# Appendix H

Geotechnical Assessment Report

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April 24, 2012
Project No. 60208355.008

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**Subject:** **Technical Memorandum**
**Addendum to Geotechnical Assessment Report**
**Proposed VA Facilities, Alameda NAS**
**Alameda, California**

Dear Ms. Allsep,

Attached please find a technical memorandum provided as an addendum to the subject AECOM Technical Services, Inc. (AECOM) report prepared to 1) update certain information contained in the subject report originally prepared in May 2009, and 2) summarize and comment on a new geotechnical report just received from HDR for alternative project facilities adjacent and directly north of the original proposed facilities addressed in the May 2009 AECOM report.

Please contact me if there are any questions.

Very truly yours,
AECOM

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Enclosure

**TECHNICAL MEMORANDUM**
**APRIL 24, 2012**
**PROJECT NO. 60208355.008**

# ADDENDUM TO GEOTECHNICAL ASSESSMENT REPORT PROPOSED VA FACILITIES, ALAMEDA NAS ALAMEDA, CALIFORNIA BY AECOM, DATED MAY 13, 2009

## GENERAL

The subject report, prepared in May 2009, was developed to provide preliminary geotechnical assessment and recommendations to support environmental assessment for the Veterans Administration (VA) Facilities Project at the Alameda Naval Air Station (NAS) in Alameda, California, as the project was conceived at that time. The project involves land transfer from the Navy to the VA for development of medical and cemetery facilities. The present ongoing environmental assessment work is currently addressing two development alternatives as follows:

- Alternative 1 - development of a 158,000 square foot outpatient clinic in the north central portion of the land transfer parcel, flanked by cemetery facilities to the west and east.

- Alternative 2 - development of a 158,000 square foot outpatient clinic directly to the north of the Alternative 1 location, in the north central portion of an expanded land transfer parcel, with cemetery facilities to the west.

Alternative 1 is a modified version of the project facilities being considered at the time of the AECOM geotechnical assessment work in 2009. With the introduction of Alternative 2, a supplemental geotechnical investigation has been performed by Allegiance Group, LLC for HDR Architects, Inc., specifically addressing geotechnical conditions and recommendations, including new field investigation work, for the Alternative 2 project, with results presented in an Interim Geotechnical Investigation Report dated April 5, 2012.

The purpose of this technical memorandum, as an addendum to the May 13, 2009 AECOM geotechnical assessment report, is to update certain information presented in the AECOM May 2009 geotechnical assessment report to reflect the current Alternative 1 project, and to summarize and comment on the new HDR geotechnical investigation report associated with the Alternative 2 project.

## ENVIRONMENTAL ASSESSMENT ALTERNATIVE 1 - PREVIOUS GEOTECHNICAL ASSESSMENT REPORT

The VA project facilities, as proposed at the time of the previous AECOM geotechnical assessment work in 2009, have been modified and refined to constitute the current Alternative 1 project, as follows:

- Elimination of the originally planned hospital facility.
- Elimination of the original planned helipad facility in support of the hospital.
- Expansion of the original planned outpatient clinic facilities to 158,000 square feet, relocated to the original planned hospital location.
- Modification of the cemetery facilities to be developed on both the west and east sides of the north central location of the outpatient clinic facilities.

The alteration of the previous project facilities plan to form the current Alternative 1 project does not substantially change the May 2009 geotechnical assessment report conclusions and recommendations addressing proposed site development. Thus, the contents of the previous geotechnical report are considered currently valid and applicable to the present environmental assessment Alternative 1 project.

## ENVIRONMENTAL ASSESSMENT ALTERNATIVE 2 - NEW GEOTECHNICAL INVESTIGATION REPORT

The critical development project structures comprising Alternative 2 are proposed to the north of their location for Alternative 1, in a northward expanded transfer parcel. The new HDR geotechnical investigation report specifically addresses and supports the north Alternative 2 project facilities area, and is based on additional, site specific new field exploration and testing work. Review of the new, current HDR geotechnical investigation report relative to and in comparison with the prior AECOM 2009 geotechnical assessment report supporting the Alternative 1 project, results in the following observations and conclusions:

- The level of detail and depth of assessment and recommendations are similar in both reports.
- Both reports have many similarities and do not vary widely in description of geotechnical site conditions and identification of geotechnical, geologic, and seismic hazards.
- Both reports have many similarities in site development, structure foundations, and site improvement recommendations.

- Site improvement to mitigate against potential liquefaction, lateral spreading, and settlement is discussed and suggested in both reports. While the HDR report presents a couple of site improvement concepts and methodologies, deep dynamic compaction appears to be the likely and most cost effective approach, as preliminarily presented. This is the methodology introduced in the AECOM report.

- Both reports discuss reasons to avoid expensive deep pile foundations. With suggested ground improvement and settlement mitigation by surcharging, conventional shallow foundations are discussed in the HDR report, but with special interconnections and reinforcement to limit differential settlement. The similar counterpart to this, introduced in the AECOM report, is consideration of using mat foundations.

- At the time of the AECOM 2009 report, site specific seismic parameters were identified based on the California Building Code (CBC), which is not the criteria followed by the VA. The applicable criteria adopted for VA facilities is associated with the International Building Code (IBC), as laid out in the HDR report, which can be considered applicable to both alternative project sites.

- Based on the above comparisons of the various geotechnical assessments and recommendations provided in the previous and new geotechnical reports, directed toward the current project development alternatives at the two adjacent site locations, the two reports are considered to be fairly interchangeable in their applicability to either of the project alternatives. This is concluded since the conditions at the two alternative sites are not expected to vary widely, and the conclusions and recommendations and level of assessment detail do not vary widely between the two reports.

AECOM

DRAFT

# GEOTECHNICAL ASSESSMENT REPORT

for Proposed VA Facilities, Alameda NAS, California

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A Background Information Used

## 1.0 INTRODUCTION

This report presents the results of our preliminary geotechnical evaluation for the proposed Veterans Administration (VA) Facilities at the former Alameda Naval Air Station (NAS) in Alameda, California. This evaluation was performed in general accordance with the inter-company agreement with EDAW | AECOM dated April 7, 2009.

