ASTM D4829-03. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hour or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined by use of ASTM D 1557-02. A soil at a selected moisture content is placed in five layers into as mold of given dimensions, with each layer compacted by 25 blows of a 10 pound hammer dropped from a distance of 18 inches subjecting the soil to a total compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted, represent a curvilinear relationship know as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve.

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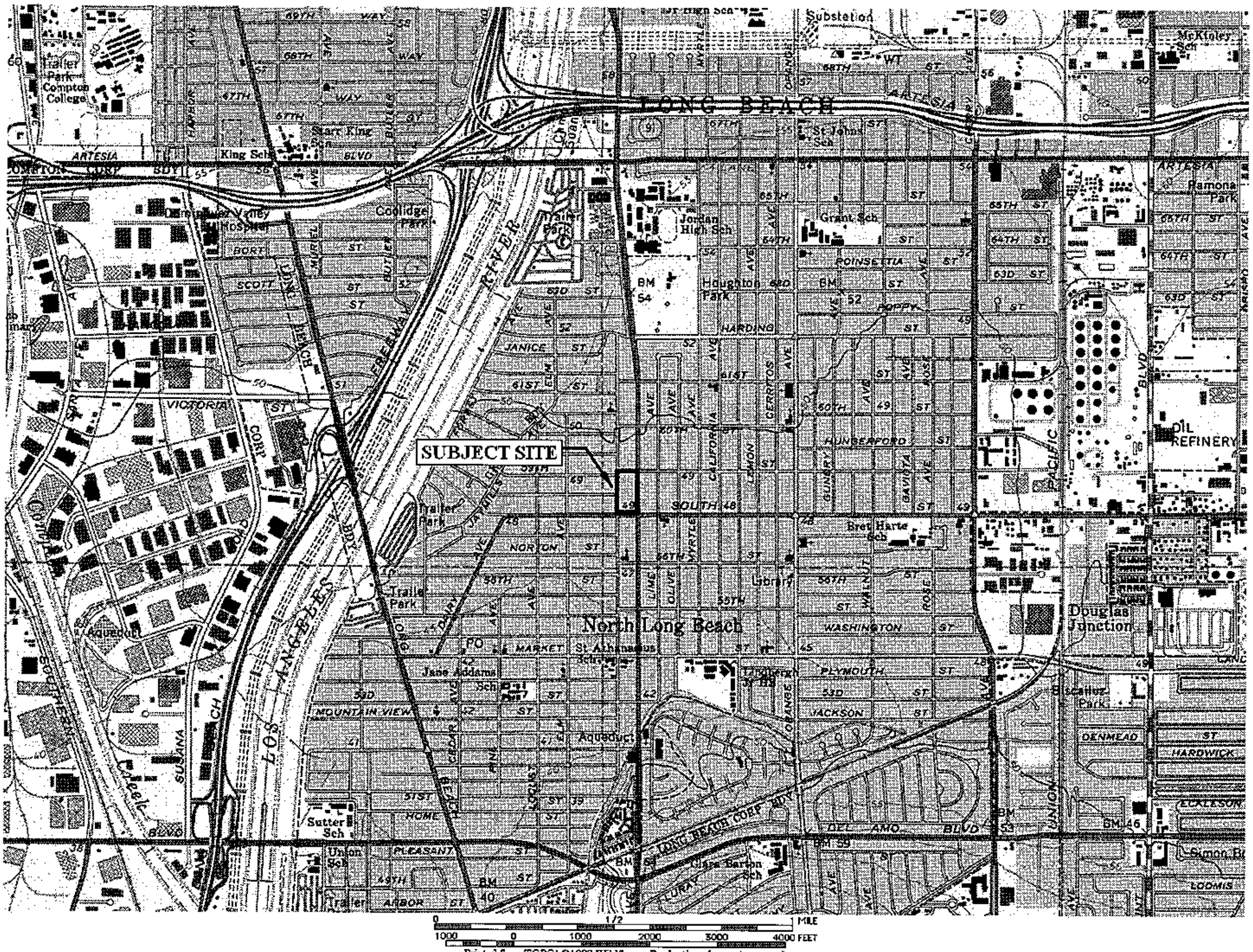

