GEOTECHNICAL DESIGN REPORT

PROPOSED LANEY COLLEGE STUDENT CENTER MODERNIZATION PROJECT – NEW ENTRANCEWAY

Prepared for

The Peralta Community College District
333 East 8th Street
Oakland, CA 94606

Prepared by

Terraphase Engineering Inc.
1404 Franklin Street, Suite 600
Oakland, California

April 25, 2012

Project Number 0034-001-001

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terraphase engineering

April 25, 2012

Robert Dias
Director of Facilities
Peralta Community College District
333 East 8th Street
Oakland, California 94606

Subject: Geotechnical Design Report, Proposed Laney College Student Center Modernization Project – New Entranceway, 900 Fallon Street, Oakland, California

Dear Mr. Dias:

Terraphase Engineering Inc. (Terraphase) is pleased to present the attached Geotechnical Design Report for the Laney College Student Center Modernization Project, to be located at Laney College in Oakland, Alameda County ("the Site"), California. Design recommendations for project foundations are presented, along with other pertinent findings and conclusions. This report updates the previous geotechnical design report for the proposed project prepared by LFR Inc. and dated December 9, 2009 and supersedes our report of March 31, 2012. The report was revised to indicate that up to 0.6 inches of differential settlement may occur between the women's locker room, where the new entranceway is supported, and the adjacent student center (see Page 5).

Terraphase observed the installation of two subsurface probes in the women's locker room at Laney College to assess the subsurface soil conditions at the Site. The results of our assessment indicate that the Site will be suitable to support the proposed development, provided that the Site is prepared in accordance with the recommendations contained within the attached report. We have made one major revision to the LFR recommendations. We are recommending that any floors demolished during installation of the new foundation for the Student Center Entranceway be replaced with structural slabs tied into the foundation elements. This appears to be how the current floors are constructed and making new floors structural slabs will alleviate concerns about soil settlement below them.

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We appreciate the opportunity to provide this service for the Peralta Community College District and look forward to being of further assistance as the project proceeds. If you have any questions concerning the contents of the attached report, please feel free to call Jeff Raines at (510) 645-1853 at any time.

Sincerely,

Jeff Raines, P.E. (C51120), G.E. (2762)
Principal Geotechnical Engineer

Edmund Medley, PhD. P.E. (47602), C.E.G. (1604)
Principal Consultant

Copy to: Brian Swanson, C.E.G., California Geological Survey

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1.0 INTRODUCTION

Terraphase Inc. (Terraphase) has prepared this report to present the results of our geotechnical engineering design study for the proposed Laney College Student Center Modernization Project to be located at 900 Fallon Street in Oakland, Santa Clara County, California ("the Site"; Figure 1). The modernization project consists of the addition of a new three-story glass atrium on the front of the building and the addition of shear walls on the building interior to stiffen the torsional resistance of the structure. This Geotechnical Design Report is based on the proposal prepared for the Peralta Community College District ("the District") by Terraphase dated October 28, 2011.

Woodward Clyde Sherard (1966) performed a preliminary geotechnical investigation of the Site in 1966 prior to the construction of the campus. Two boring logs provided by the Bay Area Rapid Transit (BART) District are included in Appendix B. The locations of the BART and Woodward Clyde Sherard borings are shown on Figure 2 along with the locations of the two subsurface probes installed in 2012.

This report was prepared in general accordance with the California Department of the State Architect (DSA) requirements for the design of a public school. The DSA consults with the California Geologic Survey (CGS) to assess whether the geotechnical work performed for a school site is sufficient. The CGS requirements for the geotechnical reports for school sites are presented in CGS Special Publication 48 (CGS 2011).

2.0 PROJECT DESCRIPTION

The proposed project consists of a new entranceway to the student center and the addition of several shear walls to the interior of the structure. The new entranceway is a glass-walled structure to be constructed on the northeastern side of the building. It is to be founded on 12 footings at the basement level of the student center inside the adjacent woman's locker room. The footing loads are estimated to be 80 kips (Kiland 2009). The new shear walls interior to the building will be supported on the existing foundation. The project structural engineer (Kiland 2009) does not believe that the new shear walls – which are to be installed to address torsional issues related to asymmetries in the building structure - will significantly change the loads on the existing foundation. However, the loads on the existing foundation were not assessed in accordance with the existing California Building Code and the seismic load demand on the existing foundation is likely greater than the original design contemplated due to changes in seismic design requirements.

3.0 SCOPE OF STUDY

Based on our understanding of the project, the following scope of services was formulated and completed:

- Terraphase observed and logged the installation of two subsurface probes in the area where the new entranceway is to be founded, one to 82 feet below the ground surface (bgs) and the other to 21 feet bgs.
- Terraphase reviewed the existing geotechnical information available from the District.
- Terraphase revised the seismic risk assessment for the building prepared by LFR (2009a) including both probabilistic and deterministic design spectra.
- Terraphase prepared foundation recommendations.
- Terraphase documented the results of the work in this report.
- Terraphase addressed comments from CGS on the LFR design report (2009b).

This report summarizes our study results and presents our design and construction recommendations, as well as the subsurface data on which they are based.

4.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

4.1 Subsurface Exploration

On March 16, 2012, Terraphase observed the installation of two subsurface borings in the women's locker room below the location of the proposed entrance. One probe was a cone penetration test (CPT) probe (CPT B-2) advanced to 82 feet below the ground surface (bgs). The second boring (Boring B-1) was a soil direct push sample advanced to 21 feet bgs for the purpose of collecting soil samples. The purpose of the deeper probe was to assess soil material properties to at least a depth 10 feet below the potential depth of deep foundation elements used to support the new entranceway. The second probe was advanced to 21 feet bgs to collect soil samples in the interval between 10 and 21 feet to assess soil index properties for liquefaction susceptibility assessment. Based on stratigraphy from the initial CPT probe, soil samples were collected at depths of 10 to 11, 11 to 12, 14.5 to 15.5 and 20 to 21 feet bgs. CPT and Boring Logs are appended to this report in Appendix D.

The stratigraphy encountered in the Terraphase CPT probe was similar to the stratigraphy encountered by Woodward Clyde Sherard in 1966 and BART prior to installing the BART tubes. The Woodward Clyde Sherard and BART boring logs are appended to this report in Appendix B along with drawings of the BART tubes and some of the original structural plans for the Student Center.

4.2 Laboratory Testing

The soil samples were analyzed by the Cooper Geotechnical Testing Laboratory in Palo Alto, California for index properties (Atterberg Limits for cohesive soil samples and particle size distributions [gradation] for cohesionless soil samples). The index tests were conducted both for material identification purposes and to assist in the assessment of liquefaction potential of the soils. The soil sample collected between 11 and 12 feet bgs was also assessed for saturated conductivity and pH to assess if the soils near the groundwater are aggressively corrosive.

Soil samples collected by LFR in 2009 had also been submitted to the Cooper Testing Laboratory in Palo Alto for analysis for corrosion characteristics and Atterberg Limits. Woodward Clyde Sherard (1966) performed tests on soil samples collected from the subsurface at the Site. Woodward Clyde Sherard testing included shear strength, index properties (e.g., Atterberg Limits), moisture density testing, and corrosion parameters. The results of the latter testing program are presented in Appendix C.

