Chapter 4. Geology and Soils

Affected Environment

Data Sources

This chapter is based on previous investigations and studies performed by others within the HAAF and neighboring areas. Primary sources of information are the following:

- draft Hamilton Wetlands Conceptual Restoration Plan (Woodward-Clyde 1998);
- Hamilton Army Airfield Reuse Plan Existing Conditions Analysis prepared by Robert Bein, William Frost & Associates (Robert Bein, William Frost & Associates 1995); and
- a comprehensive summary of existing geologic conditions prepared by Environmental Science Associates for the BMKV property (Environmental Science Associates 1993).

Regional Geology and Topography

The project site is located within California's geologically and seismically active Coast Ranges Geomorphic Province. The province is characterized by a series of northwest-trending faults, mountain ranges, and valleys (Figure 4-1) (Environmental Science Associates 1993).

Two distinct geomorphic zones, the Bay Plain and Franciscan Uplands zones, occupy the project site. The Bay Plain extends from the edge of San Pablo Bay to the foot of the hills immediately west of the HAAF parcel. Adjacent to San Pablo Bay, the nearly level site consists of former mudflats and marshlands that have been separated from tidal action by dikes and levees since the early 1900s; the site is drained by a system of trenches and pumps (Robert Bein, William Frost & Associates 1995). As the site dried out and the soil became desiccated after being removed from tidal inundation, it began to settle below its original elevation. Current ground elevations at the site range from +7 to -7 feet NGVD, with a typical ground elevation of -5 feet. (Woodward-Clyde 1998.)

The water table is typically several feet below the surface and varies somewhat seasonally. As shown in Figure 4-2, below a thin near-surface crust, the area is underlain by soft marine clays known as bay mud to depths that vary from 70 feet near San Pablo Bay to 30 feet or less at the northwestern end of the site. The crust is composed of desiccated bay mud throughout the area; in many locations, especially in the HAAF area, the crust also consists of several feet of granular fill and, in the former runway and taxiway areas, pavement (Figure 4-2).

The project site is located on soils of one primary geomorphic types (Figure 4-3):

- **Bay Mud—**The bay mud consists of thick deposits of soft, unconsolidated, water-saturated, silty clays containing vegetative remains and is up to 70 feet thick. This soil type exhibits high compressibility, low shear strength, and generally low permeability and is underlain by much stronger and less compressible soils. The HAAF runway, hangars, and main administrative buildings are situated on the Bay Plain and underlain by bay mud that extends to the historical limits of the marshland of San Pablo Bay. Before dikes were installed in the early 1900s to allow agricultural use of the land, the bay muds were inundated regularly by high tides. Artificial fill (consisting of rock, soil, and other materials) was deposited on top of the bay mud to permit construction of the runway. Artificial fill (containing rock, soil, and other materials) was deposited on top of the bay mud to permit construction of the runway.

Adjacent to the project site are several other geomorphic types:

- **Franciscan Formation—**The hills west of the HAAF parcel are formed of sandstone and shale of the Franciscan Formation, which weather to form a light sandy or silty soil that is moderately well drained. A small portion of the upland area consists of Franciscan Melange, a mixture of rock fragments of variable size in a highly-sheared clay that weathers to a hummocky topography of clay-rich, swelling soils. The upland portions of the HAAF facility are underlain by rocks of the Franciscan Formation, known locally as Hamilton Field arkose, with sandstone and shale in higher elevation areas.

- **Colluvium—**At the base of the slopes are deposits of colluvium, which consist of unsorted, unconsolidated, clay-rich soil and rock fragments. Downslope movements of weathered bedrock and melange materials have resulted in the accumulation of these colluvial deposits at the base of slopes.

- **Alluvium—**Unconsolidated clay, silt, sand, and gravel that have been deposited by streams make up the alluvial material in the northern and eastern lowland areas. Deposition by the local streams has created accumulations of clay, silt, sand, and gravel in the west-central portion of HAAF. (Robert Bein, William Frost & Associates 1995.)

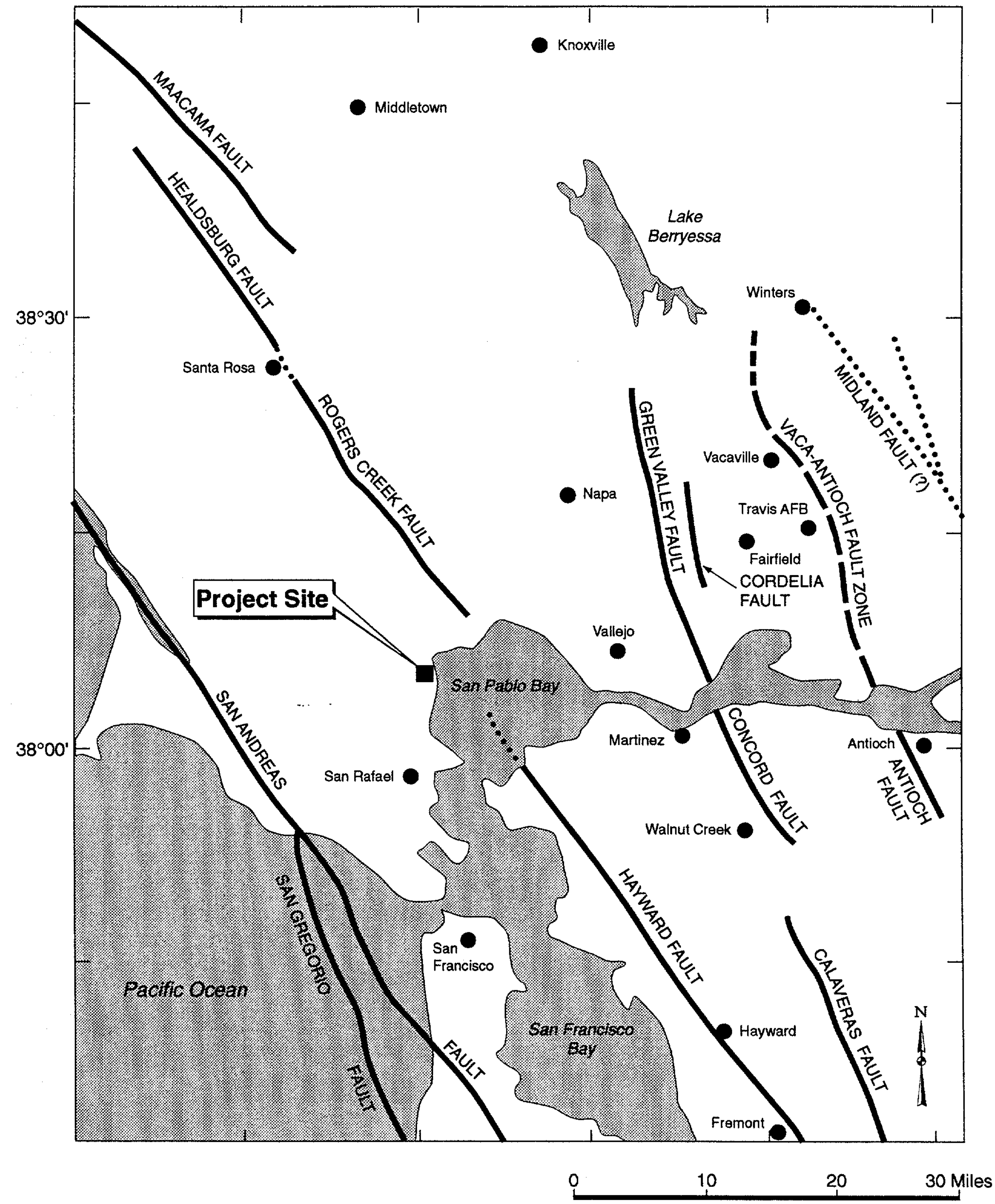

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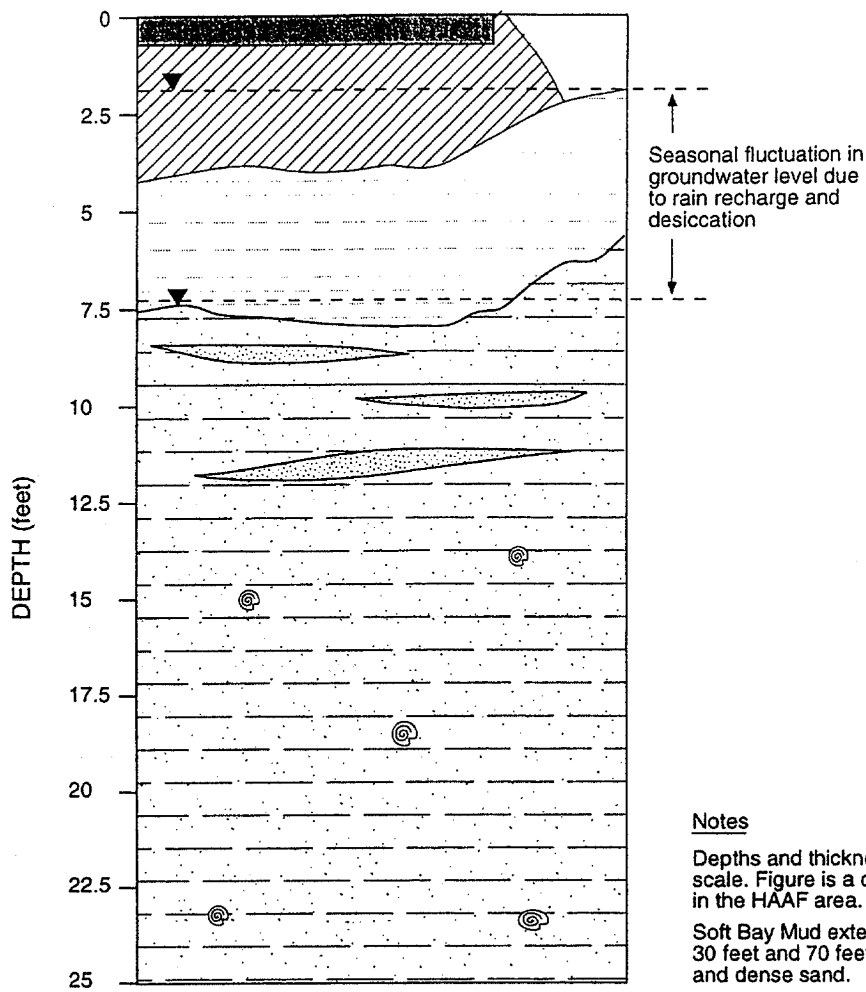
Figure 4-1
Regional Faults



Notes

Depths and thicknesses approximate, no horizontal scale. Figure is a compilation of field and boring logs in the HAAF area.

Soft Bay Mud extends to depth varying between 30 feet and 70 feet, and is underlain by stiff clay and dense sand.

LEGEND

Pavement
concrete or asphalt

Fill
Yellowish-brown (10YR 5/4) to greenish grey (10Y 5/1) gravelly sand to reworked Bay Mud

Desiccated Bay Mud
Silty clay, greenish grey (10Y 5/1) to greyish brown (10YR 3/2), strong iron oxide staining on numerous desiccation cracks

Soft Bay Mud
Silty clay, greenish grey (10Y 5/1) to dark grey (2.5Y 4/1), soft, saturated, shell fragments scattered throughout, rich in organic matter (decayed plant fragments, peat)

Sand lenses
Discontinuous lenses, 1-inch to 3-feet thick, fine to coarse grained, dark greenish grey (10G 3/1) to brown (7.5YR 4/3), clayey, generally found along the hill range

Shells and shell fragments

Source: Woodward-Clyde 1998.

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Figure 4-2
Schematic Stratigraphic Column in Shallow Soil on Hamilton Army Airfield Runway

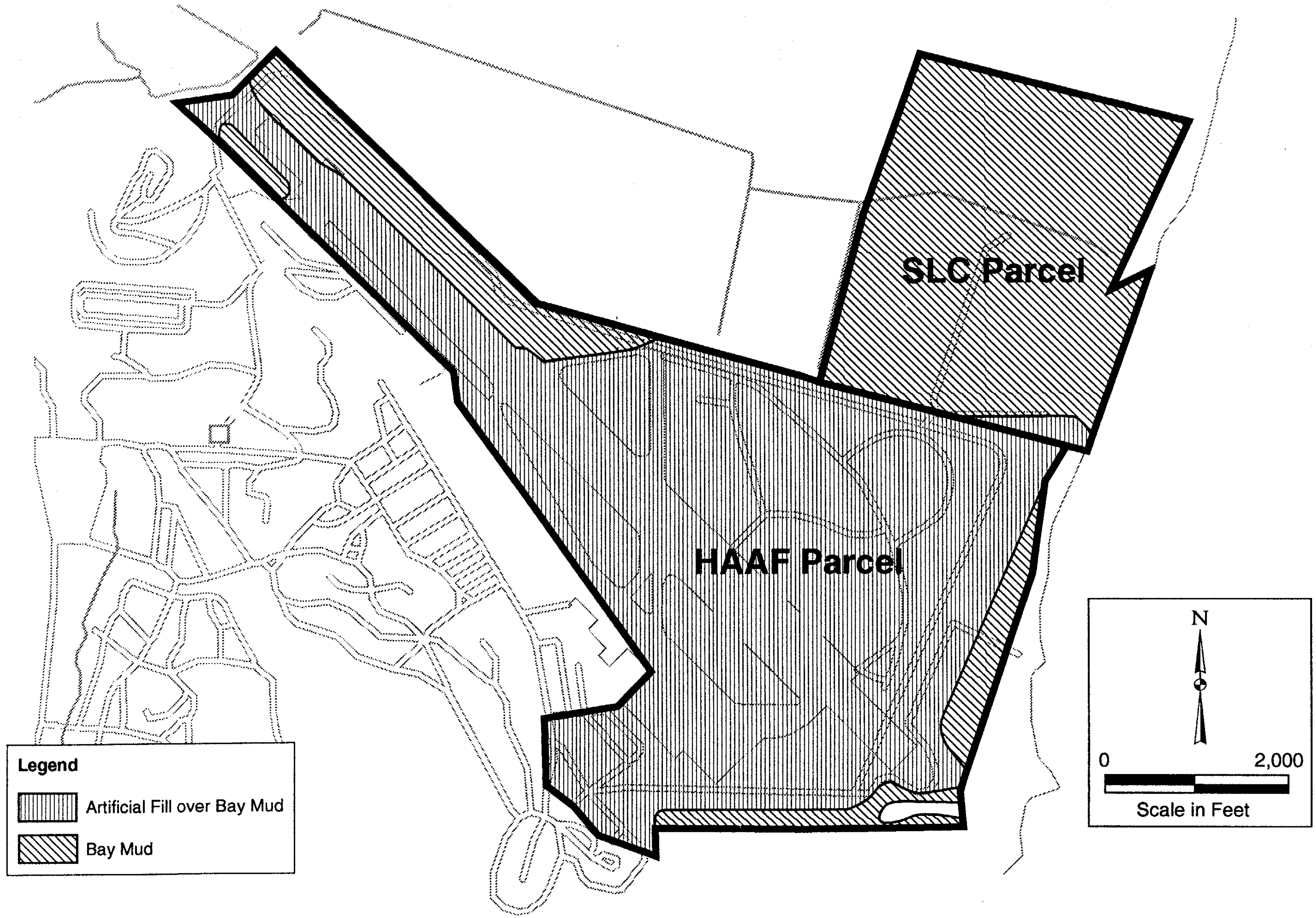

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Figure 4-3
Geology and Soils in the Project Area

Soils

Soils on hills and within existing wetlands on the project site consist primarily of naturally occurring clays, clay loams, and gravelly sandy loams. On the lower, developed portions of the HAAF area, natural soils have been extensively disturbed by grading, fill placement, and construction of buildings and paved areas. Three soil types are present: Saurin Urban Land Bonnydoon, Xerorthents-Urban Land, and Xerorthents. The Saurin series is a clay loam over sandstone bedrock, the Bonnydoon soil is a gravelly loam, and the Xerorthents type is used to describe the highly variable, disturbed urban flatlands. Surrounding areas contain Cortina gravelly sandy loam (industrial park area to the north) and Reyes clay (St. Vincent's Silveira Landholdings to the south). The native Novato soil series is now present in the HAAF area only in the salt marsh east of the levee. (Robert Bein, William Frost & Associates 1995.)

In addition to the three naturally occurring soil types, local upland soil material has been placed as fill ranging in depth from several inches to several feet. This fill has been compacted over extensive areas of Reyes soil, under the roadways and parking pads, and as berms extending into vegetated areas. The fill material is variable but is commonly a reddish-brown, very gravelly, sandy clay loam, which is typical of subsoil material from any of the four major upland soil series in the area. (U.S. Army Corps of Engineers 1996a.)

Seismicity and Geologic Hazards

The project site is located in one of the most seismically active regions in the United States. The site's seismic setting is dominated by the Hayward fault to the southeast, the San Andreas fault to the west, and the Healdsburg-Rogers Creek fault to the northeast (Figure 4-2). The maximum credible earthquake for each of these faults, measured in Richter scale magnitude (M), are as follows:

- the Hayward fault—7.5 M,
- San Andreas fault—8.3 M, and
- Healdsburg-Rogers Creek fault—7.2 M.

Two smaller, potentially active faults are near the project site. A possible trace of the Burdell Mountain fault is mapped as extending toward and terminating about 4,000 feet north and west of the project site. Estimates differ regarding the date of the last displacement on the Burdell Mountain fault. It is generally thought to have been active during the Quaternary period (the last 2.5 million years), and some evidence suggests that it may have been active during the Holocene epoch (the last 11,000 years). (Environmental Science Associates 1993.) The Tolay Fault also reaches to within 6.5 miles of the project site and may be active (Robert Bein, William Frost & Associates 1995).

The project area is likely to undergo ground shaking from a major earthquake. The U.S. Geological Survey has estimated that there is a 67% probability that there will be one or more earthquakes of magnitude 7.0 or greater in the Bay Area in the next 30 years. (Environmental Science Associates 1993.)

Four major hazards are associated with earthquakes. These are surface fault rupture, ground shaking, ground failure, and inundation resulting from earthquake-generated waves or dam failures. (Environmental Science Associates 1993.)

Ground Shaking

Three major factors affect the severity (intensity) of ground shaking at a site in an earthquake: the magnitude of the earthquake; the distance to the fault that generated the earthquake; and the geologic materials that underlie the affected site. Thick, loose soils, such as bay mud, tend to amplify and prolong groundshaking vibration. Because the project site is underlain by bay mud, ground shaking would be more intense at the site than in nearby areas underlain by bedrock. (Environmental Science Associates 1993.)

Surface Fault Rupture

Because no active or potentially active faults are known to cross the project site, the potential for surface fault rupture at the site is remote. (Environmental Science Associates 1993.) In addition, the project site is not within an Alquist-Priolo Special Studies Zone as designated by the state.

Ground Failure

Ground failure hazards of potential concern at the site include liquefaction, earthquake-induced settlement, and lurching. All of these involve displacement of the ground surface resulting from a loss of strength or failure of the underlying materials because of ground shaking.

Liquefaction is the sudden loss of strength in loose, saturated materials (predominantly sands) during an earthquake, which results in temporary fluid-like behavior of those materials (much like quicksand). Liquefaction typically occurs in areas where groundwater is shallow and materials consist of clean, poorly consolidated, fine sands. Subsurface conditions at the project site are not conducive to liquefaction because bay mud does not contain substantial amounts of granular materials.

Ground shaking can also induce settlement of loose, granular soils above the water table. Subsurface conditions at the site consist of clays and silts rather than sands and, thus, are not conducive to earthquake-induced settlement.

Lurching, or lurch cracking, is the cracking of the ground surface in soft, saturated material as a result of earthquake-induced ground shaking. The bay mud that underlies the project site is susceptible to lurching, particularly where deposits are bordered by steep channel banks or adjacent hard ground. (Environmental Science Associates 1993.)

Earthquake-Induced Inundation

Earthquakes could result in the inundation of the project site as a result of tsunamis (or tidal waves), and seismic seiches (oscillating waves in enclosed water bodies).

Tsunamis are sea waves produced by large-scale seismic disturbances of the ocean floor. Tsunamis can be generated by local offshore seismic events, as well as by events thousands of miles away. A tsunami with a 100-year recurrence interval (i.e., a 1% probability of occurrence in a given year) has an estimated runup of 3.7 feet in the vicinity of the project site (i.e., the resulting waves would wash 3.7 feet up on the levee banks in the project area). At its current elevations, the project site could be flooded by a tsunami in the event of levee failure or overtopping. (Environmental Science Associates 1993.)

Seismic seiches may be generated in tidal marsh ponds such as those currently present along the outboard tidal marsh.

Environmental Consequences and Mitigation Measures

Approach and Methods

The approach and methods used to evaluate project impacts on geology and soils consisted of reviewing available soils and geologic data for the site and updating the environmental baseline for these issues.

Impact Mechanisms

The potential for the project to have various geotechnical and geological impacts was considered; these impacts included the potential for personal injury; loss of life; and property damage to structures, utilities, or levees caused by existing geologic hazards such as:

- strong seismic ground shaking,
- liquefaction,
- seismically induced settlement, and
- site settlement under the proposed marsh restoration plan.

One of the special geologic issues associated with this project is the compressibility of the bay mud. Primarily because of its high compressibility and low strength, the soft bay mud poses considerable constraints to development of the perimeter levees, which are critical features of the restoration plan. New fill loads placed on top of areas underlain by bay mud could cause compression of the bay mud, leading to the need for more fill to be placed and causing uneven settlement of the ground surface. Depending on the depth of the bay mud, settlement may take 10–50 years to be fully apparent. Uneven settlement is desirable, however, for the evolution of the wetland topography.

In addition, fill applied over limited areas, such as levee fill, can cause shear stresses in the bay mud. If these stresses exceed the soil's shear strength, stability failure may result. Therefore, new levees should be designed to provide adequate geometric stability, which may require the use of stabilizing berms. (Woodward-Clyde 1998.)

Thresholds of Significance

According to professional criteria and judgment and applicable regulations and plans, the project would result in a significant impact if:

- the potential exists for personal injury, loss of life, or property damage to proposed structures, utilities, or levees caused by existing geological hazards;
- foundation elements, roadways, or other infrastructure elements would be degraded by chemical action or mechanical weathering of onsite soils;
- secondary effects of seismic ground motion could result in damage to proposed site improvement;
- a geologic condition (such as increased liquefaction potential) is created or allowed to persist that could cause substantial structural damage onsite or offsite;
- a substantial change in topography or destruction of any unique soil type would occur; or
- substantial degradation of physical, chemical, or biological soil quality would occur that degrades or destroys the function of the soil to support sensitive habitats, such as wetlands.

Impacts and Mitigation Measures of Alternative 1: No Action

Impact 4.1: Continuation of Existing Levee Maintenance, Pumping, and Subsidence

Under Alternative 1, the HAAF parcel would not be transferred and the Army would retain ownership. The Army would be responsible for continuing maintenance and operation of the drainage and flood control facilities. It is assumed that the Army will maintain the existing level of flood protection at HAAF. This would include monitoring of pumping facilities, drainage ditches, and levees.

Impacts and Mitigation Measures Common to Alternatives 2, 3, 4, and 5

Impact 4.2: Settlement of Soils from Fill Loads for Levees and Sedimentation

The bay mud and Bay Plain soils would be subject to settlement under Alternatives 2-5 and the BMKV Scenario from the loads imposed by the levees and natural sedimentation following levee breach. Differential settlement could lower site elevations below desired grades and damage levees unless the effects of settlement are adequately considered in project design and construction. In addition, an anticipated 0.5-foot sea-level rise could damage levees over the duration of their design lives.

New fill would be placed in the project area to raise the elevation of levees. The weight of the fill would compress the underlying bay mud, leading to possibly uneven settlement of the fill. The main settlement process would occur in the first 30-50 years after fill placement; settlement would slow appreciably after that time.

Settlement rates and amounts have been estimated for various thicknesses of bay mud and various heights of fill because these factors are mutually dependent. That is, for a given thickness of bay mud, settlement would increase in direct proportion to an increase in fill height. Conversely, for a given thickness of fill, the amount of ultimate settlement would increase with the depth of the underlying bay mud. For example, given a 20-foot-thick layer of fill, the ultimate settlement for a 20-foot-thick deposit of bay mud would be 5 feet, whereas for a 70-foot-thick deposit of bay mud, the ultimate settlement of the same 20-foot-thick fill layer would be 13 feet.

As the thickness of the bay mud increases, however, the rate of settlement decreases. In the example above, 9.5 years would be required to achieve 50% of the ultimate settlement with a 20-foot-thick deposit of bay mud, whereas more than 150 years would be required to reach 50% of the ultimate settlement with a 70-foot-thick deposit of bay mud. Using the same example, after 10 years, about 2.5 feet of settlement would occur over a 20-foot deposit of bay mud, whereas about 1.5 feet of settlement would occur over a 70-foot-thick deposit of bay mud.

Settlement can be either uniform or differential. Uniform settlement results in equal amounts of settlement over the area of concern. Differential settlement occurs when some areas settle more than others. Uniform settlement could cause problems if an entire levee or site grade settled sufficiently to allow flooding. Differential settlement could affect development if adjacent areas underwent different degrees of settlement and, as a result, improvements spanning both areas were damaged structurally.

Differential settlement could result from variations in the thickness and compressibility of bay mud and variations in the thickness of fill placed. The potential for differential settlement would be highest where the thicknesses of bay mud and fill change within relatively short horizontal distances. This would be especially true where fill would be placed over existing ditches, levees, or embankments.

To achieve a long-term (50-year) levee crest elevation of +8 feet NGVD, the conceptual plan calls for the levee to be constructed to an elevation of +12 feet initially, to accommodate an estimated 4 feet of long-term settlement. Moderate adjustments can be made to levee crest height if the levee is ultimately observed to settle more than 4 feet. The estimate of 4 feet of settlement also includes a 0.5-foot allowance for sea-level rise.

A design-level subsurface geotechnical investigation will be conducted by a qualified geologist and a comprehensive, detailed geotechnical design will be prepared for the project levee and fill placement plan (see Chapter 3, "Project Alternatives under Consideration"). This design-level investigation will address the levee and fill program with respect to site settlement, stability of slopes, soil constraints, and potential for earthquake-induced ground failure (lurching). The recommendations presented in the Preliminary Conceptual Design Report—Soil Encapsulation Berms prepared by IT Corporation (1997) will be incorporated into the comprehensive geotechnical design and fill placement plan. A comprehensive monitoring and inspection program of settlement and its effects will also be implemented.

Specifically, the subsurface investigation and design will, at a minimum, identify subsurface conditions encountered (e.g., thickness, depth, and compressibility of bay mud; presence of other underlying soil layers or sand or peat lenses) and describe how differential settlement on construction sites throughout the project site will be avoided and/or compensated for using standard engineering techniques. The specific techniques would be selected during the design phase for the project and could include, but need not be limited to:

- placing additional fill to compensate for anticipated settlement and sea-level rise, such as initial construction of levees 4 feet above long-term levee crest elevation to accommodate long-term settlement and sea-level rise;
- application of surcharge loads or other settlement acceleration techniques, such as installation of wick drains; and
- uniform placement of fill during construction and avoidance of excessive fill placement.

Because the conceptual plan addresses and the final design will address this issue and settlement will not cause adverse effects (i.e., levee failure), this impact is considered less than significant.

Impact 4.3: Potential Levee Slope Failure Resulting from Low Strength of Underlying Bay Mud

Under Alternatives 2-5, levees would be constructed on bay mud, which is structurally weak. Slope stability failures could occur, either under static conditions or during seismic shaking, if the levees are not designed and constructed appropriately. Slope stability would be particularly critical when the outboard levee is breached and the area is inundated, providing additional external force on levees. Factors influencing slope stability include strength of natural soils and fills, embankment heights and slopes, and depth of inundation. The severity of seismic shaking, in conjunction with the above factors, also affects slope stability.

Stability of levees, however, would increase over time with consolidation and settlement of material placed within the levees. The current plan proposes a long-term (50-year) levee crest of +8 feet by constructing levees to an initial elevation of +12 feet with 3:1 (horizontal to vertical) or flatter slopes and toe berms on both sides that average 6 feet high and 50 feet wide (for a required minimum stability safety factor of 1.3). This levee has a 200-foot-wide footprint. Over time, as the levee settles and the underlying bay mud consolidates and gains strength, the factor of safety would increase to well in excess of 1.5. ~~The levee is also expected to survive the maximum credible earthquake for the project area.~~ (Woodward-Clyde 1998.) A description of the guidelines that the Corps will use for design of the levees is included in Chapter 3, "Project Alternatives under Consideration".

To ensure the stability of levee slopes, a geotechnical investigation will be conducted and appropriate engineering design of levee slopes and stability, effects of placing fill against them, and the potential need for stabilizing berms will be determined. ~~Therefore, this~~ This impact is considered less than significant because this investigation and subsequent design will minimize the potential for slope failure.

Impact 4.4: Potential Seepage through or under the Levee from Materials Placed on the Bay Side of the Levee

Because water would be introduced on the bay side of the flood control levee, seepage through or under the levee could occur. This seepage may affect adjacent properties.

The conceptual plan calls for the levee to be constructed using fine-grained materials to reduce the potential for through-levee seepage. Existing granular near-surface fill from below the main body of the levee (but not below the toe berms) should be excavated, and a keyway (a trench filled with new levee fill) about 20 feet wide should be constructed through the natural clay crust. (Woodward-Clyde 1998.) Therefore, this impact is less than significant.

Impact 4.5: Potential Exposure of Sensitive Wetlands and Levees to Seismic Hazards

Critical project structures, such as cell and perimeter levees and holding pond levees, could fail or be damaged during an earthquake, releasing contaminants to the environment and delaying marsh restoration. Because no known active faults cross the project site, however, the potential for surface fault rupture at the site is remote.

The project site is likely to undergo ground shaking from a major earthquake in the Bay Area within its 50-year lifespan. Because the project site is underlain by bay mud, ground shaking would be more intense at the site than in nearby areas underlain by bedrock. Seismically induced ground shaking could damage proposed embankments, cut slopes, and levees.

The potential for liquefaction and seismically induced settlement at the site is relatively low; however, the site would be susceptible to lurching from ground shaking. Lurching could affect levee stability and would have to be considered in the design of these improvements.

A tsunami with a 100-year recurrence interval has an estimated runup of 3.7 feet in the vicinity of the project site. At current elevations, the project site could be flooded by a tsunami that resulted in levee failure or overtopping.

Critical project structures, such as levees, will be designed to the engineering standard of practice given their use, such as those recommended by the Corps. Records of the design and reconstruction of the distressed section of the levee and maintenance records will be used to develop design and maintenance criteria for project levees.

Settlement monitoring points and slope inclinometers will be initially placed and then read following an earthquake to evaluate deformation that may not be discernible by visual observation. Because the project site could be flooded by a 100-year tsunami, levee design will accommodate possible overtopping.

Because the final design will address seismic issues, this impact is considered less than significant.

lmpacts and Mitigation Measures Unique to Alternative 2

No impacts and mitigation measures are unique to Alternative 2.

lmpacts and Mitigation Measures Unique to Alternative 3

Impact 4.6: Settlement of Soils from Fill Loads for Levees, Sedimentation, and Dredged Material

This impact would be the same as Impact 4.2, although the addition of dredge materials to accelerate the creation of wetlands would also increase the rate of settlement (Geomatrix Consultants 1998) from the surcharge load. The conceptual plan calls for placing dredged material against the outboard slope of the New Hamilton Partnership levee. The fill may be placed up to the full height of the levee (elevation 8 feet) and slope gently away from the levee at a 2% grade. The dredged material is expected to be sandy soil rather than soft clay (bay mud).

The existing New Hamilton Partnership flood control levee was completed in November 1996. The levee was designed to retain water from San Pablo Bay when the existing dike system is breached. Design of the levee included an evaluation of settlement and slope stability. Placement of approximately 10 feet of dredged soil against the levee would affect settlement and stability of the levee (discussed below).

To accommodate settlement and provide adequate slope stability during a strong seismic event, New Hamilton Partnership plans to raise the level of the levee after it is allowed to settle for 3-5 years. The levee crest was constructed at elevation 8 feet. When the levee crest settles to elevation 6.5 feet (3-5 years after construction), the levee will be raised to elevation 8.5 feet. Analyses (Geomatrix Consultants 1998) indicate that another 80-100 years will be required for the levee crest to subside to elevation 6.5 feet a second time.

The weight and lateral extent of dredged soil placed against the levee will increase settlement of the levee. Analysis indicates that dredged soil brought to elevation 8 feet within 3-5 years after construction of the levee would increase settlement of the levee crest from 0.5 foot to 1 foot over 50-

100 years. Although the increased settlement of the levee is not considered to be excessive, the settlement would require raising the levee an additional time to keep the crest at or above elevation 6.5 feet. In this regard, there would be additional cost for levee maintenance over the present schedule to keep the levee crest at or above elevation 6.5 feet.

In its analysis, Geomatrix Consultants also calculated settlement, with and without dredged soil against the levee, at a point 20 feet beyond the inboard toe of the levee. This point was selected as representing the rear wall of future residential structures. The placement of dredged soil against the levee was found to also cause increased settlement 20 feet beyond the inboard toe of the levee. The increase in settlement is dependent on the height of fill placed against the levee. If 10 feet of fill is placed against the levee, the increase in settlement 20 feet from the inboard toe of the slope is estimated to be 1-2 inches over 50 years. If the thickness of fill is reduced to 6 feet, the increase in settlement is estimated to be about 0.5 inch.

To aid in evaluating how reducing the height of dredged material or moving the dredged material away from the levee would affect settlement of the levee, Geomatrix Consultants evaluated both of these conditions (Figure 4-4). Condition A assumes that new dredged material would be placed directly against the levee. The analysis was performed for fill heights of 6, 8, and 10 feet. Condition B assumes that 10 feet of new fill would be placed but that the edge of the dredged material would be moved away from the top of the levee. The analysis was performed for distances of 20, 40, and 60 feet between the levee and dredged material (distance X in Figure 4-4). The slope of the dredged material was assumed to be 3:1 (horizonal to vertical).

The results of the settlement analysis for the conditions A and B are summarized in Table 4-1. The results of the analysis indicate that reducing the height of fill placed directly against the levee would be beneficial in terms of settlement. Also, moving the fill away from the levee would reduce settlement.

Because placing dredged material against the New Hamilton Partnership levee would cause settlement of this levee and possibly of adjacent properties and this issue is not addressed or anticipated sufficiently in the conceptual plan, this impact is considered significant.

Mitigation Measure 4.6: Limit the Height of Dredged Material to 4 Feet. To reduce settlement at residential sites to acceptably small values, the surface of the dredged material placed against the levee should not extend above elevation 4 feet. This will limit the thickness of fill to about 6 feet. To bring the dredged material surface to elevation 8 feet (approximately 10 feet of fill), the fill should be moved at least 60 feet from the top of the levee (distance X in Figure 4-4). The area between the levee and dredged material would need to be sloped to an outlet to prevent water from ponding. The height of fill placed to provide drainage between the levee and dredged material should not exceed elevation 2 feet. The actual limits on the amounts of dredged material should be determined during the final design phase of the project.

In areas where structures are not located adjacent to the levee, the dredged material could be placed against the levee.

Impact 4.7: Potential for Levee Failure Resulting from Low Strength of Underlying Bay Mud

With regard to stability of the levee, the dredged soil eliminates the outboard slope and any stability considerations for the slope. Also, considerations of seepage through and beneath the levee and wave erosion of the outboard slope are eliminated. The only stability consideration is the inboard slope. Stability analyses undertaken during design of the levee (Geomatrix Consultants 1998) indicated that the factor of safety against slope failure decreased about 10% when the crest was increased from 20 feet to 40 feet in width. Although no stability analysis was undertaken for the dredged soil condition, increasing the crest width beyond 40 feet (which would be the situation if dredged soil is placed directly against the levee) may decrease the factor of safety against slope stability by not more than an additional 10%. A 10-20% reduction in the factor of safety is not a concern except for the seismic loading condition.

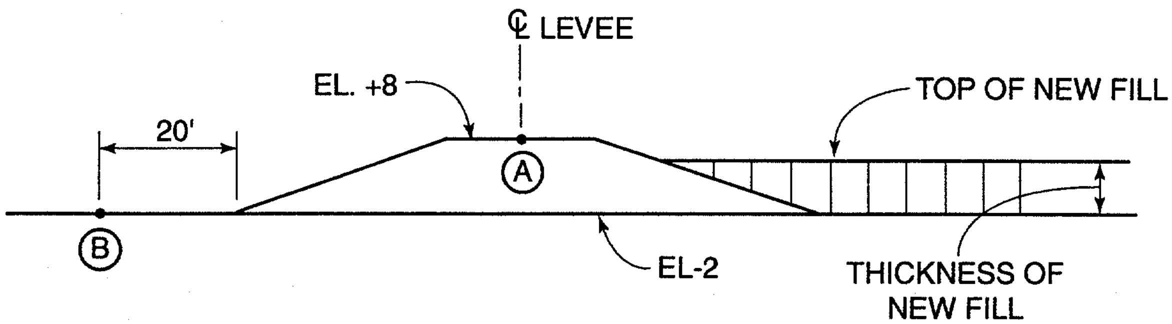
Because a thorough evaluation of the seismic stability of the inboard slope will be undertaken in conjunction with the design-level geotechnical investigation and recommendations implemented (see Impacts 4.3 and 4.5), this impact is considered less than significant.

lmpacts and Mitigation Measures Unique to Alternative 4

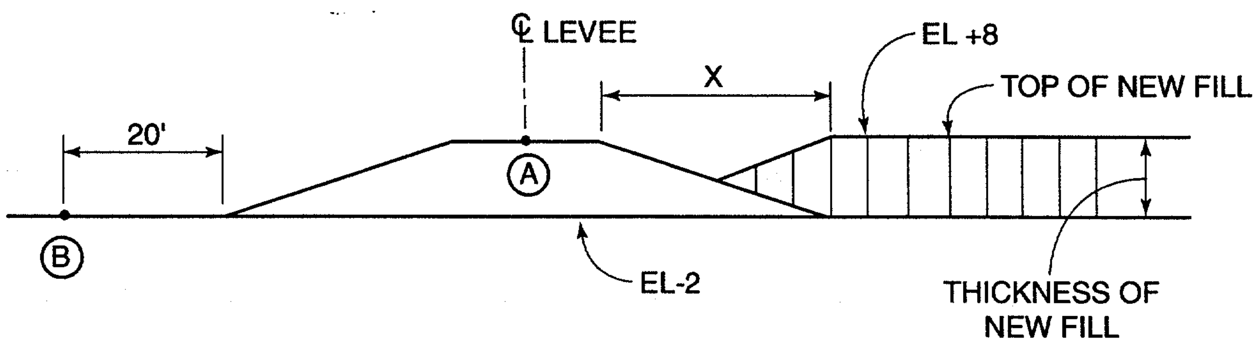
No impacts are unique to Alternative 4.

lmpacts and Mitigation Measures Unique to Alternative 5

Impacts and mitigation measures for this alternative are the same as those described for Alternative 3.



CONDITION A - FILL PLACED AGAINST THE LEVEE



CONDITION B - FILL PLACED AWAY FROM THE LEVEE

Source: Geomatrix Consultants 1998.

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Figure 4-4
Conditions Analyzed for Levee Settlement

Table 4-I.
Conditions of Settlement Analysis

Elevation at Top of New Fill	Thickness of New Fill (feet)	Distance X (feet)*	Settlement (feet)	
			Point A	Point B
Condition A—Fill Placed against the Levee				
-2	0	--	3.4	0.3
+4	6	--	3.6	0.3
+6	6	--	3.7	0.3
+8	10	--	4.0	0.4
Condition B—Fill Placed away from Levee				
0	0	--	3.4	0.3
+8	10	60	3.5	0.3
+8	10	40	3.6	0.3
+8	10	20	3.8	0.4

Note: The results of the settlement analysis are intended to indicate the general magnitude of levee settlement resulting from placing dredged material on or adjacent to the levee. Although single settlement values are given, it is more correct to assume a range of settlement values for each condition analyzed. A range of ±20% from the calculated values is reasonable for estimating long-term settlement. The final design-level investigation that will be conducted for this alternative shall also confirm these limits.

* Distance X refers to Figure 4-4.

Potential Issues and Resolutions under the Bel Marin Keys V Scenario

Potential issues and resolutions under this scenario are the same as the impacts and mitigation measures described for Alternative 3.