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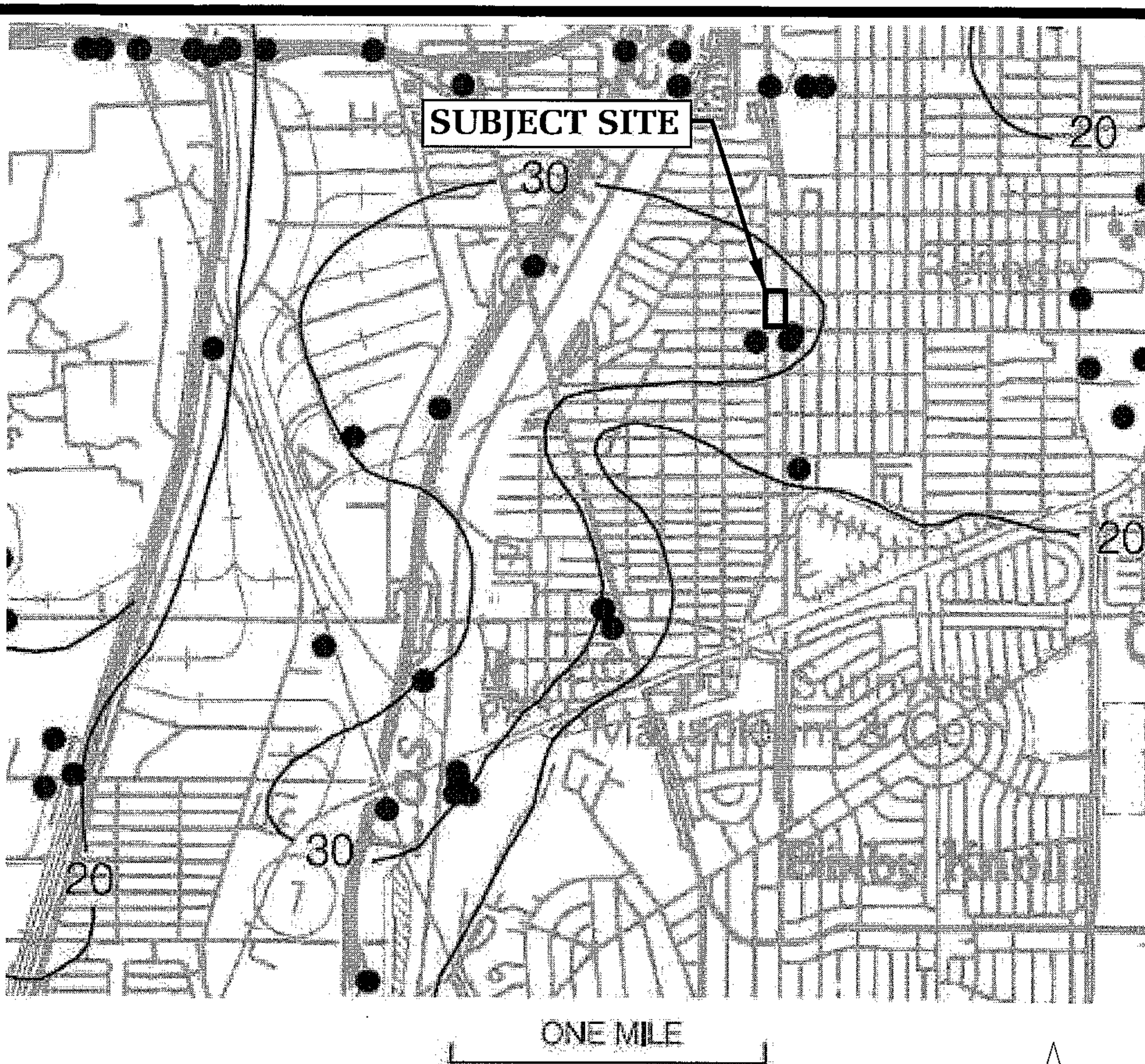
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, LONG BEACH, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

AMCAL MULTI HOUSING, INC.

FILE NO. 19593



20 Depth to groundwater in feet

REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 028
LONG BEACH 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

