APPENDIX D
GEOTECHNICAL INVESTIGATION

APPENDIX D-1
PRELIMINARY GEOTECHNICAL INVESTIGATION

GEOTECHNICAL INVESTIGATION
9-Story Condominium Complex
239 Serramonte Boulevard
Daly City, California

Prepared for:
Serramonte Terraces, LLC
888 Brannan Street, #153-155
San Francisco, California 94103

Attention: Mr. Henry Lam

December 22, 2014
Job 1130.16.00

Earth Investigations Consultants, Inc.
P.O. Box 795
Pacifica, California 94044
Phone 650-557-0262
Fax 650-557-0264
earthinvestigations@comcast.net

Earth Investigations Consultants

December 22, 2014
Job 1130.16.00

Serramonte Terraces, LLC
888 Brannan Street, #153-155
San Francisco, California 94103

Attention: Mr. Henry Lam

RE: GEOTECHNICAL INVESTIGATION
9-Story Condominium Complex
239 Serramonte Boulevard
Daly City, California

Dear Mr. Lam:

Pursuant to your authorization, we have completed the referenced project. We find the proposed project feasible from a geotechnical standpoint. Please refer to the attached report for details of our investigation, conclusions, and design-level recommendations for the proposed project.

We trust that this provides you with the information you require at this time. Please contact our office if you have any questions.

Sincerely,

Earth Investigations Consultants, Inc.

Joel E. Baldwin, II
Engineering Geologist 1132

REGISTERED GEOLOGIST
JOEL E. BALDWIN, II
No. EG 1132
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

David W. Buckley
Civil Engineer 34386

REGISTERED PROFESSIONAL ENGINEER
DAVID W. BUCKLEY
C. 34386
Exp. 9/30/15
CIVIL
STATE OF CALIFORNIA

JEB:DWB:jb:gi
Distribution: efile and 5 bound copies to addressee

Geologists & Engineers
P.O. Box 795 ● Pacifica, CA 94044 ● (650) 557-0262 ● Fax (650) 557-0264 ● earthinvestigations@comcast.net

TABLE OF CONTENTS

	Page
COVER LETTER	i
TABLE OF CONTENTS	ii
INTRODUCTION	1
Site Location and Proposed Project	1
Purpose and Scope of Services	1
Previous Work Summary	3
FINDINGS	9
Historic and Current Use of the Site	9
Topography and Drainage	10
Surficial Soils	11
Rocks and Structure	13
Ground Water	16
Landslides and Erosion	17
Faults and Seismicity	18
DISCUSSION AND CONCLUSIONS	20
RECOMMENDATIONS	22
Seismic Design	22
Site Preparation, Grading and Compaction	22
Shoring and Underpinning	24
Utility Trenches	25
Foundations	25
Mat Slab	25
Drilled Piers	26
Footings	26
Retaining Walls	27
Cantilever Conditions	27
Restrained Conditions	29
Ground-Level Exterior Slabs-on-Grade (sidewalks and courtyard patios)	29
Drive Isle and Parking Pavements	30
Earth Flow Mitigation	31
Drainage	31
Erosion Control	32
MAINTENANCE	32
INVESTIGATION LIMITATIONS	32

INVESTIGATION LIMITATIONS 32

SUPPLEMENTAL SERVICES 34

REFERENCES 34

AERIAL PHOTOGRAPHS 39

LIST OF ILLUSTRATIONS

Following Page

Frontispiece – Southwesterly oblique conceptual view of proposed project iv

Figures

Geotechnical Data Source Index Map 3
1. Paleogeographic map of the San Francisco Bay area... 13

Plates Follow Report Text

1. Vicinity Map
2. Site Plan
3A. 1956 Site Setting
3B. 1968 Site Setting
4. Geologic Map
5. Fault Activity map
6. Fault Map
7. Earthquake Probability Map
8. Generalized Cross Sections A-A', B-B', C-C', D-D' and E-E'
9. Mitigation Plan
10. Mitigation Details

Appendix A – Logs of Borings
Following Plates

A1. Log of Boring 1 (2 pgs.)
A2. Log of Boring 2 (2 pgs.)
A3. Log of Boring 3 (2 pgs.)
A4. Log of Boring 4 (2 pgs.
A5. Key to Borings (
A6. Rock Hardness Criteria

Appendix B – Laboratory Test Results

B1. Moisture Content and Dry Density Test
B2. Plasticity Test
B3. Triaxial Shear Test

Appendix C - Kleinfelder (2002) Appendix A – Logs of Borings & Appendix B – Laboratory Test Results

A1. Boring Log Legend
A2. Log of Boring B-1 (2 pgs.)
A3. Log of Boring B-2
A4. Log of Boring B-3 (2 pgs.)
A5. Log of Boring B-4 (2 pgs.)
A6. Log of Boring B-5 (2 pgs.)
A7. Log of Boring B-6
Logs of Hand Augered Test Holes
Log of Boring K-1
Log of Boring K-2
Log of Boring K-3
Log of Boring K-4

B-1. Direct Shear Test
B-2. Direct Shear Test
B-3. Direct Shear Test
Corrosivity Test Results

INTRODUCTION

Site Location and Proposed Project

The project, herein referred to as "Site", is to construct in 4, 9-story condominium buildings in 2 phases at the lower elevation of a vacant hillside property located on Serramonte Boulevard between the historic Chinese Cemetery and Serramonte Shopping Center (Plate 1, Vicinity Map; Plate 2, Site Plan; also see architectural renderings on the following this page). We understand that the project is in the planning stages, but is expected to involve significant grading to create the stepped building pads with street-level podium levels, and 2 to 3 levels of underground parking. Approximately 1150 linear feet of retaining wall to support cuts of up to 90 feet in height area planned at this time.

Purpose and Scope of Services

The purpose of this investigation was to characterize the engineering geologic setting, and geotechnical properties of the foundation soil profile in order to provide preliminary, design-level geotechnical recommendations for the proposed development. The scope of services included:

- Review of published and unpublished geologic and geotechnical literature and maps. Plates 4-7 contain excerpts of published regional geologic, fault and seismic mapping pertinent to the Site. Appendices A – D contain boring and laboratory test results data from selected geotechnical reports available at the City of Daly City Public Works Department;
- Photogeologic mapping from historic stereo imagery pre- and post-dating middle 1960's mass grading of the Site area (Plates 3A-3B);

- Site engineering geologic reconnaissance mapping onto the generalized topographic Site plan prepared by GLA Civil Engineers (2014);
- Site soil characterization by drilling and sampling four (4), borings to depths ranging from 60 to 71 feet at the approximate locations depicted on Plate 2. The borings were drilled on September 29-30, 2014 with a Semco, track-mounted, continuous flight auger equipped with a cathead and rope arrangement to facilitate sampling. Relatively undisturbed soil samples were retrieved from the selected depths of the soil profile encountered at the borings locations using Modified California and Standard Penetration Test (SPT) samplers. The samplers were driven with a 140-pound hammer free-falling from 30 inches. The number of drops (blows) required to advance the respective samplers at 6-inch intervals for a total of 18-inches are tabulated on the Boring Logs in terms of SPT values (Plates E1-E4, Appendix E). Blows to drive the modified California sampled were converted to SPT values using a conversion of factor of 0.76. Plates E5 and E6 contain descriptions of the terms and symbols used on the logs;
- Laboratory testing of samples retrieved at the boring location included moisture content, dry density and percent passing ASTM #200 sieve, Atterberg limits, and undrained Triaxial shear strength (Appendix F);
- Analysis of the data and preparation of pertinent geotechnical recommendations for the proposed project.

Previous Work Summary

We were able to review several geotechnical investigation reports associated with commercial and residential developments in the Site area (see Geotechnical Data Source Index Map on the following page). This section of the report summarizes geologic and geotechnical findings and geotechnical design parameters provided in the reports. Reports by Woodward-Clyde-Sherard & Associates (1965, 1966, 1967; locality 1 on index map), pertaining to geotechnical aspects of design and construction of Serramonte Shopping Center, were prepared before and during mass grading in the later part of 1960. Site geologic mapping and boring logs were absent from the report copies received from the City Public Works Department; however, descriptions in the report are of dissected, locally badland landscape occupied by a dairy farm. They characterized Merced formation as clayey, silty sand, well consolidated with excellent shear strength. Cut and fill slopes for the shopping center project was limited to 2H:1V. The consultant recommended that buildings be supported on spread footings and/or belled piers excavated to a minimum depth of 2 feet. Recent observations during this investigation of the shopping center development revealed good performance of building support, pavements, and drainage over the past 45 years.

William F. Jones, Inc. (1976, 1977 including Converse, Davis & Associates 1974, 1975 reports; locality 2 on index map) prepared a geotechnical update report for proposed strip mall on the west side of Gellert Boulevard near the intersection with Serramonte Boulevard, which was apparently used in design of the Alto Serramonte Townhomes on the slope above strip mall (re-named Cypress Point Townhomes, built in 1979). These properties are on east flank of the hill supporting the Site also mass graded late 1960's as a source of fill material to

Index map

reclaim dissected badlands for adjacent development of Gellert Boulevard and other distant developments. Reconnaissance geologic mapping of the cut slope exposures, and drilled borings revealed Merced formation as non-expansive, medium dense to hard, variably cemented, indistinctly bedded, medium to fine sandstone, silty sandstone, and siltstone with local gravelly lenses of the Merced formation dipping steeply to the northeast. They conducted a slope stability analysis, applying a unit weight of 120 pounds per cubic foot (pcf), phi of 34 degrees and a conservative cohesion of 250 pounds per square foot (psf), existing graded slopes inclined up to 1 ½H:1V were stable under static and pseudo-static modeling. Their analyses indicated strong earthquake shaking from a nearby earthquake would only affect unconfined, surface soils on steep slopes.

Surface soils were found to be highly susceptible to erosion by runoff. Paved ditches were recommended to control runoff on intermediate benches and abatement of rodent burrowing found to be an important mechanism in initiation of slope gully erosion. Based upon specific experience in the area, the consultant recommended an inclination of ½H:1V for temporary cut slopes up to 10 feet high, and 1H:1V for higher slopes. Well-drained, drilled pier-supported, cantilever walls, and laterally braced (tied-back) soldier pile walls were recommended retaining walls types to permanently support various cut slope conditions up to 20 feet high. Active lateral earth pressures of 32h and 65h (for triangular distribution, where h is the height of the wall) were recommended for design of cantilever walls supporting level backfill, and backfill sloping up to a maximum of 2H:1V, respectfully. Recommended active pressures given for laterally braced walls (using a trapezoidal pressure distribution) were 19h and 39h for level and backfill and backfill sloping up to a maximum of 2H:1V. They recommended an increase in the above active pressures by 18 percent to account for a horizontal acceleration of 0.1g, and by 75 percent for 0.3g. An

adhesion value of 1000 psf was recommended for design of tiebacks embedded in Merced formation. Conventional footings embedded in Merced formation were recommended for support of buildings.