5.0 SITE AND SUBSURFACE CONDITIONS

5.1 Site Description

The Site is located at 900 Fallon Street in Oakland, California (Figure 1). The Site is located at an area that was originally within the open water channel of the Oakland Estuary in the late 19th century (United States Geological Survey [USGS] 1899). By 1900 all but the extreme southern perimeter of the Site had been filled in to form a park (Soderberg 1899). By 1939 the entire Site had been filled. During World War II the Site was used for temporary housing (USGS 1942).

According to published topographic maps, the Site lies at an elevation of approximately 15 feet above mean sea level (msl) and is essentially flat. The local topography slopes to the southeast toward the Oakland Inner Harbor with a slope of approximately 1 percent.

5.2 Subsurface Conditions

The site soils consist of fill over marsh deposits over Younger San Francisco Bay Mud overlying stiffer clays and sandy clays, locally characterized as Old Bay Mud. The fill and Younger Bay Mud extend to depths between about 20 and 27 feet below the original ground surface (approximately 5 to 12 feet below msl). The floor of the women's locker room is likely at the original grade or a few feet below it. A 5-foot-thick layer of peat was encountered in Woodward Clyde Sherard Hole 1 between about 9 and 14 feet below original ground surface (bgs). Woodward Clyde Sherard Hole 1 was located at the northeastern corner of the future Student Center Site. Geologic cross sections of the Campus from the Woodward Clyde Sherard report (1966) are reproduced in Appendix B of this report. Figures 3 and 4 in this report present updated cross-sections specific to the Student Center Entranceway project incorporating the latest and historical subsurface data. Locations of the BART tubes are based on drawings supplied by BART which are included in Appendix B.

LFR installed two hand auger borings through the floor of the women's locker room in 2009 (HA-1 at the location of Boring B-1, and HA-2 at the location of CPT B-2). Soil encountered in HA-1 consisted primarily of a brown silty-clayey sand (SC) of low plasticity (Plasticity Index of 7 and Liquid Limit of 24.5). Soil encountered in HA-2 consisted primarily of a uniform sand and likely represents trench bedding material.

5.3 Groundwater

Groundwater was encountered at approximately 16 feet bgs (approximately -4 msl) in the Woodward Clyde Sherard borings. Terraphase encountered groundwater at approximately 10 feet bgs in Boring B-2. Groundwater at ten feet bgs is consistent with the groundwater map presented in CGS 2003b. A groundwater elevation of +2 feet msl (NGVD29) was used in the liquefaction analyses. A sensitivity analysis was performed to assess the impact of a higher groundwater elevation (+7 feet msl NGVD29) on liquefaction settlements. The higher groundwater elevation did not change the predicted liquefaction settlements.

6.0 DISCUSSION

6.1 Site Conditions

The soil conditions near the surface at the Site consist of compressible clays and organic soils and undocumented fill. Because of the predicted high settlements for buildings founded on shallow foundations upon such materials the Student Center was originally constructed on piles. Structural drawings of the women's locker room could not be located, but it is also likely founded on piles given that there does not appear to be any differential settlement between the women's locker room and the Student Center.

6.2 Foundations

Because differential settlements between the new entranceway (which is to be mostly glass) and the Student Center would be unacceptable, Terraphase has not considered shallow foundations for the entranceway. The Site is an open plaza in front of the Student Center and is the roof of the women's locker room below. Because a pile driving rig cannot be mobilized to the Site due to weight restrictions on the plaza, the only feasible deep foundation options are micropiles and hand-dug caissons. Micropiles are 2- to 4-inch-diameter steel pipes that are pushed into the ground using hydraulic jacks or small-diameter cast-in-drilled-hole (CIDH) concrete piers with a single high-strength reinforcing rod in the center. The hole for the CIDH micropiles is typically installed using a lowoverhead drill rig. Hand-dug caissons are holes excavated by laborers using shovels and other hand tools. Because the holes are hand dug, they must be at least 3 feet in diameter. Because of the high cost and hazard, hand-dug caissons are not recommended and not further considered herein.

6.3 Settlement Estimates (Including Seismic Shakedown)

Static settlements for a foundation supported on micropiles are estimated to be less than ¼ inch for design loads of 25 kips per pile.

Total seismic settlements may be as large as 1 inch with differential settlements of up to ½ inch over the longitudinal axis of the new entranceway. The Entranceway should be designed to accommodate up to 0.6 inches of differential settlement between the new entranceway and the Student Center.

7.0 DESIGN RECOMMENDATIONS

7.1 Site Preparation

It is our understanding that the concrete slab at the floor of the women’s locker room below the new entranceway will be completely removed. This will also require the removal of the showers and associated water pipelines and drains.

7.2 Lateral Loads

Resistance to lateral loads from wind or seismic forces would be obtained from passive resistance on the vertical faces of the grade beams and pile caps. We recommend an equivalent fluid pressure of 240 per cubic foot (pcf) be used for a passive resistance value acting on faces of embedded foundation members. The resistance posed by the top foot of soil should be neglected in these calculations. The friction on the bottoms of footings and nonstructural slabs-on-grade also may be included in the design. A friction coefficient of 0.3 (NAVFAC 1986) can be used for calculating base friction for pile caps and grade beams. Where a vapor barrier is used between slab-on-grade and soil, a friction coefficient of 0.20 is recommended. These friction coefficient values do not include a factor of safety. If base friction and passive resistance are combined, the passive resistance should be reduced by 1/3.

7.3 Excavations and Backfill

The slab of the women’s locker room is up to 18 inches thick and likely will also require the relocation of a number of utilities – the entranceway footprint includes the locker room shower area, which contains numerous floor drains and water lines.

Trenches should be backfilled above the pipe bedding sand with compacted fill, in accordance with the stricter of the recommendations contained in this section or in accordance with local requirements. Fill material should be placed in lifts no greater than 8 inches in loose thickness and compacted by mechanical means. Trench backfill should be compacted to at least 95 percent relative compaction.

Imported fill materials should be approved by the Engineer before being brought to the Site. Imported fill shall be certified as clean from the source (not from former industrial sites or similar locations; and not chemically contaminated). Requirements for environmental testing of fill to be used for school sites are contained in California Department of Toxic Substances Control (2001). Imported fill should be nonexpansive, granular in nature, and meet the following requirements: minimum R-Value of 35 (Caltrans 301), maximum expansion index of 25 (UBC 18-2), and maximum plasticity index of 12 (ASTM D4318). The soil should be compacted in lifts no greater than 8 inches loose to a minimum of 95 percent relative compaction. A representative of the geotechnical engineer should observe site grading, including stripping, scarifying, and placing and compacting of fill and backfill.

Controlled density fill, if used, shall be composed of cementitious materials, aggregate, water, and an air-entraining admixture, as follows:

1. Cementitious materials shall be Portland cement in combination with fly ash.

2. Admixture shall be an air-entraining agent.

3. Aggregate Content: CDF mixture shall contain no aggregate larger than 3/8 inch. Amount passing a No. 200 sieve shall not exceed 12 percent. No plastic fines shall be present.

