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# **Preliminary Foundation Evaluation Oakland Army Base Reuse Study**

Oakland, California

*Prepared for:*

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Oakland, California

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## **EXECUTIVE SUMMARY OAKLAND ARMY BASE REUSE STUDY**

### **ES-1.0 INTRODUCTION**

#### **ES-1.1 SITE HISTORY**

The Oakland Army Base (OARB) is a 422-acre site located on the Oakland waterfront, about 2 miles northwest of downtown Oakland and the Oakland City Hall. It was a major Pacific port and shipping center for the military from World War II through the first Persian Gulf conflict. It currently contains approximately 50 buildings and structures (warehouses, offices, commissary, and military housing). The OARB was closed September 30, 1999.

The OARB is situated in an area that was reclaimed from San Francisco Bay. The original San Francisco Bay shoreline was approximately 500 to 3,000 feet east of the eastern edge of the base. Prior to development, several streams emptied into the bay at or near the current location of West Grand Avenue on the east side of the base. These former streambeds produced varying thicknesses and characteristics of the natural subsurface materials and varying depths of artificial fill.

Filling occurred from 1894 to 1915 to develop the Oakland mole. Most of the OARB was filled and the current shoreline established between 1915 and the mid-1940s. Originally, the western area of the OARB was in shallow waters of the bay, and the eastern part consisted of tidal mud flats. Early building on the site took place between 1918 and 1940. The Army authorized the base in 1940 and construction started in June of 1941, with most of the current buildings constructed by 1946. Records indicate that 6.5 million yards of fill was placed to reclaim the base area from the bay, primarily with hydraulically dredged silty sand.

#### **ES-1.2 SITE DEVELOPMENT PLANS**

The gross development area is 198 acres, with 177 acres of the total being currently available for building. The Oakland Base Reuse Authority (OBRA) is in the process of creating a new development plan for the site. Development may include office buildings; a business park/light industrial area; industrial buildings; and relocation of Maritime Street to a location parallel to but approximately 500 feet east of the existing street.

Earthwork is expected to include minor to moderate grading to develop drainage and site landscaping.

## **ES-2.0 SITE CONDITIONS**

### **ES-2.1 SUBSURFACE CONDITIONS**

The generalized subsurface conditions can be divided into four strata consisting of hydraulically-placed sandy fill; interfingered Young Bay Mud and sand; dense sand and stiff sandy clays (San Antonio Formation); and stiff to very stiff silty clays (Old Bay Mud). The Franciscan bedrock is estimated to lie at depths of several hundred feet below the site. The thickness of the four major soil strata encountered in the borings vary across the site, but in general the fill is about 5 to 35 feet thick, and the underlying Young Bay Mud and sand is on the order of 6 to 60 feet thick.

### **ES-2.2 GEOLOGIC HAZARDS**

The primary geotechnical concerns are the potential for the upper sandy fill to liquefy during strong ground shaking, which could lead to settlement, loss of bearing capacity, and (in some locations) lateral spreading; and the compressibility of the Young Bay Mud that underlies the existing fill. In addition, the site will be subject to strong ground shaking during a significant earthquake.

### **ES-3.0 FOUNDATION DEVELOPMENT OPTIONS**

Selection of foundation types should consider the ability of the site soils to support the planned structure under both static and dynamic (earthquake) loads, and on the settlement of the foundation due to building loads. It should also consider potential settlement due to site grading. For the purposes of this preliminary foundation evaluation, the site has been divided into five areas (as shown on Figure ES-1) based on differing subsurface conditions with consequently differing preliminary foundation recommendations.

Shallow foundations will generally settle several inches, as summarized in Table ES-1, which is typically more than can be tolerated by most structures. Liquefaction-induced settlement and reduction in bearing capacity pose additional hazards to shallow foundations. Deep foundation will provide acceptable building support, but are more expensive than shallow foundations. As an alternative to deep foundations, ground improvement measures may be taken to mitigate consolidation settlement and liquefaction effects at the site, and the buildings then supported on shallow foundations. Although ground improvement is not inexpensive, where there is adequate time and space around a proposed building footprint, these may provide the most cost-effective foundation approach.

The type of foundations to be used should be selected in consideration of the anticipated building loads and the amount of settlement that can be tolerated on a project-specific basis during final design phase evaluations.

A design-level geotechnical investigation should be performed for any planned structure to assess the site-specific subsurface conditions and relate them to the building-specific design criteria. Further site-specific evaluations of shallow and/or deep foundations should be included in these investigations. The following recommendations are for planning purposes only, and should not be used for the final design of any building.

### **ES-3.1 SHALLOW FOUNDATIONS WITH NO GROUND IMPROVEMENT**

Shallow foundations (such as spread footings or slab-on-grade) generally provide the least expensive approach where they are capable of supporting a building with acceptable levels of total settlement and differential settlement. This is generally assumed to mean no more than about 1 inch of total settlement, and about ½ inch of differential settlement between adjacent columns or in any 20 to 30 foot span. Some types of structures may be able to tolerate somewhat larger magnitudes of settlement than these values. For preliminary planning purposes, Table ES-1 shows a summary of the maximum footing dimensions and allowable bearing pressure for shallow spread footings, assuming a maximum consolidation settlement of about 3 inches.

Although some buildings have been constructed in the Bay Area in locations where anticipated settlement was as great as 3 inches, this magnitude of potential settlement will usually result in either the selection of deep foundations, or the decision to perform ground improvement prior to building construction. The potential for liquefaction-induced settlement provides further incentive to consider deep foundations or ground improvement.

In addition to the settlement expected for a building on shallow footings, the following points should be considered:

- The consolidation settlement of a building on a mat or structural slab foundation can be approximated by calculating the net weight of the structure (i.e., not including any portion of the slab that is below original grade) divided by the footprint area of the building to get the average contact pressure, divided the contact pressure by 125 to get a roughly equivalent height of fill, and estimate the consolidation settlement under the equivalent thickness of fill.

- If a site is to be graded before the building is constructed, the amount of consolidation settlement under the weight of any new fill (as estimated from Table ES-1) should be added to the consolidation settlement under the footings.
- To account for performance following an earthquake, the magnitude of liquefaction-induced settlement shown on Table ES-1 should be added to the consolidation settlement under the footings.
- For preliminary planning purposes, differential settlement may be approximated as half of the total settlement from all sources.

### **ES-3.2 DEEP FOUNDATIONS**

Piles will provide good foundation support in any portion of the site with only minimal settlement. However, in Area E (Figure ES-1) there is the potential for liquefaction-induced lateral spreading due to a large earthquake, so in this area measures should be considered to mitigate this hazard, such as ground modification.