Recent observations during this investigation revealed good performance over the past 40 years of the building foundations, pavements, and shotcrete retaining walls up to 20 feet high.

John V. Lowney & Associates (1979; locality 3 on index map) conducted a geotechnical investigation for a one-story, and two-story with basement office complex located approximately 1600 feet south of the Site on former reclaimed (in late 1960's) badlands terrain at the southeast corner of the Gellert and Hickey Boulevard intersection. Dense to very dense, weathered Merced formation sandstone was encountered beneath from 10 to 20 feet of fill; on which numerous sieve analyses were run to distinguish fine-grained sand from silt. They recommended temporary cut slopes be no greater than 1H:1V and permanent cut slopes no greater than 2H:1V.

Recommended for 2-story building foundations were minimum 30-inch diameter piers extending at least 5 feet into bedrock, and designed for 600 psf skin friction and lateral resistance of 500 pcf in the fill, and 1200 psf skin friction in the bedrock. One story buildings were recommended to be supported on 12-inch wide footings embedded at least 18 inches engineered fill or Merced formation designed for an allowable bearing of 2500 psf for dead+live loads, with 1/3 increase for wind and seismic. A mat slab, designed for 1500 psf for dead+live loads with 1/3 increase to account for wind and seismic loads, was recommended for the basement (vault) foundation.

The drilled pier design above was recommended for support of fully drained, restrained retaining walls designed to resist an active equivalent fluid pressure of 35 pcf plus a uniform pressure of 5H psf, where H is the height of the wall in feet, and an additional uniform pressure of ½ of any surcharge load(s) at the surface. Walls free to deflect were designed for an equivalent fluid pressure of 35 pcf plus 1/3 of any surcharge load(s) applied at the surface.

Kleinfelder, Inc. (2000-2002; locality 4 on index map) conducted a geotechnical investigation on the Site for a previously proposed 4-story hotel and condominium complex featuring 2 to 3 levels of underground parking. Site characterization for the proposed project was based upon sampling of 6 borings and 4 shallow hand auger holes, in addition to 4 borings drilled for a 2000 investigation to depths of 11 ½ to 16 ½ feet. Merced formation was encountered generally within less than 5 feet below the ground surface at their boring locations. It was represented by poorly cemented, medium dense to hard, fine-grained, friable sandstones, and sandy siltstones mantled by a thin veneer of loose to medium dense, clayey sand. They suggested Merced formation bedding is nearly horizontal. Ground water was encountered in the bedrock materials at variable depths ranging from 4 feet to 50 feet. They concluded the existing slope is generally stable at the average gradient of 2 ½H:1V – 2H:1V.

They mentioned a small rotational slide that at an undisclosed location on the Site that would be removed given the depth of proposed grading. They emphasized the need for slope drainage and erosion control measures to mitigate gully erosion of the bedrock materials, and to maintain long-term slope stability. Temporary cut slopes no greater than 1 ½H:1V were recommended. Various temporary and permanent foundation and retention system were provided, including:

- Soil-nail wall(s) (to be designed by others) staged during excavation was considered to accommodate continuous support of the cut slope (conventional "top-down" retaining wall construction). The following strength parameters were provided for design of soil nails:
    - Drained Conditions (Long Term): C = 320 psf; Phi = 32 degrees; Unit Weight = 125 pcf
    - Undrained Conditions (Short Term: C = 400 psf; Phi = 34 degrees; Unit Weight = 125 pcf.

  It was recommended that the designer of the soil nail wall use at least an "at rest" earth pressure and seismic coefficient of 0.35 for pseudo-static analyses for loads from design basis earthquake (e.g., 10% probability of exceedance in 50 years), and an assumed depth of 15 feet to ground water. The design was to apply a rectangular distribution over the entire depth of the wall with a pressure equal to 1/3 of the surcharge load imposed at the top of the wall. Double Corrosion Protection bars encapsulated by polyethylene corrugated tubing were recommended for soil nails. The following wall design concepts were offered:
- Temporary or permanent, restrained or cantilever soldier beam and lagging wall with separate foundation system, and potential for permanent wall construction in the front of lagging wall;
- Shotcrete or formed concrete foundation retaining wall supported by closely-spaced drilled piers, with tie-backs where needed;

Recommended lateral earth pressures acting in a triangular distribution provided for design of fully drained poured in place or masonry block walls up to 6 feet included:

- Cantilever Walls: 40 pcf for backfill for a slope up to 6H:1V; 50 pcf for 3H:1V backfill; and 55 pcf for 2H:1V backfill

- Restrained Walls: 55 pcf for 6H:1V backfill; 65 for 3H:1V backfill; and 70 for 2H:1V backfill

Conventional spread footings supported embedded in Merced formation were recommended provided they were constructed with a minimum of 5 feet of horizontal confinement from a slope steeper than 5H:1V. They recommended an allowable bearing capacity of 6000 psf (dead+live loads, increase by 1/3 to account for wind and seismic loads) for an 18-inch wide footing bearing on lean concrete of bedrock.

Interior slabs on grade design was for a modulus of subgrade reaction of 150 pounds per square inch per inch of deflection provided they are supported on Merced formation.

FINDINGS

Historic and Current Use of the Site

Historic topographic maps and aerial photographs reveal that the northern flank of the linear hill occupied by the Site remained relatively pristine until widespread mass grading commenced in the middle to late 1960's for commercial and residential development of the Serramonte area (Plates 3A and 3B). By that time, sandstone and siltstone of the Merced formation was well known as a reliable source of easily excavable, quality fill soil.

During this period of mass grading, a large hill (old landslide depicted on Plate 3A, discussed later in this report) was removed to a depth of at least 120 feet as a source of fill to reclaim gullied land for development of Serramonte Shopping Center and Serramonte Boulevard. In addition to the eventual cut "daylight" line extending onto the northern side of the Site, quarrying for fill on the eastern side

of the linear hill (see preceding locality 2 on index map) to reclaim badlands for development of Gellert Boulevard and other distant projects (locality 3 on index map) resulted in cut slopes, and intermediate benches extending onto the north flank of the hill and onto the eastern third of the Site. The intermediate benches graded on the Site remain present today, as depicted on Plate 2. During this period, an apparent "haul road" was graded between northern edge of the hill crest in the south-central part of the Chinese Cemetery and the upper intermediate bench in the upper east side of the Site. We suspect there has been progressive grading of the cemetery area since before 1915. There was also grading for the cemetery service road that extends along the full length of the edge of the hillcrest with the eastern part extending down the hill to the southeastern corner of the Site. Grading for eventual commercial development on the northwest and northeast sides of the Site also took place during this period. By the late 1979, the Cypress Point Townhomes were constructed on the graded northeastern flank of the linear hill (locality 2 on index map).

Topography and Drainage

The northerly facing hillside the Site occupies is within the Colma Creek drainage basin at an average elevation of 350 feet above mean sea level. Slope morphology across the eastern quarter of the Site is visible given grass cover and scattered stands of mature pine trees, while the ground surface elsewhere is obscured by dense standing and locally abundant uprooted, mature pine trees and dense understory brush. The generalized Site plan contours reflect slope gradients averaging approximately 2H:1V over a vertical rise in elevation approximately 130 feet on the eastern side and approximately 40 feet on the western side. The slope profile in the eastern half is interrupted by intermediate drainage 10-to 40-foot wide benches cut during the early 1960's mass grading.

Elevation of the Site gently rises approximately 100 feet between the northeast and northwest ends.

Field reconnaissance and interpretation of historic topographic maps and aerial photographs indicate the current curvilinear topographic contour of a broad, concave-upward inflection between the upper and lower intermediate benches across the eastern half of the Site was formed at the headward part of an old landslide removed by the historic mass grading (Plate 2; Plates 3A and 3B). The inflection is intersected by two tributary, seasonal drainage swales of the 3 draining the northern flank of the hills prior to the late 1960's.

The headwaters of the 3, swales, modified by mass grading, drain the north flank of the hill across the Site (S1, S2 and S3 on Plate 2, Plate 3A and 3B). Swale S1, an indistinct, brush and tree covered feature in the southwestern part of the Site has been obscured in the headward part by grading on the cemetery property. Runoff and seepage into the head of the swale is influenced by a closed depression adjacent to the pools service road.

Swale S2 is a dense brush and tree covered feature having an obvious concave upward profile between the cemetery property and the southern property line of the Site; however, the headward part of the swale has been modified by historic mass grading and by more recent placed of oversteepened of grave excavation spoil. On the western side, this feature receives on the western side considerable runoff from the cemetery service road. The upper bench that crosses the middle reach has been completely obscured by uprooted, mature pine trees that have been densely overgrown by vines and brush. Its northerly projection toward the location of Proposed Building C is inferred by the dense growth and uprooted trees that obscure visibility of the ground surface.

Swale S3, projected across the eastern quarter of the Site, appears to have been removed by mass grading (cut and fill in the middle and lower reaches). Grading in the headward reach of the swale is represented by a relatively smooth, broad concave upward remnant on the cemetery. It is grass covered and bordered by a dense stand of mature trees and understory brush. Concentrated runoff from an artery of the cemetery service road extends has eroded a notch in the graded slope where swale intersects the southeastern part of the Site, just uphill of the upper bench.

Site runoff is generally uncontrolled. It appears that considerable infiltration of runoff is introduced into the subsurface from pooling of runoff on the irregular bench surfaces, and abundant surface depressions caused by rodent burrowing and uprooted trees. An eroded, approximately 2- to 3-foot deep, linear ditch and a timber diversion dike we observed in the lower-middle part of the Site above proposed Building C, was apparently constructed to redirect concentrated cemetery service road runoff away from the cut slope between the lower bench and Serramonte Boulevard. An earthen berm was constructed along the downslope edge of the bench in an effort to convey surface runoff to a 12-inch diameter corrugated metal pipe (CMP) in a small, concentric depression partially filled by homeless encampment debris near the proposed Building A location. The CMP slopes diagonally and apparently discharges into the municipal storm drain beneath the boulevard.

Surface runoff has abundant opportunity to infiltrate the surface soil mantle through rodent burrows; particularly on the poorly-drained surfaces that characterize the graded benches. While seepage onto the ground surface was active at the time of our investigation, we observed numerous piping voids associated with rodent burrows that episodically produce surface seepage during prolonged rainfall.

Surficial Soils

The entire Site is mantled by a relatively thin (2 to 5 feet thick) veneer of low to moderate plasticity, generally loose/soft silty and clayey sand derived from weathering of the underlying Merced formation, and locally from the historic grading described earlier. Aside from the earthen berm, notable thicknesses of fill slopes were not readily apparent from surface morphology; however, downhill sides of the intermediate benches are likely to be underlain by wedges of undocumented fill.