4. Air Content: Total calculated air content of the sample, prepared in accordance with ASTM C231, shall not exceed 30 percent.

5. Strength: Controlled density fill shall have an unconfined compressive strength at 28 days of from 50 pounds per square inch (psi) to a maximum of 150 psi.

Pile cap and grade beam concrete should be poured neat against properly moisture-conditioned, undisturbed soil; compacted soil; or compacted fill. Any disturbed or softened material encountered at the bottom of the pile cap or grade beam excavations should be removed to expose firm bearing material. Excavations should be kept moist before concrete placement.

Before issuing the construction bids, the geotechnical engineer should review the foundation plans and prepare a review letter. In addition, the geotechnical engineer should observe foundation installation operations.

7.4 Micropiles

7.4.1 General

Presented below is a preliminary design for steel push. Push micropiles are small-diameter pipes pushed into the ground using hydraulic jacks.

The design presented below is preliminary for feasibility purposes. Final designs will be the responsibility of the specialty Contractor selected for the work.

7.4.2 Steel Push Micropiles

7.4.2.1 General

Suitable piles for this project include 3.5 and 4.5-inch outside diameter schedule 80 steel pipe piles (micropiles). The piles should be galvanized for corrosion protection. The piles are to be pushed into place using a hydraulic jack equipped with a load cell to verify the piles have reached the required capacity. Micropiles are frequently used in end bearing and are equipped with a “shear breaker” on their ends to eliminate side friction. Terraphase recommends that shear breakers not be used for this installation.

7.4.*2.2* *Micropile Capacity*

Capacity of the micropiles will be obtained from side friction in the bearing strata (Old Bay Mud stiff clay) located approximately 25 feet below the base of the locker room. Frictional capacities of the micropiles based on CPT results (Appendix D) are presented in Table 1 below. The frictional resistance is based on an average undrained shear strength of 5,000 pounds per square foot (psf) in the top 20 feet of the bearing strata. The soil undrained shear strengths were divided by 2 (alpha = 0.5), and these values were applied as adhesions to the surface areas of the piles. End bearing was taken as equal to 9 (Su)A, where Su is the undrained shear strength (5,000 psf) and A is the cross-sectional area of the micropiles. There is no reason to expect that there will be any settlement in the Younger Bay Mud above the bearing strata as no fill is being placed on them, however, the full capacity of the Younger Bay Mud was applied to the micropiles as a downdrag load (negative capacity).

The capacities shown in Table 1 are theoretical capacities presented for the convenience of the Contractor. Actual jacking loads will be used in the field to determine final pile capacities. The Contractor is responsible for selecting the push pile to be used in the work to obtain the required design capacity (30 kips ultimate).

Table 1
Theoretical Pile Length Requirements for 20 kip Capacity Piles
Laney College Student Center
Oakland, California

Pile Diameter (in)	3.5	4.5
Surface Area/ft (ft)	0.92	1.18
Probable Load per foot upper strata (lbs)	460	590
Probable Load per foot bearing strata (lbs)	2291	2945
Feet required in bearing strata (ft)	17	14
Total Length per Pile (ft)	39	36
End Bearing Capacity (lbs)	3,007	4,970
Required Installed Capacity (lbs)	41,000	45,000

The capacities presented above are theoretical. The values in "Feet required in bearing strata" row in Table 1 represent the amounts of penetration the micropiles will require in the bearing strata, which results in a total resisting force of 30,000 pounds. Applying a factor of safety of 1.5 to the 30,000 pound ultimate capacity results in a 20 kip allowable capacity micropile. A factor of safety of 1.5 was selected because every pile capacity is required to be measured directly in the field during installation. Actual push micropile lengths will probably vary due to heterogeneity of the site soils.

The final row in Table 1 presents the required resistance of the micropiles during installation. These values are obtained by adding the resistance of the upper 25 feet of the piles to the required design capacity.

Even though the piles are jacked into place slowly, excess pore pressures will be generated in the saturated clays below the Site, which will reduce the frictional resistance of the micropiles. Hence, the long-term installed capacity of the micropiles will be larger than the instantaneous to short-term capacities measured during installation.

7.4.*2.3* *Tensile Capacity*

Tensile capacity of the piles can be taken to be 20 kips minus the end-bearing capacity given in Table 1.

7.4.*2.4* *Pile Testing*

Terraphase recommends that two of the micropiles be tested by loading them to twice their design capacity. The tests can be conducted in either tension or compression.

7.5 California Building Code (CBC) Seismic Design Criteria

Appendix A of this report contains the seismic design criteria for the project.

7.6 Soil Corrosivity

One soil sample was analyzed for sulfate, chloride, pH, and resistivity in 2009 by LFR. A soil sample collected from 11 to 12 feet bgs during the current effort was analyzed for pH and resistivity.

The sulfate concentration detected in the 2009 soil sample was 354 parts per million (ppm). Per the American Concrete Institute (1995), a sulfate concentration of less than 1,000 ppm is "negligible". Hence sulfate resistant concrete is not necessary at the Site.

The chloride content detected in the 2009 soil sample was 42 ppm. According to the California Department of Transportation (Caltrans) Corrosion Guidelines (Caltrans 2003), a chloride content of less than 500 ppm does not indicate a corrosive environment.

The 2009 and 2012 soil pHs were measured to be 7.8. According to the Caltrans Corrosion Guidelines (Caltrans 2003), a pH greater than 5.5 does not indicate a corrosive environment.

The minimum electrical resistivity of the 2009 soil sample was 870 ohm-centimeters while the minimum electrical resistivity of the 2012 soil sample was 1500 ohm-centimeters. Caltrans considers soils with electrical resistivity of less than 1,500 ohm-centimeters to be corrosive.

Woodward Clyde & Associates (1968) performed an extensive boring program at Laney College to assess soil corrosivity potential. They concluded that the fill and upper soil strata at the Site were "generally corrosive for unprotected metal piles by galvanic action." Woodward Clyde's sulfate concentration resulted ranged from 68 to 1,320 ppm. ACI considers sulfate concentrations between 1,000 and 2,000 ppm to be "Moderate" and recommends that Type II Portland Cement be used for concrete to be in contact with soils with moderate sulfate concentrations. The Woodward Clyde report is appended to this report at the end of Appendix B.

We are not corrosion engineers. Nevertheless, because the suite of available data indicates conflicting indications of corrosion potential, it appears that corrosion protection <u>is warranted if steel pipe piles are selected</u>.

7.7 Expansive Clays

As the Site is very close to the Oakland Inner Harbor, there should be little change in the elevation of the groundwater table with the seasons. In addition, the Site has been completely covered by concrete for almost 50 years. Hence, significant changes in soil water contents are not expected and expansion or contraction of the site soils is unlikely. The soil sample collected from HA1 had a low plasticity index (7) which does not indicate significant expansive potential. The soil sample collected at 20 to 21 feet bgs was found to have a plasticity index of 41 which is indicative of expansive clays. This soil sample was collected to be characteristic of the sensitive clay found in CPT B-2 between 17 and 21 feet bgs. As this strata is below the likely lowest level of the groundwater table, the moisture content is unlikely to ever change and hence the clay is unlikely to ever shrink or swell significantly.