Geotechnologies, Inc.
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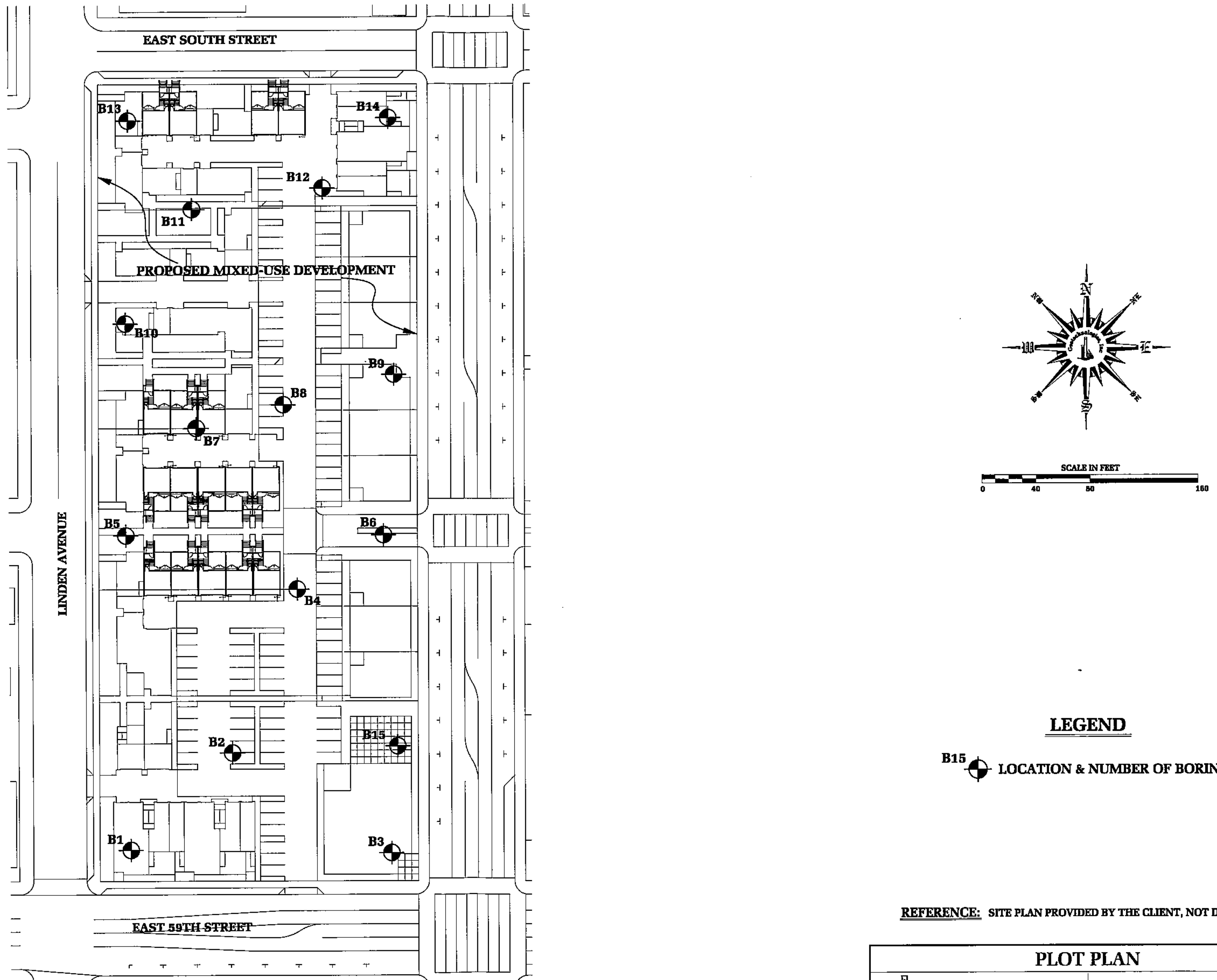
AMCAL MULTI HOUSING, INC.

FILE No. 19593



LEGEND

B15 LOCATION & NUMBER OF BORING

REFERENCE: SITE PLAN PROVIDED BY THE CLIENT, NOT DATED

PLOT PLAN		
Geotechnologies, Inc. Consulting Geotechnical Engineers	AMCAL MULTI HOUSING, INC.	
	FILE No. 19593	DRAWN BY: BA
	DATE: March 08'	SHEET: 1 of 1

BORING LOG NUMBER 1

Drilling Date: 01/25/08

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				-		
1	22	13.6	96.5	1 --		
				-	SM	Silty Sand, yellowish-brown, moist, medium dense, fine grained
				2 --		
				-		
3	14	15.7	89.0	3 --		
				-		Silty Sand, yellow and dark brown, moist, medium dense, fine grained
				4 --		
				-		
5	14	8.4	96.3	5 --		
				-		
				6 --		
				-		
7	12	26.7	90.7	7 --		
				-	ML	Sandy to Clayey Silt, dark brown, moist, firm, slightly porous
				8 --		
				-		
				9 --		
				-		
10	26	21.0	101.6	10 --		
				-	SM	Silty Sand, dark and yellowish-brown, moist, medium dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	21	26.1	101.0	15 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark and yellowish-brown, moist, medium dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	28	26.9	97.8	20 --		
				-	ML	Sandy Silt, dark and yellowish-brown, moist, stiff
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	39	30.1	90.5	25 --		
				-		

BORING LOG NUMBER 1

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	30	26.9	97.4	30 --		
				-		Total depth: 30 feet
				31 --		No Water
				-		Fill to 1 foot
				32 --		
				-		
				33 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual
				-		
				34 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				35 --		140-lb. Slide Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				36 --		
				-		SPT=Standard Penetration Test
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 2

Drilling Date: 01/25/08

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 6-inch Asphalt over 2-inch Base
				0 --		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				1 --		
				-		
2	12	18.4	No Recovery	2 --		
				-	SM	Silty Sand, grayish-brown, moist, medium dense, fine grained
				3 --		
				-		
				4 --		
				-		
5	10	7.5	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	20	22.4	94.5	-		
				8 --	ML	Sandy to Clayey Silt, gray and yellowish-brown mottling, moist, fine grained
				-		
				9 --		
				-		
10	12	22.6	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	26	20.2	104.7	-		
				13 --		
				-		
				14 --		
				-		
15	17	10.6	SPT	15 --		
				-	SM/ML	Silty Sand to Sandy Silt, yellow and grayish-brown mottling, moist, firm to medium dense, fine grained
				16 --		
				-		
				17 --		
17.5	28	30.1	91.8	-		
				18 --	CL/SM	Sandy Clay to Silty Sand, yellowish and grayish-brown mottling, moist, stiff to medium dense, fine grained
				-		
				19 --		
				-		
20	20	25.8	SPT	20 --		
				-	ML	Sandy to Clayey Silt, yellow and grayish-brown mottling, moist, stiff
				21 --		
				-		
				22 --		
22.5	28	15.2	107.2	-		
				23 --	SM	Silty Sand, yellow and grayish-brown mottling, moist, medium dense, fine grained
				-		
				24 --		
				-		
25	17	27.6	SPT	25 --		
				-		