Anticipated pile lengths for each area are shown in the main body of this report (Figures 6 through 13), and summarized in Table ES-1, for ultimate pile capacities of 50 and 100 tons. Piles typically cost in the range of \$35/foot for 12-inch piles, and \$45/foot for 14-inch piles. Preliminary foundation costs can be estimated based on these unit costs, times the approximate number of piles that will be required for a structure, times the length of piles for a given area.

If grading is performed in or around a proposed building, the ground may settle away from a pile-supported building, requiring mitigation measures for utilities and approach walkways.

### **ES-3.3 GROUND IMPROVEMENT**

It may be feasible to improve the subsurface conditions prior to construction, and thereby reduce the potential for consolidation settlement and/or liquefaction. Although these methods are not inexpensive, they may reduce settlement estimates enough that a building can be supported on shallow foundations, thus saving the cost increment of using a deep foundation. The following methods may be considered:

1. Surcharging may be used, either with or without wick drains, to reduce post-construction settlement related to consolidation of Young Bay Mud. The cost of surcharging without wick drains is a function of the availability of fill soils and ability to dispose of them afterwards, and the impact on the development schedule. Surcharging with wick drains will cost on the order of \$0.60 per foot of wick drain

plus \$0.50 per square foot of area to be surcharged, with wicks spaced on a 3- to 5-foot triangular grid. Surcharging may require a significant amount of time (as little as 3 months or up to 18-months with wick drains, and possibly longer than this without wick drains) before construction can begin.

2. Geofoam may be used to replace existing soil, to compensate for the added weight of a proposed building. This could result in no net increase in load to the underlying soils, thus reducing consolidation settlement. It will not affect liquefaction.
3. If liquefaction potential is to be mitigated, four methods of ground improvement of the fill should be considered:
  - i. Stone Columns.
  - ii. Deep Dynamic Compaction (DDC)
  - iii. Rapid impact Compaction (RIC)
  - iv. Earthquake drains.

We discussed the first two of these methods with Hayward Baker, and the last with Moore and Tabor, both of whom are ground improvement specialists. We discussed the fourth method with Nilex Corporation. Some of the pros and cons of each approach are discussed below.

Each of these techniques is discussed further in the following sections.

### **ES-3.3.1 Stone Columns**

Stone columns are a method in which a mandrel is first vibrated into the loose soil and then gravel is introduced into the hole and vibrated so that the gravel column becomes denser and larger in diameter, thereby densifying the surrounding ground and essentially eliminating the potential for liquefaction within areas that are treated.

For preliminary costing, assuming sand fill depths on the order of 15 to 20 feet deep, and assuming approximately 3-foot-diameter stone columns spaced in a square grid of 9 feet on centers both ways, treatment of 50,000 square feet would require about 617 columns and cost about \$250,000, or about \$5/square foot.

### **ES-3.3.2 Deep Dynamic Compaction**

DDC involves dropping a heavy weight from a crane onto the ground surface, resulting in compaction of the soil to depth of about 15 to 20 feet. For most of the types of fill soils encountered at the Gateway Development Area, we anticipate that this method could densify

the fills to the point where they would have a low susceptibility to liquefaction, and if seismic settlements occurred, they would be small.

It is likely that treatment would cost on the order of about \$2 per square foot of treated ground, or about \$100,000 to treat 50,000 square feet.

### **ES-3.3.3 Rapid Impact Compaction**

RIC is a method in which a 7.5 ton weight is lifted hydraulically and dropped about 1 meter, at a rate of about 40 to 60 blows per minute. It is a relatively new technology to the United States, but based on preliminary information it is expected to have a similar effectiveness to that of DDC.

The cost of RIC is expected to be about \$2.30 per square foot. To treat 50,000 square feet, the cost would be about \$115,000.

### **ES-3.3.4 Earthquake Drains**

Earthquake drains are vertical pipe drains that are inserted by pushing with a mandrel to the depth of concern. They mitigate liquefaction by providing a vertical drainage path for water to escape, thus reducing the increase in pore pressures generated by the earthquake. The installation of earthquake drains also typically causes a minor amount of ground densification, but some seismically-induced settlement may still occur.

For this application we expect they should be installed in a triangular grid with a spacing of about 4 ½ to 6 feet on centers, to a depth of the bottom of existing fill. The costs shown on Table ES-2 assume a depth of loose fill of 15 feet, but the actual depths of fill across the site vary from about 10 to over 30 feet, so earthquake drain costs would vary as well.

## **ES-3.4 RELATIVE COSTS AND PERFORMANCE**

Relative costs and performance of the various alternatives are summarized on Table ES-2.

**TABLE ES-1  
SUMMARY OF ANTICIPATED FOUNDATION PERFORMANCE WITH NO GROUND IMPROVEMENT**

Location <sup>1</sup>	Estimated Consolidation Settlement under 1 ft of fill (in)	Estimated Liquefaction Induced Settlement (in)	Square Footings		Strip Footings		Driven Piles		
			Footing Widths (ft)	Allowable Bearing Capacity <sup>2</sup> (psf)	Footing Widths (ft)	Allowable Bearing Capacity <sup>2</sup> (psf)	Pile Size (in)	Ultimate Capacity (tons)	Length <sup>3</sup> (ft)
A	1	3	2 to 5	2,000	2 to 3	2,000	12	50	27
								100	37
							14	50	24
								100	34
B	2-4	3	2 to 5	2,000	2	1,000	12	50	42-67
								100	62-87
							14	50	37-62
								100	52-77
C	2-5	2	2 to 2½	2,000	2	500	12	50	52-67
								100	72-87
							14	50	47-62
								100	62-77
D	2-4	6- 12	2 to 5	2,000	2 to 3	2,000	12	50	62-97
								100	82-117
							14	50	57-92
								100	72-107
E	2-4	Liquefaction-induced lateral spreading likely; shallow spread footings not recommended without site specific evaluation. Alternatively, ground improvement may be considered to mitigate liquefaction potential.					12	50	42-107
								100	47-127
							14	50	42-102
								100	47-117

**Notes:**

- <sup>1</sup> See Figure ES-1 for location of areas
- <sup>2</sup> Allowable bearing capacity for dead plus long-term live load, consolidation induced settlement less than 3-inches.
- <sup>3</sup> See Figures 6 to 13 for variation of preliminary pile lengths within an area.