7.8 Slabs on Grade

The existing floors in the Women's Locker Room are unusually thick and appear to be designed as structural slabs bridging between pile caps. If existing floors are demolished as part of the foundation installation project, they should be replaced with similar structural slabs bridging between pile caps. Terraphase recommends that the replacement floors be underlain by a moisture barrier. A typical moisture barrier should include a capillary moisture break consisting of at least 4 inches of clean, free-draining gravel or crushed rock (1/2- to ¾-inch gradation) overlain by a moisture-proof membrane of at least 10 mils thick (Grace FlorPrufe or equivalent). The vapor retarder should be covered with 2 inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. Water should not be allowed to accumulate in the capillary break or sand prior to casting the slab.

The vapor retarder should meet the requirements for Class C vapor retarders as given in ASTM Standard E1745-97. The vapor retarder should be installed in general accordance with the methodology documented in ASTM Standard E1643-98. These requirements include overlapping seams by at least 6 inches, taping seams, and sealing penetration through the vapor retarder. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in the following table.

Material underlying the slabs should conform to the following gradation specifications:

Table 2
Subslab Foundation Materials
Laney College Student Center
Oakland, California

	Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	1 inch	90 – 100
	¾ inch	30 – 100
	½ inch	5 – 25
	3/8 inch	0 – 6
Sand	No. 4	100
	No. 200	0 – 5

The sand overlying the membrane should be moist at the time concrete is placed. There should be no free liquid in the sand.

8.0 DESIGN REVIEW AND CONSTRUCTION MONITORING

Terraphase recommends that the geotechnical aspects of the project be reviewed by Terraphase during the design process. The scope of services may include:

- assisting the design team in providing specific recommendations for special cases
- reviewing the foundation design and evaluating the overall applicability of our recommendations
- reviewing the proposed construction techniques to evaluate whether they satisfy the intent of our recommendations
- reviewing and stamping drawings

Terraphase recommends that foundation construction and earthwork performed during construction be monitored by a qualified representative from our office, including:

- site preparation (stripping and grading)
- installation of micropiles

Terraphase's representative should be present to observe the soil conditions encountered during construction to evaluate the applicability of the recommendations presented in this report to the soil conditions encountered and to recommend appropriate changes in design or construction procedures, if conditions differ from those described herein.

9.0 Geological Hazards

9.1 Introduction

This section of the report presents the results of the geologic hazards evaluation conducted by Terraphase for the proposed modification to the Laney College Student Center.

This report includes our opinions concerning potential geologic hazards that may have an impact on site development and could potentially impede the performance of the proposed project. This report was prepared in general accordance with California Educational Code Section 17212.5. Conclusions presented in this report are based in part on the published data discussed in this report, and on our experience with the types of geologic hazards applicable to sites located in Northern California. These conclusions should not be extrapolated to other areas outside the Site without our prior review.

This report addresses the geological hazards included on the California Geological Survey (CGS) Note 48 (Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings).

9.2 Location And Site Description

The Site (Figures 1, 2 and 5) is located in Oakland, California, in the County of Alameda. The Laney College Student Center ("the Student Center") is an existing four-story building. It was constructed in late 1968 and is one of the original campus buildings. An open plaza lies to the north of the building (where the new entranceway will be located). The Women's Locker Room is located below the plaza. According to the building plans (Skidmore, Owings & Merrill 1968) the Student Center was to be founded on piles. The choice of pile type was left up to the contractor and could have been 14-inch-square reinforced, prestressed concrete piles, step-taper concrete piles or steel pipe pile/concrete composite piles. The Woodward Clyde files on the project were reviewed. The files indicated that the student center was partially constructed on 14-inch-square prestressed (but not reinforced) concrete piles and partially on 24-inch-diameter drilled piers. The drilled piers were installed next to the Bay Area Rapid Transit (BART) tubes. The building overlies the BART system subway tunnels. The footprint of the building is approximately 10,000 square feet in area.

The Site was located in open water in the Oakland Estuary in the late 19th century (USGS 1899). By 1900 (Soderberg 1899) all but the extreme southern perimeter of the Site had been filled in to form a park (attached Figure 6 is a photograph of the estuary being filled). By 1939 (USGS 1942) the entire Site had been filled. During World War II the Site was used for temporary housing.

The center of the Student Center is located at a latitude of approximately 37.79604° North, and a longitude of approximately 122.26299° West. According to published topographic maps (Figure 5), it lies at an elevation of approximately 15 feet above mean sea level (msl), and is essentially flat. The local topography slopes to the southeast toward the Oakland Inner Harbor with a slope of approximately 1%.

The Site was inspected by Mr. Jeffery Raines, P.E. (51120), G.E. (2762), on August 28, 2009 and March 16, 2012. No obvious evidence of potential geological hazards was observed at the Site on that date.

9.3 Purpose And Scope Of Services

The purpose of this evaluation was to identify major geological and seismic hazards that could potentially preclude the proposed school siting or make the proposed construction economically unfeasible. Terraphase performed the following scope of services for this geologic hazard evaluation:

- conducted a review of geologic hazards data
- conducted a site inspection
- prepared a report of pertinent findings with respect to seismic, geologic, and geotechnical engineering issues, including:
    - pertinent site maps showing the approximate project location
    - local geologic setting, faulting, and seismicity
    - site liquefaction potential, ground rupture potential, and other geologic and seismic hazards
    - flood inundation potential

9.4 SITE CONDITIONS

The Site was extensively studied in 1966 by Woodward Clyde Sherard during the initial design of the campus. The site soils consist of fill over marsh deposits over Younger San Francisco Bay Mud overlying stiffer clays and sandy clays. The fill and Younger Bay Mud extended to depths between 20 and 27 feet below the original ground surface (approximately to 8 feet below msl). A 5-foot-thick layer of peat was encountered in Woodward Clyde Sherard Boring 1 between 9 and 14 feet below ground surface (bgs). Boring 1 was located at the northeastern corner of the future Student Center Site. Most of this peat was likely removed during the construction of the BART tubes. Groundwater was encountered at approximately -4 feet mean sea level in the Woodward Clyde Sherard borings.

9.4.1 Geology and Soils

9.4.1.1 Geology

A geological map of the Site (based on Graymer et al. 1996) is presented on Figure 7. The topography of the San Francisco Bay Area consists of north- to northwest-trending mountain ranges and intervening valleys that are characteristic of the Coast Range geomorphic province. The Coast Ranges consist of the Mendocino Range to the north of the Bay, the Santa Cruz Mountains west of the Bay, and the Diablo Range to the east of the Bay. The San Andreas Fault Zone lies to the west, and represents a major boundary that separates Franciscan Complex rocks on the North American Plate from Salinian basement rocks of the Pacific Plate.

The geology of the Bay Area is made up primarily of three different geologic provinces: the Salinian block, the Franciscan Complex, and the Great Valley sequence. The Salinian block is located west of the San Andreas Fault and is composed primarily of granitic plutonic rocks.

The Mesozoic Franciscan Complex is bounded on the eastern side by the Hayward Fault and on the western side by the San Andreas Fault. The Franciscan rocks represent pieces of former oceanic crust that have been accreted to North America by subduction and collision. These rocks are primarily deep marine sandstone and shale. However, chert and limestone are also found within the assemblage. The rocks of the Franciscan Complex are prone to landslides.