BORING LOG NUMBER 2

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --	ML	Sandy to Clayey Silt, gray and yellowish-brown mottling, moist, fine grained
27.5	23	30.5	88.9	-		
				28 --	ML/CL	Clayey Silt to Silty Clay, gray to dark gray
				29 --		
30	20	30.7	SPT	30 --		
				31 --	SM/ML	Silty Sand interbedded with Sandy Silt, yellow and grayish-brown mottling, moist, medium dense, fine grained and stiff
				32 --		
32.5	26	19.0	112.4	-		
				33 --	CL	Sandy to Silty Clay, gray to dark gray, moist, stiff
				34 --		
35	16	13.6	SPT	35 --		
				36 --		
				37 --		
37.5	56	10.6	126.7	-		
				38 --	SP	Sand, gray, wet, dense, fine to medium grained, minor gravel
				39 --		
40	66	11.0	SPT	40 --		
				-		Sand, yellowish-brown, moist, dense, fine grained, minor gravel
				41 --		
				42 --		
42.5	38 50/3"	9.5	126.0	-		
				43 --	SP/SW	Sand with Gravel, yellowish-brown, wet, very dense, fine to coarse grained
				44 --		
45	86	8.7	SPT	45 --		
				46 --		
				47 --		
47.5	40 50/4"	19.0	109.1	-		
				48 --	SP	Sand, yellow to olive-brown, wet, very dense, fine grained
				49 --		
50	41	33.8	SPT	50 --		
				-	CL	Silty Clay, yellowish-brown, moist, stiff

BORING LOG NUMBER 2

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
				52	SM/SP	Silty Sand to Sand, yellowish-brown, wet, dense, fine grained
52.5	39 50/5"	6.0	124.3	53	SP/SW	Sand with Gravel, yellowish-brown, wet, very dense, fine to coarse grained
				54		
55	88	35.6	SPT	55	ML	Sandy to Clayey Silt, yellow and reddish-brown, moist, very stiff
				56	SP	Sand, yellowish-brown, wet, very dense, fine to medium grained
				57		
57.5	89	7.0	124.1	58		
				59		
60	56	11.9	SPT	60		
				61		Total depth: 60 feet Water at 37 feet Fill to 2 feet
				62		
				63		
				64		
				65		
				66		
				67		
				68		
				69		
				70		
				71		
				72		
				73		
				74		
				75		

BORING LOG NUMBER 3

Drilling Date: 01/25/08

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Silty Sand, dark to yellowish-brown, moist, medium dense, fine grained
				-		
				1 --		
				-		
2	24	11.9	100.6	2 --		
				-	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				3 --		
				-		
4	15	11.1	91.3	4 --		
				-		
				5 --		
				-		
				6 --		
				-		
7	12	29.5	85.9	7 --		
				-	CL	Sandy to Silty Clay, dark brown, moist, firm
				8 --		
				-		
				9 --		
				-		
10	25	18.5	95.2	10 --		
				-	SM	Silty Sand, yellowish-brown, moist, dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	39	13.2	102.0	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	22	34.0	91.7	20 --		
				-		
				21 --		
				-	CL	Sandy to Silty Clay, dark and yellowish-brown mottling, moist, stiff
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	24	19.2	92.9	25 --		
				-		Sandy to Silty Clay, gray and yellowish-brown mottling, moist, stiff

BORING LOG NUMBER 3

Project: File No. 19593 — **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	29	23.6	99.8	30	ML/SM	Sandy Silt to Silty Sand, yellow and grayish-brown mottling, moist, stiff to medium dense, fine grained
				31		
				32		Total depth: 30 feet No Water Fill to 2 feet
				33		
				34		
				35		
				36		
				37		
				38		
				39		
				40		
				41		
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 4

Drilling Date: 02/01/08

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 6-inch Asphalt, No Base
				0 --		FILL: Silty Sand, yellowish-brown, moist, medium dense, fine grained
				-		
1	13	11.1	88.8	1 --		
				-		asphalt and gravel, moist
				2 --		
				-	SM	Silty Sand, yellowish-brown, moist, medium dense, fine grained
3	16	24.8	93.2	3 --		
				-	ML/SM	Sandy Silt to Silty Sand, yellowish-brown, moist, medium dense, fine grained
				4 --		
				-		
5	26	12.5	87.0	5 --		
				-	SM	Silty Sand, yellowish-brown, moist, medium dense, fine grained
				6 --		
				-		
7	38	25.2	91.8	7 --		
				-		light grayish-brown to yellowish-brown, slightly porous, moist
				8 --		
				-		
				9 --		
				-		
10	57	21.2	103.3	10 --		
				-		yellowish-brown with gray mottling, slightly porous, moist
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	53	31.1	92.4	15 --		
				-	ML/CL	Sandy Silt to Silty Clay, yellowish-brown with gray mottling, moist, medium dense, fine grained, firm
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	50	23.2	103.0	20 --		
				-	SM	Silty Sand, light gray with reddish-orange mottling, moist, medium dense, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	76	26.3	97.2	25 --		
				-	CL	Silty Clay, gray with yellowish-brown mottling, moist, stiff