Rocks and Structure

The Site area is underlain by Plio-Pleistocene Merced formation (Bonilla, 1971, 1998; Plate 4). It represents over 5000 feet of marine and aeolian sand, silt, alluvial gravel, sandy mudstone, and layers of volcanic ash that accumulated in a shallow, geologic structure-controlled narrow seaway (Colma Strait, fig. 1) extending along the northeast side of the San Andreas fault between Lake Merced and the eastern margin of Montara Mountain (Oakshott, 1959; Clifton and Hunter, 1987). This interpretation stems from seacliff exposures near between Mussel Rock Fleishacher Zoo (Hall 1966).

Poorly to semi-consolidated, and well cemented, conglomerates, medium- to coarse-grained sandstones, silty, fine-grained sandstones, siltstones, and mudstones that comprise the Merced formation are bounded by the in San Andreas fault and Juro-Cretaceous Franciscan basement rocks on the southwest side, and on the northeast side by the Serra fault and weakly consolidated, late Pleistocene near-shore, beach and dune deposits that comprise the Colma formation (Plate 4). Bonilla's early field mapping of the Merced formation in the Site area benefited from seemingly abundant exposures in the highly dissected

Colma straight map

landscape visible on 1956 aerial photographs. Tectonic deformation across the bounding faults has resulted in tightly folded rocks. Bonilla (1971) mapped mostly steep, northeasterly dipping beds in the Site area. Converse, Davis and Associates (1974, 1975) during a geotechnical investigation for the Cypress Point townhome development adjoining the southeast corner of the Site, reported faint, steep northeast bedding adjacent to the site area (see locality index map presented earlier). Bonilla (1998) later interpreted a local, northwesterly trending synclinal fold trough the Site, although there was no structural data was presented to support this interpretation (Plate 4).

Tectonic deformation has created a distinctive, northwest trending linearity to the Site area terrain, as interpreted from 1956 aerial photographs. We detected relatively thinly stratified Merced formation represented on aerial photographs as rhythmic, linear tonal contrasts exposed in the bank of an drainage on southwest side of the linear hill supporting the Chinese Cemetery. The tonal contrasts also subparallel, relatively short linear hills in the area, including the one supporting the Site. We infer northwest and northeast trending linear spur ridges, intervening drainages, and aligned saddles visible on aerial photographs are controlled, at least in part, by joint sets in the Merced formation locally exposed in voids formed by erosional scarps, and voids formed by piping and uprooted trees that generally mark disturbed/eroded ground of the Site (Plate 2; Plate 3A).

The few Site exposures of resistant, massive, fine-grained silty fine-grained sandstone and siltstone revealed thin iron-oxide-lined and indistinct joint surfaces steeply inclined to the northwest and northeast with variable shallow to steep dips to the north, northeast and northwest. One closely-spaced joint set, oriented subparallel to the inferred strike of bedding and steeply inclined to the southwest, was measured in massive, silty fine-grained sandstone exposed in the westerly part of the Site. An indistinct contact between very friable, fine- to medium-

grained sand on the north side and silty fine-grained sandstone south side was observed in the same area; however a bedding reliable attitude was unobtainable.

Merced formation encountered in our and Kleinfelder, Inc. (2002) borings was comprised of weak to strong, thin- to relatively thick-bedded, silty coarse sandstone, silty fine-grained sandstone, siltstone, low- to moderate-plasticity clayey siltstone and sandstone, mudstone (Appendix A and B, Plate 7).

Ground Water

Seepage was encountered during drilling of Borings 1 and 2 at depths of 39 feet and 47 feet, respectively. Similar occurrence of seepage was reported by Kleinfelder, Inc. (2002) during drilling of Boring HB-1 upslope for proposed Building A at 34 ½ feet, Boring HB-2 adjacent to Building A at 36 feet, and Boring HB-4 at 45 feet below the ground surface (Plate 7, Appendices A and B). The average depth to stabilized ground water across the Site was approximately 45 feet. Two intact piezometer locations from a previous unknown investigation were discovered on the uppermost bench in the southeastern half of the Site. They were labeled as P1 and P2 on the exterior side of the galvanized metal box standpipe caps; suggesting they were constructed during a past geotechnical investigation. (The expansion screw cap at the top of the 2-inch diameter PVC casing was marked with the Geostore inscription). P-1, located in S3 near the southeast corner of the site was measured to be 100 feet deep without measurable water. P-2, located in the divide between the easterly swale and middle swale was measured to be 89 feet deep with water encountered at a depth of 75 feet.

Landslides and Erosion

We judge that the old (late Pleistocene) landslide, mapped from aerial photographs between the Site and middle of the Serramonte Shopping Center, was effectively removed during mass grading based upon comparative analysis of pre- and post-mass grading topographic mapping (Plate 3A and 3B). The approximate headward extent of this feature on the Site prior to mass grading is depicted on Plate 2. There was no observed evidence of active bedrock landsliding constraining the Site.

Uncontrolled, concentrated cemetery service road runoff onto the Site from at least 3 locations above the southern property line has caused considerable erosion, soil creep, and localized surficial debris sliding (defined as Eroded/Disturbed Ground) between the upper bench and downhill side of the lower bench where Building Sites B-D are proposed, and the southeast corner of the Site above proposed Building A (Plate 2). As mentioned above, this condition resulted in attempts to redirect lower bench runoff with temporary drainage measures depicted on Plate 2). Remnants of visqueen embedded in the rooting structure of a mature juniper tree at the slope transition onto the lower bench, adjacent to the site of proposed Building C, is interpreted as erosion control installed 20 or more years ago. The scalloped ground surface along the downslope side of the upper bench in the eastern part of the Site can be related to overtopping runoff and piping by seepage from flooding of the undrained upper bench ground surface during intense and prolonged rainfall.

Current surface and subsurface drainage patterns represent a real hazard to Site safety by weakening the tree rooting zone, which appear to be limited to the 2- to 5-foot thick, loose and soft surface soil horizon. This has resulted in locally high occurrence of mature tree toppling in the middle and western part of the Site, and

creation of large undrained depressions where trees have been rotated or uprooted.

Faults and Seismicity

The Site lays the western margin of the San Andreas fault system, which in central and northern California is represented by a series of northwest trending, right-lateral strike and subparallel, range front thrust faults (Jennings and Saucedo, 2002). The Site lies on a tectonic block approximately ¾ of a mile northeast of the active San Francisco Peninsula segment of the San Andreas fault, and approximately 1/3 of mile southwest of the Serra fault. The Serra fault is believed to be an active thrust fault within the northernmost, landward segment of the Peninsula Fold and Thrust Belt (PFTB) based upon Holocene deformation observed in strata exposed in the seacliff at Fort Funston northwest of the Site (Kennedy, 2002), and paleoseismic trenching across the trace mapped in Junipero County Park between San Bruno and Millbrae southeast of the Site (William Hengesh and others, 2004). Trench exposures indicate active faulting occurs across a 5-to 12-foot wide zone that dips 56 degrees to the southwest to merge at depth with the San Andreas fault. A little more than a mile separates the Serra fault from the San Andreas fault in the Site area. The Serra fault is not considered a source for future earthquakes. Based upon ground deformation along the PFTB in Santa Clara Valley during the 1989 Loma Prieta earthquake, the Serra fault is judged to pose a local, surface rupture hazard (Haugerud and Ellen, 1990; Hengesh and others, 2004).

There are no active faults constraining the Site; hence, potential for fault offset across the Site is considered nil. While the Site is located on the hanging wall of the Serra trust fault, based upon the discussion above and distance to the fault

zone, we judge low the potential for coseismic ground deformation to affect the Site during a nearby major earthquake on the San Andreas fault.

Plate 5 illustrated major active faults that have produced significant Bay Area earthquakes. The magnitude 7.9, 1906 San Francisco earthquake was centered on the northern end of the San Francisco Peninsula Segment of San Andreas fault. It caused very strong to very violent ground shaking with attendant ground rupture extending between Mendocino County to San Juan Bautista (Lawson, 1908). At the time of the earthquake, the Site area was only inhabited by farms. However, there was no reported deformation in the Site area from coseismic movement on the Serra fault. There was no evidence of seismically-induced landsliding on the Site area detected from interpretation of historic aerial photographs.

A magnitude 5.3 earthquake in 1957 was centered west of the site on the San Andreas fault near Mussel Rock west of the site (Bonilla, 1959. It caused landsliding from the steep coastal bluffs, and concentrated damage to buildings and other man-made structures in the adjoining Westlake Palisades area in western Daly City. There was no reported damage to structures or landslides east of the fault.

The 1989 Loma Prieta earthquake, centered near Bean Hill in the southern Santa Cruz Mountains approximately 60 miles to the south, caused moderately strong shaking in the site area. There was no reported slope deformation on the site or reported damage to structures in nearby developments.

A major earthquake on the San Andreas described above, and the Hayward and Calaveras faults, mapped approximately 15 northeast and 42 miles southeast of the Site, respectively, is expected to cause very strong to very violent ground

shaking at the Site (Petersen and others, 1999). The chance of a magnitude earthquake of 6.7 earthquake or greater by the year 2036 on one of the major active Bay Area faults is illustrated on Plate 6.

DISCUSSION AND CONCLUSIONS

It is our opinion the proposed project is feasible from a geotechnical standpoint, provided the erosion and siltation hazard from concentrated off-site runoff and uncontrolled grading is understood, and the mitigation recommendations contained in this report are carefully followed.

The northerly facing hillside to support the project is underlain by steep, northeasterly dipping Merced formation comprised of medium dense to hard, but highly erodible, sandstone, siltstone and claystone of the Merced formation on a tectonic block between the active San Andreas and Serra faults. Subsequent mapping of a synclinal fold across the Site is unsupported by the structural data presented on Bonilla's 1971 and 1998 mapping. We conclude, therefore, the southwesterly dip of strata on the north limb of the mapped feature and north margin of the site is likely related to back-rotation of Merced strata from an ancient landslide removed during late 1960's mass grading and presents no treat to site stability.

The Site is constrained by steep, poorly-drained, graded, and native slopes, and localized flooding from the cemetery service road runoff. It is our opinion the risk of global instability is low provided the recommendations in this report be adhered to in project design and construction. This conclusion is based upon observed slope conditions and historic performance of the Site and surrounding area during the past earthquakes, particularly the magnitude 7.9, 1906 earthquake centered a few miles to the northwest.

The ancient landslide event, as well as widespread bedrock landsliding in the coastal bluffs of Daly City and San Francisco by wave undercutting, demonstrates the sensitivity of Merced rocks, and propensity for failure when slopes are saturated and/or oversteepened. Thus, the proposed grading, with proposed cuts up to 90 feet, requires strict adherence to the drainage, grading, and retaining wall recommendations in the following section. We anticipate the northeasterly dipping strata could present adverse conditions limiting the height of temporary cut slopes to a height of 5 feet for each successive lift of top-down wall construction. Perched groundwater seepage encountered in the borings will require that excavations be provided with adequate dewatering technique by the specialty contractor.