**TABLE ES-2  
SUMMARY OF RELATIVE COSTS FOR FOUNDATION AND GROUND IMPROVEMENT OPTIONS**

Mitigation for Consolidation Settlement	Mitigation for Liquefaction Potential	Typical Ground Improvement Cost	Shallow Spread Footings		Mat or Structural Slab, or Slab with Thickened Grid		Deep Foundations (driven piles)	
			Cost	Performance	Cost	Performance	Cost	Performance
None	None	\$0.00/sf	Low	Poor	Moderate	Poor	<b>Mod High</b>	<b>Good</b>
None	DDC	\$2.00/sf	Mod Low	Poor	Mod	Mod Poor	<b>High</b>	<b>V. Good</b>
None	RIC	\$2.30/sf	Mod Low	Poor	Mod	Mod Poor	<b>High</b>	<b>V. Good</b>
None	EQ Drains <sup>3</sup>	\$3.30-5.00/sf	Mod High	Poor	High	Mod Poor	<b>High</b>	<b>V. Good</b>
None	Stone Columns	\$5.00/sf	Mod High	Poor	High	Mod Poor	V. High	V. Good
Surcharge w/o Wick Drains	None	\$1.00/sf	Mod Low	Poor in EQ	Mod High	Mod in EQ	High	Good
Surcharge w/o Wick Drains	DDC	\$3.00/sf	<b>Mod</b>	<b>Good</b>	Mod High	Good	<b>High</b>	<b>V. Good</b>
Surcharge w/o Wick Drains	RIC	\$3.30/sf	<b>Mod</b>	<b>Good</b>	Mod High	Good	<b>High</b>	<b>V. Good</b>
Surcharge w/o Wick Drains	EQ Drains <sup>3</sup>	\$4.30-6.00/sf	<b>Mod High</b>	<b>Fair</b>	High	Good	<b>V. High</b>	<b>V. Good</b>
Surcharge w/o Wick Drains	Stone Columns	\$6.00/sf	Mod High	Good	High	Good	V. High	V. Good
Surcharge w/ Wick Drains <sup>4</sup>	None	\$3.10/sf	Mod Low	Poor in EQ	Mod High	Mod in EQ	High	Good
Surcharge w/ Wick Drains <sup>4</sup>	DDC	\$5.10/sf	<b>High</b>	<b>Good</b>	High	Good	V. High	V. Good
Surcharge w/ Wick Drains <sup>4</sup>	RIC	\$5.40/sf	<b>High</b>	<b>Good</b>	High	Good	V. High	V. Good
Surcharge w/ Wick Drains <sup>4</sup>	EQ Drains <sup>3</sup>	\$6.40-8.10/sf	High	Fair	High	Good	V. High	V. Good
Surcharge w/ Wick Drains <sup>4</sup>	Stone Columns	\$8.10/sf	V. high	Good	V. High	Good	V. High	V. Good
Geofoam <sup>5</sup>	None	\$2.00/sf	Mod	Poor in EQ	High	Mod in EQ	V. High	Good
Geofoam <sup>5</sup>	DDC	\$4.00/sf	<b>Mod High</b>	<b>Good</b>	High	Good	V. High	V. Good
Geofoam <sup>5</sup>	RIC	\$4.30/sf	<b>Mod High</b>	<b>Good</b>	High	Good	V. High	V. Good
Geofoam <sup>5</sup>	EQ Drains <sup>3</sup>	\$5.30-7.00/sf	Mod High	Fair	High	Good	V. High	V. Good
Geofoam <sup>5</sup>	Stone Columns	\$7.00/sf	Mod High	Good	High	Good	V. High	V. Good

**Notes:**

(see next page)

**Notes for Table 2: SUMMARY OF RELATIVE COSTS FOR FOUNDATION AND GROUND IMPROVEMENT OPTIONS**

1. Likely preferred alternatives are shown in **Bold and are Highlighted**. Other alternatives may be selected based on project-specific criteria.
2. These estimated relative costs are based on previous experience on similar projects. They should be considered approximate and should only be used for preliminary planning evaluations. Actual costs may be significantly different due to site-specific considerations, time of year of construction, the general state of the local economy at the time of construction, and other factors.
3. Cost assumes Earthquake Drains would be 15 feet deep to reach the bottom of existing fill, spaced on triangular grid of 6 feet to 4 ½ feet. Where existing fill extends deeper or shallower, or if closer spacing is needed, costs will be proportionally greater or less.
4. Cost assumes wick drains would be 50 feet deep to reach the bottom of Bay Mud. Where Bay Mud extends deeper or shallower (as shown on Figures 2 to 4), costs will be proportionally greater or less.
5. Geofoam cost is for a building of about 125 pounds per square foot net dead plus live load (likely range for 2 story building); geofoam for lighter or heavier (e.g., shorter or taller) buildings would be proportionally more expensive.

# **PRELIMINARY FOUNDATION EVALUATION**

## **Oakland Army Base Reuse Study**

### **Oakland, California**

#### **1.0 INTRODUCTION**

This report summarizes the preliminary foundation evaluation performed by Geomatrix Consultants, Inc. (Geomatrix), for Kimley-Horn and Associates (Kimley-Horn) and the Oakland Base Reuse Authority (OBRA) for development of the Oakland Army Base, located in Oakland, California. The Oakland Army Base (OARB) has been decommissioned and will be transferred to the City of Oakland. OBRA is the agency responsible for the reuse planning and implementation of the Base Reuse Plan and is currently responsible for the redevelopment of the Gateway Development Area (GDA) at the northeastern end of the site. Our services were performed in accordance with our proposal dated September 19, 2005.

#### **1.1 PURPOSE**

The purpose of the geotechnical study is to aid Kimely-Horn and OBRA in the evaluation of the current condition of the OARB and to assist in preliminary planning for redevelopment of the base property. The objective of the preliminary foundation evaluation was to evaluate and characterize geotechnical conditions at the site in order to develop planning level foundation recommendations for development.

#### **1.2 SCOPE OF WORK**

The scope of work for Geomatrix's evaluation included the following tasks.

##### **Review of Available Data**

This task included reviewing and summarizing the geotechnical data compiled during our previous study at the site (Geomatrix, 2001) in light of the current development plans. The applicable data was extracted from our project files and used in the current evaluation. No additional subsurface exploration was included in our current scope.

##### **Develop Preliminary Foundation Recommendations**

The data gathered and reviewed was used to divide the OARB into areas with similar subsurface conditions, and to develop preliminary foundation recommendations for each of these areas. For each area, preliminary foundation recommendations for single- and multi-story buildings were developed for the anticipated subsurface conditions of

that area, taking into consideration fill thicknesses and characteristics of the underlying native soils, especially estimated strength and compressibility. The preliminary foundation recommendations include foundation type and dimensions (e.g., minimum footing widths or pile lengths), as well as commentary on performance issues (e.g., long-term total and differential settlement).

### **Prepare Summary Report**

The results of the preliminary foundation evaluation are summarized in this report.

### **Attend Meeting and Project Management**

We will attend a meeting to discuss the preliminary conclusions presented in a draft of this report.

Our scope of work did not include an evaluation of environmental considerations such as soil or groundwater contamination, which we understand is being addressed by a separate consultant.

## **1.3 REPORT ORGANIZATION**

The report is organized into the following sections: Section 2 provides a description of the project and previous work performed at the site; Section 3 describes the subsurface conditions at the site; and Section 4 presents our preliminary foundation recommendations for the site.

## **2.0 PROJECT DESCRIPTION**

This section presents a summary of the current project setting, the site history, the proposed development plans for the site, and the previous work performed at the site.