### 1.1 Project Description

The entire former Alameda NAS will eventually be transferred to the City of Alameda for various uses. This report concerns the Alameda Point area, shown on Figure 1, where certain VA facilities are being proposed. The property is generally located south of the previously proposed Alameda Point Golf Course and north of the IR Site 2 landfill and the protected Least Tern bird colony.

It is our understanding that the proposed VA facilities will generally consist of a 2-story hospital structure (250,000 square feet) and associated helipad, a 2-story outpatient clinic (107,000 square feet), two 2-story office buildings (100,000 square feet), a nature center, and a 53-acre Columbaria development for cremated remains. In addition to the structures, necessary access roads, parking, and underground utilities will also be included. Structural loads are not yet available, but should be typical for 2-story structures, although the proposed hospital structure may incorporate higher loads. It is also anticipated that some low additional fills will be necessary to create positive drainage gradient across the site.

### 1.2 Purpose

The purpose of this report is to provide preliminary geotechnical assessment and recommendations to support environmental assessment for the VA facilities project. The discussions, conclusions, and recommendations presented in this report are based upon available geologic mapping, subsurface data, and review of work by other consultants. The subsurface data and geologic mapping are limited and preliminary in nature. They should be supplemented during final design of the project, when specific plans and structural loads are known, in order to provide design-level geotechnical recommendations.

### 1.3 Scope of Work

We have reviewed geotechnical related information and reports provided for the Alameda Point area, inclusive of the proposed VA facilities project site, particularly the October 2001 Harding ESE report "Geotechnical Investigation for Preparation of the Draft Environmental Impact Report, Alameda Point Golf Course, Alameda, California". Based upon our understanding of the type of geotechnical information and level of detail needed to support the current environmental assessment effort, it is our opinion that review and use of existing data and information near and surrounding the proposed VA facilities project site is sufficient basis for development of the current desired geotechnical input. The following geotechnical scope of work was established to support environmental assessment of the VA facilities project.

**Task 1 - Site Visit / Meeting**

A brief site visit was conducted for familiarization with existing conditions of the area of the proposed VA facilities, combined with meeting other project / client staff for project discussions.

**Task 2 - Data Review**

Existing data review was undertaken to provide a basis for geotechnical input to the environmental assessment. This task included the following:

- Review of pertinent existing geotechnical related data and reports made available for our use in the geotechnical evaluation.
- Obtaining additional geotechnical related information that would appear useful, based on the review of initially available information.
- Assessment and identification of the geotechnical related material considered applicable for use in and providing basis for geotechnical input to the environmental assessment.
- Development of general characterization of geotechnical project site conditions.

**Task 3 - Geotechnical Engineering Evaluation**

Based upon the review and selection of applicable existing available geotechnical related data, geotechnical engineering evaluation at the conceptual / preliminary level was made to develop input and recommendations to support the environmental assessment, addressing the following site development geotechnical considerations, including adverse conditions and mitigation:

- Geologic / seismic hazards
- Liquefaction
- Structure foundation support
- Site earthwork and grading
- Ancillary site development aspects, including underground utilities, pavement and drainage

**Task 4 - Report**

Results of the data review and geotechnical engineering evaluation, and associated preliminary geotechnical recommendations, are presented in this report for use to support the environmental assessment of the proposed VA facilities project.

### 1.4 Limitations

The data, information, interpretations, and conclusions contained in this report are presented solely as bases and guides to the existing conditions at the site of the proposed VA facilities. The conceptual stage preliminary conclusions and professional opinions presented herein were developed by AECOM Technical Services, Inc. (AECOM) in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied. These data, conclusions, and recommendations should be considered to relate only to the specific project and locations discussed herein. AECOM is not responsible for any conclusions and recommendations that may be made by others.

This report has been prepared for the express use of EDAW | AECOM and its clients. It may not contain sufficient information for the purposes of other parties for other uses. If any changes are made in the project as described in this report, the conclusions and recommendations contained herein shall not be considered valid, unless the changes are reviewed by AECOM, and the conclusions and recommendations of this report are modified or approved in writing.

## 2.0 BACKGROUND

Historical records indicate that Alameda Point was formerly a shallow mud flat consisting of young Bay Mud with depths generally ranging from 20 to over 100 feet thick. The area has been filled to create land over a protracted period of time from 1906 to about 1956.

### 2.1 Site History

In general, the fill sequence indicated above has consisted of periodic sandy fills, placed by hydraulic dredging methods in several phases. Historical data indicates that sandy fill obtained from dredging was placed along the Oakland Channel between 1906 and 1915 for the Southern Pacific Narrow Gauge Railroad. Further filling was conducted between 1915 and 1934, between 1938 and 1947, and between 1949 and 1956 in areas proceeding from east to west across the property. The westerly fill consists of heterogeneous landfill materials consisting of a wide variety of waste material and construction debris. A fill history is shown on Figure 2.

As part of the fill sequence, a series of scuttled hulls of small warships were used as a breakwater for a yacht harbor between the years 1934 and 1938. Figure 2 shows the approximate location of these scuttled hulls in relation to the proposed VA facilities. Present information suggests that these hulls are still present beneath the ground surface, but may have drifted during past storms, and therefore, their exact locations are not known. They may, however, have an impact on any development of the site, particularly if pile support for structures is considered.

### 2.2 Previous Investigations

In conjunction with plans to close the Alameda NAS, a number of previous studies have been undertaken by engineering consultants. Harding ESE performed a geotechnical investigation for the proposed Alameda Point Golf Course, just north of the subject VA facilities site. Their study is titled "Geotechnical Investigation for Preparation of the Draft Environmental Impact Report, Alameda Point Golf Course, Alameda, California", dated October 9, 2001. AECOM has used the information supplied by this report to supplement our research for the VA facilities site. Also, we have reviewed and used data contained in a number of other reports and letters provided to us for our use. The information that we have used is primarily contained in reports by Tetra Tech EM, Inc., Subsurface Consultants, Inc., Battelle, and the United States Geological Survey (USGS). The specific references are detailed in the references section of this report.