Engineering geologic mapping of the Site was constrained by the generalized topographic contours of the Site Plan. It will be important to better detail the topography for necessary supplemental engineering geologic mapping of drainage and surficial features affecting site performance. Given much of the ground surface is hidden by dense vegetal cover, tree and brush supplemental mapping should be undertaken after grubbing. Before grubbing, it will be important for the project arborist to make a critical evaluation of the obvious unstable rooting habit of the locally dense, mature pine forest. After grubbing, the engineering geologist should conduct a supplemental reconnaissance to identify potential problem areas for mitigation.

If it becomes impractical for mitigate adverse runoff, grading and siltation on the cemetery property, it would be necessary to implement measure to mitigate the existing adverse runoff and siltation conditions on the Site. Therefore, we have recommended construction of a heavy-duty chain-link fence across the potential source area track. The mitigation, including remedial grading and surface and subsurface drainage improvements on the upper intermediate bench near the

southern property line, should be implemented during the initial stages of project mass grading.

RECOMMENDATIONS

Seismic Design

The proposed structures should be designed for the following seismic design criteria derived from review of previous Site geotechnical explorations, our subsurface exploration data, and guidelines offered in the 2013 California Building Code (ASCE 7-10):

- Site Location: Latitude 37.670; Longitude = -122.470
- Site Soil Class: C
- Spectral Response Acceleration Values (g): Fv= 1.3; Ss = 2.595; S1 = 1.246; SDs = 1.730; SD1 = 1.080

Site Preparation, Grading and Compaction

Areas to be developed should be stripped of all existing vegetation and disturbed soil. Once the vegetation is removed, it will necessary for the engineering geologist from our office to verify areas uphill of the proposed Site retaining wall that require remedial grading (Plates 2 and 9). Areas affected by erosion, soil creep and localized debris sliding should be overexcavated with regularly spaced benches cut into undisturbed Merced formation (Plate 10). Bench subdrainage requirements will be assessed during grading, however, for planning assume installation of a subdrain at the rear of every other bench elevation above the key subdrain. After the bedrock is exposed, it should be scarified at least 8 inches deep, moisture conditioned to near optimum and compacted to at least 90 percent of the maximum dry density of the materials. Once the bench surface is

prepared, Site soil suitable for engineered fill, as assessed by our field engineer, shall be placed in loose maximum 8-inch thick lifts, moisture conditioned to near optimum and compacted to at least 90 percent relative to the maximum dry density of the soil used (as assessed by the ASTM D1557 laboratory test procedure). Benches cut into undisturbed bedrock and inclined into the slope shall be constructed as the fill repair rises up the slope. Localized overexcavation may be required where soft soil is encountered in the swale areas. The field engineer should assess repair subdrainage requirements during grading. Anticipate subdrainage will be required on alternating benches and where repair fill is greater than 5 feet in thickness, and elsewhere. Distribution of the surface and subsurface drainage mitigation for offsite runoff illustrated on Plate 10 will be assessed by the soil engineer following completion of the remedial grading.

The maximum fill slope shall be 2H:1V unless reinforced by an approved geogrid (i.e., Miragrid or equal), spaced horizontally 2 feet apart in the compacted fill; in which case finished fill slopes of 1 1/2H:1V may be approved. Preliminarily, we recommend that cut slopes in bedrock have a maximum gradient of 2H:1V, and 3H:1V, where surficial deposits are encountered in cut areas. However, the actual material quality at the cut slope location will dictate maximum inclinations (e.g., a 5-foot high cut slope in massive sandstone may be acceptable with a finish slope of 1 ½H:1V). Steeper slopes should be retained by engineered retaining walls.

We anticipate the proposed grading plan for pad and below grade parking basements will be achieved by "top-down" grading. This technology involves sequential grading of benches to create 4- to 6-foot high, vertical or near cuts, and installation of reinforced shotcrete supported by soil nails on the cut face prior to excavation of the lower lift. Our engineering geologist should observe

each cut face for potential instability, and evaluate the need for segmented excavation support of each cut lift.

We recommend that exposed basement and street-level subgrades to receive slabs be scarified to a depth of 8 inches, moisture conditioned to near optimum and recompacted to at least 90 percent relative to the maximum dry density of the materials used (ASTM D1557).

It is possible that proposed excavations deeper than 15 feet will encounter perched ground water. Therefore, the grading contractor must be prepared to manage the likely occurrence of seepage into the excavation. Conventionally, removal of accumulated water can be achieved by constructing a temporary sump(s) equipped with an appropriate pumping system(s) that convey water to an approved discharge location. A bed of 2- to 3-inch clean crushed rock on the floor of the excavation can facilitate positive movement of water to the temporary sump/pump system, and provide a durable surface for movement of personnel and equipment during until the permanent basement drainage system is completed.

Shoring and Underpinning

Shoring and underpinning is not expected given sequential grading and retaining wall offered by the "top-down" construction technique. However, we recommend that the project be carefully reviewed by the underpinning/shoring project engineer and underpinning specialty contractor.

Utility Trenches

Vertical trench excavations up to 3 feet deep should be capable of standing with minimal bracing for short construction periods. Local conditions may require that trenches less than 3 feet be cut and braced as specified in the State of California Safety Ordinance dealing with Excavations and Trenches.

Utility trenches should be designed to prevent the transportation of water into foundations, slabs or pavement subgrade soils. In particular, where utilities cross foundations, trenches should be plugged with compacted soil or concrete for their full depth, and for a distance of at least 2 feet on either side of the foundations. On-Site, inorganic soil may be used as utility trench backfill. Special compaction of trench backfill will be necessary under and adjacent to the proposed structures, concrete slabs, and engineered fill. In these areas, backfill should be conditioned with water to produce a soil-water content near the optimum value, and placed in horizontal layers, each not exceeding 6 inches in loose thickness. Each layer should be compacted to a density equivalent to at least 90 percent of the maximum dry density of the soil as determined by ASTM test D1557. The top two feet of trench backfill under slabs and pavements should consist of non-expansive, granular soils compacted to at least 95 percent of maximum dry density.

Foundations

Mat Slab

We anticipate that the main buildings and parking basement retaining walls will be supported by a thickened, reinforced concrete mat slab. The following parameters are offered for structural design:

- Allowable bearing value of 4000 pounds per square foot (psf) below a depth of 2 feet in bedrock.

- Modulus of subgrade reaction of 250 kips per cubic foot (kpf).
- Resistance to lateral loads can be achieved through a coefficient of sliding friction of 0.35, and a passive equivalent pressure of 300 pcf, beginning a depth of 1 foot.

Drilled Piers

- Minimum diameter of 16 inches and embedment of 10 feet into undisturbed bedrock.
- Allowable skin friction of 500 psf
- Passive equivalent fluid pressure of 600 pcf acting over 1 ½ pier diameters, beginning at a depth of 1 foot below. This value should increased by 1/3 to account for all loads.
- Lateral soil creep load of 50 pcf acting over 1 ½ pier diameters for piers constructed within 10 feet of a slope inclined greater than 2H:1V

Footings

- Minimum width of 12 inches and 24 inches of embedment into undisturbed bedrock of engineered fill;
- Allowable bearing value of 4000 psf beginning at a depth of 1 foot below the top of bedrock. Increase this value by 1/3 to account for wind and seismic forces.
- Passive equivalent fluid pressure of 600 pcf beginning at a depth of 1 foot below the ground surface.

Retaining Walls

Cantilever Conditions

Cantilever Site walls to support ground level parking and landscaped areas. The foundation type specified for slope conditions in *Foundation* section should support retaining walls.

Retaining walls should be designed for an active equivalent fluid pressure of 50 pcf for level backfill and 65 pcf for backfill sloping up to maximum of 2H:1V. In the event cantilever walls are to be restrained from rotation, they should be designed to resist an additional uniform pressure of 100 psf. Unrestrained walls supporting roadways should be designed to resist an additional surcharge of 1/3 the applied load acting on the top portion of the wall. Where seismic parameters are required by the structural, they should be designed for a pressure equal to 8H psf, where H is the height of the retained soil. The seismic component should be considered a load acting 0.5 times the wall height above the wall base.

The pressures described above are contingent upon the walls being constructed with a backdrainage system. We recommend that the backdrain pipe be located at least 1 foot below the adjacent lowest grade to mitigate underseepage toward the basement house foundation. The backdrain should consist of a geosynthetic drainage mat (i.e., Miradrain 5000 or equivalent) integrated with a minimum 4-inch diameter, perforated Schedule 40 PVC pipe (or better) in accordance with the manufacture's specifications, and sloped to drain by gravity to a sump and pump system designed by the project civil engineer.

Site retaining walls should also be fully backdrained. Retaining wall backdrains should consist of either a geosynthetic drainage mat and properly placed perforated pipe (as specified by the manufacturer, i.e., Miradrain 5000 or equivalent), or a minimum 12-inch wide prism of ¾- to 1 ½-inch clean crushed

separated from adjacent soil by Mirafi 140N (or better) filter fabric and extending to within a 18 inches of the finished backfill surface. Either backdrain design option should be constructed with a minimum 4-inch diameter, perforated Schedule 40 PVC pipe, oriented holes down placed at least 8 inches below the wall stem to mitigate underseepage, and with a minimum 2 percent gradient to offer gravity drainage to the approved discharge location. The upper foot of the crushed rock should be backfilled with compacted soil, or if used, the drainage mat should be cut at least 18 inches below the finished ground surface to mitigate infiltration of surface runoff.

Walls should be backfilled before construction next to them as a measure to mitigate potential damage to pavements and buildings from slight deflection during compaction. The finished backfill surface behind walls should be carefully sloped to drain in a positive manner so that ponding and erosion does not occur. Open, reinforced, concrete lined Vv-ditches should be designed to provide surface drainage control behind the retaining walls. They should be tied to minimum 12-inch square, steel grate covered catch basins constructed with trash racks to mitigate clogging from soil derived from burrowing animals, and falling vegetal debris. In turn, the basin should be connected to a solid pipe that carries water from the catch basin to the storm drain. Surface water should not be diverted into subdrains.

It would be prudent to consider thorough waterproofing of walls to prevent detrimental migration of moisture and potential development of unsightly precipitation on the wall face. Waterproofing details should be provided by the architect.

Restrained Conditions

Soil nail walls to support deep excavations should be designed and installed by a highly qualified design/build, specialty contractor (e.g., DrillTech, Inc., Antioch, CA). We provide the following soil parameters for use in design:

- Drained Conditions (Long Term): C = 320 psf; Phi = 32 degrees; Unit Weight = 125 pcf
- Undrained Conditions (Short Term: C = 400 psf; Phi = 34 degrees; Unit Weight = 125 pcf.

Soil-nail walls require comprehensive backdrainage, including placement of vertical, minimum 12-inch wide panels of Miradrain 5000 (or better) on the cut face at intervals between the nail array to intercept seepage and convey it to a 4-inch diameter Schedule 40, perforated pipe sloped at least 2 percent to the discharge point. The pipe should be encased in filter fabric-wrapped prism of clean crushed rock that extends at least 6 inches up the base of the cut. As a measure to mitigate underseepage, we recommend that pipe subdrain arrangement be confined in trench that extends at least 8 inches below the lowest adjacent grade.