### **2.1 SITE SETTING**

The OARB is a 422-acre site located on the Oakland waterfront, about 2 miles northwest of downtown Oakland and the Oakland City Hall. It was a major Pacific port and shipping center for the military from World War II through the first Persian Gulf conflict. It currently contains approximately 50 buildings and structures (warehouses, offices, commissary, and military housing). The OARB was closed September 30, 1999.

The OARB, as shown on Figure 1, is bounded on the north by the eastern approach to the Oakland-San Francisco Bay Bridge, on the northeast by the East Bay Municipal Utility District (EBMUD) wastewater treatment facility, on the east by the Union Pacific railroad yard, on the south by 7th Street, and on the west by Maritime Street and the Port of Oakland Outer Harbor facilities.

### **2.2 SITE HISTORY**

The OARB is situated in an area that was reclaimed from San Francisco Bay. The earliest available map of the Oakland area, the San Antonio Creek Map of 1857 (Bache, 1857), suggests that the San Francisco Bay shoreline was approximately 500 to 3,000 feet east of the eastern edge of the base. The 1857 map shows several streams emptying into the bay at or near the current location of West Grand Avenue on the east side of the base. These former streambeds produced varying thicknesses and characteristics of the natural subsurface materials in those areas, and varying channel depths resulted in varying depths of artificial fill being placed across the site.

A map showing the historical bay shorelines along the Oakland waterfront (Rogers and Figuers, 1991) suggests that some parts of the Middle Harbor and Outer Harbor areas of the Port of Oakland were filled between 1894 and 1915 to develop the Oakland mole (at approximately the east end of the Port's outer harbor terminal) and marine terminals in the current Berth 22 to 25 area. However, as of 1915, the OARB area had not been developed and was outside (i.e., on the bay side) of the Oakland shoreline. It appears that most of the OARB was filled and the current shoreline established between 1915 and the mid-1940s.

Originally, the western area of the OARB was in shallow waters of the bay, and the eastern part consisted of tidal mud flats. The earliest development at the site began in 1918, when Building 99 (east of current Berth 10, which is identified as “Outer Harbor Terminal 10” on Figure 1) opened for ship building operations. Between 1918 and 1940, the western part of the base was the site of a variety of operations, including construction works, structural steel and pipe works, the Key Route electric railroad to the Bay Bridge, and a waste oil reclaiming plant (IT Corporation, 2000).

Based on a review of newspaper clippings obtained from the Oakland Public Library, we understand that the Army authorized the base in 1940 and construction started in June of 1941. The base was commissioned on December 8, 1941, with most of the current buildings constructed by 1946. Most of the current base was developed in the early 1940s by filling tidal flats and shallow waters of the bay, and most of the fill placement was completed by the mid-1940s. Records indicate that 6.5 million yards of fill was placed to reclaim the base area from the bay, primarily with hydraulically dredged silty sand. In general, the extent of fill has not changed at the base since the late 1940s.

### **2.3 PLANNED REDEVELOPMENT**

OBRA has developed several Base Reuse Plans. A recent potential development plan set forth at a public meeting with the Port of Oakland on August 19, 2000, consists of the northern portion of the OARB being used for City redevelopment and the southern area being used for Port of Oakland marine terminal operations. The western portion of the OARB may be used as an intermodal rail terminal. However, that reuse plan is in a state of flux and subject to change as the approval process advances. The geotechnical evaluation and recommendations described herein are based on general development concepts, and are not specific to any proposed facilities.

The OARB area to be redeveloped by the City has been termed the Gateway Development Area (GDA) due to its proximity to the Bay Bridge. The GDA conceptual plans include development of office buildings at the OARB Burma Road Terminal (located at the far west end of the GDA), a business park/light industrial area located north of Chung King Street and west of the existing Maritime Street; industrial buildings on the east side of the existing Maritime Street, north of about 17<sup>th</sup> Street; and relocation of Maritime Street to a location parallel to but approximately 500 feet east of the existing street.

The gross development area is 198 acres, with 177 acres of the total being currently available for building. OBRA is in the process of creating a new development plan for the GDA.

#### **2.4 PREVIOUS INVESTIGATIONS**

Numerous soil reports, site plans, boring logs, and laboratory test results from projects at or near the OARB site were collected and reviewed to obtain information on the development of the base, subsurface conditions, and current site topography. Our previous report (Geomatrix, 2001) presents a summary of the historic data, as well as additional data obtained as part of that study. A list of the historic reports is included in Appendix A. The work performed for the current study was based on the existing data; no field exploration was performed as part of this study.

### **3.0 SUBSURFACE CONDITIONS**

The following sections describe general subsurface conditions at the site, then specific characteristics of the deposits. Groundwater conditions also are summarized.

#### **3.1 GENERAL**

The generalized subsurface conditions, based on explorations performed at the OARB to as much as 200 feet in depth, can be divided into four strata consisting of fill; interfingering Young Bay Mud and sand; dense sand and stiff sandy clays; and stiff to very stiff silty clays. The three strata encountered below the fill correspond to the Recent Bay Deposits, San Antonio Formation, and Yerba Buena (Old Bay Mud) units, respectively. Soil layers corresponding to the Alameda Formation were not identified in the previous exploratory borings. The Franciscan bedrock is estimated to lie at depths of several hundred feet at the site. The thickness of the four major soil strata encountered in the borings vary across the OARB, and individual strata may be absent in some areas. Brief descriptions of the four general soil strata encountered at the project site are given in the following sections.

#### **3.2 FILL**

All the existing land within the OARB development area is underlain by fill placed in old tidelands or shallow bay water. Fill has been reported in all the onshore exploratory boring logs and CPTs reviewed by Geomatrix for this study. With the exception of ball fields, landscape areas and the rail yards, most of the base is covered by asphalt concrete or Portland cement concrete pavement, or structures. As previously discussed, the area surrounding the OARB was filled over many decades. Some of the land in the wharf area (west end of base and generally west of Maritime Street) was filled between 1894 and 1930 using dredged material (generally silty sands) from nearby borrow areas. The dredged material was hydraulically placed. The OARB area east of Maritime Street was filled in the early 1940s. Historical records indicate that 6.5 million yards of fill was placed to complete the Army Base, primarily with hydraulically placed, dredged silty sand. Dry fill materials, ranging from clayey sands to angular gravel, were placed in the upper portions of fill. Thus, the fill tends to vary in composition, from imported gravels to dredge spoils that include high-plasticity clays, such as Bay Mud, and clean to silty and clayey sands. In general, the consistency or density of the fill material ranges from dense or stiff at the surface to loose or soft at depth. The thickness of the fill was found to vary over the site, and the approximate thickness of fill is shown by the cross sections on Figures 2 through 4.