## 3.0 SITE CONDITIONS

### 3.1 Regional Geology

The VA facilities site at Alameda Point is located on the eastern side of San Francisco Bay in the Coast Range geomorphic province. The Coast Range is characterized by northwest trending mountains and valleys, and is dominated by northwest trending faults, folds, and geologic structures. The site is bordered on the west by San Francisco Bay, a

drowned, northwest trending structural depression. The Bay and much of its margins are underlain by the late Mesozoic age rocks of the Franciscan complex. The Franciscan rocks commonly consist of sheared shale and interbedded sandstone, with serpentinite and other metamorphic rocks. Tertiary and Quaternary formations occur locally in unconformity on the Franciscan, while other Mesazoic formations occur in fault contact with the Franciscan complex.

Beneath San Francisco Bay and its margins, the Franciscan bedrock is overlain by a young, geologically unconsolidated sedimentary sequence, which in places exceeds 400 feet in thickness. The sequence is divided into three units, older Bay sediments of the Yerba Buena formation, Merritt sands of the San Antonio formation, and younger Bay Mud. Artificial fill of variable thickness, quality, and density has been placed along the margins of the Bay to reclaim marshland and land once covered by shallow water. A generalized geologic cross section across the site is shown on Figure 3 of this report.

### 3.2 Seismicity and Faulting

The project area is located in a seismically active region which has been subjected to a number of strong earthquakes during historic time. The San Andreas fault system dominates the tectonics, geology, and physiography of the San Francisco Bay region. The San Andreas fault system passes about 12 miles west of the site. The Hayward fault is approximately 6 miles east of the site. Other major faults which could cause significant shaking at the site include the San Gregorio, Calaveras, Concord, Rogers Creek, and Greenville faults. The major fault systems in the area are shown on Figure 4.

### 3.3 Surface Conditions

A field visit was made to the project site for observation of current conditions. The site visit was conducted on April 3, 2009. Based on the site visit and observations made, the site surface conditions can be summarized as follows:

- Generally flat site conditions.

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- Some localized, depressed areas with standing water and associated vegetation.
- Significant pavement areas crossing portions of the site along air strip runways, with some degree of deterioration and weed growth.
- Some existing bunkers on the proposed VA facilities site.

### 3.4 Subsurface Conditions

AECOM has reviewed the subsurface information from various borings and CPT data, extracted from existing studies

and reports made available to us, and we have identified the pertinent borings located either on or in the immediate vicinity of the proposed VA facilities property. This existing subsurface data has been used as a basis for our assessment of geotechnical conditions. The applicable borings that we have used for our studies are shown on Figure 2. For reference, the applicable logs and data used is provided in Appendix A.

Our research has indicated that the VA facilities site is probably underlain by from 15 to as much as 30 feet of artificial fill consisting of loose to medium dense sands, overlying from as little as 30 feet to as much as about 65 feet of very soft, compressible younger Bay Mud deposits. In turn, the younger Bay Mud is underlain by about 30 feet of dense to very dense sands of the San Antonio formation, including Merritt and Posey sands. These sands overly stiff to very stiff, older Bay Mud (clay) deposits with a similar origin as the younger Bay Mud.

Based upon information secured from Special Report 97, "Geologic and Engineering Aspects of San Francisco Bay Fill" by the California Division of Mines and Geology, 1969, a deep linear deposit of younger Bay Mud extends beneath the proposed VA facilities site, with the base of the younger Bay Mud as deep as 80 to 90 feet. Areas of the former Alameda NAS, both north and south of the proposed VA facilities site, show the base of the younger Bay Mud at much shallower depths. This thicker deposit of younger Bay Mud will have an effect on development of the proposed VA facilities, as detailed later in this report.

Special Report 97, referenced above, also indicates that bedrock was encountered along the northern boundary of the Alameda Point property (north of the VA facilities property) at a depth of 433 feet.

### 3.5 Groundwater

Groundwater has been encountered quite close to the present ground surface. Our research has indicated a groundwater depth of on the order of about 3 to 6 feet, and is apparently influenced by tides.

## 4.0 SITE GEOTECHNICAL ISSUES AND HAZARDS

### 4.1 Fault Rupture

No earthquake faults have been geologically mapped beneath the subject site. Therefore, it is our opinion that the potential for fault rupture beneath the proposed development is low to negligible.

### 4.2 Ground Shaking

The site is likely to be subjected to strong ground shaking during the life of the project. Previous work for the Port of Oakland (Geomatrix and Harza, 1997) has developed recommendations for design-level ground shaking for four hazard levels corresponding to 50, 20, 10, and 5 percent probability of exceedance in 50 years. The associated peak ground acceleration (PGA) for each of these levels was found to be 0.29g, 0.45g, 0.57g, and 0.68g, respectively. These levels are considered to be appropriate for preliminary planning of the subject project.

### 4.3 Liquefaction

Liquefaction occurs in loose, saturated cohesionless soils during earthquake shaking. Hydraulically placed sandy fill is particularly prone to liquefaction. During liquefaction, rapid loss of strength due to pore water buildup occurs. The results of liquefaction include settlement of the ground surface, buried utilities, and structures founded on shallow foundations. Available geologic information (e.g., Youd and Hoose, 1978) show evidence of ground settlement during past earthquakes.

Based upon review of a limited number of boring logs, on and near the project site, by others and our research, we performed a preliminary liquefaction analyses for the site. Our analyses show that the San Andreas fault is the controlling fault with a moment magnitude of 7.9. For an expected peak ground acceleration of 0.57g (10 percent exceedance in 50 years), the saturated sandy fill materials on the site below the water table are prone to liquefaction, with calculated liquefaction settlements on the order of 4.4 inches to 9.4 inches for saturated fill thicknesses of 10 feet and 25 feet, respectively.