Ground-Level Exterior Slab-on-Grade (sidewalks and courtyard patios)

Any exterior, ground level slab section should be constructed on subgrade prepared subgrade as described in the *Site Preparation, and Compaction* section of this report. Minimum 5-inch thick slabs should be supported on a minimum of 6 inches of Class 2 aggregate baserock compacted to at least 95 percent. Where migration of moisture vapor through slabs would be detrimental, an impermeable moisture vapor barrier should be provided between the baserock section and slab. It may be prudent to place an additional 2 inches of clean sand over the membrane to protect it during construction.

Drive Isle and Parking Pavements

Pavements should be placed on a uniform surface of bedrock or competent soil. Where pavements span a soil-bedrock interface, we recommend that the bedrock be over-excavated a minimum of 12 inches and replaced as engineered fill. Soil pavement subgrades and areas of overexcavation, prior to placement of fill, should be scarified to a depth of 8 inches and re-compacted to a minimum of 90 percent, and a moisture content of approximately 3 percent above optimum. Final pavement design will be dependent upon the civil engineer's anticipated traffic and the materials exposed at the subgrade levels. Table 1 defines preliminary, conservative pavement sections, in inches, for various traffic indices. Final pavement design can be evaluated after R-value testing of representative samples retrieved from the subgrade exposed during mass grading.

Table 1. Preliminary Pavement Design

Traffic Index	Asphaltic Concrete	Aggregate Base	Aggregate Sub-Base
4.0	2.0"	8.5"	--
4.5	2.5"	9.0"	--
5.0	2.5"	10.0"	--
5.5	3.0"	11.5"	--
6.0	3.0"	6.0"	8.0"
6.5	3.5"	6.5"	9.0"
7.0	3.5"	7.0"	10.0"

Earth Flow Mitigation

It will be necessary to mitigate the perceived potential for off-Site earth flow and siltation onto the property from the loose, spoil pile at the top of the slope on the north-central part of the Chinese Cemetery. We recommend construction of a minimum 6-foot high, chain-link fence above the upper bench, as illustrated on Plate 2. It should consist of 2, continuous strands of galvanized, No. 9 wire chain link supported by steel posts (I-beams), spaced 6 feet apart, and chosen by the structural engineer to provide resistance from deformation by an equivalent fluid pressure of 80 pounds pcf. The posts should be encased in minimum 12-inch diameter piers that extend at least 12 feet below the ground surface.

Drainage

We recommend that the proposed structure be provided with roof gutters and downspouts. The downspouts should be connected to solid PVC pipes and these pipes should discharge into the storm drainage system.

Positive surface drainage gradients of at least 2 percent should be provided for a distance of at least 5 feet away from all structures. The driveway isles and parking areas should drain to inlets that carry water by properly sized, solid PVC (Schedule 40 or equivalent) pipes to the storm drainage system. Surface runoff on the basement driveway ramps should be intercepted by properly strip drains designed by the civil engineer for anticipated conditions. Captured runoff will require pumping to the storm drainage system. Water can then be pumped to the storm drainage system.

After remedial grading to produce positive surface gradients, we recommend the intermediate benches uphill of the development, be fitted with reinforced concrete

V-ditches atop a subdrain to mitigate future degradation of the slopes above the proposed Site development (Plates 9 and 10).

Erosion Control

The civil engineer should prepare, and the project landscape contractor should implement a comprehensive erosion control plan to account for seasonal rainfall during and following construction. We recommend that the project engineering geologist make periodic inspections of the Site drainage and erosion control features for a period of 2 years.

MAINTENANCE

Surface and subsurface drainage facilities should be checked frequently, and cleaned and maintained as necessary. We recommend the project account for periodic engineering geologic inspections of the project drainage and siltation mitigations for a period of at least 5 years to assure they have remained effective given the anticipated continued uncontrolled grading on the cemetery property. The length of time to continue the inspections beyond 5 years should be carefully evaluated by the project management on the basis of reports provided by the engineering geologist.

INVESTIGATION LIMITATIONS

This report has been prepared in accordance with generally accepted geotechnical engineering principles and practices, and is in accordance with the standards and practices set by the geotechnical consultants in the area. This acknowledgment is in lieu of any warranty. We offer no guarantees.

This report was prepared using preliminary architectural drawings and without the limit of structural loads and other details. Although the recommendations presented herein are suitable for final design, we should collaborate, and eventually review final design details with the architect, and structural and civil engineers.

Subsurface conditions could vary between those indicated by the explorations and interpreted from surface features. Hence, recommendations presented in this report are only valid if we are retained to provide plan review and construction observation services; in order to verify the exposed geotechnical conditions, to modify recommendations if necessary, and to ascertain that the project is constructed in accordance with the recommendations.

This report is submitted with the understanding that it is the responsibility of the Client (Owner) to ensure that the applicable provisions of the recommendations contained herein are made known to all design professionals involved with the project; that the recommendations are incorporated into the construction drawings; and that the necessary steps are taken to see that the contractor and subcontractors carry out the recommendations in the field.

If conditions different from those described in this report are encountered during construction, or if the project is revised, we should be notified immediately so that we may modify our recommendations, if warranted.

The practice of geotechnical engineering changes, and, therefore, we should be consulted to update this report if construction is not performed within 12 months.

SUPPLEMENTAL SERVICES

We recommend that we review the final foundation, grading and drainage plans for conformance with the intent of our recommendations. During construction, we should observe the rough and finished grading operations, foundation excavations prior to steel placement, and the installation of all drainage facilities prior to burial to ascertain that our recommendations are followed. Upon completion of the project, we should perform a Site observation and report the results of our work in a final report. These services are outside the present scope and will be billed on a time and materials basis, in accordance with the fee schedule current at that time. These services will be performed only if we are provided with sufficient notice to perform the work. We do not accept responsibility for items that we are not notified to observe. We recommend that the Owner be responsible for notification, no less than 48 hours before the requested Site visit.

REFERENCES

Bonilla, M.G., 1959, geologic observations in the epicentral area of the San Francisco earthquake of March 22, 1957, *in*, San Francisco earthquake of March 1957. California Division of Mines and Geology Special Report 57.

_____, 1960, Map of landslides in the San Francisco south quadrangle and part of the Hunters Point quadrangle, California: U.S. Geological Survey, scale 1:24,000.

_____, 1971, Preliminary geologic map of the San Francisco south quadrangle and part of the Hunters Point quadrangle, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-311, scale 1:24,000.

_____, 1998, Preliminary geologic map of the San Francisco south 7 ½ minute quadrangle and part of the Hunters Point 7 ½ minute quadrangle, San Francisco Bay Area, California, a digital database: U.S. Geological Survey Open File Report 98-354.

California Division of Mines and Geology, 1982, Earthquake fault zones, San Francisco south 7 ½ minute quadrangle: State of California Department of Conservation, map scale 1:24,000.

Clifton, H.E. and Hunter, R.E., 1987, The Merced formation and related beds: A mile thick succession of late Cenozoic coastal and shelf deposits in the sea cliffs of San Francisco, California: Geological Society of America Centennial Field Guide, Cordilleran Section.

Converse, Davis and Associates, 1974, Soil and Foundation and earthquake engineering study, Alto Serramonte Development – Phase 1, Gellert Blvd. at Hickey Blvd, Daly City, California: Geotechnical consultant's February 27 report Job S-73-45, 7 pgs.

_____, 1975, Foundation investigation, Alto Serramonte Center, Gellert Boulevard, Daly City, California: Geotechnical consultant's February 27 report Job S-7-401-A 17 pgs. with illustrations.

Graymer, R.W., Bryant, William, McCahn, C.A., Hecker, Suzanne, Prentice, C.S., 2006, Map of Quaternary-active faults in the San Francisco Bay regions, U.S. Geological Survey Scientific Investigations Map 2919, scale 1:270,000.

Hengesh, J.V., Nolan, J.M. and Wakabayashi, 2004, Seismic hazards associated with the Serra fault, San Francisco Peninsula, California: Association of Engineering Geologists, Field Trip Guidebook, Seismic hazards of the Range Front Thrust Faults, northeastern Santa Cruz Mountains/Southwestern Santa Clara Valley, California, March 27.

Hall, N.T., 1966, Late Cenozoic stratigraphy between Mussel Rock and Fleishacker Zoo, San Francisco Peninsula: California Division of Mines and Geology, Mineral Information Service, v. 19, no. 11.

Jennings, C.W. and Saucedo, G.J., 2002, Simplified fault activity map of California: California Geological Survey, Map Sheet 54, 1:2,500,000.

John V. Lowney & Associates, 1980, Geotechnical investigation, Serramonte office building complex, Daly City, California: Geotechnical consultant's July 18 report, Job 277-22, PA 8891, 13 pgs. with illustrations and appendices.

Kennedy, D.G., 2002, Neotectonic character of the Serra fault, northern San Francisco Peninsula, California: Unpublished M.S. thesis, San Francisco State University, 117 pgs.

Lawson, A.C. (ed.), 1908, The California earthquake of April 18, 1906: Report of the California State Earthquake Investigation Commission: Carnegie Institution, Washington, D.C., v. 1, 451 pgs.

Oakeshott, G.B., 1959, San Andreas fault in Marin and San Mateo Counties, *in*, Oakeshott, G.B. (ed.), San Francisco earthquakes of March 1957: California Division of Mines, Special Report 57

Petersen, M, and others, 1999, Seismic shaking maps of California: California Division of Mines and Geology Map 48.

Smith, T.C., 1981, Serra fault, northern San Mateo County,: California Division of Mines and Geology, Fault Evaluation Report, FER-119, 8 pgs. with map.

Wagner, D.L., Bortugno, E.J. and McJunkin, R.D., 1991, Geologic map of the San Francisco – San Jose quadrangle, California Division of Mines and Geology, map scale 1:250,000.

William F. Jones, Inc., 1976, Soil investigation, Alto Serramonte Apartments, Phase 1, Hickey Boulevard at Gellert Boulevard, Daly City, California: Geotechnical consultant's September 16 report, Job 1/76084/1, 7 pgs. with illustrations.

William F. Jones, Inc., 1977, A geotechnical assessment for The Alto Serramonte Townhomes (Cypress Point Townhomes), Daly City, California: Geotechnical consultant's August 30 summary report, Job 1.77100/1, 5 pgs. with appendices.

Woodward-Clyde-Sherard & Associates, 1965, Soil investigation of the fill area for Serramonte Shopping Center, Phase II, northeastern portion of San Mateo County: Geotechnical consultant's April 13 report, Job No. 3-5940-A,15 pgs. with illustrations.

_____, 1966, Soil and foundation investigation for Serramonte Shopping Center, northeastern portion of San Mateo County: Geotechnical consultant's February 2 report, Job No. 5940B-1, 4 pgs.

_____, 1967, Engineering observation of earthwork, Serramonte Regional Shopping Center, Daly City, California: Geotechnical consultant's July 20 report, Job No. 5940AF, 2 pgs.