### **3.3 RECENT BAY DEPOSITS (INTERFINGERED YOUNG BAY MUD AND SAND)**

Interfingered Young Bay Mud and sand was encountered in subsurface explorations throughout the project site. For discussion purposes, the interfingered Young Bay Mud and sand will be referred to as Bay Mud or recent bay deposits. The Bay Mud generally consists of very soft to soft silty clay. Bay Mud generally is characterized as having low density, high water content, high plasticity, and high compressibility under imposed loads. The interfingered sands encountered typically consist of gray, loose to medium dense, fine to medium grained, clean to silty and clayey sands having shell beds of varying extents and thicknesses. The relative proportions of the two materials were found to vary greatly within short distances.

Recent bay deposits were encountered in all explorations reviewed for this project and ranged in thickness from approximately 6 feet at the south end of the site (near Building 590) to approximately 60 feet at Burma Road near the Toll Plaza. The approximate thickness of recent bay deposits is shown by the cross sections on Figure 2 through 4.

### **3.4 DENSE SANDS AND STIFF SANDY CLAYS**

The recent bay deposits generally are underlain by a stratum of dense to very dense sand containing layers of silty or sandy clay. These sands, referred to as the Merritt/Posey Sand of the San Antonio Formation, are generally fine to medium grained. The base of the San Antonio Formation typically extends down to elevations between -40 and -70 feet (Port of Oakland Datum or approximately Mean Low-Low Water [MLLW]) within the OARB. Subsurface exploration west of Maritime Street did not encounter sand in some locations; in other areas thin layers of medium dense to dense sand and silty sandy clay were encountered.

### **3.5 STIFF TO VERY STIFF SILTY CLAYS**

The sands and clays described above are underlain by a stratum of stiff to very stiff silty clays, which is referred to locally as Old Bay Clay, and is part of the Yerba Buena Formation. These are over-consolidated clays with relatively low densities and high water contents. Some of the stiff to very stiff silty clays that were encountered in the borings had water contents that were lower and densities that were higher than those typically associated with Old Bay Clay deposits.

### **3.6 GROUNDWATER**

Investigations performed within the study area indicate that groundwater lies between elevations +2 and +8 feet (Port of Oakland Datum). Based on current ground surface elevations, this is equivalent to depths of about 4 to 10 feet. It is generally higher toward the

southern portions of the site. Groundwater levels may fluctuate in response to changes in seasons, variations in rainfall, tidal influences, and other factors.

## **4.0 PRELIMINARY FOUNDATION RECOMMENDATIONS**

From a geotechnical perspective, it is our opinion that the site is suitable for the types of development that are proposed. The primary geotechnical concerns are the potential for the upper sandy fill to liquefy during strong ground shaking, and the compressibility of the Young Bay Mud that underlies the existing fill. In addition, the site will be subject to strong ground shaking during a significant earthquake. The following sections discuss these and other concerns and describe the preliminary foundation recommendations for the OARB site. Preliminary recommendations for both single- and multiple-story buildings are included.

### **4.1 DISCUSSIONS**

Selection of a foundation system for structures is dependent on the soil's ability to support the planned structure under both static and dynamic (earthquake) loads, on the settlement of the foundation due to building loads, and on the settlement of the site due to areal filling.

Structures that are properly supported on deep foundations that extend below the existing fill and Bay Mud will settle less than similar structures supported by shallow foundations that are founded above the Bay Mud. Alternatively, ground improvement measures may be taken to mitigate settlement and liquefaction effects at the site, as discussed below. Hence, the type of foundations to be used should be selected in consideration of the anticipated building loads and the amount of settlement that can be tolerated on a project-specific basis during final design phase evaluations.

For areas of the OARB closer to the bay waterfront, lateral spreading movements may be anticipated. The magnitude of such movements will be strongly affected by the stability of the shoreline slopes and waterfront dikes and/or retaining structures. Because shoreline/waterfront conditions have been modified locally, it is difficult to accurately predict the potential for lateral spreading at any specific location. However, we have developed preliminary, planning level, recommendations for addressing lateral spreading caused by liquefaction.

Another consideration in selecting the most appropriate foundation system for new buildings is the potential need to excavate and possibly dispose of soil or groundwater that may contain hazardous materials. In areas where hazardous materials are suspected, it may be more cost-effective to use a driven pile foundation, which may generate less excavated soil than shallow foundation excavations or drilled pier foundations. Deep foundations may also mitigate liquefaction-related foundation movement. Exploration and evaluation of potential contamination is being performed by another consultant. We recommend that selection of

appropriate foundation types for specific buildings be performed in consultation with the design team and the environmental consultant during preliminary design.

In several sites around San Francisco Bay Area that are underlain by fill over Bay Mud, lightweight, single-story buildings have been supported on shallow foundations in fill. This approach has the advantage of a lower construction cost compared with a deep foundation. However, buildings supported on shallow spread footings will be subject to both long term consolidation settlement and to seismically induced settlement due to the liquefaction and compaction of the sandy fill. At the OARB, shallow foundations for the planned industrial buildings that are lightweight, single-story structures may be appropriate depending on the desired performance of the buildings, i.e., they may be acceptable if the anticipated settlement and possible lateral spreading can be tolerated, or if the repair costs are factored into the life-cycle building costs.

Given the presence of Bay Mud and the variability in fill composition and density across the site, it is our opinion that future structures that are two or more stories in height should be supported on deep pile foundations that extend through the Bay Mud and derive their support in the underlying San Antonio Formation. In the Bay Area, 12- to 14-inch prestressed concrete piles are typically used at sites having subsurface conditions similar to the OARB. Ultimate pile capacities of 50 to 100 tons can be achieved if piles are embedded in the soils underlying the Bay Mud, as discussed below.

In some situations, drilled piers may be an attractive alternative deep foundation type rather than driven piles. As with driven piles, drilled piers would derive their axial capacity primarily from side friction in the dense sands and stiff clays of the San Antonio Formation below the Bay Mud. An advantage of large-diameter piers over piles can be the large compressive and uplift load capacities that can be developed in a single large-diameter pier, whereas multiple piles would be required to provide comparable capacities. The disadvantages of piers include disposal of the large quantity of soil and water generated during excavation (drilling), and challenging construction considerations associated with drilling below groundwater and drilling through the frequently-cohesionless fill soils. Significant cave-in problems in the underlying sand layers are likely if the holes are not properly cased or filled with slurry. Because of these concerns, drilled piers have not commonly been used in the Port of Oakland or OARB, and it is our opinion that they are not likely to be the preferred foundation type for future development at the OARB.

A design-level geotechnical investigation should be performed for any planned structure to assess the site-specific subsurface conditions and relate them to the building-specific design criteria. Further site-specific evaluations of shallow and/or deep foundations should be included in these investigations. The following recommendations are for planning purposes only, and should not be used for the final design of any building.

## **4.2 SHALLOW SPREAD FOOTINGS**

Where anticipated moderate settlement can be tolerated, lightly-loaded, single-story buildings may be supported on shallow spread footings bearing on the existing fill. For the purposes of this evaluation, we have assumed that only buildings with less than about 125 psf gross loading can be supported on shallow spread footings. Footings should have a minimum width of 2 feet and should extend about 2 feet below lowest adjacent finished grade. Preliminary recommendations for footing dimensions and allowable capacities are presented in Table 1 and on Figure 5. Allowable bearing capacities for total load, including wind and seismic loads, can be increased by one-third. Because of the possibility of earthquake-induced lateral spreading, buildings should not be supported on shallow spread footings at the west end of the site along Burma Road, as delineated on Figure 5.

Estimates of ultimate consolidation settlements for footings designed in accordance with the recommendations are also presented on Table 1. These estimates of consolidation settlement do not include settlement induced by placing site fills. Estimates of settlement induced by placing one foot of fill across the site are also shown on Table 1.

Potential liquefaction-induced settlement will be directly related the thickness of the existing sandy fill and the relative density of this fill. The estimated liquefaction-induced settlements are also presented in Table 1.

## **4.3 DRIVEN PILE FOUNDATIONS**

Multiple-story buildings as well as single-story buildings where the expected settlements cannot be tolerated should be supported on driven, pre-cast concrete piles. The driven piles will derive their support from the stiff clayey and dense sandy soils of the San Antonio Formation underlying the recent bay deposits. Based on our review of the available subsurface data, there are two likely scenarios for driven piles based on subsurface conditions. It is our experience that in areas where the San Antonio Formation is primarily dense sand, driven piles will reach refusal at relatively shallow embedment depths and derive the majority of their axial capacity through end-bearing in the dense sands. In areas where the San Antonio Formation is

primarily stiff clay, longer piles will be required to develop adequate skin friction to support the design loads. Review of the data indicates that end-bearing piles may be possible in an area south of 14<sup>th</sup> Street and an area west of the Caltrans Access Road (see Figures 6 to 9), and that friction piles will likely be needed in the area north of 14<sup>th</sup> Street and east of the Caltrans Access Road (see Figures 10 to 13). The sandy San Antonio Formation west of the Caltrans Access Road may be variable and locally discontinuous. Site-specific investigations should be performed to confirm the presence of the dense sand for each proposed building. If friction piles are required in some areas currently shown for end-bearing piles, the design pile tip elevations may be 20 to 35 feet deeper than those shown on Figures 6 to 9.

Preliminary estimates of design pile tip elevations for 12-inch and 14-inch square concrete piles have been estimated across the site. For the purposes of this preliminary evaluation, we have developed the estimated pile tip elevations for 50-ton and 100-ton ultimate capacities. The allowable dead plus long term live loads can be assumed to be about one-half the ultimate capacity. Figures 6 through 9 show the estimated pile tip elevation for end-bearing piles. Figures 10 through 13 show the estimated pile tip elevation for friction piles. Piles typically cost in the range of \$35/ft for 12-inch piles, and \$45/ft for 14-inch piles. Preliminary foundation costs can be estimated based on these unit costs, times the approximate number of piles that will be required for a structure, times the length of the piles in a given area.

Because of the variability of the subsurface conditions, design-level site-specific geotechnical investigations should be performed once actual building configurations have been determined. After a design-level geotechnical investigation has been performed and preliminary pile capacities calculated, an indicator pile program should be performed. The indicator pile program should consist of driving piles at various locations across the building site and evaluating the drivability and capacity of the proposed pile designs. We recommend that the capacity of the indicator piles be evaluated using a dynamic field monitoring procedure such as a pile driving analyzer (PDA) or the dynamic Pile Load Test (PLT). The final design pile capacities should then be evaluated based on the results of the indicator pile program.

Because of the history of the site, it is likely that the existing fill will contain obstructions that may cause complications during construction. These obstructions may include abandoned utilities, building footings and floor slabs, pavements, railroad tracks, and other construction debris. The possible effects of obstructions on the pile driving should be addressed in the site-specific geotechnical evaluations for each building.

Utility lines leading into pile-supported buildings must be designed to accommodate settlement of the ground relative to the building. Connections to the building may have to be flexible. Also, if the utility lines are suspended from the concrete floor slab, the hangers should be designed to support the weight of pipe and soil backfill over the pipe. Use of loose sand backfill over the suspended pipelines will reduce the earth loads on the pipeline hangers.

#### **4.4 GROUND IMPROVEMENT**

It may also be feasible to improve the subsurface conditions prior to construction, and thereby reduce the potential for consolidation settlement and/or liquefaction. Although these methods are not inexpensive, they may reduce settlement estimates enough that a building can be supported on shallow foundations, thus saving the cost increment of using a deep foundation. The following methods may be considered:

1. Surcharging may be used, either with or without wick drains, to reduce post-construction settlement related to consolidation of Young Bay Mud.
2. Geofoam may be used in place of site fills to reduce consolidation settlement.
3. If liquefaction potential is to be mitigated, four methods of ground improvement of the fill should be considered: Stone columns, Deep Dynamic Compaction (DDC), Rapid impact Compaction (RIC), and earthquake drains. We discussed the first two of these methods with Hayward Baker, the third with Moore and Tabor, and the last with Nilex Corporation, all of whom are ground improvement specialists. Some of the pros and cons of each approach are discussed below.

Each of these techniques is discussed further in the following sections. A summary of the relative cost and performance of each foundation type and ground improvement technique is shown on Table 2.

##### **4.4.1 Surcharging**

The post-construction consolidation settlement can be reduced by preloading or surcharging the site with a large fill. When properly implemented, the surcharge load will cause the site settlement to occur prior to building construction. The surcharge load is then removed and construction begins. Typically, prefabricated wick drains are installed prior to the placement of the surcharge load to facilitate drainage of the compressible soil, thereby accelerating consolidation and decreasing the time required to complete the surcharge program. Typical

wick drain spacing is about 5 feet, with wick drains extending from the ground surface to the bottom of the compressible layer (i.e., the recent bay deposits).

Although surcharging the site will reduce the potential for consolidation settlement, it typically does not reduce the potential for liquefaction or liquefaction-induced settlement. Surcharging with wick drains can also be expensive (on the order of \$0.60 per foot of wick drain plus \$0.50 per square foot of area to be surcharged). Surcharging may require a significant amount of time (as little as 3 months or up to 18-months with wick drains, and possibly longer than this without wick drains) before construction can begin.

#### **4.4.2 Geof foam**

Geof foam is a lightweight alternative to earth fill. Geof foam is a cellular plastic material that is strong but very lightweight, typically about 1 to 2 pounds per cubic foot. It is manufactured in large blocks that can be placed across a site to raise the grades without significantly increasing the stresses in the underlying soils. Geof foam may also be used to replace existing soil, to compensate for the added weight of a proposed building. This could result in no net increase in load to the underlying soils, thus reducing consolidation settlement. It will not affect liquefaction. The cost for Geof foam is about \$2 per cubic foot.

#### **4.4.3 Stone Columns**

Stone columns are a method in which a mandrel is first vibrated into the loose soil and then gravel is introduced into the hole and vibrated so that the gravel column becomes denser and larger in diameter, thereby densifying the surrounding ground. Stone columns can probably provide the greatest degree of compaction for the treated area and for the full depth of fill. With this method, it should be possible to essentially eliminate the potential for liquefaction within areas that are treated.

For preliminary costing, assuming sand fill depths on the order of 15 to 20 feet deep, and assuming approximately 3-foot-diameter stone columns spaced in a square grid of 9 feet on centers both ways, treatment of 50,000 square feet would require about 617 columns and cost about \$250,000, or about \$5 per square foot.

It should be noted that the stone columns normally extend beyond the footprint of the buildings by about half the thickness of the liquefiable layers. This is because the outer row of stone columns and the soil between them tends to soften during and following seismic shaking due to migration of excess pore pressures from outside the stabilized mass.

#### **4.4.4 Deep Dynamic Compaction**

DDC involves dropping a heavy weight from a crane onto the ground surface, resulting in compaction of the soil to depth of about 15 to 20 feet. For most of the types of fill soils encountered at the OARB, we anticipate that this method could densify the fills to the point where they would have a low susceptibility to liquefaction, and if seismic settlements occurred, they would be small.

It is likely that treatment would cost on the order of about \$2 per square foot of treated ground, or about \$100,000 to treat 50,000 square feet.

As with stone columns, it should be noted that the DDC treatment should normally extend beyond the footprint of the buildings by about half the thickness of the liquefiable layers, i.e., about 5 to 10 feet where fill is 10 to 20 feet thick (which is roughly the range of applicability for stone columns).

#### **4.4.5 Rapid Impact Compaction**

RIC is a method in which a 7.5 ton weight is lifted hydraulically and dropped about 1 meter, at a rate of about 40 to 60 blows per minute. It is a relatively new technology to the United States, but based on preliminary information it is expected to have a similar effectiveness to that of DDC. Prior to final selection of this approach, we would recommend a test program be implemented to treat sample areas on the site. Moore and Tabor indicates it would cost about \$12,000 for a day of testing in which several 20' x 20' test areas could be treated. Each area should have several CPT probes pushed before treatment as well as after to evaluate the effectiveness of the method.

The cost of RIC is expected to be about \$2.30 per square foot . To treat 50,000 square feet, the cost would be about \$115,000.

As with stone columns, it should be noted that the RIC treatment should normally extend beyond the footprint of the buildings by about half the thickness of the liquefiable layers, i.e., about 5 to 10 feet where fill is 10 to 20 feet thick.

#### **4.4.6 Earthquake Drains**

Earthquake drains are vertical pipe drains that are inserted by pushing with a mandrel to the depth of concern. They mitigate liquefaction by providing a vertical drainage path for water to escape, thus reducing the increase in pore pressures generated by the earthquake. The

installation of earthquake drains also typically causes a minor amount of ground densification, but some seismically-induced settlement may still occur.

Earthquake drains consist of perforated rigid plastic pipe wrapped in a geotextile filter sock. The tops of drains should be connected to a manifold and directed to drain to the storm drain system. For this application we expect they should be installed in a triangular grid with a spacing of about 4 ½ to 6 feet on centers, to a depth of the bottom of existing fill. Earthquake drains typically cost on the order of \$4 per linear foot installed. The costs shown on Table 2 assume a depth of loose fill of 15 feet, but the actual depths of fill across the site vary from about 10 to over 30 feet, so earthquake drain costs would vary as well.

#### **4.4.7 Settlement Considerations for the Ground Modification Alternatives**

It should be noted that the ground surface will still settle, even after improvement of the existing fill, due to consolidation of the underlying Bay Mud under the weight of new fill and structures. Any of the treatment options will result in a densification of the fill over the Bay Mud, and this increased density will result in some increased load on the Bay Mud. In addition, new fill placed for site grading and the weight of the buildings will also cause settlement of the ground surface.

Site settlements will be larger in the building areas that have been treated because of the building weight, because the existing fill density will be increased by the ground treatment, and because additional fill may need to be added to fill areas that have settled due to the densification (by DDC or RIC; the stone column methods effectively adds fill within the material being densified). The settlement should be relatively uniform under any given building, but some differential settlement should still be anticipated. In order to reduce the effects of differential settlement across a building, the foundations could consist of a structural slab, post-tensioned concrete, or a slab stiffened with a grid of grade beams.

It may be possible to partially mitigate the differential settlement between the buildings (which will settle more) and the adjacent paved or landscaped areas (which will settle less) by placing a layer of geogrids or woven geotextile fabric at least 1.5 feet below the finished grade, spanning between the building and non-building areas. This will help to distribute the load somewhat and will resist the development of abrupt breaks in the surface expression of settlement. It will also reduce differential settlements during an earthquake. Buildings pads should also be built high enough to maintain positive drainage away from the buildings as settlements occur.

## 5.0 LIMITATIONS

The primary objectives of this preliminary foundation evaluation were to compile and evaluate subsurface data and to provide preliminary foundation recommendations for the Oakland Army Base redevelopment. The discussions and preliminary recommendations are based on the results of limited field investigations and evaluations. They are intended to assist OBRA in planning elements of the project and in evaluating alternatives. Although the data are sufficient to carry out the anticipated preliminary design of specific projects, supplemental and more detailed site-specific investigations and analyses are needed for final design and construction recommendations for all structures planned for the OARB site.

The required extent of future geotechnical studies will depend on the type and extent of proposed improvements, as well as on their proximity to areas for which subsurface data are already available. A thorough geotechnical study for a typical building indicated in OBRA's conceptual plan likely would include drilling 4 to 6 exploratory borings and possibly performing several CPTs, evaluating the engineering properties of the subsurface materials, and developing site grading and foundation recommendations for the planned structure. Foundation design for new structures will be heavily influenced by structure loads, the extent of site grading, and the thickness of recent bay deposits underlying the site. Methods to mitigate liquefaction hazard should also be evaluated.

The extent of liquefiable deposits within Oakland Army Base is known only in general terms. Additional subsurface information is required to provide a more detailed assessment of the location and possible consequences of liquefaction within given areas. Where the consequences of liquefaction are unacceptable, further studies are required to evaluate the most cost-effective means of mitigating potential liquefaction.

## **6.0 BASIS FOR RECOMMENDATIONS**

The evaluations made in this report are based on the assumption that soil conditions at the site do not deviate appreciably from those described herein, and are disclosed in the exploratory borings. In the performance of our professional services, Geomatrix, its employees, and its agents comply with the standards of care and skill ordinarily exercised by members of our profession practicing in the same or similar localities. We are responsible for the evaluations contained in this report, which are based on data related only to the specific project and location discussed herein. In the event conclusions based on these data are made by others, such conclusions are not our responsibility unless we have been given an opportunity to review and concur in writing with such conclusions.

## 7.0 REFERENCES

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- Rogers, J.D., and Figuers, S.H., 1991, Engineering geologic site characterization of the greater Oakland-Alameda area, Alameda and San Francisco counties, California: Report to the National Science Foundation, Grant No. BCS-9003785, 52 p., December.

**TABLE 1**  
**SUMMARY OF SHALLOW SPREAD FOOTING RECOMMENDATIONS**  
Oakland Army Base Reuse Study  
Preliminary Foundation Evaluation  
Oakland, California

Location <sup>1</sup>	Estimated Consolidation Settlement Under 1 ft of Fill <sup>2</sup> (in)	Estimated Liquefaction Induced Settlement (in)	Square Footings			Strip Footings		
			Footing Widths	Allowable Bearing Capacity <sup>3</sup> (psf)	Estimated Consolidation Settlement (in)	Footing Widths	Allowable Bearing Capacity <sup>3</sup> (psf)	Estimated Consolidation Settlement (in)
A	1	3	2 to 5 feet	2,000	3	2 to 3 feet	2,000	3
B	2 to 4	3	2 to 4 feet	2,000	3	2 feet	1,000	3
C	2 to 5	2	2 to 2½ feet	2,000	3	2 feet	600	3
D	2 to 4	6 to 12	2 to 5 feet	2,000	1	2 to 3 feet	2,000	3
E	2 to 4	Liquefaction-induced lateral spreading likely; shallow spread footings not recommended without site specific evaluation. Alternatively, ground improvement may be considered to mitigate liquefaction potential.						

1. See Figure 5 for location of areas
2. See Figure 12 of Geomatrix (2001) for additional details of distribution of settlement potential
3. Allowable bearing capacity for dead plus long-term live load.

**TABLE 2**  
**SUMMARY OF RELATIVE COSTS FOR FOUNDATION AND GROUND IMPROVEMENT OPTIONS**

Mitigation for Consolidation Settlement	Mitigation for Liquefaction Potential	Typical Ground Improvement Cost	Shallow Spread Footings		Mat or Structural Slab, or Slab with Thickened Grid		Deep Foundations (driven piles)	
			Cost	Performance	Cost	Performance	Cost	Performance
None	None	\$0.00/sf	Low	Poor	Moderate	Poor	<b>Mod High</b>	<b>Good</b>
None	DDC	\$2.00/sf	Mod Low	Poor	Mod	Mod Poor	<b>High</b>	<b>V. Good</b>
None	RIC	\$2.30/sf	Mod Low	Poor	Mod	Mod Poor	<b>High</b>	<b>V. Good</b>
None	EQ Drains <sup>3</sup>	\$3.30-5.00/sf	Mod High	Poor	High	Mod Poor	<b>High</b>	<b>V. Good</b>
None	Stone Columns	\$5.00/sf	Mod High	Poor	High	Mod Poor	V. High	V. Good
Surcharge w/o Wick Drains	None	\$1.00/sf	Mod Low	Poor in EQ	Mod High	Mod in EQ	High	Good
Surcharge w/o Wick Drains	DDC	\$3.00/sf	<b>Mod</b>	<b>Good</b>	Mod High	Good	<b>High</b>	<b>V. Good</b>
Surcharge w/o Wick Drains	RIC	\$3.30/sf	<b>Mod</b>	<b>Good</b>	Mod High	Good	<b>High</b>	<b>V. Good</b>
Surcharge w/o Wick Drains	EQ Drains <sup>3</sup>	\$4.30-6.00/sf	<b>Mod High</b>	<b>Fair</b>	High	Good	<b>V. High</b>	<b>V. Good</b>
Surcharge w/o Wick Drains	Stone Columns	\$6.00/sf	Mod High	Good	High	Good	V. High	V. Good
Surcharge w/ Wick Drains <sup>4</sup>	None	\$3.10/sf	Mod Low	Poor in EQ	Mod High	Mod in EQ	High	Good
Surcharge w/ Wick Drains <sup>4</sup>	DDC	\$5.10/sf	<b>High</b>	<b>Good</b>	High	Good	V. High	V. Good
Surcharge w/ Wick Drains <sup>4</sup>	RIC	\$5.40/sf	<b>High</b>	<b>Good</b>	High	Good	V. High	V. Good
Surcharge w/ Wick Drains <sup>4</sup>	EQ Drains <sup>3</sup>	\$6.40-8.10/sf	High	Fair	High	Good	V. High	V. Good
Surcharge w/ Wick Drains <sup>4</sup>	Stone Columns	\$8.10/sf	V. high	Good	V. High	Good	V. High	V. Good
Geofoam <sup>5</sup>	None	\$2.00/sf	Mod	Poor in EQ	High	Mod in EQ	V. High	Good
Geofoam <sup>5</sup>	DDC	\$4.00/sf	<b>Mod High</b>	<b>Good</b>	High	Good	V. High	V. Good
Geofoam <sup>5</sup>	RIC	\$4.30/sf	<b>Mod High</b>	<b>Good</b>	High	Good	V. High	V. Good
Geofoam <sup>5</sup>	EQ Drains <sup>3</sup>	\$5.30-7.00/sf	Mod High	Fair	High	Good	V. High	V. Good
Geofoam <sup>5</sup>	Stone Columns	\$7.00/sf	Mod High	Good	High	Good	V. High	V. Good

**Notes:**

(see next page)

**Notes for Table 2: SUMMARY OF RELATIVE COSTS FOR FOUNDATION AND GROUND IMPROVEMENT OPTIONS**

1. Likely preferred alternatives are shown in **Bold and are Highlighted**. Other alternatives may be selected based on project-specific criteria.
2. These estimated relative costs are based on previous experience on similar projects. They should be considered approximate and should only be used for preliminary planning evaluations. Actual costs may be significantly different due to site-specific considerations, time of year of construction, the general state of the local economy at the time of construction, and other factors.
3. Cost assumes Earthquake Drains would be 15 feet deep to reach the bottom of existing fill, spaced on triangular grid of 6 feet to 4 ½ feet. Where existing fill extends deeper or shallower, or if closer spacing is needed, costs will be proportionally greater or less.
4. Cost assumes wick drains would be 50 feet deep to reach the bottom of Bay Mud. Where Bay Mud extends deeper or shallower (as shown on Figures 2 to 4), costs will be proportionally greater or less.
5. Geofoam cost is for a building of about 125 pounds per square foot net dead plus live load (likely range for 2 story building); geofoam for lighter or heavier (e.g., shorter or taller) buildings would be proportionally more expensive.