East of the Hayward Fault is the Great Valley sequence, which is composed primarily of Cretaceous and Tertiary marine sedimentary rocks in the Bay Area. These rocks are also prone to landsliding.

The Coast Ranges represent northwest-southeast-trending structural blocks comprised of a variety of basement lithologies that are juxtaposed by major geologic structures. The Coast Ranges-Sierran Block boundary zone lies to the east of the Site. To the west, the major boundary is the San Andreas Fault Zone, which separates Franciscan Complex rocks of the North American plate from the Salinian basement rocks on the Pacific plate. The Coast Ranges ophiolites within the Franciscan Complex have been deformed by a series of thrust faults, most of which appear to be inactive.

The Diablo Range extends from the Sacramento River Delta, south along the western side of the San Joaquin Valley. Rocks of the Mesozoic Great Valley are thrust upon Franciscan basement along the San Joaquin Valley margin, and are covered locally by younger sediments of Paleocene to Pleistocene age.

Faults of the San Andreas system separate the Diablo Range from the remainder of the Coast Ranges. Mount Diablo is separated from the western East Bay hills by the Calaveras Fault and from the southern extension of the Diablo Range by the Livermore Valley, an east-west-trending Cenozoic basin. The Diablo Range is bounded to the east by the Coast Range-Sierran Block boundary zone, which typically is represented by a series of blind and partially concealed thrust faults (Wong et al. 1988; Unruh and Moores 1992). The eastern side of Mount Diablo is bounded by the San Joaquin Fault (Sowers et al. 1992).

The Diablo Range comprises a series of large asymmetrical anticlines, with intervening synclines. The anticlines are composed of Franciscan Complex rocks, while the synclines contain younger rocks. The folds are frequently cut by east- and west-verging thrust faults. These thrust faults are displaced or truncated by strike-slip movement on the northwest-striking, right-lateral faults of the San Andreas fault system.

9.4.1.2 Soils

Woodward Clyde Sherard presented three cross-sections of the campus subsurface in their report (1966), which are included in this report in Appendix B. The soils underlying the Student Center and adjacent Women’s Locker Room consist of up to 5 to 7 feet of fill overlying marsh deposits (peat) overlying Younger Bay Mud overlying stiffer clay and sandy clay. Woodward Clyde Sherard encountered 4 feet of loose, very-clayey sand between 21 and 25 feet bgs in Boring 1 adjacent to the Student Center.

9.4.2 Hydrology and Hydrogeology

Due to the proximity of the Oakland Inner Harbor and Lake Merritt, groundwater is expected to be very close to mean sea level.

9.4.3 Faulting and Seismicity

The known regionally active faults within 50 kilometers of the Site that are capable of producing significant ground shaking at the Site are listed in Table 1 and shown on Figure 8. Activity was determined by slip rates, as per the CGS (Petersen et al. 1996).

Table 3 includes an estimate of the mean peak ground acceleration (PGA) and the Modified Mercalli Intensity (MMI) likely to be felt at the Site due to earthquakes on the individual faults. The MMI scale is described in Table 4. The calculated MMI should be considered to be a rough order of magnitude estimate; it is presented here because it is easier to interpret than PGAs.

MMI was evaluated using EQFAULT software (Blake 2000a). EQFAULT uses the inverse of the Murphy and O'Brian (1978) acceleration – intensity equation to calculate the MMI:

$$I_{mm} = [\log_{10}(980.7 * a_{Hg}) - 0.29]/0.24$$
$$a_{Hg} = horizontal\ acceleration\ (g)$$

Appendix A contains a probabilistic and deterministic seismic hazard assessment for the Site.

Table 5 presents the significant historical earthquakes that have occurred in the site vicinity.

Table 3
Known Active Earthquake Faults within 50 Kilometers of the Site
Laney College Student Center
Oakland, California

Abbreviated Fault Name	Approx. Distance, miles (km)	Maximum Earthquake Mag. (Mw)	Peak Ground Accel. (g)	Est. Site Intensity, Modified Mercalli
HAYWARD (HN+RC)	3.5 (5.6)	7.1	0.416	X
HAYWARD (HS+HN+RC)	3.5 (5.6)	7.3	0.426	X
HAYWARD (FLOATING)	3.5 (5.6)	6.9	0.402	X
HAYWARD (HS+HN)	3.5 (5.6)	6.9	0.403	X
HAYWARD (HS)	3.5 (5.6)	6.7	0.387	X
HAYWARD (HN)	3.5 (5.6)	6.5	0.376	IX

Abbreviated Fault Name	Approx. Distance, miles (km)	Maximum Earthquake Mag. (Mw)	Peak Ground Accel. (g)	Est. Site Intensity, Modified Mercalli
CALAVERAS (FLOATING)	13.9 (22.4)	6.2	0.118	VII
CALAVERAS (CC+CN)	13.9 (22.4)	6.2	0.121	VII
CALAVERAS (CS+CC+CN)	13.9 (22.4)	6.9	0.165	VIII
CALAVERAS (CN)	13.9 (22.4)	6.8	0.158	VIII
MOUNT DIABLO (MTD)	14.4 (23.1)	6.7	0.182	VIII
SAN ANDREAS (SAP+SAN+SAO)	14.5 (23.4)	7.8	0.213	VIII
SAN ANDREAS (SAS+SAP)	14.5 (23.4)	7.4	0.187	VIII
SAN ANDREAS (SAP+SAN)	14.5 (23.4)	7.7	0.201	VIII
SAN ANDREAS (SAP)	14.5 (23.4)	7.2	0.171	VIII
SAN ANDREAS	14.5 (23.4)	7.9	0.217	VIII
SAN ANDREAS (FLOATING)	14.5 (23.4)	6.9	0.158	VIII
SAN ANDREAS (SAS+SAP+SAN)	14.5 (23.4)	7.8	0.208	VIII
CONCORD/GV (FLOATING)	16.6 (26.7)	6.2	0.1	VII
CONCORD/GV (CON)	16.6 (26.7)	6.3	0.105	VII
CONCORD/GV (CON+GVS+GVN)	16.6 (26.7)	6.7	0.133	VIII
CONCORD/GV (CON+GVS)	16.6 (26.7)	6.6	0.127	VIII
SAN ANDREAS (SAN)	17.0 (27.4)	7.5	0.168	VIII
SAN ANDREAS (SAN+SAO)	17.0 (27.4)	7.7	0.183	VIII
SAN GREGORIO (SGS+SGN)	18.8 (30.2)	7.4	0.155	VIII
SAN GREGORIO (FLOATING)	18.8 (30.2)	6.9	0.128	VIII
SAN GREGORIO (SGN)	18.8 (30.2)	7.2	0.144	VIII
CONCORD/GV (GVS)	19.6 (31.6)	6.2	0.089	VII
CONCORD/GV (GVS+GVN)	19.6 (31.6)	6.5	0.106	VII
HAYWARD (RC)	22.2 (35.7)	7	0.116	VII
GREENVILLE (GN)	24.2 (39.0)	6.7	0.094	VII
MONTE VISTA - SHANNON	24.8 (39.9)	6.7	0.116	VII
WEST NAPA	25.5 (41.0)	6.5	0.084	VII
GREAT VALLEY 5	29.0 (46.6)	6.5	0.093	VII