Working Group on California earthquake probabilities, 2008, The uniform California earthquake rupture forecast, version 2 (UCERF 2): U.S. Geological Survey Open File Report 2007-1437.

AERIAL PHOTOGRAPHS

Date	Job No	Flight Line	Frames	Scale
7/29/46	GSCP	2	126-127	1:24,000++
5/27/56	DDB	1R	26-27	1:20,000+++
7/9/63	AV 5500	4	24-25	1;20,000+
6/17/68	GS-VBZJ	1	9	1;20,000++
4/30/70	GSVCM1	2	182-183	1:80,700 #
4/22/73	3567	2	25-26	1:12,000++
6/17/74		7	28-29	1:20.000++

Sources:

+Pacific Aerial Photographs, Oakland, California

++U.S. Geological Survey, Menlo Park, California

+++U.S Soil Conservation Service, Ogden, Utah

U.S. Air Force

APPENDIX C

Kleinfelder (2002) – Appendix A – Logs of Borings and Appendix B – Laboratory Test Results:

Kleinfelder (2002) Appendix A – Logs of Borings

- A1. Boring Log Legend
- A2. Log of Boring B-1 (2 pgs.)
- A3. Log of Boring B-2
- A4. Log of Boring B-3 (2 pgs.)
- A5. Log of Boring B-4 (2 pgs.)
- A6. Log of Boring B-5 (2 pgs.)
- A7. Log of Boring B-6
- Logs of Hand Augered Test Holes
- Log of Boring K-1
- Log of Boring K-2
- Log of Boring K-3
- Log of Boring K-4

Klenifelder (2002) Appendix B – Laboratory Test Results

- B-1. Direct Shear Test
- B-2. Direct Shear Test
- B-3. Direct Shear Test
- Corrosivity Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY	GW		Well-graded gravels or gravel with sand, little or no fines.
		GP		Poorly-graded gravels or gravel with sand, little or no fines.
		GM		Silty gravels, silty gravel with sand mixture.
		GC		Clayey gravels, clayey gravel with sand mixture.
	SAND AND SANDY	SW		Well-graded sands or gravelly sands, little or no fines.
		SP		Poorly-graded sands or gravelly sands, little or no fines.
		SM		Silty sand.
		SC		Clayey sand.

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
FINE GRAINED SOILS	SILTS AND CLAYS	ML		Inorganic silts and very fine sands, rock flour or clayey silts with slight plasticity.
		CL		Inorganic lean clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
		OL		Organic silts and organic silt-clays of low plasticity.
	SILTS AND CLAYS	MH		Inorganic elastic silts, micaceous or diatomaceous or silty soils.
		CH		Inorganic fat clays (high plasticity).
		OH		Organic clays of medium high to high plasticity.
HIGHLY ORGANIC SOILS		Pt		Peat and other highly organic soils.

Standard Penetration Split Spoon Sampler 2.0 inch, 1.4 inch I.D.

Modified California Sampler 2.5 inch O.D., 2.0 inch I.D.

Bulk Sample

California Sampler, 3.0 inch O.D., 2.5 inch I.D.

Shelby Tube 3.0 inch O.D.

0745, 5/31 Approximate water level first observed in boring. Time recorded in reference to a 24 hour clock.

0800, 5/31 Approximate water level observed in boring following drilling

PEN Pocket Penetrometer reading, in tsf

TV:Su Torvane shear strength, in ksf

LL	LIQUID LIMIT	TX	TRIAXIAL SHEAR
PI	PLASTICITY INDEX	CONSOL	CONSOLIDATION
%-#200	SIEVE ANALYSIS (#200 SCREEN)	R-Value	RESISTANCE VALUE
DS	DIRECT SHEAR	SE	SAND EQUIVALENT
C	COHESION (PSF)	EI	EXPANSION INDEX
PHI	FRICTION ANGLE	FS	FREE SWELL (U.S.B.R.)

Notes: Blow counts represent the number of blows a 140-pound hammer falling 30 inches required to drive a sampler through the last 12 inches of an 18 inch penetration, unless otherwise noted.

The lines separating strata on the logs represent approximate boundaries only. The actual transition may be gradual. No warranty is provided as to the continuity of soil strata between borings. Logs represent the soil section observed at the boring location on the date of drilling only.

KLEINFELDER

PROJECT NO. 10-3011-46/GEO

BORING LOG LEGEND

Hansen PSC
Serramonte Boulevard
Daly City, California

PLATE

A-1

4/8/02 1:44:26 PM

Date Completed:	2/11/02	Drilling Method:	4" Solid Stem Auger
Logged By:	E. Appel	Hammer Wt.:	140 lbs., 30" drop
Total Depth:	65.3 ft	Notes:	

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION
								Surface Elevation: Estimated 175.0 feet (Above MSL)
								POORLY GRADED SAND WITH CLAY (SP) - light olive brown, loose to medium dense, fine grained with trace medium grained sand, 20% clay, moist
5								**SANDSTONE** - light brown with rust brown mottling, fine grained, completely to highly weathered, friable, facturing not apparent [MERCED FORMATION]
10		50/5"						-gravel lens - medium to coarse grained sand, gravel up to 0.5 inch in diameter -gray-brown, weak, moderately weathered
15								
20		50/4"	98	20				
25								
30		50/3"	99	12				-occasional lenses of better cementation
35			9.15					

KLEINFELDER

LOG OF BORING NO. B-1
Hansen PSC
Serramonte Boulevard
Daly City, California

PLATE
A-2

PROJECT NO. 10-3011-46/GEO

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION (Continued from previous plate)
		13:0						-gray, weak, slightly weathered
40		50/5"						-very wet
45								
50								
		50/2"						
55								
60		50/3"						-weakly cemented
65		50/3"	99	28				Boring terminated at 65.25 feet Groundwater observed at 34.5 feet Boring backfilled with grout
70								

KLEINFELDER

PROJECT NO. 10-3011-46/GEO

LOG OF BORING NO. B-1
Hansen PSC
Serramonte Boulevard
Daly City, California

PLATE
A-2
(cont'd)

L:\2000\02\PROJ\10301\46\10301146.GPJ

Wed 11/5/03 1:03PM

Date Completed:	2/12/02	*Drilling Method:*	4" Solid Stem Auger
Logged By:	E. Appel	*Hammer Wt.:*	140 lbs., 30" drop
Total Depth:	30.3 ft	*Notes:*	

Depth,ft	FIELD Sample	FIELD Blows/ft	LABORATORY Dry Density pcf	LABORATORY Moisture Content %	LABORATORY Compress. Strength tsf	LABORATORY Other Tests	Pen, tsf	DESCRIPTION
								Surface Elevation: **Estimated 125.0 feet (Above MSL)**
								SAND WITH CLAY (SC) - light olive-brown with rust brown mottling, medium dense, fine grained with trace medium grained, moist
		8:50						**SANDSTONE** - light brown with rust brown mottling, completely to moderately weathered, friable, fractures not apparent [MERCED FORMATION]
5								
10		50/5"	107	23				
15								
20		50/5.5"						
25								
30		50/4"						Boring terminated at 30.33 feet Groundwater observed at 4.0 feet Boring backfilled with grout
35								

KLEINFELDER

PROJECT NO. 10-3011-46/GEO

LOG OF BORING NO. B-2
Hansen PSC
Serramonte Boulevard
Daly City, California

PLATE
A-3

Date Completed:	2/12/02	Drilling Method:	4" Solid Stem Auger
Logged By:	E. Appel	Hammer Wt.:	140 lbs., 30" drop
Total Depth:	50.2 ft	Notes:	

Depth, ft	FIELD Sample	FIELD Blows/ft	LABORATORY Dry Density pcf	LABORATORY Moisture Content %	LABORATORY Compress. Strength tsf	LABORATORY Other Tests	Pen, tsf	DESCRIPTION
								Surface Elevation: Estimated 185.0 feet (Above MSL)
								POORLY GRADED SAND (SP)- light orange- brown, medium dense, fine grained with 15-20% medium grained, clay <10%, moist
								SANDSTONE - light brown with rust brown mottling, fine grained, friable, completely weathered, fracturing not apparent [MERCED FORMATION]
5								
								-approximately 10% medium grained, light brown, moderately weathered
10		92/9"	91	45				
								-weak
15								
20		59						
25								**CLAYEY SILTSTONE** - gray, some fine grained sand, moderately weathered, very closely fractured, friable to weak, moist
30								

KLEINFELDER

PROJECT NO. 10-3011-46/GEO

LOG OF BORING NO. B-3
Hansen PSC
Serramonte Boulevard
Daly City, California

PLATE
A-4

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION
								(Continued from previous plate)
		90/10"	101	23				-moderately cemented, slightly weathered
35								
40		95/8"						
45			9:00					
50		50/2" 2:45	98	10				Boring terminated at 50.17 feet Groundwater observed at 45 feet Boring backfilled with grout
55								
60								

KLEINFELDER

PROJECT NO. 10-3011-46/GEO

LOG OF BORING NO. B-3
Hansen PSC
Serramonte Boulevard
Daly City, California

PLATE
A-4
(cont'd)

L:\0602\02\PROJ\10\3011\46\10301146.GPJ

Date Completed:	2/13/02	Drilling Method:	4" Solid Stem Auger
Logged By:	E. Appel	Hammer Wt.:	140 lbs., 30" drop
Total Depth:	45.2 ft	Notes:	

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen. tsf	DESCRIPTION
								Surface Elevation: Estimated 165.0 feet (Above MSL)
								SAND WITH CLAY (SC) - olive to orange brown, medium dense, fine to grained sand, trace to medium grained sand, moist
5								**SANDSTONE** - light brown, highly to completely weathered, friable, fine grained, fracturing not apparent [MERCED FORMATION]
10		50/5"						**SANDY SILTSTONE**- brown with rust brown mottling, friable, fine grained sand, moderately weathered, fracturing not apparent, moist -weak
15								
20		81/11.5"						
25								
								CLAYEY SILTSTONE - gray, weak, slightly weathered, moderately cemented, closely fractured, moist
30								

KLEINFELDER

PROJECT NO. 10-3011-46/GEO

LOG OF BORING NO. B-4

Hansen PSC
Serramonte Boulevard
Daly City, California

PLATE

A-5

L:\2002\GP\RCJ\10301146\10301146.GPJ

4/8/02 1:38:06 PM

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION
								(Continued from previous plate)
		87/9"	102	22				
35								
40		60						
45		50/2"12.30						Boring terminated at 45.17 feet Groundwater observed at 45 feet Boring backfilled with grout
50								
55								
60								

KLEINFELDER

PROJECT NO. 10-3011-46/GEO

LOG OF BORING NO. B-4
Hansen PSC
Serramonte Boulevard
Daly City, California

PLATE
A-5
(cont'd)

Date Completed:	2/13/02	Drilling Method:	4" Solid Stem Auger
Logged By:	E. Appel	Hammer Wt.:	140 lbs., 30" drop
Total Depth:	45.0 ft	Notes:	