Additional liquefaction studies should be performed during the design level phase to get a more accurate estimate of liquefaction settlement beneath the site.

### 4.4 Lateral Spreading

Lateral spreading is generally caused by liquefaction of granular soils behind a free face or below sloping ground. As soil liquefies, land loses shear strength, it tends to move laterally and vertically down the slope or towards a free face, such as a shoreline or channel.

Harding ESE has indicated that liquefaction of the fills on the previously proposed Alameda Point Golf Course site (north of the VA facilities site) could cause over 10 feet and possibly as much as 40 feet of horizontal displacement at the

northern shoreline (Oakland Channel), and up to about 3 feet of horizontal movement near the south edge of the golf course site, which is the northern boundary of the VA facilities site.

The proposed VA facilities hospital structure, clinic, and office buildings are located still further away from the Oakland Channel (on the order of 1,500 feet). Therefore, it is our opinion that the potential for lateral spreading for the proposed VA facilities site will generally be limited to less than about 3 feet.

A somewhat larger potential for lateral spreading does exist at the west end of the proposed Columbaria property near the shoreline of San Francisco Bay. Potential lateral displacements on the order of those indicated by Harding ESE should initially be assumed in this area. Structures associated with the Columbaria development should be kept well back from the shoreline in this area due to the possibility of lateral displacement.

Additional studies of lateral spreading should be performed during a design level investigation to obtain more exact data regarding the potential for lateral spreading.

### 4.5 Seismic Settlement

Seismically induced settlement resulting from densification of the loose sandy soils above the water table will also occur. It is estimated that this settlement could be on the order of several inches and will be in addition to the liquefaction settlements below the water table indicate above.

### 4.6 Consolidation Settlement

Based upon our research of available geologic and boring data, the VA facilities site is believed to be underlain by soft younger Bay Mud varying in thickness from about 30 feet to as much as about 65 feet. Placing additional fill or constructing buildings with shallow foundations will place additional weight on the Bay Mud, which will cause consolidation of the Bay Mud layer, resulting in settlement at the ground surface. Consolidation will occur relatively slowly as excess pore pressures dissipate. The amount of consolidation settlement will depend on the thickness of the existing fill, thickness of the soft Bay Mud, and the imposed loads from the new fill and buildings.

A preliminary analysis of consolidation settlement was performed, based upon the addition of up to an additional fill thickness of 5 feet to allow for positive drainage on the site. Consolidation settlement will increase with increasing thicknesses of fill. Therefore, it is important that the additional fill thickness be kept to a minimum across the site. Our analyses indicate that for a soft Bay Mud thickness of 30 feet, the additional expected consolidation settlement of a 5-foot

fill will be on the order of about 17 inches in 50 years, and for a Bay Mud thickness of 65 feet, the additional expected consolidation settlement will be on the order of about 18 inches in 50 years.

Building loads are not yet known; however, additional loading of the Bay Mud due to building loads will increase the estimated settlements indicated above. When building loads are defined, additional consolidation settlement analyses should be performed to determine the magnitude of settlement due to the building loads.

## 5.0 PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

### 5.1 Site Preparation, Earthwork, and Grading

The site should be stripped of vegetation, organic soils, and any debris, and the upper 12 inches of soil should be scarified, brought to a moisture content near optimum moisture content, and compacted to a minimum of 90 percent of the laboratory maximum dry density, determined in accordance with ASTM D1557, latest edition. It is anticipated that in areas covered by runway concrete, the pavement and any base rock may be removed and reused as base materials for the project parking lots.

Any excavations for debris pockets may encounter groundwater. This may require dewatering to remove the undesirable materials in these areas. It is also anticipated that a geotextile layer may be required for base stability where excavations extend to near the shallow water table.

Any new fill should be granular in nature, non expansive, and should be compacted in loose lifts limited to 8 inches, to a minimum of 90 percent of the laboratory maximum dry density, determined in accordance with ASTM D1557, latest edition. If dredged material is considered, only sandy dredged materials should be allowed. Compaction near the groundwater level may be a problem. However, it is important that measures be taken to achieve a firm and non-yielding fill.

### 5.2 Structure Foundation Support

For preliminary design, we believe that any building requiring a foundation that is not subject to considerable settlements should be founded on driven, concrete piles. However, because of the possibility that the base of the soft Bay Mud material could extend to as much as about 95 feet below the ground surface, long pile lengths could be required, and the allowable pile capacity will be limited.

Structural concrete mats could be a viable alternative to driven piles. Although subject to settlements indicated earlier, structural concrete mats can spread the building loads over large areas, and settlements tend to be more uniform than conventional spread footings.

Conventional shallow spread footings could be considered for very lightly loaded structures founded on soils that are improved in situ, as discussed later in this report. Structural mats can also derive benefit from preparatory ground improvement methods discussed below.

### 5.2.1 Deep Foundations

Driven prestressed, precast concrete piles are expected to extend to required depths from approximately 50 to 60 feet and to as much as 100 to 110 feet, depending on the thickness of soft Bay Mud encountered on the site at the specific structure location. The piles should extend into the very dense sands below the soft Bay Mud materials.

For preliminary design, we have calculated the allowable compressive load capacity of 12-inch by 12-inch square, concrete piles. For the base of the soft Bay Mud at approximately 45 feet, the allowable compressive capacity for the indicated pile is 45 kips at a depth of 50 feet (5-foot penetration into the dense sands) and 120 kips at a depth of 60 feet (15-foot penetration into the dense sands). Similarly, for the base of the soft Bay Mud at approximately 95 feet, the allowable compressive capacity for the indicated pile is 83 kips at a depth of 100 feet (5-foot penetration into the dense sands) and 150 kips at a depth of 110 feet (15-foot penetration into the dense sands). During design level studies, a more accurate picture of the variable depth to dense sand bearing stratum beneath proposed structures should be obtained with sufficient drilling and soil sampling

The existing fills on the site have been in place for over 50 years. Therefore, it is our opinion that severe down drag on the piles will not occur, if the thickness of any additional fill is limited to a maximum of 5 feet, and preferably less.

For preliminary design, uplift capacity of the piles should be taken as two thirds the compressive capacity of the piles. Pile capacities could also be further limited by pile head connection details and structural material strengths. In particular, the structural engineer should determine if the long pile lengths through the soft Bay Mud materials will cause any pile buckling.

In areas where the scuttled ship hulls apparently exist, as shown on Figure 2, pre-drilling may be necessary. It may also be impossible to penetrate the hull with drilling equipment. This eventuality must be assessed during any pile driving. It would be best to site any building well away from the mapped location of these hulls to avoid construction difficulties.

Indicator piles and possibly pile load tests should be considered to evaluate pile driving hammer performance, and to confirm production pile lengths, driving criteria, and capacity. Piles should have a minimum spacing of three times the pile width, center to center.

### 5.2.2 Shallow Foundations

Because of the variability in depth to the dense sandy layer possibly requiring some relatively long piles with limited allowable load capacity, a stiff structural concrete mat could be considered as an alternative to piles. Mat foundations should rest on at least 3 feet of adequately compacted granular structural fill and should be embedded at least 2 feet below the ground surface. The excavations should be cleared of debris and loose soils, prior to structural fill and concrete placement, and the excavations may require dewatering due to shallow groundwater.

For preliminary design, we recommend a modulus of subgrade reaction of 100 kips per cubic foot (kcf) for a 1-foot square bearing plate. The structural engineer should make appropriate correction for the size of the loaded mat area.

A stiff structural mat conforming to the above criteria can be designed using an allowable bearing pressure of 2,000 pounds per square foot (psf) for dead loads, 2,500 psf for dead plus sustained live load, and 3,300 psf for total loads, including wind and earthquake forces. These values are net allowable values, and footing weight can be ignored. Mat foundations will settle under the new applied loads due to consolidation of the underlying Bay Mud. The effect of settlement should be made for specific buildings and facilities during final design studies.

Small buildings of limited size with light structural loading could possibly be founded on shallow spread footings. Any spread footings should be supported on at least 3 feet of adequately compacted fill; should be founded at least 18 inches below the lowest adjacent grade; and should be a minimum of 18 inches in width.

Provided that the small, lightly loaded buildings are founded on spread footings as indicated above, they may be designed for an allowable bearing pressure of 1,500 psf for dead loads, 2,500 psf for dead plus sustained live loads, and 3,300 psf for total loads, including wind and earthquake forces. Because of the variability of the foundation conditions below the fill layer, differential settlement should be anticipated and some cracking within the structures should be anticipated. Therefore, no important structures should be considered for shallow, spread footings. Settlement estimates due to footing loads should be made for individual buildings during design phase studies. With very light loading, settlements of small lightly loaded footings on a compacted granular soil pad could be on the order of several inches, with differential settlement exceeding one inch between footings. These structural settlements are in addition to anticipated liquefaction

and consolidation settlements discussed above.

Resistance of shallow foundations to lateral loads can be provided by passive resistance acting on the vertical faces of the excavations and friction acting between the bottom of the footings and the supporting grade. We recommend using a preliminary equivalent fluid pressure of 350 pounds per cubic foot (pcf) for passive resistance and a friction coefficient of 0.35 for base friction. These values assume adequately compacted materials adjacent to and below the footings. For shallow footings not surrounded by concrete or pavement, the upper 18 inches of soil below finished grade should be ignored when determining passive resistance.

### 5.3 Seismic Design Parameters

We have developed site-specific earthquake design parameters based on the procedures described in Chapter 16, Section 1613 of the 2007 California Building Code (CBC). These procedures utilize State standardized spectral acceleration values for maximum considered earthquake ground motion, taking into account historical seismicity, available paleoseismic data, and activity rates along known traces, as well as site-specific soil and bedrock response characteristics.

Contour maps of Class B bedrock horizontal spectral acceleration values for the State of California are included as figures in Chapter 16 of the 2007 CBC, representing both short (0.2 second) and long (1.0 second) periods of spectral response and taking into account 5 percent of critical damping.

The USGS has developed a ground motion parameter procedure for building design under their Earthquake Hazards Program that allows for site-specific adjustments of acceleration values for different subsurface conditions, which are defined by site classes. Given a representative latitude of 37.79062 and longitude of -122.32083, in accordance with guidelines presented in the 2007 CBC, the following seismic design parameters will apply for the subject site:

- Site Class E - Soft Soil Profile
- SDs = 0.900
- SD1 = 0.960
- SMs = 1.350
- SM1 = 1.440

### 5.4 Ground Improvement

Seismic settlement and the limited lateral spreading due to liquefaction discussed previously could cause ground displacement at the site. Therefore, it may be appropriate to consider several ground improvement techniques to mitigate against displacement from potential liquefaction, at least below the more important structures and other facilities, where large ground movements cannot be accepted.

Dynamic compaction could be considered to improve the existing sandy fill materials beneath structures. Dynamic compaction methods generally consist of dropping a large weight, generally about 15 tons, from a crane, typically from heights of 70 to 100 feet. The weight is dropped several times per location to densify the soil. Preliminarily, we recommend that if dynamic compaction is considered, densification should achieve an increase in the standard penetration test (SPT) blow count of the loose to medium dense sands to a value of on the order of 25 to 30 (SPT blows per foot). An initial spacing should be on the order of 10 feet and may require from 5 to 10 drops per location. If dynamic compaction is pursued, a bridging layer of dense compacted granular fill with a minimum cover over the water table of 5 feet should be considered for stability of the equipment.

If the consolidation settlements indicated earlier present a problem, the installation of wick drains could be considered to accelerate the anticipated settlements.

### 5.5 Concrete Slab on Grade

Buildings supported on piles should incorporate structural slab floors that are also founded on piles. For other small, lightly loaded buildings, floors may be slabs-on-grade or structurally supported. For interior slab-on-grade floors and exterior concrete slabs, subgrades should be moisture conditioned to near optimum moisture content and compacted to at least 90 percent relative compaction, in accordance with ASTM D1557, latest edition. All building floor slabs should be underlain by a minimum of 24 inches of sandy fill and should be adequately reinforced to control cracking due to settlement of the Bay Mud.

Where floor slabs are to resist dampness caused by moisture vapor transmission through the concrete, an adequate moisture barrier consisting of 4 inches of compacted, free draining crushed rock overlain by a moisture membrane at least 10 mils thick is required. A 2-inch thick sand cover should be used to protect the membrane during construction.

### 5.6 Pavement

The upper 6 inches of subgrade in paved areas should be scarified, moisture conditioned and re-compacted to at least 95 percent of the laboratory maximum dry density as determined by ASTM D1557, latest edition in order to provide a smooth nonyielding surface. The subgrade should not be allowed to dry out and crack prior to any pavement construction.

We estimate that the R-value of the existing sandy fill on the site is approximately 40. Based upon this R value for preliminary design purposes, preliminary pavement design sections have been calculated using Caltrans methods for various Traffic Indices (TI), as summarized in Table 1. The pavement sections assume a design life of 20 years. We believe that a TI of about 5 is appropriate for pavements subject to occasional heavy vehicles, and TI's of 6 to 7 are appropriate for access roads and driveways subject to relatively heavy traffic. The aggregate base materials should conform to the criteria for Class 2 Aggregate Base in the current Caltrans Standard Specifications.

### 5.7 Utilities

Utility trench excavations should be performed in accordance with OSHA excavation and trench safety requirements. As indicated earlier, groundwater is anticipated to be relatively shallow, on the order of about 5 feet, but influenced by tides. We therefore recommend that underground utility trenches be kept as shallow as is feasible. Where deeper trenches are required, dewatering will probably be necessary.

Utility line bedding should consist of clean sand, which should extend from the bottom of the trench to 12 inches above the top of the pipe. The sand bedding should be placed in a dry trench and mechanically compacted to the satisfaction of the geotechnical engineer. Utility trenches that pass through the building pad should be plugged with non-permeable clay or lean concrete at the perimeter to prevent infiltration of water into the building pad.

Trench backfill should meet the criteria for fill materials indicated in Section 5.1. In general, trench backfill should be placed in uniform layers not exceeding 8 inches in thickness, moisture conditioned to near optimum moisture content, and compacted to 90 percent relative compaction, as determined by ASTM D1557, latest edition. Jetting of the backfill with water for compaction is not considered to be satisfactory.

### 5.8 Drainage

The minimum recommended surface drainage gradient is 2 percent. Steeper gradients may be required if no ground improvement is performed to compact the on-site fill materials. However, steeper gradients would require more new fill,

which would lead to additional settlements. In order to keep additional fills to a minimum, the use of pump stations for adequate drainage may be necessary.

Buildings should be provided with roof gutters, and roof downspouts should be connected to a closed-pipe system for draining the runoff away from the buildings.

### 5.9 Additional Work

This report was prepared using boring and CPT data and geologic mapping performed by various agencies and other consultants. AECOM did not perform any additional subsurface investigation for this preliminary assessment of geotechnical conditions for input to the project environmental assessment. It should therefore only be considered as preliminary in nature for planning purposes. A design-level geotechnical investigation should be performed at the appropriate time with on-site specific geotechnical borings and laboratory testing to form the basis for design-level geotechnical recommendations.

The recommendations of this report should be revised as necessary during design-level studies, when site specific geotechnical information has been secured.

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**TABLE 1**

**PRELIMINARY PAVEMENT DESIGN**
**VA FACILITIES PROPERTY**
**FORMER ALAMEDA NAVAL AIR STATION**

| Traffic Index | Asphaltic Concrete (in) | Class 2 Aggregate Base (in) |
|---|---|---|
| 5 | 3.0 | 4.0 |
| 6 | 3.5 | 5.5 |
| 7 | 4.0 | 7.0 |

# APPENDIX A

# BACKGROUND INFORMATION USED

L:\work\112362\cad\04 Alameda Point\Figure 1.dwg May 12, 2009

**Preferred Site Concept**

- Building
- Trail
- Landscape/Hardscape
- Existing Bunker
- Parking

0 0.25 0.5 Miles

SOURCE: FROM EDAW | AECOM, PROVIDED ON 3/20/09

AECOM

**FIGURE 1**

**SITE MAP**

VA FACILITIES AT FORMER NAS ALAMEDA
ALAMEDA POINT, CALIFORNIA

MAY 2009

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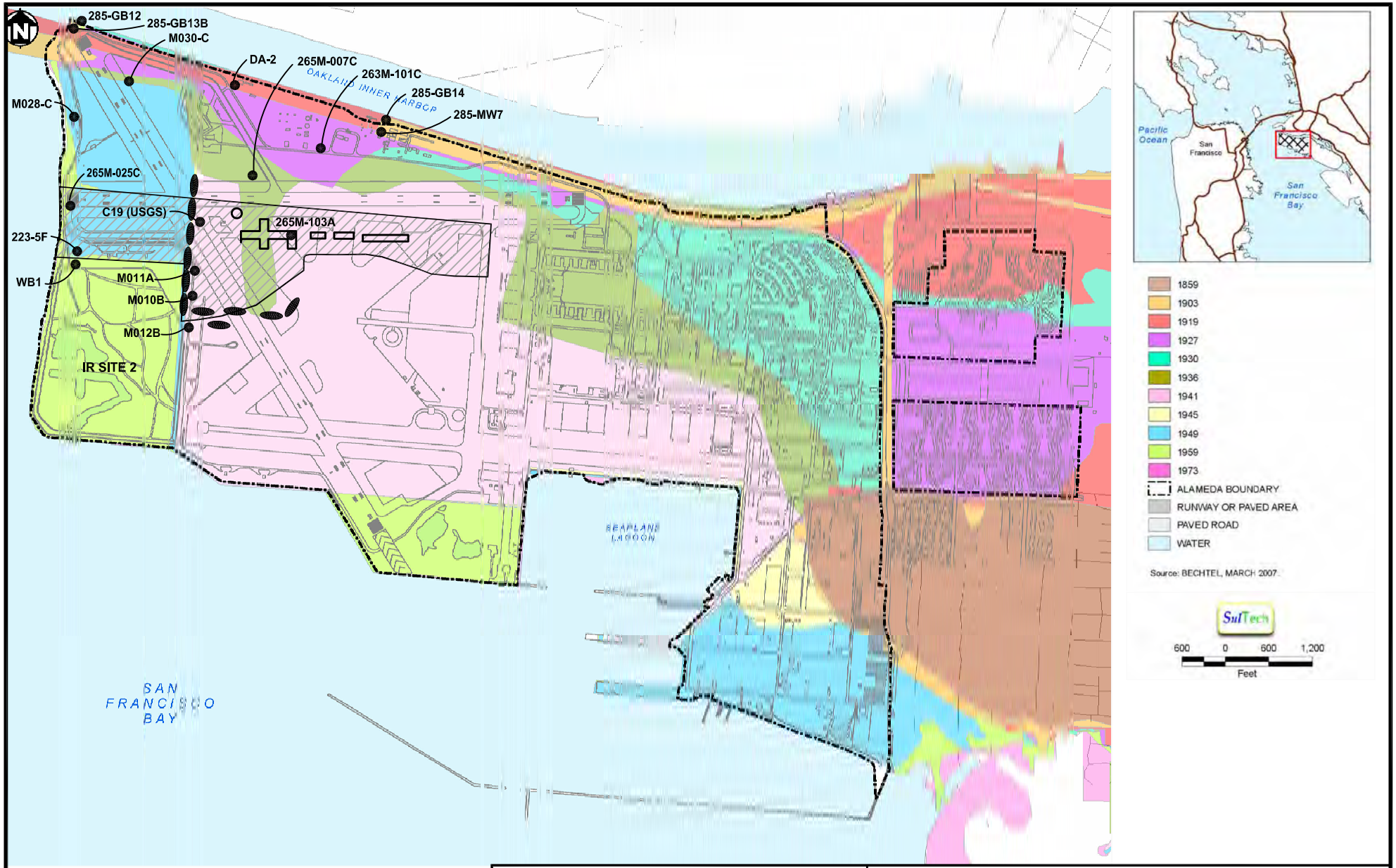
L:\work\112362\cad\04 Alameda Point\Figure 2.dwg May 12, 2009




**LEGEND**

- PROPOSED VA FACILITIES AREA
- PREVIOUS BORING, WELL, OR CPT (HARDING ESE, 2001, BATTELLE, 2008)
- SCUTTLED WARSHIP HULL (HARDING ESE, 2001)

SOURCE: BASE FROM EDAW | AECOM, PROVIDED ON 4/3/09

AECOM

**FIGURE 2**

**FILL HISTORY**

VA FACILITIES AT FORMER NAS ALAMEDA
ALAMEDA POINT, CALIFORNIA

MAY 2009

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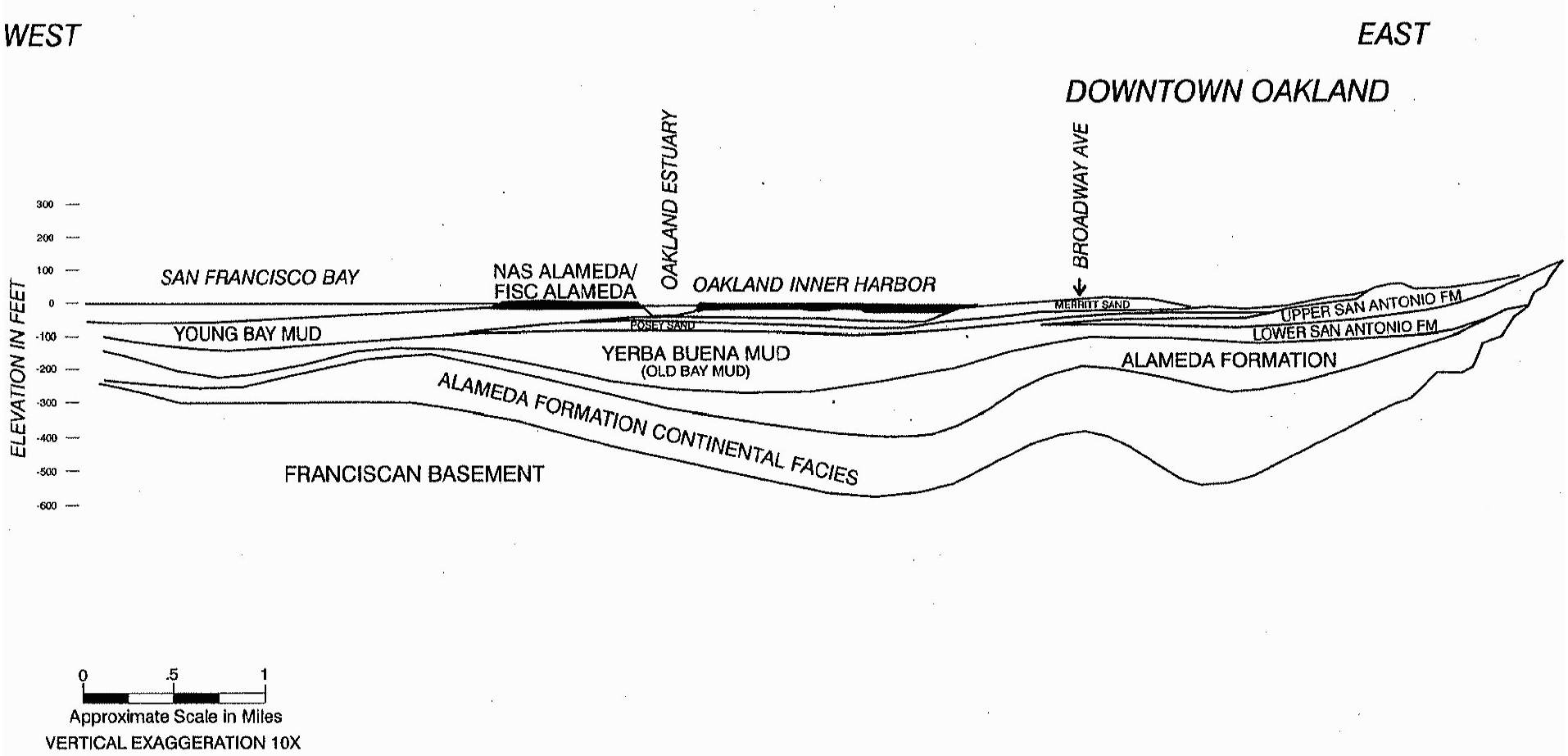


NAS Alameda and FISC Alameda are underlain by Bay Mud sediments deposited on a west-thinning layer of Merritt-Posey sand.

Source: Carlisle and Rollins 1994;
modified from Rogers and Figuers 1991.

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**FIGURE 3**

**GENERAL GEOLOGIC SECTION**

VA FACILITIES AT FORMER NAS ALAMEDA
ALAMEDA POINT, CALIFORNIA

MAY 2009

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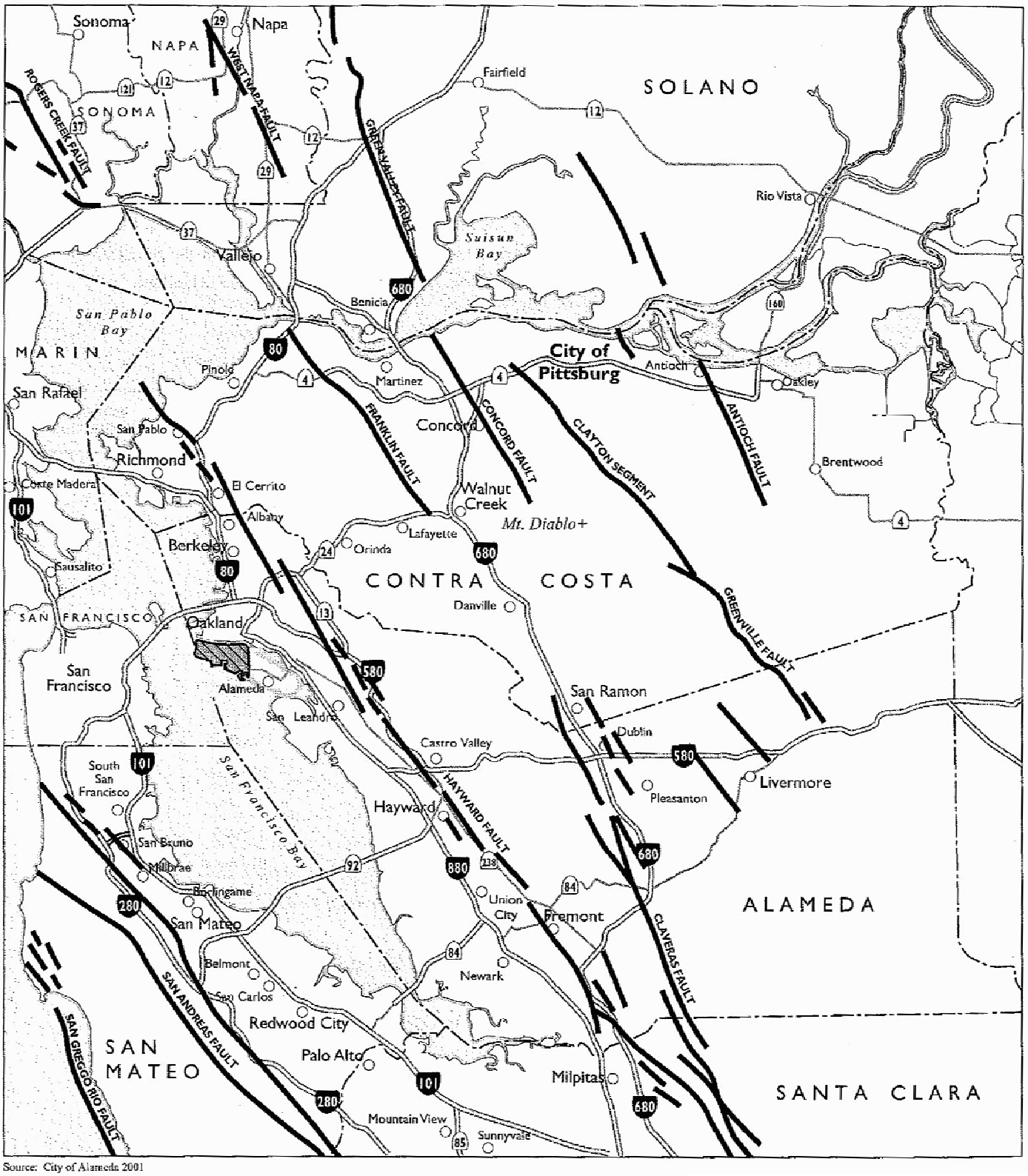


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**FIGURE 4**

**FAULT MAP**

VA FACILITIES AT FORMER NAS ALAMEDA
ALAMEDA POINT, CALIFORNIA

MAY 2009

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