Notes: The expected peak ground acceleration (PGA) is the mean value based on Abrahamson & Silva (1997)

CON =Concord
GVS =Green Valley South
GVN =Green Valley North
GV = Green Valley
CS = Calaveras South
HS = Hayward South
RC = Rodgers Creek
SAO = San Andreas Offshore
SAS = San Andreas South
SGN= San Gregorio North
GN = Greenville North
GS = Greenville South
CC = Calaveras Central
CN = Calaveras North
HN = Hayward North
MTD = Mount Diablo Thrust
SAN = San Andreas North
SAP = San Andreas Peninsula
SGS= San Gregorio South
km = kilometers

Table 4
Applicable Portions of Modified Mercalli Intensity Scale
Laney College Student Center
Oakland, California

Intensity	Shaking	Summary	Description
VII	Strong	Nonstructural Damage	Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D, including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices (also unbraced parapets and architectural ornaments). Some cracks in masonry C. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.
VIII	Very Strong	Moderate Damage	Steering of motor cars affected. Damage to masonry C; partial collapse. Some damage to masonry B; none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.
IX	Violent	Heavy Damage	General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. (General damage to foundations.) Frame structures, if not bolted, shifted off foundations. Frames racked. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluvial areas sand and mud ejected, earthquake fountains, sand craters.
X	Very Violent	Extreme Damage	Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.

Notes:

Masonry A: Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces.

Masonry B: Good workmanship and mortar; reinforced, but not designed in detail to resist lateral forces.

Masonry C: Ordinary workmanship and mortar; no extreme weaknesses like failing to tie in at corners, but neither reinforced nor designed against horizontal forces.

Masonry D: Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.

Table 5
Historical Earthquakes in Campus Vicinity, Magnitude > 6
Laney College Student Center
Oakland, California

Latitude	Longitude	Date	Magnitude	PGA (g)	MM	Distance in miles (km)
37.8	122.2	06/10/1836	6.8	0.544	X	3.4 (5.5)
37.7	122.1	10/21/1868	6.8	0.314	IX	11.1 (17.8)
37.8	122.5	06/21/1808	6.3	0.204	VIII	12.9 (20.8)
37.7	122.5	4/18/1906	8.25	0.319	IX	14.5 (23.4)
37.6	122.4	06/01/1838	7	0.193	VIII	15.5 (24.9)
38	121.9	05/19/1889	6	0.087	VII	24.3 (39.1)
37.5	121.9	11/26/1858	6.1	0.078	VII	28.5 (45.8)
38.2	122.4	03/31/1898	6.2	0.081	VII	28.9 (46.5)
37.3	121.9	10/08/1865	6.3	0.063	VI	39.6 (63.7)
38.4	122	04/19/1892	6.4	0.06	VI	44.1 (70.9)
37.32	121.698	4/24/1984	6.2	0.052	VI	45.1 (72.6)
37.25	121.75	7/1/1911	6.6	0.062	VI	47.0 (75.7)
38.5	121.9	04/21/1892	6.2	0.045	VI	52.4 (84.4)
37.036	121.883	10/18/1989	7	0.064	VI	56.5 (90.9)

Notes: Source: Blake 2000c
Latitude and Longitude are the locations of the assumed epicenters.
MM = Mercalli Magnitude (please see Table 2)
Acceleration is the mean expected acceleration at the Campus due to the historical earthquake calculated using the Abrahamson and Silva (2008) attenuation relationship.

9.4.4 Ground Rupture Potential

The Site is not located within an Alquist-Priolo Special Studies Earthquake Fault Zone. The nearest Alquist-Priolo Zone is associated with the Hayward Fault located 3.5 miles to the east-northeast of the Site.

There does not appear to be a significant risk of surface rupture during the expected service life of the proposed school buildings.

9.4.5 Liquefaction Potential

Liquefaction can be induced by cyclic loading (shaking) from an earthquake, which can cause granular materials to lose their inherent shear strength due to increased pore-water pressures. Some of the factors that typically contribute to liquefaction risk include a shallow water table, low relative density of granular materials below the groundwater table, low soil cohesion or plasticity, low percentage of fine-grained material in soil, relatively long seismic shaking duration, and high ground acceleration during earthquakes.

At the Site, the water table occurs at a reported (Woodward Clyde Sherard 1966) elevation of approximately 0 to 2 feet msl. The California Geological Survey (CGS 2003b) maps the Student Center vicinity as being at risk for liquefaction.

Liquefaction potential beneath the location of the new Student Center entranceway was assessed using the computer program C-Liq (Geologismiki 2006). C-Liq applies the NCEER method (Youd et al, 2001) along with the calibrated procedures for post-earthquake displacements by Zhang et al. (2002) to cone penetration test data. The C-Liq results are appended to this report in Appendix E.

C-Liq reports that up to 1.5 inches of total liquefaction settlement are possible at the location of the new entranceway with about 1 inch of the settlement taking place between 34 and 50 feet bgs where it is likely to cause settlement of the micropiles. Assuming the entire 1 inch of settlement is transmitted to the surface (which is the worst case) the new entranceway footings should be designed to withstand up to ½ inch of settlement across the length of the foundation.

There may be some settlement in the dry soils above the water table. However as the soils above 25 feet bgs are not accounted for in the design of the pile supports, any settlements in the dry silts, clays and fill above the water table will not result in settlement being transmitted into the new entranceway supports. Terraphase has recommended that any floors replaced during the installation of the new entranceway be structural slabs (not supported by the underlying soils) tied into the new pile caps.

The existing piles were not designed in accordance with the current building code and may not have sufficient lateral stiffness in the event of an earthquake.

9.4.6 Flood Inundation Potential

9.4.6.1 Flood Zonation

The local Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps indicate that the Site (FEMA 2009) is not located within a 100-year flood zone (Figure 9). The nearest 100-year flood zone ("Zone A") is located approximately 400 feet east of the Site (the Lake Merritt Outflow Channel).

9.4.6.2 Dam Inundation

The Association of Bay Area Governments dam inundation system (2012) indicates that there are several reservoirs that discharge to Lake Merritt; however, none of the reservoirs is large enough to cause flooding downstream of Lake Merritt.

9.4.7 Land Subsidence

The Safety Element of the Oakland General Plan (2004) addresses the possibility of land subsidence, but does not indicate that it has occurred in Oakland. The primary cause of land subsidence in the Bay Area is groundwater extraction. Given the Site's proximity to the Oakland Inner Harbor, groundwater extraction in the vicinity of the Site is unlikely as the groundwater would likely be saline. Hence, groundwater-extraction-induced subsidence at the Site is unlikely. Even if subsidence were to occur at the Site, the building is installed on piles and would be unlikely to be damaged by the subsidence. Hence, land subsidence is not considered to be a significant hazard for the Site.