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION
								Surface Elevation: Estimated 165.0 feet (Above MSL)
								SAND WITH SILT (SP) - light brown with rust brown mottling, medium dense to dense, fine grained sand with trace gravel, moist
5				9.00				**SANDSTONE** - light brown with rust brown mottling, friable, completely weathered, fracturing not apparent, fine grained -yellow-brown -light yellow-brown, moderately weathered
10		50/5.5"						
15								**SANDY SILTSTONE** - gray, weak, slightly weathered, fine grained sand, fracturing not apparent, moist
				14.00				
20		50/6"						
25								
30								

KLEINFELDER

PROJECT NO. 10-3011-46/GEO

LOG OF BORING NO. B-5
Hansen PSC
Serramonte Boulevard
Daly City, California

PLATE
A-6

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION
								(Continued from previous plate)
		85						
			86	31				
35								
40		50/5"						
45		50/5"	106	18				
								Boring terminated at 45.5 feet Groundwater observed at 8 feet Boring backfilled with grout
50								
55								
60								

KLEINFELDER

PROJECT NO. 10-3011-46/GEO

LOG OF BORING NO. B-5
Hansen PSC
Serramonte Boulevard
Daly City, California

PLATE
A-6
(cont'd)

Date Completed:	2/14/02	Drilling Method:	4" Solid Stem Auger
Logged By:	E. Appel	Hammer Wt.:	140 lbs., 30" drop
Total Depth:	35.3 ft	Notes:	

Depth, ft	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests	Pen, tsf	DESCRIPTION
								Surface Elevation: Estimated 135.0 feet (Above MSL)
								SAND WITH CLAY (SC) - brown with orange mottling, medium dense, fine grained with medium grained, moist
								-dense
5								SANDY SILTSTONE - gray, fine grained sand, friable, moderately weathered
10		50/4"	99	26				
15								
20		82						
25								
30		50/5"						-occasional lenses of medium grained sand
35		50/4"						Boring terminated at 35.33 feet No free water observed Boring backfilled with grout
40								

KLEINFELDER

PROJECT NO. 10-3011-46/GEO

LOG OF BORING NO. B-6
Hansen PSC
Serramonte Boulevard
Daly City, California

PLATE
A-7

L:\0302GPROJ\10301146\10301146.GPJ

4/9/07 1:43:34 PM

LOG OF HAND AUGERED TEST HOLES
SERRAMONTE OFFICE BUILDING

Test Hole	Approx. Elevation (feet)	Approx. Depth (feet)	Classification
1	180	0-4	Light brown well graded sand, moist to wet.
		4-6	Light brown to orange well graded sandstone , uncemented, weathered, moist.
			Total Depth approx. 6'
2	200	0-0.5	Organics with dark brown poorly graded sand, moist.
		0.5-2	Light brown well graded sand, moist to very moist, dense to moderately dense.
		2-6	Orange brown well graded sandstone, weathered, moist, dense to very dense.
			Total Depth approx. 4'
3	206	0-0.5	Organics with dark brown poorly graded sand, moist.
		0.5-2	Light brown well graded sand, moderately dense to dense, moist to very moist.
		2-4	Orange/light brown well graded sandstone, weathered, moist, dense to very dense.
			Total Depth approx. 4'

Holes excavated by Chris McCarty on March 15, 2000.

PLATE H-1

© 2002 Kleinfelder, Inc

Date Completed:	3/10/00	Drilling Method:	
Logged By:	E. Appel	Hammer Wt.:	140 lbs., 30" drop
Total Depth:	11.5 ft	Notes:	

Depth, ft	FIELD Sample	FIELD Blows/ft	LABORATORY Dry Density pcf	LABORATORY Moisture Content %	LABORATORY Compress. Strength tsf	LABORATORY Other Tests	Pen, tsf	DESCRIPTION
								Surface Elevation: **Estimated 118.0 feet (Above MSL)**
	■	12	98	26				**CLAYEY SAND (SC)** - dark gray brown, loose, fine grained, slightly moist
5	■	68	109	20			3.50	**SANDSTONE** - gray to blue-gray, fractures not evident, friable, slightly weathered, fine grained
								-occasional lenses of medium to coarse sand
10	■	90	116 149	14 24			4.00	
								End of Boring No free water encountered Boring backfilled with grout
15								
20								
25								
30								

KLEINFELDER

PROJECT NO. 10-3011-46/GEO

LOG OF BORING NO. K-1
Hanson PSC Serramonte
Daly City, California

PLATE
A-2

Date Completed:	3/10/00	*Drilling Method:*	
Logged By:	E. Appel	*Hammer Wt.:*	140 lbs., 30" drop
Total Depth:	16.5 ft	*Notes:*	

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION
								Surface Elevation: **Estimated 134.0 feet (Above MSL)**
	■	51	105	20				**CLAYEY SAND (SC)** - brown, dense, fine grained, very moist -light brown
5	■	30	99	26		DS: C=400 psf; Phi=32° (See Plate B-1)		**SAND WITH CLAY** - gray brown with rust mottling, dense, fine grained, moist
10	■	68	102	23			>4.5	**SANDSTONE** - yellow brown with rust mottling, friable and dense, moist
15	■	66	103 133 99	23 28 19				
20								End of Boring No free water encountered Boring backfilled with grout
25								
30								

KLEINFELDER

PROJECT NO. 10-3011-46/GEO

LOG OF BORING NO. K-2
Hanson PSC Serramonte
Daly City, California

PLATE
A-3

L:\2001\PROJ\10\3011\46\10301146.GPJ

Date Completed:	3/10/00	Drilling Method:	
Logged By:	E. Appel	Hammer Wt.:	140 lbs., 30" drop
Total Depth:	11.5 ft	Notes:	

Depth, ft	FIELD Sample	FIELD Blows/ft	LABORATORY Dry Density pcf	LABORATORY Moisture Content %	LABORATORY Compress. Strength tsf	LABORATORY Other Tests	Pen, tsf	DESCRIPTION (Surface Elevation: Estimated 154.0 feet (Above MSL))
0								**SAND WITH LEAN CLAY** - brown gray, medium dense, fine grained, moist
4	■	35						-medium dense to dense
5–6	■	37	105	20				-dense
7								**SANDSTONE** - gray with orange mottling, dense, friable, moist
10–11	■	50	117	51		DS: C=220 psf; Phi=34° (See Plate B-2)	3.75	
11.5			103	18				End of Boring No free water encountered Boring backfilled with grout
15								
20								
25								
30								

KLEINFELDER	**LOG OF BORING NO. K-3** Hanson PSC Serramonte Daly City, California	PLATE **A-4**
PROJECT NO. 10-3011-46/GEO		

4/8/02 1:49:13 PM

Date Completed: 3/10/00

Logged By: E. Appel

Total Depth: 16.5 ft

Drilling Method:

Hammer Wt.: 140 lbs., 30" drop

Notes:

Depth, ft	FIELD Sample	FIELD Blows/ft	LABORATORY Dry Density pcf	LABORATORY Moisture Content %	LABORATORY Compress. Strength tsf	LABORATORY Other Tests	Pen, tsf	DESCRIPTION
								Surface Elevation: Estimated 172.0 feet (Above MSL)
								CLAYEY SAND (SC) - dark brown to orange brown with depth, medium dense, fine grained, moist
		21	95	28				**CLAYEY SAND (SC)** - brown gray with orange mottling, medium dense, fine grained, moist
5		22					3.00	
10		32	101	24				**SANDSTONE** - gray with orange fracture staining, friable, medium dense, moist
15		32	102 147 97	25 26 21			3.00	
								End of Boring No free water encountered Boring backfilled with grout
20								
25								
30								

KLEINFELDER

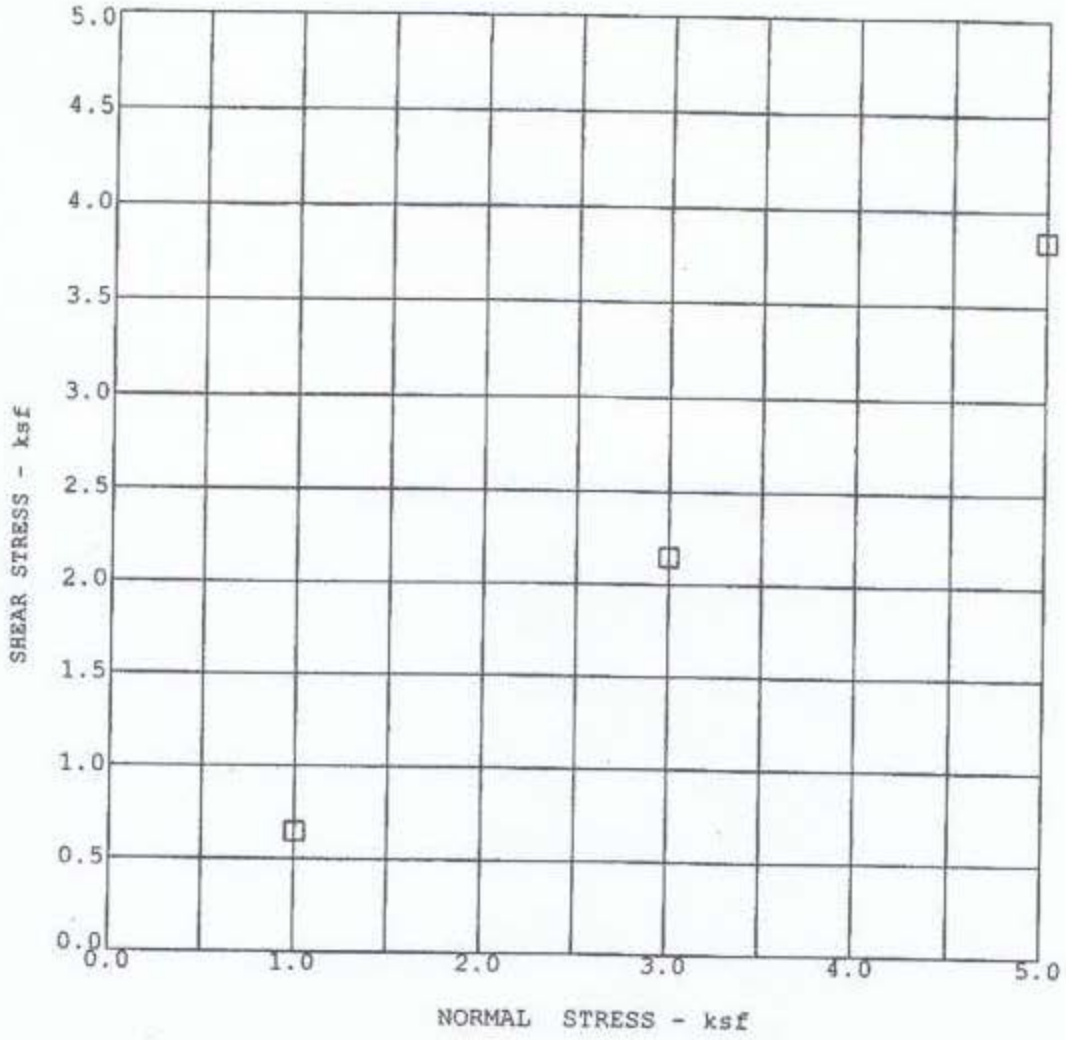
PROJECT NO. 10-3011-46/GEO

LOG OF BORING NO. K-4
Hanson PSC Serramonte
Daly City, California

PLATE A-5

L:\2001\01PROJ\10301146\10301146\10301146.GPJ

4/8/02 1:49:15 PM



TEST TYPE: CD/WET/STAGED

RATE OF SHEAR: 0.00192 in/min.