BORING LOG NUMBER 4

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	60	25.0	102.5	30 --	SC/SM	Clayey Sand to Silty Sand, gray, moist, dense, fine grained, stiff
				-		
				31 --		Total depth: 30 feet
				-		No Water
				32 --		Fill to 1½ feet
				-		
				33 --		
				-		
				34 --		
				-		
				35 --		
				-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 5

Drilling Date: 02/01/08

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Silty Sand, yellowish-brown, moist, medium dense, fine grained
				1 --		
2	74	6.1	98.1	2 --		moist
				3 --	SM	Silty Sand, yellowish-brown, moist, dense, fine grained
4	83	7.4	92.5	4 --		slightly porous, moist, very dense, fine grained
				5 --		
				6 --		
7	46 50/6"	21.2	100.5	7 --		yellowish-brown with gray mottling, moist, dense, fine grained
				8 --		
				9 --		
10	64	12.1	93.9	10 --	SM/SP	Silty Sand to Sand, yellowish-brown with gray mottling, slightly porous, moist, dense, fine grained
				11 --		
				12 --		
				13 --		
				14 --		
15	80	12.4	92.4	15 --	SM/ML	Silty Sand to Sandy Silt, yellowish-brown with light gray mottling, moist, very dense, fine grained
				16 --		
				17 --		
				18 --		
				19 --		
20	58	23.9	101.8	20 --	SC	Clayey Sand, yellowish-brown with gray mottling, moist, dense, fine grained, stiff
				21 --		
				22 --		
				23 --		
				24 --		
25	76 50/5"	27.5	98.3	25 --	SM	Silty Sand, gray with yellowish-brown mottling, moist, very dense, fine grained

BORING LOG NUMBER 5

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	53	21.4	110.6	30	ML/CL	Sandy Silt to Sandy Clay, gray, slightly porous, moist, medium dense, fine grained, firm
				31		
				32		Total depth: 30 feet No Water Fill to 2½ feet
				33		
				34		
				35		
				36		
				37		
				38		
				39		
				40		
				41		
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 6

Drilling Date: 02/01/08

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0		FILL: Silty Sand, yellowish-brown, moist, medium dense, fine grained
				1		
2	35	16.8	97.8	2		moist
				3	SM	Silty Sand, yellowish-brown with gray mottling, moist, medium dense, fine grained
4	43	9.8	95.2	4		
				5	SM/SP	Silty Sand to Sand, light grayish-brown to yellowish-brown, slightly porous, moist, medium dense, fine grained
				6		
7	35	26.3	93.9	7	ML	Clayey Silt, light grayish-brown with yellowish-brown mottling, moist, firm
				8		
				9		
10	47	9.7	107.3	10	SP/SC	Sand to Clayey Sand, yellowish-brown with gray mottling, moist, medium dense, fine grained, medium stiff, firm
				11		
				12		
				13		
				14		
15	42	27.3	92.7	15	ML/CL	Sandy Silt to Silty Clay, yellowish-brown with gray mottling, moist, medium dense, fine grained, firm
				16		
				17		
				18		
				19		
20	67	25.9	95.4	20	SM	Silty Sand, yellowish-brown with gray mottling, moist, dense, fine grained
				21		
				22		
				23		
				24		
25	73	30.0	91.7	25	CL/ML	Silty Clay to Clayey Silt, light greenish-gray to gray, moist, stiff

BORING LOG NUMBER 6

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	66	19.0	110.9	30 --	SM/CL	Silty Sand to Sandy Clay, gray, moist, dense, fine grained, stiff
				-		
				31 --		Total depth: 30 feet
				-		No Water
				32 --		Fill to 2½ feet
				-		
				33 --		
				-		
				34 --		
				-		
				35 --		
				-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 7

Drilling Date: 02/01/08

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0		FILL: Silty Sand, light gray, moist, medium dense, fine grained
1	45	12.7	101.1	1		
				2	SP	Sand, yellowish-brown, moist, medium dense, fine grained
3	25	17.4	86.5	3		
				4	SM	Silty Sand, light grayish-brown to yellowish-brown, porous, moist, medium dense, fine grained
5	36	29.2	89.9	5		
				6	SP/ML	Sandy to Clayey Silt, yellowish-brown with light gray mottling, moist, medium dense, fine grained, firm
7	35	25.4	96.3	7		
				8	ML	Clayey Silt, gray with yellowish-brown mottling, slightly porous, moist, firm
				9		
10	45	20.8	102.7	10		
				11		Clayey to Sandy Silt, gray with yellowish-brown mottling, moist, medium dense, fine grained, firm
				12		
				13		
				14		
15	88	23.1	91.8	15		
				16	SM/ML	Silty Sand to Sandy Silt, yellowish-brown with light gray mottling, slightly porous, moist, very dense, fine grained
				17		
				18		
				19		
20	40	19.0	107.2	20		
				21	SC/SM	Clayey Sand to Silty Sand, yellowish-brown with gray mottling, moist, dense, fine grained, stiff
				22		
				23		
				24		
25	77	33.9	91.9	25	SM/CL	Silty Sand to Silty Clay, medium brown with gray mottling to dark gray, moist, dense, fine grained, stiff

BORING LOG NUMBER 7

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
				28 --		
				29 --		
30	45 50/6"	18.0	111.5	30 --	SC/CL	Clayey Sand to Sandy Clay, gray, moist, dense, fine grained, stiff
				31 --		Total depth: 30 feet No Water Fill to 1 foot
				32 --		
				33 --		
				34 --		
				35 --		
				36 --		
				37 --		
				38 --		
				39 --		
				40 --		
				41 --		
				42 --		
				43 --		
				44 --		
				45 --		
				46 --		
				47 --		
				48 --		
				49 --		
				50 --		

BORING LOG NUMBER 8

Drilling Date: 01/28/08

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt Area
				0 --		FILL: Silty Sand, yellowish-brown, moist, medium dense, fine grained
				-		
				1 --		
				-	SP	Sand, light gray, moist, medium dense, fine grained
2	19	25.0	85.0	2 --		
				-	ML/SM	Sandy Silt to Silty Sand, olive-brown with gray mottling to olive-brown, moist, medium dense, fine grained
				3 --		
				-		
4	14	8.1	86.6	4 --		
				-	SP	Sand, yellowish-brown with light gray mottling, moist, medium dense, fine grained
				5 --		
				-		
				6 --		
				-		
7	33	22.4	104.2	7 --		
				-	SC	Clayey Sand, olive-brown, moist, medium dense, fine grained, medium stiff
				8 --		
				-		
				9 --		
				-		
10	43	22.5	100.7	10 --		
				-	SC/SM	Clayey to Silty Sand, yellowish-brown with light gray mottling, moist, medium dense, fine grained, medium stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	38	26.8	94.6	15 --		
				-	ML	Clayey Silt, yellowish-brown with light gray mottling, moist, medium stiff, some gravel
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	33	24.5	99.3	20 --		
				-	SM/CL	Silty Sand to Sandy Clay, yellowish-brown with light gray mottling, slightly porous, moist, medium dense, fine grained, medium stiff
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	35	31.2	93.6	25 --		Sand to Sandy Silt, gray with yellowish-brown mottling, moist, medium dense, fine grained
				-	SP/ML	

BORING LOG NUMBER 8

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
				28 --		
				29 --		
30	30	16.6	114.8	30 --	CL	Sandy Clay, gray to gray with yellowish-brown mottling, moist, medium stiff
				31 --		Total depth: 30 feet No Water Fill to 1 foot
				32 --		
				33 --		
				34 --		
				35 --		
				36 --		
				37 --		
				38 --		
				39 --		
				40 --		
				41 --		
				42 --		
				43 --		
				44 --		
				45 --		
				46 --		
				47 --		
				48 --		
				49 --		
				50 --		

BORING LOG NUMBER 9

Drilling Date: 01/28/08

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 3-inch Asphalt over 2-inch Base
				0 --		FILL: Silty Sand, yellowish-brown, moist, medium dense, fine grained
1	13	27.4	91.8	1 --		
				2 --	ML/SM	Clayey Silt to Silty Sand, olive-brown with light gray mottling, moist, medium dense, fine grained, medium stiff'
3	12	24.8	80.2	3 --		
				4 --	ML	Sandy Silt, yellowish-brown with light gray mottling, moist, medium dense, fine grained
5	31	23.0	86.1	5 --		
				6 --		olive-gray to light gray with yellowish-brown mottling, moist, some gravel
7	37	21.2	101.5	7 --		
				8 --	ML/SM	Clayey Silt to Silty Sand, light gray with medium brown mottling, moist, medium dense, fine grained, medium stiff'
				9 --		
10	34	19.8	105.0	10 --		
				11 --	SP/SC	Sand with Clay to Clayey Sand, yellowish-brown with gray mottling, moist, medium dense, fine grained, medium stiff
				12 --		
				13 --		
				14 --		
15	48	31.4	92.6	15 --		
				16 --	ML/CL	Clayey Silt to Silty Clay, yellowish-brown with light gray mottling, moist, medium stiff
				17 --		
				18 --		
				19 --		
20	45	20.8	106.3	20 --		
				21 --	SC/SM	Clayey Sand to Silty Sand, light gray, moist, medium dense, fine grained, medium stiff
				22 --		
				23 --		
				24 --		
25	34	35.4	90.0	25 --		
				-	CL	Silty Clay, gray to dark gray, moist, medium stiff

BORING LOG NUMBER 9

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	31	18.1	112.2	30		Sandy Clay, gray, moist, medium stiff
				31		Total depth: 30 feet No Water Fill to 1 foot
				32		
				33		
				34		
				35		
				36		
				37		
				38		
				39		
				40		
				41		
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 10

Drilling Date: 01/28/08

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Silty Sand, yellowish-brown, moist, medium dense, fine grained
				-		
				1 --		
				-	SM/SP	Silty Sand to Sand, yellowish-brown, moist, medium dense, fine grained
2	20	4.2	97.5	2 --		
				-	SP	Sand, yellowish-brown to light gray, moist, medium dense, fine grained
				3 --		
				-		
4	32	5.9	97.4	4 --		
				-	SP/SM	Sand to Silty Sand, yellowish-brown, moist, medium dense, fine grained
				5 --		
				-		
				6 --		
				-		
7	38	16.6	92.5	7 --		
				-	ML	Clayey to Sandy Silt, gray to gray with yellowish-brown mottling, moist, medium dense, fine grained, medium stiff
				8 --		
				-		
				9 --		
				-		
10	50	15.6	100.5	10 --		
				-	SM/SP	Silty Sand, light gray with yellowish-brown mottling, moist, medium dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	49	25.7	93.7	15 --		
				-	SM/ML	Silty Sand to Clayey Silt, light gray with yellowish-brown mottling, slightly porous, moist, medium dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	42	18.1	98.1	20 --		
				-	SM	Silty Sand, yellowish-brown with gray mottling, slightly porous, moist, medium dense, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	39	32.5	89.3	25 --	SM/ML	Silty Sand to Clayey Silt, olive-gray with yellowish-brown mottling to gray, moist, medium dense, fine grained, medium stiff
				-		

BORING LOG NUMBER 10

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	41	17.5	111.2	30 --	ML/SC	Sandy Silt to Clayey Sand, olive-gray, moist, medium dense, fine grained, medium stiff
				-		
				31 --		
				-		Total depth: 30 feet
				32 --		No Water
				-		Fill to 1 foot
				33 --		
				-		
				34 --		
				-		
				35 --		
				-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 11

Drilling Date: 02/01/08

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0		FILL: Silty Sand, yellowish-brown, moist, medium dense, fine grained
				1		
2	65	4.6	95.2	2	SP	Sand, light gray, moist, medium dense, fine grained
				3	SP/ML	Sand to Sandy Silt, light gray with yellowish-brown mottling, moist, dense, fine grained
4	25	10.5	80.8	4		
					SM	Silty Sand, light gray with olive-gray mottling, moist, medium dense, fine grained
				5		
				6		
7	37	27.2	93.4	7		
					ML	Clayey Silt, gray with yellowish-brown mottling, moist, firm
				8		
				9		
10	38	28.0	95.8	10		
						moist
				11		
					SM	Silty Sand, gray with yellowish-brown mottling, moist, medium dense, fine grained
				12		
				13		
				14		
15	60	19.7	105.2	15		
						light gray to light gray with yellowish-brown mottling, moist, dense, fine grained
				16		
				17		
				18		
				19		
20	35	29.7	91.8	20		
					CL/ML	Silty Clay to Clayey Silt, gray, moist, firm
				21		
				22		
				23		
				24		
25	25 50/6"	18.6	104.9	25	SM/SP	Silty Sand to Sand, greenish-gray to gray, slightly porous, moist, dense, fine grained

BORING LOG NUMBER 11

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	47	19.6	110.6	30	ML/SC	Sandy Silt to Clayey Sand, gray to gray with yellowish-brown mottling, moist, dense, fine grained, firm
				31		Total depth: 30 feet No Water Fill to 1½ feet
				32		
				33		
				34		
				35		
				36		
				37		
				38		
				39		
				40		
				41		
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 12

Drilling Date: 01/28/08

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0		FILL: Silty Sand, grayish-brown, moist, medium dense, fine grained
				1		
					SP	Sand, light grayish-brown, moist, medium dense, fine grained
2	20	3.4	97.3	2		
						light gray, moist
				3		
4	28	2.9	96.4	4		
						moist
				5		
				6		
7	33	18.2	93.1	7		
					ML/CL	Sandy Silt to Silty Clay, yellowish-brown to olive-brown with light gray mottling, moist, medium dense, fine grained, medium stiff
				8		
				9		
10	53	26.1	98.2	10		
					SM/CL	Silty Sand to Sandy Clay, gray with yellowish-brown mottling to olive-gray, slightly porous, moist, medium dense, fine grained, medium stiff
				11		
				12		
				13		
				14		
15	51	21.0	94.8	15		
					ML	Sandy to Clayey Silt, light gray with yellowish-brown mottling, moist, medium dense, fine grained, medium stiff
				16		
				17		
				18		
				19		
20	44	16.4	109.4	20		
					SM	Silty Sand, light grayish-brown with reddish-orange mottling, moist, medium dense, fine grained
				21		
				22		
				23		
				24		
25	37	24.0	96.8	25		
					SP	Sand, gray, moist, medium dense, fine grained

BORING LOG NUMBER 12

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
				28 --		
				29 --		
30	30	19.7	109.1	30 --	CL	Sandy Clay, gray with yellowish-brown mottling, moist, medium stiff
				31 --		Total depth: 30 feet No Water
				32 --		Fill to 1 foot
				33 --		
				34 --		
				35 --		
				36 --		
				37 --		
				38 --		
				39 --		
				40 --		
				41 --		
				42 --		
				43 --		
				44 --		
				45 --		
				46 --		
				47 --		
				48 --		
				49 --		
				50 --		

BORING LOG NUMBER 13

Drilling Date: 02/01/08

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0		FILL: Silty Sand, light grayish-brown, moist, medium dense, fine grained
1	32	9.7	85.4	1		Sand, light gray, moist, medium dense, fine grained
				2	SP	Sand, light gray, moist, medium dense, fine grained
3	18	4.8	81.6	3		moist
				4		
5	30	16.0	91.0	5	SP/CL	Sandy to Silty Clay, light gray to gray, moist, medium dense, fine grained, firm
				6		
7	44	23.2	99.4	7	ML	Clayey Silt, light gray with yellowish-brown mottling, slightly porous, moist, medium dense, fine grained, firm
				8		
				9		
10	68	14.0	96.9	10	SM	Silty Sand, yellowish-brown with gray mottling, moist, dense, fine grained
				11		
				12		
				13		
				14		
15	65	17.6	108.8	15		slightly Clayey, gray with yellowish-brown mottling, slightly porous, moist, dense, fine grained
				16		
				17		
				18		
				19		
20	50	28.7	94.0	20	ML	Clayey Silt, gray, slightly porous, moist, firm
				21		
				22		
				23		
				24		
25	25 50/5"	3.4	96.8	25	SP	Sand, light gray to gray, moist, very dense, fine grained

GEOTECHNOLOGIES, INC.

BORING LOG NUMBER 13

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
						gray, very moist
30	25 50/5"	24.0	98.6	30		
						Total depth: 30 feet No Water Fill to 2 feet
				31		
				32		
				33		
				34		
				35		
				36		
				37		
				38		
				39		
				40		
				41		
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 14

Drilling Date: 01/28/08

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Silty Sand, yellowish-brown, moist, medium dense, fine grained
1	22	13.0	102.1	1 --		
				2 --	SM/SP	Silty Sand to Sand, light grayish-brown to olive-gray, moist, medium dense, fine grained
3	27	4.8	96.7	3 --		
				4 --	SP	Sand, yellowish-brown, moist, medium dense, fine grained
5	31	28.6	92.5	5 --		
				6 --	CL/ML	Silty Clay to Sandy Silt, light gray with yellowish-brown mottling, moist, medium dense, fine grained, medium stiff
7	41	30.2	89.3	7 --		
				8 --	SC	Clayey Sand, gray, moist
				9 --		
10	39	17.9	101.0	10 --		
				11 --	SM/SC	Silty to Clayey Sand, gray with reddish-orange mottling, slightly porous, moist, medium dense, fine grained, medium stiff
				12 --		
				13 --		
				14 --		
15	42	21.6	100.7	15 --		
				16 --	SM	Silty Sand, yellowish-brown with light gray mottling, moist, medium dense, fine grained
				17 --		
				18 --		
				19 --		
20	47	20.4	979.0	20 --		
				21 --		gray with reddish-orange mottling to yellowish-orange, moist
				22 --		
				23 --		
				24 --		
25	32	19.8	92.8	25 --		
				-	SP	Sand, gray, moist, medium dense, fine grained

BORING LOG NUMBER 14

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
						wet, medium dense, fine to coarse grained
30	34	26.4	100.1	30		
						Total depth: 30 feet Water at 29 feet Fill to 1 foot
				31		
				32		
				33		
				34		
				35		
				36		
				37		
				38		
				39		
				40		
				41		
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 15

Drilling Date: 01/28/08

Project: File No. 19593 **AMCAL Multi-Housing**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 3-inch Asphalt, No Base
				0 --		FILL: Silty Sand, yellowish-brown, moist, medium dense, fine grained
				-		
1	27	15.4	88.4	1 --		
				-		Silty Clay, medium brown, moist, medium stiff
				2 --		
				-	SP/ML	Sand to Sandy Silt, light gray to yellowish-brown, moist, medium dense, fine grained
3	24	17.9	93.3	3 --		
				-		
				4 --	ML	Sandy Silt, medium brown with light gray mottling, porous, moist, medium dense, fine grained
				-		
5	42	23.1	93.0	5 --		
				-		Clayey Silt, olive-brown, moist, medium stiff
				6 --		
				-		
7	49	23.4	100.2	7 --		
				-		olive-brown with light gray mottling, slightly porous, moist
				8 --		
				-		
				9 --		
				-		
10	54	13.4	97.0	10 --		
				-	SM	Silty Sand, light gray with yellowish-brown mottling, moist, medium dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	42	26.4	94.6	15 --		
				-	SM/CL	Silty Sand to Silty Clay, light gray with yellowish-brown mottling, moist, medium dense, fine grained, medium stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	45	21.9	107.1	20 --		
				-	SC/SM	Clayey Sand to Silty Sand, gray with medium brown mottling, medium dense, fine grained, medium stiff
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	45	36.4	86.3	25 --		Clayey Silt to Silty Clay, greenish-gray with yellowish-brown mottling, moist, medium stiff
				-	ML/CL	