9.4.8 Naturally Occurring Asbestos

Graymer et al. (1996) map a significant deposit of serpentinite in the foothills directly east of the Site (in Joaquin Miller Park). The deposit was mined commercially for asbestos in 1915 (USGS 2009a) but the deposit was of such limited quantity that mining was abandoned after one year. Since serpentinite rocks are located within a 10-mile radius of the Site and this deposit is known to contain asbestos, and the deposit is in the hydrologically upgradient direction, it represents a potential source of naturally occurring asbestos at the Site (DTSC 2004). The Site is completely paved over and does not have any significant areas of exposed soil (there is essentially no landscaping at the Student Center). Hence, naturally-occurring asbestos is not a significant hazard for the Student Center.

9.4.9 Other Hazards

Certain other potential geologic hazards, including tsunamis, seiches, naturally occurring radon, and oil and gas fields, do not appear to pose significant risks at the Site, for the reasons discussed briefly below.

- Tsunamis and Seiches. Tsunamis do not pose an appreciable risk at the Site as it is sheltered by Alameda Island (Ritter and Dupre 1972).
- Naturally Occurring Radon. The California Department of Health Services (DHS) (2010) maintains a database of radon measurements in California, based on zip code. No elevated radon results (greater than or equal to 4.0 picoCuries per liter [pCi/L]) have been reported in thirteen measurements from the 94606 zip code, which includes the Site. Three hundred and fifty radon measurements have been made in Oakland zip codes and reported to DHS. Ten of those measurements exceeded the 4.0 pCi/L DHS action level.
- Oil and Gas Fields. The Site is not located within an oil or gas field, as recognized by the California Department of Oil, Gas, and Geothermal Resources (DOGGR 2009). Several wells have been drilled in the Oakland area, but all were dry holes, which have been plugged and abandoned. The nearest dry hole was approximately 4 miles south of the Site.

- Mineral Resources and Producers. The Site is not located near a mineral resource producer, as listed by the Mineral Resources Data System (MRDS) of the US Geological Survey (USGS 2009a). No MRDS localities are listed within 1 mile of the Site.
- Volcanoes – White et al. (2011) do not place Oakland in an area where there is a significant threat of volcanic eruption.

9.4.10 CONCLUSIONS

The conclusions reached by Terraphase are based on the published data reviewed for this geologic hazards evaluation. Our findings are summarized below.

- The Site is not located within or near an Alquist-Priolo Special Studies Earthquake Fault Zone. Surface rupture should not reasonably be expected during the life of the proposed school buildings.
- The Site is not located within the 100-year flood zone. The nearest such zone lies approximately 300 feet to `the east.
- The Site will likely be subjected to strong shaking during future earthquakes.
- The site area may be susceptible to liquefaction hazards, due to the presence of relatively shallow groundwater and unconsolidated sediments. However the building is constructed on piles and hence liquefaction settlements are not expected to significantly affect the building.
- Serpentinite has been mapped upgradient from the Site and within 10 miles of it. However, the entire Site has been paved and hence students, faculty, and staff at the college are not likely to be exposed to naturally occurring asbestos.
- A pipeline risk assessment is not required for community colleges and one was not performed as part of this effort.

Based on the above findings, it is Terraphase’s opinion that the Site is not subjected to geologic hazards that cannot be mitigated through normal engineering design.

10.0 LIMITATIONS

The opinions and recommendations presented in this report are based upon the scope of services, information obtained through the performance of the services, and the schedule as agreed upon by Terraphase and the party for whom this report was originally prepared. This report is an instrument of professional service and was prepared in accordance with the generally accepted standards and level of skill and care under similar conditions and circumstances established by the geotechnical consulting industry. No representation, warranty, or guarantee, expressed or implied, is intended or given. To the extent that Terraphase relied upon any information prepared by other parties not under contract to Terraphase, Terraphase makes no representation as to the accuracy or completeness of such information. This report is expressly for the sole and exclusive use of the party for whom this report was originally prepared for a particular purpose and only in it's entirely. Only the party for whom this report was originally prepared and/or other specifically named parties have the right to make use of and rely upon this report. Reuse of this report or any portion thereof for other than its intended purpose, or if modified, or if used by third parties, shall be at the user's sole risk.

Furthermore, nothing contained in this report shall relieve any other party of its responsibility to abide by contract documents and applicable laws, codes, regulations, or standards.

Subsurface Explorations and Testing

Results of any observations, subsurface exploration, or testing, and any findings presented in this report apply solely to conditions existing at the time when Terraphase's exploratory work was performed. It must be recognized that any such observations and exploratory or testing activities are inherently limited and do not represent a conclusive or complete characterization. Conditions in other parts of the Site may vary from those at the locations where data were collected and conditions can change with time. Terraphase's ability to interpret exploratory and test results is related to the availability of the data and the extent of the exploratory and testing activities.

The findings and recommendations submitted in this report are based, in part, on data obtained from subsurface borings, test pits, and specific, discrete sampling locations. The nature and extent of variation between these test locations, which may be widely spaced, may not become evident until construction. If variations are subsequently encountered, it will be necessary to reevaluate the conclusions and recommendations of this report.

Correlations and descriptions of subsurface conditions presented in boring logs, test pit logs, subsurface profiles, and other materials are approximate only. Subsurface conditions may vary significantly from those encountered in borings, and sampling locations and transitions between subsurface materials may be gradual or highly variable.

Conditions at the time water-level measurements and other subsurface observations were made are presented in the boring logs or other sampling forms. These field data have been reviewed and interpretations provided in this report. However, groundwater levels may be variable and may fluctuate due to variations in precipitation, temperature, and other factors. Therefore, groundwater levels at the site at any time may be different than stated in this report.

Review

In the event that any change in the nature, design, or location of the proposed structure(s) is planned, the conclusions and recommendations in this report shall not be considered valid nor relied upon unless the changes are reviewed and the conclusions and recommendations of this report are modified or verified in writing.

Terraphase should be provided the opportunity for a general review of final design plans and specifications to assess that our recommendations have been properly interpreted and included in the design and construction documents.

Construction

To verify conditions presented in this report and modify recommendations based on field conditions encountered in the field, Terraphase should be retained to provide geotechnical engineering services during the construction phase of the project. This is to observe compliance with design concepts, specifications, and recommendations contained in this report, and to verify and refine our recommendations as necessary in the event that subsurface conditions differ from those anticipated prior to the start of construction.

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APPENDIX A
SITE SPECIFIC SEISMIC HAZARD ASSESSMENT

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A.1 Introduction

This Appendix provides general ground motion parameters, as required by the 2007 California Building Code (CBC). It also includes a site-specific ground motion analysis, as required by 2007 CBC 1614A.1.2 for sites located within 10 kilometers (km) of an active fault. The ground motion analysis was conducted in accordance with American Society of Civil Engineers (ASCE) Standard 7-05, Sections 21.2 to 21.4 (ASCE 2005).

The evaluation addressed fundamental building periods ranging from 0 to 4 seconds.

A.2 ASCE 7-05, Section 11.4: Seismic Ground Motion Parameters

The Site Class was determined based on previous investigations conducted by Woodward Clyde (1966) and interpretation of the 82 foot deep CPT probe installed by Terraphase in 2012. Other ground motion parameters were obtained using the U.S. Geological Survey's "Earthquake Ground Motion Tool", version 5.09 (USGS 2008a), using the following options:

- Geographic Region = 48 Conterminous States
- Data Edition= 2005 ASCE 7 Standard
- Latitude (Degrees) = 37.79604° North
- Longitude (Degrees) = 122.26299° West

The resulting ground motion parameters may be summarized as follows:

- $\mathbf{S_S}$ = 1.505
- $\mathbf{S_1}$ = 0.600
- **Site Class: E** (soft clay soil, see note below on Site Classification)
- $\mathbf{F_a}$ = 0.9
- $\mathbf{F_v}$ = 2.4
- $\mathbf{S_{MS}} = F_a \times S_S$ = 1.35 (This number changes – see Section A.7)
- $\mathbf{S_{M1}} = F_v \times S_1$ = 1.44
- $\mathbf{S_{DS}} = (2/3) \times S_{MS} = (2/3) \times 1.35 = 0.90$ (This number changes – see Section A.7)
- $\mathbf{S_{D1}} = (2/3) \times S_{M1} = (2/3) \times 1.44 = 0.96$
- $\mathbf{T_0 = 0.2* S_{D1}/S_{DS}}$ = 0.21 seconds
- $\mathbf{T_s = S_{D1}/S_{DS}}$ = 1.063 seconds
- **Average Shear Wave Velocity of top 30 meters:** 175 meters/second (see note)

- **Depth to 1,000 meter per second shear wave velocity:** 213 meters (Rogers 1997)

Revised Values per ASCE 7 Section 21.4

- $\mathbf{S_{DS}}$ = 1.14 (spectral acceleration at 0.2 second not less than 90% of all higher period spectral accelerations)
- $\mathbf{S_{D1}}$ = 1.0 (the greater of the spectral acceleration at 1 second or twice the spectral acceleration at 2.0 seconds)
- $\mathbf{S_{MS}}$ = 1.71
- $\mathbf{S_{M1}}$ = 1.51

The soil at the Site was classified as Site Class E (see Table 20.3-1 in the CBC) because the average equivalent SPT blow counts in the top 100 feet of the soil profile were less than 15. A shear wave velocity of 175 meters per second (upper end of the building code range for Site Class E soils) was used in the Next Generation of Attenuation (NGA) attenuation relations that required a shear wave velocity.

A.3 Probabilistic MCE Response Spectrum

Section 21.2.1 of ASCE (2005) requires a probabilistic maximum considered earthquake (MCE) response spectrum. A probabilistic response spectra was generated using the program EZ-Frisk version 7.62 (Risk Engineering 2012). The resulting spectra is shown in Table A-1 and on Figure A-1. Appendix B contains an abridged copy of the EZ-Frisk output.

This analysis was performed using the three NGA Equations: Abrahamson and Silva (2008), Chiou and Youngs (2008), and Boore et al. (2008) – the output selected was the maximum rotated component. The USGS (2008b) seismic model was used to delineate active earthquake faults within 200 kilometers of the Site, including gridded faults representing earthquakes occurring on non-mapped faults.

A.4 ASCE 7-05, Section 21.2.2: Deterministic MCE Response Spectrum

Section 21.2.2 of ASCE (2005) requires a deterministic MCE response spectrum, based on 150% of the largest spectral accelerations associated with active faults within the region. DSA (2009) has modified this requirement by requiring that, when using the NGA equations, the 84th percentile spectra be reported. The local faults with the highest associated peak ground accelerations ($\geq$ 0.17 g) and Modified Mercalli Intensities ($\geq$ VIII) are summarized in Table 3 in the main text. The USGS (2008b) model includes gridded seismic sources intended to represent earthquakes occurring in places other than on mapped faults. These gridded faults are included in recognition that not all significant earthquakes occur on mapped faults.

A deterministic response spectra (84^{th} percentile maximum rotated horizontal component) was generated for the Site using EZ-Frisk, the USGS (2008b) seismic model and the three NGA relations listed above. Different attenuation relations were used for the gridded deep earthquake source. The seismic hazard at the Site was dominated by the nearby Hayward Fault (5.6 km from the Site) and the gridded characteristic reverse fault (5.0 km from the Site).

The 84^{th} percentile Deterministic Response Spectra (maximum rotated horizontal component) for the Site is presented in Table A-2 and shown on Figure A-2A. A lower limit deterministic MCE response spectra were then generated, in accordance with Section 21.2.2 and Figure 21.2-1 of ASCE (2005) based on the previously determined values of F_a and F_v (Section A.2) with $S_s = 1.5$ and $S_1 = 0.6$. The resulting Lower Limit Deterministic MCE Response Spectrum is shown in Table A-2 and on Figure A-2B. A Final Deterministic MCE Response Spectrum was then generated, using the greater of the Maximum and Lower Limit Deterministic MCE Response Spectra. The resulting Final Deterministic MCE Response Spectrum is shown in Table A-2 and on Figure A-2B.

A.5 ASCE 7-05, Section 21.2.3: Site Specific MCE Response Spectrum

Section 21.2.3 of ASCE (2005) requires a Site-Specific MCE Response Spectrum, to be taken as the lesser of the Probabilistic MCE Response Spectrum (from Figure A-1) and the Deterministic MCE Response Spectrum (from Figure A-2B). These two spectra are plotted together on Figure A-3.

The deterministic MCE spectral accelerations are typically lower, and therefore govern the Site-Specific MCE Response Spectrum at most periods (Figure A-3). The probabilistic spectra is lower than the deterministic spectra below spectral periods of 0.03 second.

A.6 ASCE 7-05, Section 21.2.3: Design Response Spectrum

Section 21.3 of ASCE (2005) requires a design response spectrum, where the design response accelerations are two-thirds of the site-specific MCE response accelerations (from Figure 3). The Site-Specific MCE Response Spectrum and the corresponding Final Design Response Spectrum are shown in Table A-4 and on Figure A-4.

Section 21.3 of ASCE (2005) defines a lower limit on the Final Design Response Spectrum, based on the Lower Limit Probabilistic Design Response Spectrum generated using the parameters presented in Section A.2. The Lower Limit Design Response Spectrum is 80% of the Probabilistic Design Response Spectrum; it is shown on Table A-4 and on Figure A-4.

A.7 ASCE 7-05, Section 21.4: Design Acceleration Parameters

Section 21.4 of ASCE (2005) requires recalculation of design acceleration parameters, based on the final design response accelerations at periods of 0.2, 1, and 2 seconds.

The design acceleration parameters as required by ASCE (2005), Section 21.4 are slightly different than those defined above in Section A.1 and are greater than 80% of the previously calculated values:

- $\mathbf{S_{DS}}$ = 1.14 (spectral acceleration at 0.2 second not less than 90% of all higher period spectral accelerations)
- $\mathbf{S_{D1}}$ = 1.0 (the greater of the spectral acceleration at 1 second or twice the spectral acceleration at 2.0 seconds)
- $\mathbf{S_{MS}}$ = 1.71
- $\mathbf{S_{M1}}$ = 1.51

APPENDIX B
HISTORICAL DOCUMENTS

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APPENDIX C
GEOTECHNICAL LABORATORY RESULTS

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APPENDIX D
CURRENT BORING LOGS AND CPT RESULTS

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APPENDIX E
LIQUEFACTION RESULTS

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