BORING NO: B-1

DEPTH: 20.0 ft

SOIL DESCRIPTION: Olive-Brown Sand

DRY DENSITY - pcf	98.2		
INITIAL WATER CONTENT - %	12.1		
FINAL WATER CONTENT - %	20.3		
NORMAL STRESS - psf	1000	3000	5000
MAXIMUM SHEAR - psf	649	2144	3838

FRICTION ANGLE = deg.

COHESION = ksf

KLEINFELDER

PROJECT NO. 10-3011-46/GEO

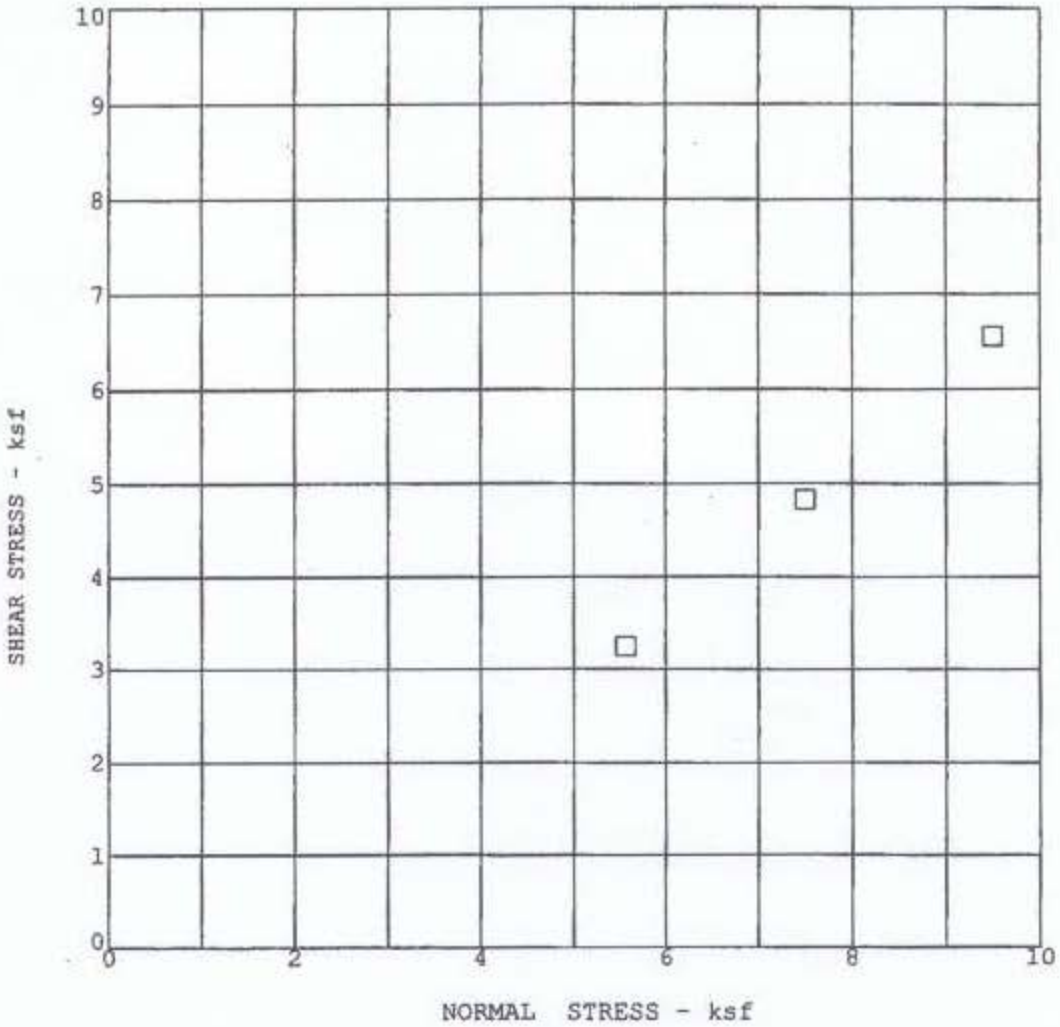
Hansen PSC
Serramonte Boulevard
Daly City, California
DIRECT SHEAR TEST

PLATE

B-1

L:\2002\02\PROJ\3100\3011\46\10301146.GPJ

4/8/02 1:46:58 PM



TEST TYPE: CD/WET/STAGED

RATE OF SHEAR: 0.00192 in/min.

BORING NO: B-1

DEPTH: 65.0 ft

SOIL DESCRIPTION: Gray Cemented Sand

DRY DENSITY - pcf	98.7		
INITIAL WATER CONTENT - %	24.5		
FINAL WATER CONTENT - %	27.7		
NORMAL STRESS - psf	5600	7500	9500
MAXIMUM SHEAR - psf	3243	4829	6559

FRICTION ANGLE = deg.

COHESION = ksf

KLEINFELDER

PROJECT NO. 10-3011-46/GEO

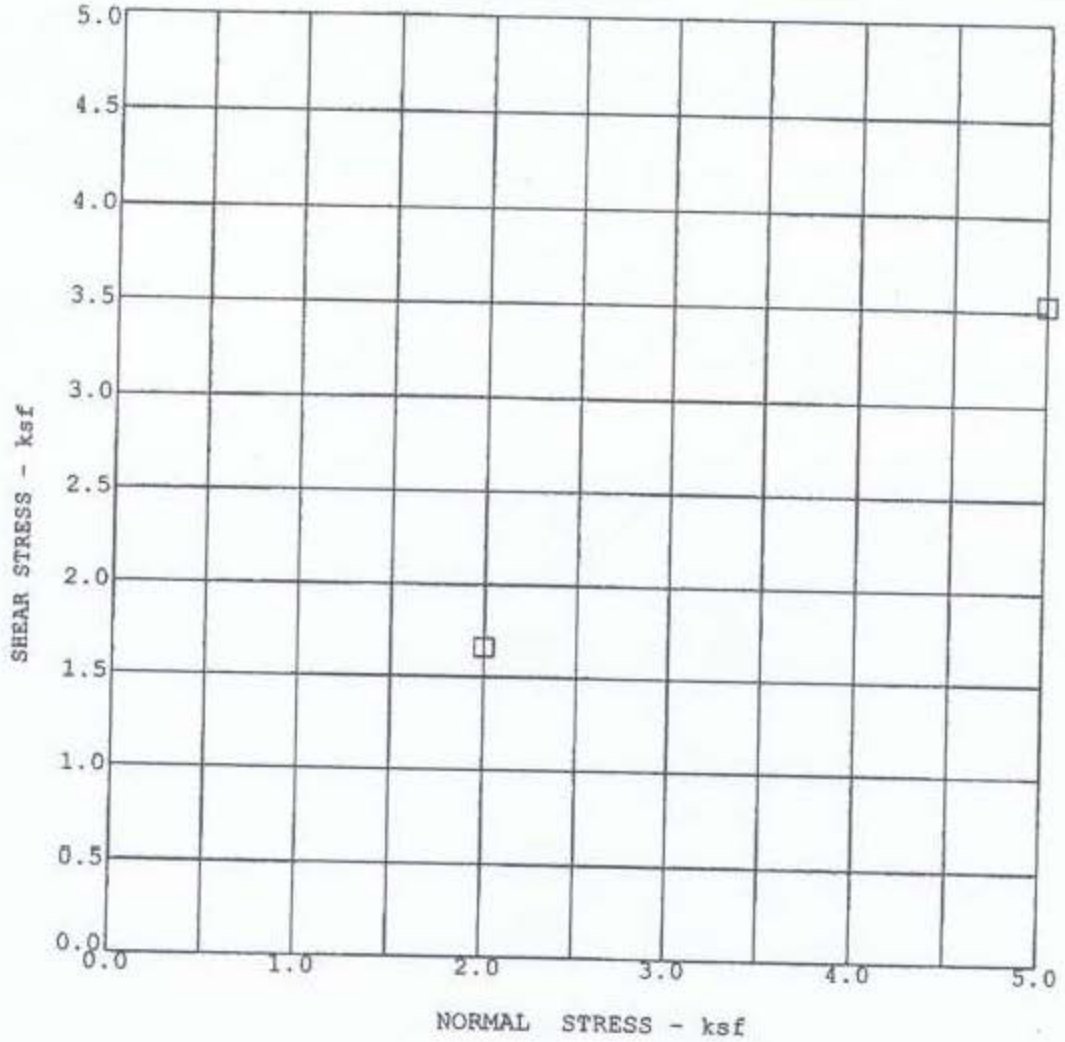
Hansen PSC
Serramonte Boulevard
Daly City, California
DIRECT SHEAR TEST

PLATE

B-2

L:\2002\02PROJ\1\3011461\3011461\3011146.GPJ

4/8/02 1:46:15 PM



TEST TYPE: CD/WET/STAGED

RATE OF SHEAR: 0.048 in/min.

BORING NO: B-5

DEPTH: 31.0 ft

SOIL DESCRIPTION: Light Yellow Brown Sand

DRY DENSITY - pcf	85.5		
INITIAL WATER CONTENT - %	41		
FINAL WATER CONTENT - %	30.8		
NORMAL STRESS - psf	2000	5000	
MAXIMUM SHEAR - psf	1658	3532	

FRICTION ANGLE = deg.

COHESION = ksf

KLEINFELDER

PROJECT NO. 10-3011-46/GEO

Hansen PSC
Serramonte Boulevard
Daly City, California
DIRECT SHEAR TEST

PLATE

B-4

L:\2002\02PROJ\10301146\10301146.GPJ

4/8/02 1:47:21 PM

California State Certified Laboratory No.2153

C E R C O
analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
Tel: 925.462.2771
Fax: 925.462.2775

14 March, 2002

Job No.0202187
Cust. No.11287

RECEIVED
18

Mr. Robert Ellis
Kleinfelder
1970 Broadway, Suite 710
Oakland, CA 94612

Subject: Project No.: 11-3011-46/GEO
Project Name: Hansen, PSC
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Ellis:

Pursuant to your request, Cerco Analytical has analyzed the soil sample submitted on February 22, 2002. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurement, this sample is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration reflects none detected with a detection limit of 15 mg/kg.

The sulfate ion concentration reflects none detected with a detection limit of and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at this location.

The pH of the soil is 7.4 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 310-mV, which is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard Jr.

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure