4.E Geology, Soils, and Seismicity

4.E.1 Introduction

This section describes the geology, soils, and seismicity of the Project Site and vicinity. It also analyzes and evaluates the impacts of Project Site development in relation to these resource areas. Feasible mitigation measures are identified as necessary to minimize significant impacts.

4.E.2 Environmental Setting

The following discussion describes the Project Site's regional topographic, geologic, and seismic setting, as well as potential geologic and seismic hazards that may affect the Project Site based upon the site conditions and location.

Landform History of the Project Site

Originally part of San Francisco Bay, the area that now makes up the Brisbane Baylands was transformed into its present-day condition through progressive filling of tidal marshlands and the resultant eastern advancement of the shoreline to its present location east of US Highway 101. In general, Bayshore Boulevard traces the early Bay shoreline. In the early 1900s, the Southern Pacific Railroad (SPRR) constructed railroad tracks across the Bay. Following the 1906 San Francisco earthquake, the area west of this rail corridor was filled in, primarily with demolition rubble. In 1914, this area became the main SPRR yard and remained so until 1960, when active rail operations temporarily ceased. Caltrain currently operates on the main line.

In the area east of the railroad tracks, Bay infilling continued up through the mid-1950s, further extending the shoreline to the east. Land filling operations were initiated in the area east of the railroad tracks, which served as the local municipal landfill for San Francisco from 1933 through 1967. Municipal waste was placed directly on tidal flats and waters at the margin of San Francisco Bay. The edge of the refuse pile was open to direct wave action from San Francisco Bay until construction of US Highway 101 began in 1959 (BKF, 2011). US Highway 101 was formed by placement of crushed rock directly in the Bay along a narrow strip east of the site. Waste and other fill has since been placed up to the US Highway 101 road base, which bounds the Project Site.

According to the Regional Water Quality Control Board (RWQCB), the former landfill portion of the Project Site was used for the disposal of primarily non-hazardous solid wastes including domestic, industrial, and shipyard waste; sewage; and construction rubble (RWQCB, 2001). The total volume of waste disposed at the landfill has been estimated to be 12.5 million cubic yards (Burns & McDonnell, 2002).

At the time of closure of the landfill in 1967, a soil cap was placed over the landfill and additional clean soil has also been placed over much of the site (BKF, 2011). **Figure 4.E-1** illustrates the history of fill placement. Current land use includes soil and aggregate material recycling operations and non-irrigated open space.

Note: Source of fill placement history is Army Corps of Engineers (1959) combined with original mapping for this study based on historic topographic maps.

SOURCE: Fugro Consultants, Inc., 2011

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Figure 4.E-1
Land Reclamation Sequence
Historic Fill and Shoreline of the Project Site

The two recycling companies currently operating on an interim basis in this portion of the site are Brisbane Recycling Company Inc. and Baylands Soil Processing, LLC. Brisbane Recycling Company Inc. maintains a concrete recycling operation in the northern portion of the site. In the southern portion of the site, Baylands Soil Processing, LLC maintains a soil recycling operation. Materials from the recycling operations are kept in stockpiles, which have contributed to consolidation of underlying refuse and Bay Mud.

Topography

As discussed above, the Bay margin natural topography of the site has been covered by rubble, solid waste, and soil fill. The elevation of the flat-lying portion of the Project Site ranges from approximately 10 to 50 feet above mean sea level (msl), with the majority of the site being flat or gently sloping toward the Bay (see **Figure 4.E-2**). Icehouse Hill, located in the southwestern portion of the Project Site, rises to approximately 200 feet with steep cuts adjacent to the existing railroad tracks and more gently sloping cuts along Bayshore Boulevard.

Soils and Geology

This subsection describes geologic and seismic hazards as well as soil and mineral resources in the Project Site vicinity. The Project Site's geologic environment is assessed based on the evaluation of current site conditions and review of published and unpublished geologic reports and maps.

Soils

As discussed above, the majority of the Project Site has been heavily modified over the last 100 years, and the native soils have been covered with rubble, solid waste, and imported fill (see Figure 4.E-1). Although the Bay Mud as a soil unit is no longer visible at the surface due to the placement of fill, the Bay Mud unit is present at shallow depths, primarily along the Bay shoreline and lagoon perimeter. Because future construction activities would be expected to encounter the Bay Mud in places, more information on this unit is provided below. The following soil types are described in the United States Department of Agriculture Web Soil Survey site (USDA, 1991).

The soils mapped on the Project Site and in the vicinity include the Urban Land-Orthents and the Barnabe-Candlestick-Buri Buri. Urban Land-Orthents soils are developed on the coastal terraces and hills north of where Interstate 280 (I-280) and Skyline Boulevard diverge. These soils encompass all developed areas of San Bruno, Colma, and Daly City. Urban Land-Orthents, Smoothed soils are highly variable with respect to depth of development and steepness of slope on which they occur. The Urban Land-Orthents soils category includes generally well-drained soils underlain by soft sandstone, whereas the Urban Land-Orthents Smoothed category comprises very shallow to very deep, well-drained, fine sandy loam over loam. The Barnabe-Candlestick-Buri Buri soil is developed on the sandstone uplands of San Bruno Mountain, and is located southwest of the Project Site and on Icehouse Hill. The unit consists of well-drained, gravelly sandy loams to fine loams.

Geology

San Francisco Bay formed during the past 10,000 years during sea level rise associated with the melting of extensive continental glaciers. The Bay is relatively shallow and has filled with mud and sand to a depth of about 300 feet. These sediments overlay bedrock of the Franciscan Complex at the Project Site (Bonilla, et al., 1998). As previously discussed, multiple man-made infilling events were carried out to reclaim Bay margin lands (see Figure 4.E-1).

Geologic cross-sections developed by Geosyntec (2006, 2008) and geology reviewed by Treadwell & Rollo, Inc. (2008) were evaluated to describe the general Project Site stratigraphy.[1] The stratigraphy from top (youngest) to bottom (oldest) for the Project Site consists of Artificial Fill comprised of construction debris and landfill waste, Young Bay Mud (which includes lenses of sand), and Old Bay Mud (which includes layers of silty sand and silts and clays within/over the bedrock). This sequence of alluvial sediments and Bay Mud records the relative rise and fall of sea levels as San Francisco Bay subsided. **Figure 4.E-3** provides a general block diagram and a cross-section that illustrate the subsurface stratigraphy of the local area. The block diagram shows the geologic units draped on the bedrock sloping eastward into the Bay. The cross-section shows the inter-fingering of the geologic units (e.g., the boundary between the Bay Margin deposits and the A-Sand) from the periodic rises and falls of sea level.

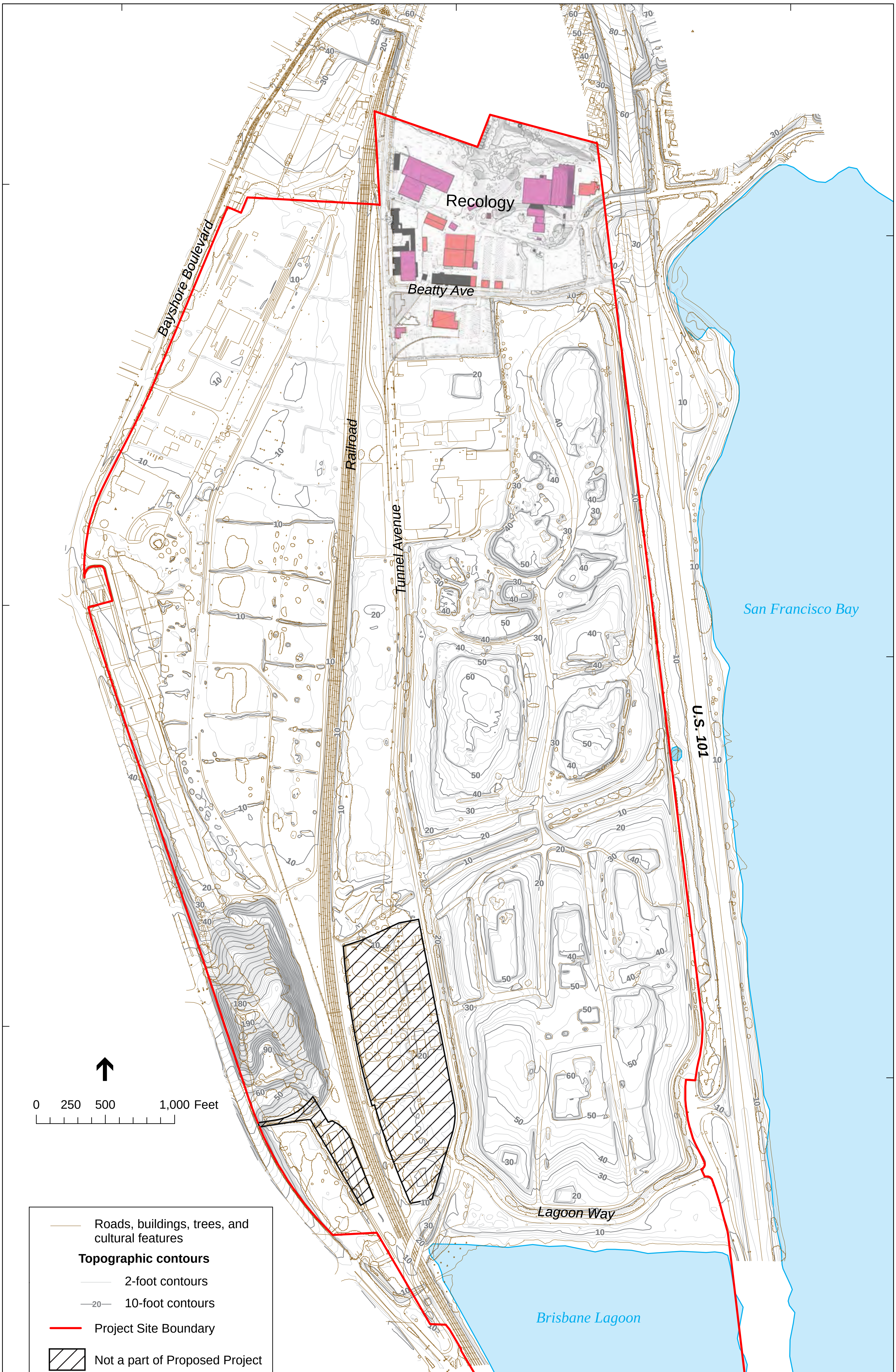
A summary of geologic materials found on the Project Site is provided in **Table 4.E-1**, and **Figure 4E-6** provides a geologic map of the Project Site vicinity.

TABLE 4.E-1
SUMMARY OF GEOLOGIC MATERIALS ON PROJECT SITE

Geologic Unit	Geologic Age	General Description
Artificial Fill	Recent (Historic)	Non-engineered fill material includes soils, concrete, bricks, tires, steel, and wood. The soil types range from sandy clay to gravel with sand and range in thickness from 6 to 40 feet. The majority of fill was composed of silty clayey sand and concrete matrix. A clean soil layer approximately 10 feet thick overlies the waste.
Waste	Recent (Historic)	Wood, paper, plastic, glass, wires, metals, and gravelly soils. Thickness ranges from 20 to 35 feet.
Young Bay Mud (YBM)	Holocene (less than 11,000 years old)	Elastic silt or fat clay. Thickness ranges from 10 to 50 feet.
Old Bay Mud (OBM)	Holocene and late Pleistocene (less than 120,000 years old)	Classified as low-to-high plasticity clays and clayey sands. In the northwest portion of the site a sand layer ranging from 88 to 93 feet in thickness underlies the base of the YBM.
Franciscan Assemblage (Bedrock)	Cretaceous-Jurassic (65 to 208 million years old)	Sandstones, shale, siltstones, chert, greenstone, and schist. Partially recrystallized and intruded by serpentine. Slope stability characteristics highly variable. Subject to sliding where highly sheared.

SOURCE: Treadwell & Rollo, 2008; Geosyntec, 2008.

[1] Stratigraphy is the vertical arrangement or sequencing of underlying materials that can be interpreted to describe the geologic history or for geotechnical purposes to design building foundations.



SOURCE: UPC, 2011, Recology, 2011

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Figure 4.E-2
Exisitng site Topography

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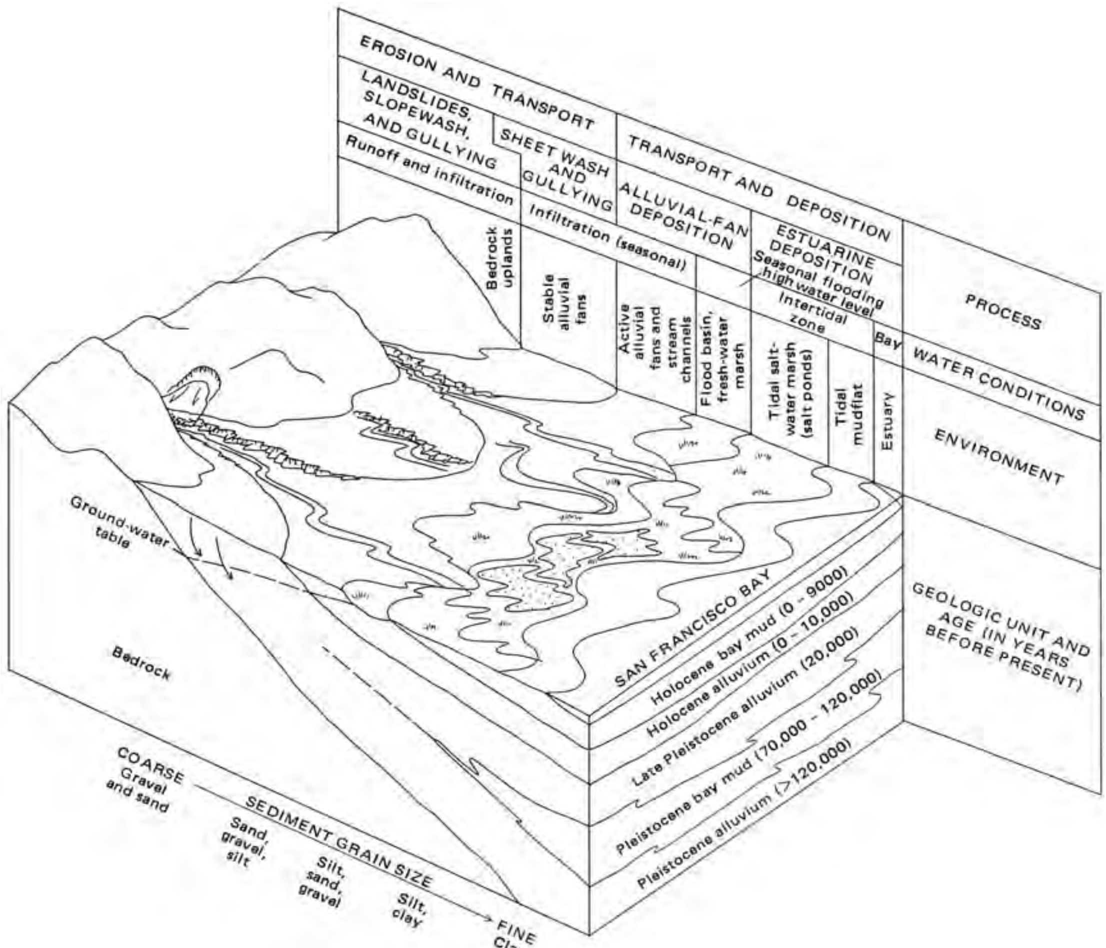
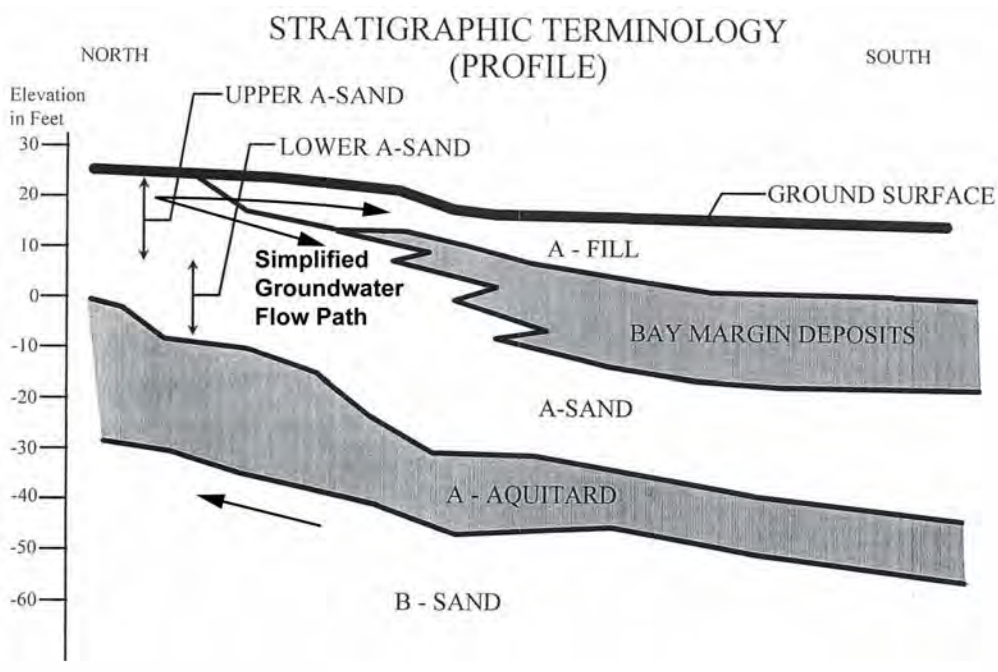


Source: Helley and Lajoie, 1979



Source: Burns & McDonnell, 2002

Figure 4.E-3
Regional and Local Stratigraphic Cross Sections

Artificial Fill, Waste, and Clean Soil Cover

Artificial fill material at the Project Site includes soils, concrete, bricks, tires, steel, and wood. Soil types range from sandy clay to gravel with sand. The majority of emplaced fill consists of silty clayey sand with a concrete matrix. At a few locations, steel was encountered within the concrete matrix. In addition, tires and wood were encountered at depths near the fill and waste boundary.

In the southern portion of the former landfill area, artificial fill thickness ranges from approximately 20 to 40 feet. In the northern portion of the former landfill area, artificial fill thickness ranges from 10 to 40 feet, with the greatest thickness of fill located near the center of the Project Site. The former railyard area is underlain by 6 to 22 feet of artificial fill, deposited on mudflats along the Bay margin in the early 1900s (Treadwell & Rollo, 2008). The fill is composed of mixtures of clay, silt, sand, rock fragments, organic matter, and other man-made debris. Geotechnical testing suggests that clay and silt within the fill are soft to very stiff and sandy fill is loose to dense (Geosyntec, 2008). The fill is underlain by Bay mud, which is a very soft to soft compressible marine clay.

Within the former landfill area, the waste material consists primarily of wood, paper, plastic, glass, wires, metals, and gravelly soils (Geosyntec, 2008).[2] The majority of waste material was composed of wood and paper. Approximately half of the waste was found to be below the water table. The waste thickness in the southern portion of the former landfill ranges from 20 to 30 feet, with deposits in the northern portion of the Project Site five feet thicker on average.

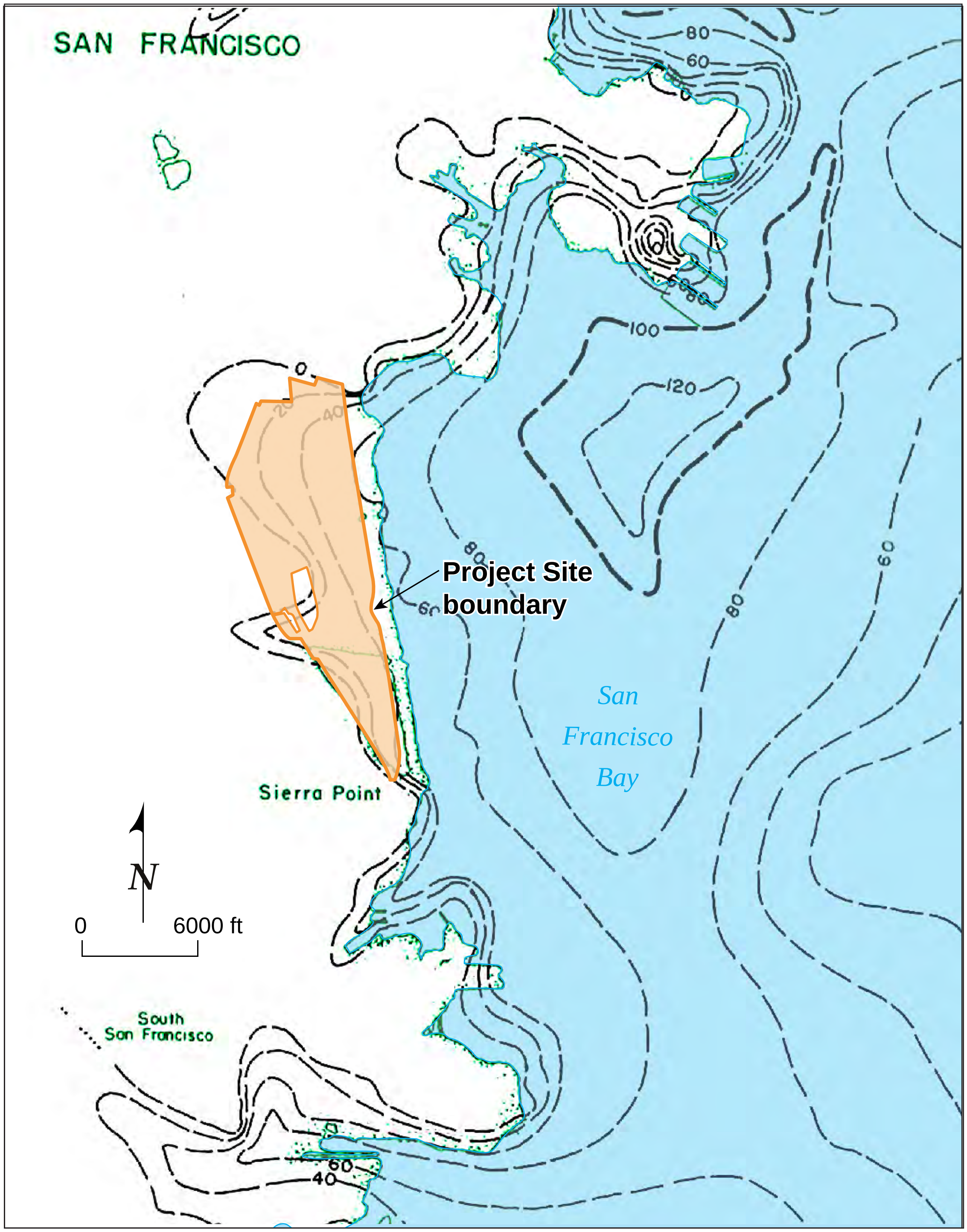
Consistent with landfill closure requirements at the time the landfill stopped accepting waste in 1967, the landfill operator placed a clean soil layer over the waste (Geosyntec, 2008). A Solid Waste Assessment Test (SWAT) report prepared by Kleinfelder for the former landfill site characterized the cover soil as primarily gravelly silt (CDM, 2005). Cover material has been added to the landfill through operations conducted by Baylands Soil Processing, LLC and Brisbane Recycling Company Inc. Recent soil cover investigations identified variability in soil cover thickness ranging from a few feet to more than 30 feet. However, the soil cover thickness over much of the landfill surface is reported to exceed 10 feet (Burns & McDonnell, 2002).

Young Bay Mud and Old Bay Mud

Young Bay Mud (YBM) is classified as elastic silt or fat clay. The total thickness of YBM deposits on the Project Site ranges from zero to up to approximately 50 feet (CDMG, 1966; Treadwell & Rollo, 2008; Geosyntec, 2008). **Figure 4.E-4** illustrates the thickness of the Young Bay Mud in the local area.

Old Bay Mud (OBM) is classified as low to high plasticity clays and clayey sands. The estimated thickness of OBM ranges from 50 feet in the west to more than 200 feet in the east. OBM thickness is estimated based on bedrock contours (see **Figure 4.E-5**), the elevation at the Project

2 The landfill was used primarily for nonhazardous wastes, but is also reported to have had some hazardous wastes deposited. For more discussion and analysis of hazardous materials associated with the former landfill see Section 4.G, *Hazards and Hazardous Materials* of this EIR.



Project Site

Coastline 1969

Notes: 1. Contour interval is 20 feet.
2. Datum is mean lower low water.

Thicknesses of Younger Bay Mud
Isopach line, dashed where approximate, dotted where inferred

SOURCE: California Division of Mines and Geology - 1966
Compiled by James E. Kahle and Harold B. Goldman
Complied from data contained in U.S. Army Corps of Engineers, Appendix "E." 1963, Radbruch, D.H., 1957; Schlocker, J., et al., 1958; Weaver, M.A., and Radbruch, D.H., 1960; and others.

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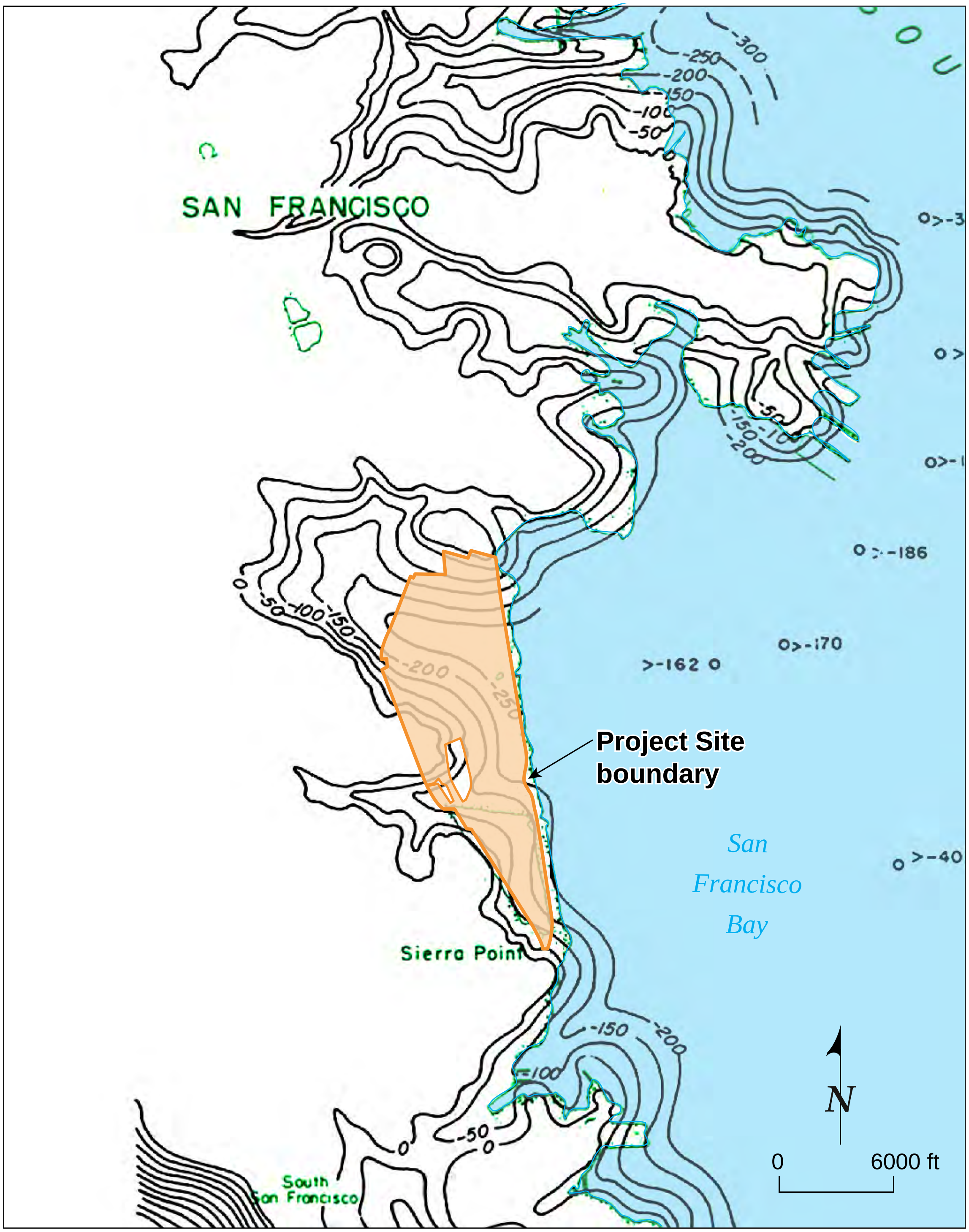
Figure 4.E-4
Young Bay Mud Isopach Map
for the Project Site Vicinity



Project Site

Coastline 1969

Top of Rock

Structure contours, solid where well located, dashed where approximate

Notes: 1. Contour interval is 20 feet.
2. Datum is mean lower low water.

Sources: California Division of Mines and Geology - 1966
Compiled by James E. Kahle and Harold B. Goldman
Complied from data contained in U.S. Army Corps of Engineers, Appendix "E." 1963. Revisions from Schlocker, J., et al., 1958; Radbruch, D.H., 1957; Weaver, M.A., and Radbruch, D.H., 1960; and others.

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Figure 4.E-5
Top of Bedrock Contour Map for the Project Site Vicinity

Site shown in topographic profile from 2007 topographic maps (UPC, 2011; see Figure 4.E-2), and the thickness of the YBM (see Figure 4.E-4). The OBM consists of two sub-units on the Project Site: high plasticity clays interpreted as OBM and OBM containing a sand layer within the northern portion of the former landfill area. The OBM with the sand layer varies from 88 to 93 feet in thickness beneath the base of the YBM (Geosyntec, 2008). OBM is poorly characterized beneath the former railyard area but was encountered in several borings (Treadwell & Rollo, 2008).

The sediment sequence consists of a staggered and layered package of beach deposits (B-sand), back-bay mud flats (A/B-aquitard),[3] and offshore barrier bar (A-sand) deposits. As shown in Figure 4.E-3, this framework is initiated as a laterally flat relationship between beach deposits (B-sand), back-bay mud flats (A-aquitard), and offshore barrier bar (A-sand). With marine transgression (increase in relative sea level), this lateral relationship migrates landward, westward, and upward. With infilling, the above sediment sequence would compact over time. Typically, compaction is greater basinward (east) than shoreward (west) due to the increasing amount of Bay Mud. On the Project Site, bedrock elevations based on this regional map indicate a subsurface ridgeline running roughly east-west, dividing the northern sand rich deposits of the OBM from the fines rich OBM to the south, creating a depth range from approximately -50 to -250 feet msl (see Figure 4.E-5).

Bedrock

Bedrock of the Franciscan Complex underlies the Project Site (see Figure 4.E-5, **Figure 4.E-6,** and **Figure 4.E-7**) (Bonilla et al., 1998). The Franciscan Complex generally includes sedimentary and igneous rocks, including consolidated sandstone and shale encountered at depth in borings (Geosyntec, 2008).

This unit consists of layers of consolidated sandstone and shale, which have been tilted by tectonic action. The sandstone and shale, in about equal amounts, are about 3,000 feet thick on nearby San Bruno Mountain (Bonilla et al, 1998). To the north of the Project Site, near the Cow Palace, a valley has been eroded in the mélange, which consists of basalt, serpentine, and sandstone blocks in a sheared shale matrix. Many of these rocks were deposited in a coastal marine environment and have been deformed and uplifted by tectonic activity associated with displacement along the San Andreas fault.

Groundwater

The regional groundwater in the vicinity of the Project Site has been divided into two zones (A and B) (Burns & McDonnell, 2010). Zone A is comprised of shallow water-bearing sediments encountered from the ground surface to depths of approximately 20 feet below ground surface (bgs). The Zone A water-bearing sediments are typically encountered above the Younger Bay Mud. The relatively coarse-grained water-bearing Zone B sediments are encountered beneath the Younger Bay Mud, which is reported to act as an aquitard between the two zones.

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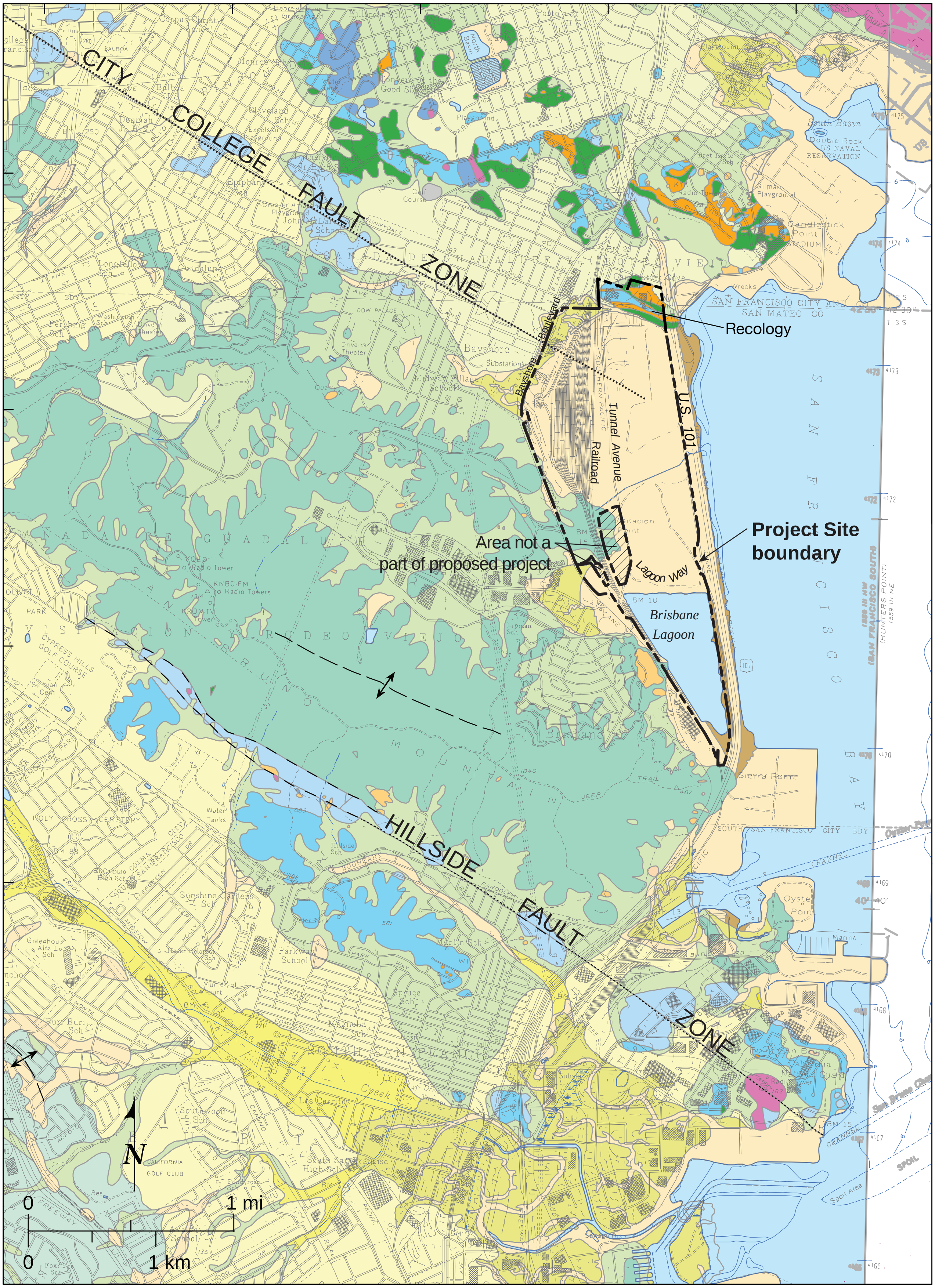
[3] An aquitard is a bed of low permeability adjacent to an aquifer; may serve as a storage unit for groundwater, although it does not yield water readily.



NOTE: See Figure 4.E-7 for explanation of map colors

SOURCE: Bonilla et al., 1998

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Figure 4.E-6
Geologic Map of the Project Site Vicinity

EXPLANATION

GEOLOGIC UNIT DESCRIPTIONS

water

QUATERNARY
Holocene

Qaf Artificial fill
Clay, silt, sand, rock fragments, organic matter, and man-made debris.

Qaf/tf Artificial fill over tidal flat
Clay, silt, sand, rock fragments, organic matter, and man-made debris, placed over tidal flats.

Ql Landslide deposits
Composition and structure depend on the geologic formation involved and type of landslide.

Qal Alluvium
Mostly sand and silt but locally contains clay, gravel, or boulders; generally gray to brown.

Qm Bay mud
Soft (moist) to firm (dry) clay and silt; locally contains shell fragments, plant remains, and thin beds of sand.

Qd Dune sand
Well sorted fine-grained sand, gray and loose in most places, grayish orange to reddish brown and firm in a few places. Age extends into Pleistocene.

Pleistocene

Qsr Slope debris and ravine fill
Stony silty to sandy clay; locally silty to clayey sand or gravel; yellowish-orange to medium gray, unstratified or poorly stratified. Where it overlies the Merced or Colma Formation it is commonly a silty to clayey sand, or gravel.

Qc Colma Formation
In northwest and central parts of area, friable well sorted fine to medium sand containing a few beds of sandy silt, clay, and gravel. In southeast part of area, mostly sandy clay and silty sand; yellowish orange to gray.

Qu Sedimentary deposits, undifferentiated

CRETACEOUS AND JURASSIC
Franciscan Complex and associated rocks

KJs Sandstone and shale
Interbedded sandstone and shale, hard where fresh and intact, soft where weathered or sheared. Commonly medium dark gray where fresh, olive gray to yellowish brown where moderately weathered, and yellowish orange to yellowish gray where highly weathered.

KJsk Sandstone and shale
Sandstone generally containing more than two percent potassium feldspar.

KJc Chert
Hard chert interbedded with firm shale; chert layers generally two or three inches thick, shale layers less than one inch thick; generally grayish red.

KJg Greenstone
Altered volcanic rocks, fine grained, mostly basalt; hard where fresh, but weathered and firm to soft in most exposures; commonly grayish olive to moderate olive gray where moderately weathered, dark yellowish orange to light brown where highly weathered.

sp Serpentine
Hard to soft, generally greenish gray; contains small bodies of grabbro and diabase.

KJu Sheared rocks
Small to large fragments of hard rock in matrix of sheared rock. Matrix generally coherent and firm, but soft in places, especially where weathered. Dark gray where fresh, yellowish brown where weathered. Derived mostly from shale and sandstone of Franciscan Complex and serpentine.

SYMBOLS

- Contact, certain
- Contact, approximately located
- Contact, concealed
- Contact, gradational or inferred
- Fault, certain
- Fault, approximately located
- Fault, concealed
- ? Fault, concealed, queried
- Fault, inferred
- Top of landslide scarp
- Anticline, approximately located

SOURCE: Bonilla et al., 1998

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Figure 4.E-7
Explanation for Geologic Map of the Project Site Vicinity

The direction of groundwater flow in the shallow water-bearing zone is generally a combination of east toward San Francisco Bay to south toward Brisbane Lagoon, depending on localized conditions. As groundwater flow reaches the margins of the Bay, the flow intersects the waters of the Bay, either through aquifers discharging directly into Bay waters or from reaching aquifers beneath the Bay. At this intersection, local groundwater flow directions on the site become highly variable. Please see Section 4.G, *Hazards and Hazardous Materials* for additional discussion of groundwater.

Regional Earthquake Faults

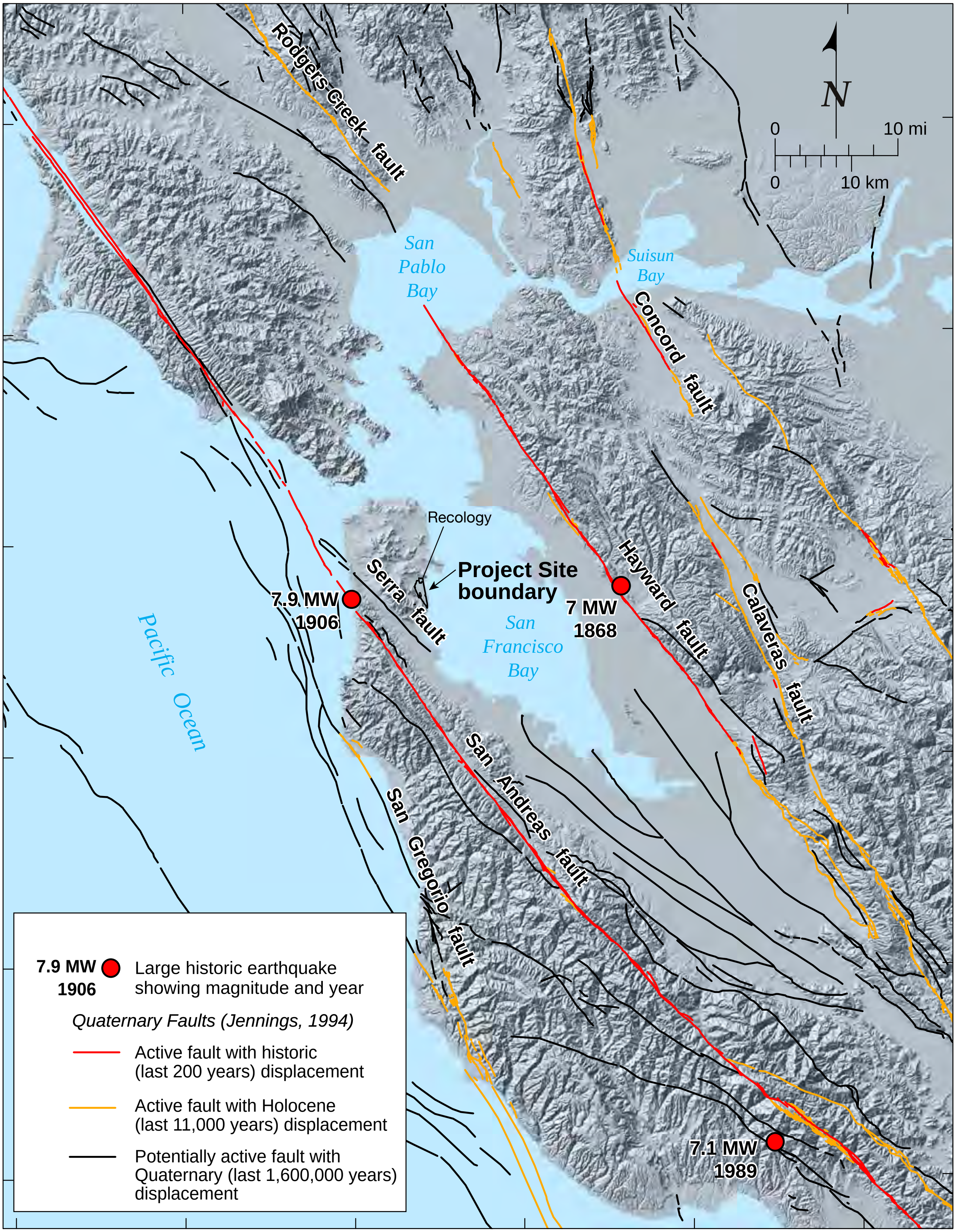
The Project Site, along with the entire San Francisco Bay Area, is dominated seismically by the active San Andreas fault system (see **Figure 4.E-8**). The San Andreas fault system forms the boundary between the northward-moving Pacific Plate (west of the fault) and the southward-moving North American Plate (east of the fault). In the San Francisco Bay Area, this movement is distributed across a complex system of subparallel right-lateral strike-slip faults, which include the San Andreas, San Gregorio, Hayward, Rogers Creek, and Calaveras faults, among others. Significant active and potentially active faults and seismic sources (earthquake) zones within 60 miles of the Project Site are listed in **Table 4.E-2**.

TABLE 4.E-2
ACTIVE FAULTS IN THE VICINITY OF THE PROJECT SITE

Fault	Approximate Distance and Direction from Project Site	Recency of Movement	Fault Classification[1,2]	Historical Seismicity[3]	Probability of at Least One M 6.7 or Larger Earthquake in 2007-2036[4]
San Andreas	5 miles southwest	Historic (1906; 1989) Holocene	Active	M 7.1, 1989 M 7.9, 1906 M 7.0, 1838 Many M 5	0.21
Hayward	13 miles east	Historic (1886, southern segment)	Active	M 6.8, 1868 Many <M 4.5	0.31
San Gregorio	11 miles southwest	Holocene-Late Quaternary	Active	Many M 3-6.4	0.07
Calaveras	24 miles east	Historic (1861) Holocene	Active	M 5.6-6.4, 1861 M 4-4.5 swarms, 1970, 1990	0.07
Monterey Bay	East of Bay (5)	(5)	(5)	M 6.5 1836	(5)

NOTES:
1 Fault activity rating as defined by the State of California (Hart and Bryant, 1997).
2 Fault designation by the USGS (OFR 96-08).
3 Earthquake Moment Magnitude (M).
4 The published background values are not explicitly stated by the WGCEP (2008) and thus the WGCEP (2003) values were used.
5 Information not available for this event.

SOURCES: Hart and Bryant, 1997; WGCEP, 2003.



SOURCE: Jennings, 1994

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Figure 4.E-8
Regional Map of Active and Potentially Active Faults

These faults are all considered active or potentially active and capable of producing significant intensities and durations of groundshaking at the site. Historically, the area has been subject to intense seismic activity (Hart and Bryant, 1997) and it will likely be subjected to a high degree of groundshaking in the future from earthquakes generated on active faults in the San Francisco Bay Area.

San Andreas Fault

The San Andreas fault zone is located approximately five miles southwest of the Project Site. The San Andreas fault is the longest active fault system in the state (see Figure 4.E-8). Within the Bay Area, the main trace of the San Andreas fault trends northwest through the Santa Cruz Mountains and the eastern side of the San Francisco Peninsula. The San Andreas fault zone was the source of the two major seismic events in recent history that resulted in widespread damage throughout the San Francisco Bay region: the 1906 San Francisco earthquake (magnitude [M] 7.9) and the 1989 Loma Prieta earthquake (M 7.1). The United States Geological Survey (USGS) Working Group on California Earthquake Probabilities (WGCEP) estimates there is a 21-percent chance that the San Andreas fault will produce an earthquake of M 6.7 or greater by 2036 (USGS, 2007). An earthquake of this magnitude would result in substantial structural damage and loss of life.

Hayward Fault

The Hayward fault zone is located approximately 13 miles east of the Project Site (see Figure 4.E-8). The Hayward fault zone is the southern extension of a fault zone that includes the Rodgers Creek fault (north of San Pablo Bay), the Healdsburg fault (in Sonoma County), and the Mayacama fault (in Mendocino County). The Hayward fault trends northwest within the East Bay, extending from San Jose 60 miles north to San Pablo Bay in Richmond. Historically, the southern portion of the Hayward fault generated a large to major earthquake in 1868. The USGS WGCEP estimates there is a 31-percent chance that the Hayward-Rodgers Creek fault system will produce an earthquake of M 6.7 or greater by 2036 (USGS, 2007).

San Gregorio Fault

The San Gregorio fault zone is located approximately 11 miles southwest of the Project Site (see Figure 4.E-8). The fault zone trends northwest and lies mostly within the Pacific Ocean in the vicinity of the Project Site, with right-lateral strike-slip motion on a near-vertical fault plane geometry. Historically, the fault has generated multiple M 3 to M 6.4 earthquakes. The USGS WGCEP estimates there is a seven-percent chance that the San Gregorio fault will produce an earthquake of M 6.7 or greater by 2032 (USGS/CGS, 2002).

Calaveras Fault

The Calaveras fault is located approximately 24 miles east of the Project Site (see Figure 4.E-8). The fault zone trends northwest within the East Bay, with right-lateral strike-slip motion on a near-vertical fault plane geometry. Historically, the fault generated a M 5.6 to 6.4 earthquake in 1861 and swarms of M4 to 4.5 earthquakes in 1970 and 1990. The USGS WGCEP estimates there is a seven-percent chance that the Calaveras fault system will produce an earthquake of M 6.7 or greater by 2036 (USGS, 2007).

Other Faults

Several smaller faults, including the Serra, City College, and Hillside faults, that are not considered active, are located within an approximately five-mile radius of the Project Site (see Figure 4.E-6, Figure 4.E-7, and Figure 4.E-8).

The Serra fault, located approximately 4.8 miles west of the Project Site, is listed as potentially active with activity in the Pleistocene (i.e., active displacement 1.8 million years before present [BP]) but not after Holocene time (11,000 years BP) (Jennings, 1994).

The City College and Hillside faults are both pre-Quaternary (i.e., active displacement greater than 1.8 million years BP) and are therefore considered inactive (Bonilla et al., 1998). The City College fault is inferred to transect the northern portion of the landfill on the Project Site, while the Hillside fault is located approximately 1.8 miles to the southwest of the Project Site (see Figure 4.E-6). The shear zone of the City College fault is estimated to be several thousand feet wide. However, since the most recent activity along these faults has been estimated to be pre-Quaternary (i.e., older than 1.8 million years), these faults are considered inactive (BKF, 2011). No active shear zones (areas of strong deformation caused by movement along a fault) are known to exist at the Project Site.

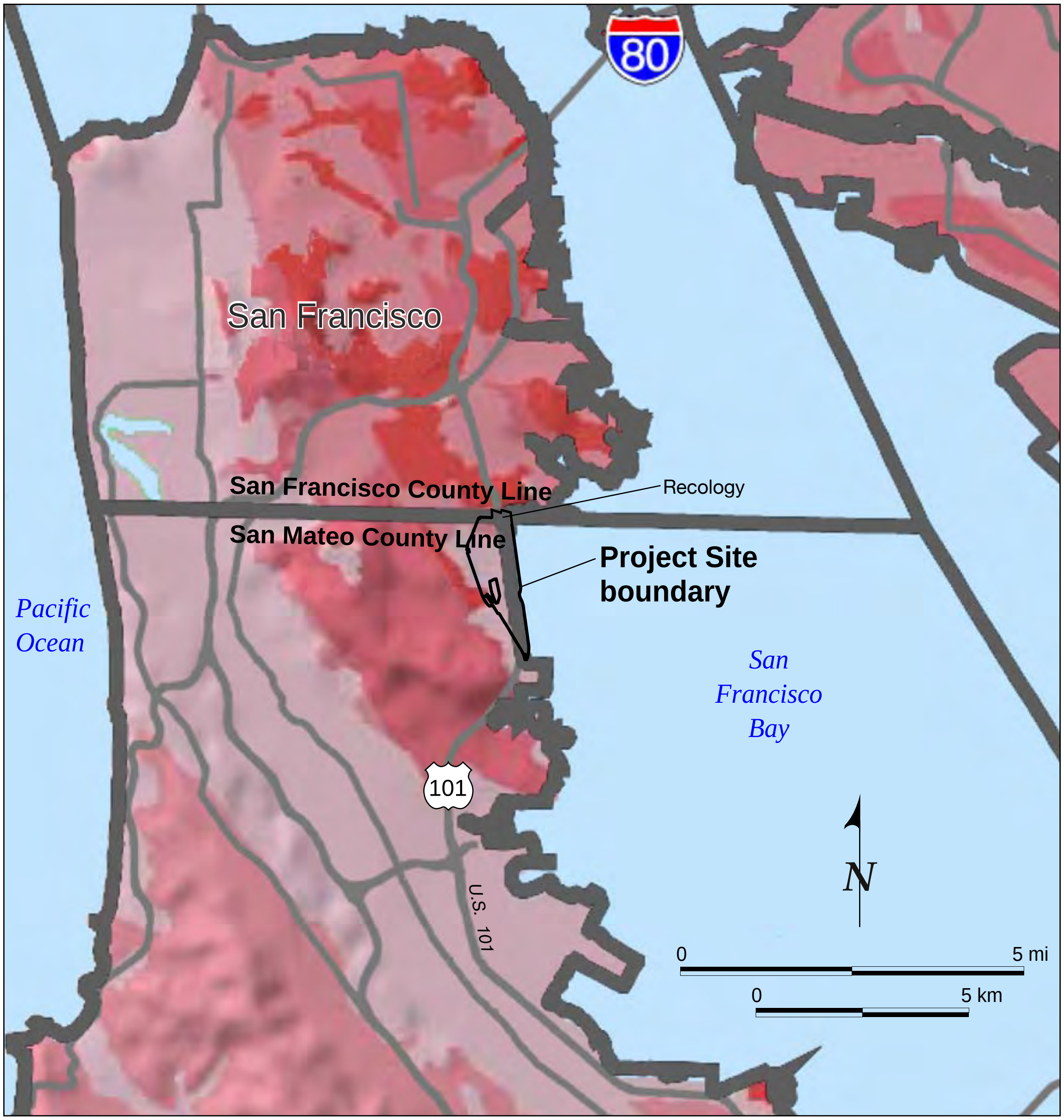
Seismicity

The San Francisco Bay Area contains both active and potentially active faults, and is considered a region of high seismic activity (see Figure 4.E-8, **Figure 4.E-9**, and **Figure 4.E-10**). A major earthquake can occur at any time, in any part of this densely populated region. The epicenter of the 1989 Loma Prieta earthquake was in the Santa Cruz Mountains, some 30 to 70 miles away from Bay Area cities. The USGS warns that should an earthquake strike at one of the closer fault areas, such as the Hayward fault, shaking in local cities can be expected to be 5 to 12 times stronger than it was in 1989.

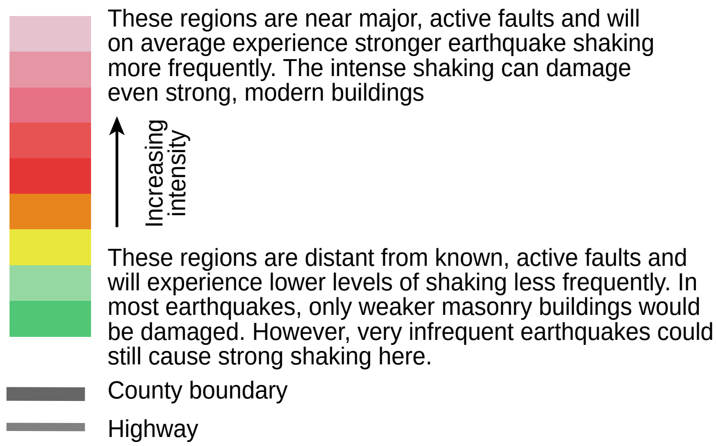
Earthquake Magnitudes

The greater San Francisco Bay Area region has historically experienced strong groundshaking from large earthquakes and will continue to do so in the future. A map showing the locations and magnitudes of the two largest historical earthquakes in the San Francisco Bay Area is presented in Figure 4.E-8. Since 1800, five earthquakes of magnitude (M) greater than 6.5 have occurred in the Bay Area (see Table 4.E-2): the 1836 M 6.5 event east of Monterey Bay, the 1838 M 7.0 event on the Peninsula section of the San Andreas fault, the 1868 M 6.8 Hayward event on the southern Hayward fault, the 1906 M 7.9 San Francisco earthquake on the San Andreas fault, and the 1989 M 7.1 Loma Prieta event in the Santa Cruz Mountains.

On the basis of research conducted since the 1989 Loma Prieta earthquake, the USGS and other scientists, comprising the Working Group on California Earthquake Probabilities (WGCEP), have



Level of Earthquake Hazard



SOURCE: USGS and CGS, 2003

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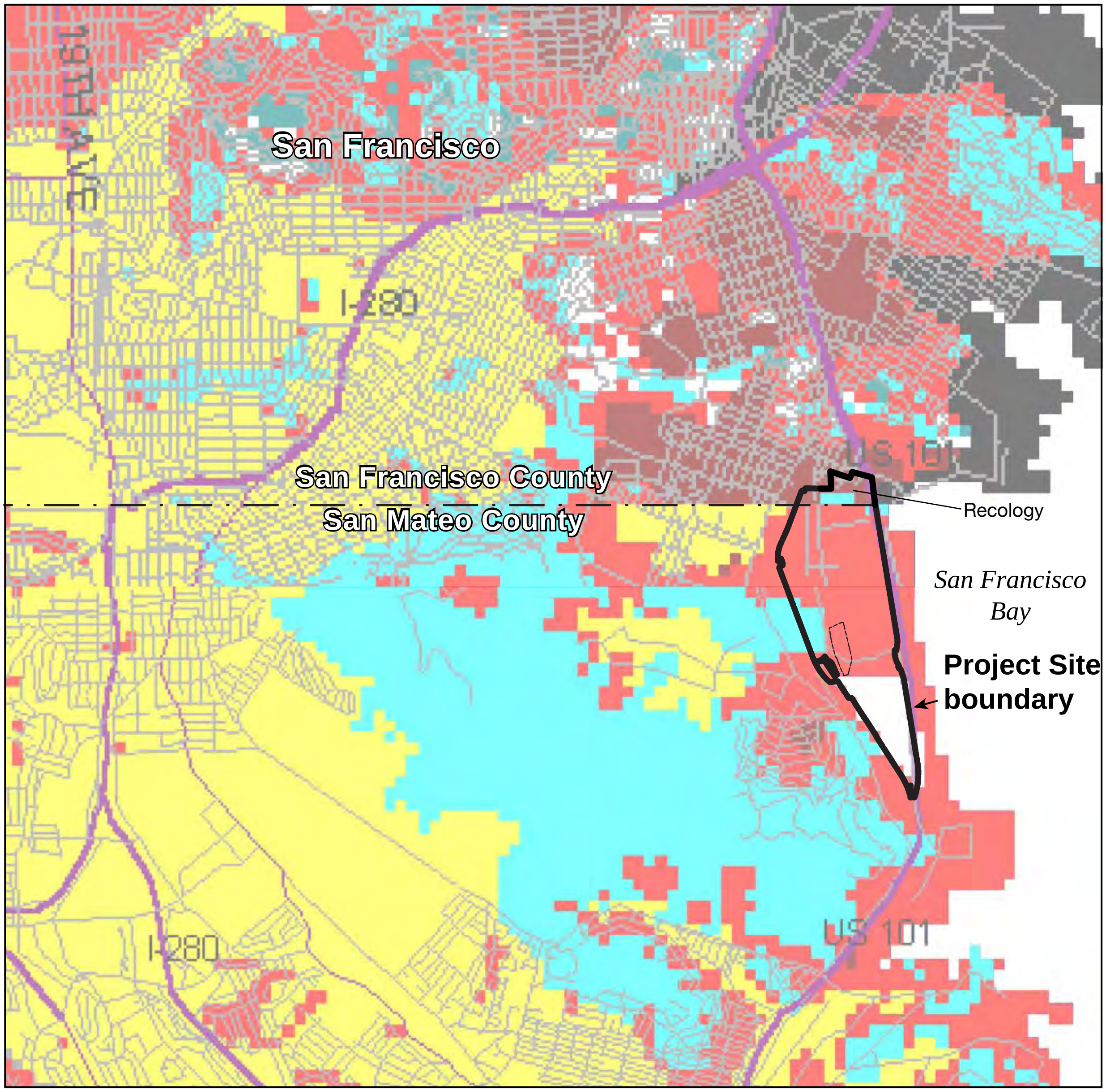
Figure 4.E-9
Level of Earthquake Hazard in the Project Site Vicinity



Source: ABAG, 2007

Shaking Amplification for Geologic Materials

- Extremely high
- Very high
- High
- Moderately high
- Moderate
- Moderately low
- Low
- Very low
- Highway
- Street

SOURCE: ABAG, 2007

Brisbane Baylands . 206069

Figure 4.E-10
Ground Shaking Amplification Map of the Project Site Vicinity

concluded that there is a 66-percent probability of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay Area before 2036 (USGS, 2007). The findings of the WGCEP (2008) report are summarized in the Table 4.E-2.

Earthquake Intensity

While magnitude is a measure of the energy released in an earthquake, intensity is a measure of the groundshaking effects at a particular location. Ground movement during an earthquake can vary depending on the overall magnitude of the earthquake, distance from a site to the fault, focus of earthquake energy, and type of geologic material. The composition of underlying soils, even those relatively distant from faults, can intensify groundshaking. The Project Site is underlain by both National Earthquake Hazard Reduction Program (NEHRP) Soil Classifications E and D, suggesting that significant amplification of strong groundshaking could occur (see **Table 4.E-3**).

TABLE 4.E-3
NATIONAL EARTHQUAKE HAZARD REDUCTION PROGRAM (NEHRP) SOIL CLASSIFICATIONS

Soil Classification	Shear-Wave Velocity (Vs)	Soil Description
Soil Type A	Vs > 1500 meters/second	Includes unweathered intrusive igneous rock. Occurs infrequently in the Bay Area. Considered with Type B. Soil types A and B do not contribute greatly to shaking amplification.
Soil Type B	1500 meters/second > Vs > 750 meters/second	Includes volcanic, mostly Mesozoic bedrock, and some Franciscan bedrock. (Mesozoic rocks are between 245 and 64 million years old. The Franciscan Complex is a Mesozoic unit that is common in the Bay Area.)
Soil Type C	750 meters/second > Vs > 350 meters/second	Includes some Quaternary (less than 1.8 million years old) sands, sandstones, and mudstones; some Upper Tertiary (1.8 to 24 million years old) sandstones, mudstones, and limestones; some Lower Tertiary (24 to 64 million years old) mudstones, and sandstones; and Franciscan mélange and serpentinite.
Soil Type D[1]	350 meters/second > Vs > 200 meters/second	Includes some Quaternary muds, sands, gravels, silts, and mud. Significant amplification of shaking by these soils is generally expected.
Soil Type E[1]	200 meters/second > Vs	Includes water-saturated mud and artificial fill. The strongest amplification of shaking is expected for this soil type.

NOTE:
1 As described in the text, the soil underlying the Project Site is NEHRP Soil Type D and/or E.

SOURCE: USGS, 2006a.

The Modified Mercalli (MM) intensity scale (see **Table 4.E-4**) is commonly used to measure earthquake effects due to groundshaking (CGS, 2007). The MM values range from I (earthquake not felt) to XII (damage nearly total). Values ranging from IV to X could cause moderate to significant structural damage. Maximum groundshaking resulting from an earthquake generated on the San Andreas fault, as discussed below, is anticipated to be violent to very violent (MM IX to MM X) at the Project Site (ABAG, 2007).

TABLE 4.E-4
MODIFIED MERCALLI INTENSITY SCALE

Intensity Value	Intensity Description	Average Peak Acceleration[1]
I	Not felt except by a very few under especially favorable circumstances.	<0.0017g
II	Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.	<0.014g
III	Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing of truck. Duration estimated.	<0.014g
IV	During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.	0.014-0.039g
V	Felt by nearly everyone, many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbances of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop.	0.039-0.092g
VI	Felt by all, many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.	0.092-0.18g
VII	Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motor cars.	0.19-0.34g
VIII	Damage slight in specially designed structures; considerable in ordinary substantial buildings, with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, and walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motor cars disturbed.	0.34-0.65g
IX	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.	0.65-1.24g
X	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks.	>1.24g
XI	Few, if any, (masonry) structures remain standing. Bridges destroyed. Board fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.	>1.24g
XII	Damage total. Practically all works of construction are damaged greatly or destroyed. Waves seen on ground surface. Lines of sight and level are distorted. Objects are thrown upward into the air.	>1.24g

NOTE:
[1] g (gravity) = 580 centimeters per second squared. 1.0 g of acceleration is a rate of increase in speed equivalent to a car traveling 328 feet from rest in 4.5 seconds.

SOURCE: CGS, 2007.

In 2002, the USGS teamed with the California Geological Survey (CGS) to complete an update of the national seismic hazard maps that depict the probabilistic groundshaking hazard for the entire United States (USGS/CGS, 2002). The hazard was calculated at a series of gridded locations (spaced 0.05 kilometer apart) across the country using probabilistic seismic hazard analysis techniques. The USGS maps display contoured ground motion parameters for a standardized probability scenario. As shown in **Table 4.E-5**, the estimate of the range of peak ground accelerations (PGA) expected in the vicinity of the Project Site is between 0.5 to 0.6 g, and spectral accelerations are expected to be between 0.58 to 1.251 g within a 475-year period (10 percent in 50 years; CGS, 2012).[4]

TABLE 4.E-5
ESTIMATED PEAK GROUND ACCELERATIONS (PGA) FOR PROJECT SITE[1]

Ground Motion	Firm Rock	Soft Rock	Alluvium
PGA	0.56	0.56	0.56
SA 0.2 Sec	1.251	1.251	1.251
SA 1.0 Sec	0.58	0.656	0.757

NOTE:
1 Calculations based on user specified point in the center of the Project Site (latitude 37.6977,longitude -122.4). Values expressed as a fraction of the acceleration due to gravity (g). Abbreviations: PGA = peak ground acceleration, SA = spectral acceleration. Ground motion values were interpolated from a grid of calculated values (0.05 degree spacing), not intended for design. NEHRP Soil corrections were calculated by USGS/CGS (2002) and used to calculate soft rock alluvium.

SOURCE: USGS/CGS, 2002, CGS, 2012.

Mineral Resources

The California Department of Conservation, CGS has classified lands within the San Francisco-Monterey Bay Region into Aggregate and Mineral Resource Zones (MRZs) based on guidelines adopted by the California State Mining and Geology Board, as mandated by the Surface Mining and Reclamation Act of 1974 (Stinson et al., 1983).

No known mineral resources are located within or near the Project Site. Mineral resource extraction activities have not taken place within or around the Project Site during recent history. The Project Site is mapped by CGS as MRZ-1, a zone where no significant mineral or aggregate deposits are present (Kohler-Antablin, 1996).

Geologic Hazards

Slope Failure

Background

Slope failure can occur in the form of creep, slumps, large progressive translation or rotational failures, rockfall, or debris flows. Soil creep is the slow continuous deformation of soil or rock.

4 Spectral accelerations refers to what might be experienced by a building during an earthquake as opposed to what might be experienced on the ground.

Slumps refer to a mass movement of materials that slide on a curved plane, and are generally smaller than what would be considered to be a large translational or rotational failure, which would also occur along a curved plane of motion. Rockfalls and debris flows are more general references to the downward movement of rock or soil materials.

Landslides can occur during earthquakes, triggered by the strain induced in soil and rock by the groundshaking vibrations. During non-earthquake (static) conditions, slope failures occur most frequently during the rainy season when high groundwater conditions persist. Landslides typically occur most frequently during or following large storms and in years with significant precipitation.

Landslides are most likely to occur in areas where they have previously occurred. Landslide mapping, therefore, provides a basis for estimating the most likely locations for future slope failure. Steep slopes are often prone to sliding. Slides may occur slowly or suddenly, at times without apparent provocation. Possible landslide causes include gradual disintegration of the structure of the soil, an increase in pore water pressure, liquefaction of underlying soil, or horizontal acceleration due to earthquake groundshaking. Evaluation of the stability of a slope is performed by calculating "factors of safety;" the factors of safety are calculated for both static and dynamic (earthquake-induced groundshaking) conditions.

Project Site Conditions

The review of existing maps, including landslide inventory maps, confirmed that no landslides are mapped within the Project Site. The associated landslide hazard for the former landfill area is nil to very low because surface gradients are very gentle (see Figure 4.E-2). Along the southwestern boundary of the former railyard area, moderate to locally steep relief is associated with bedrock outcrops in the vicinity of Icehouse Hill, near the Kinder Morgan Energy Tank Farm; however, no landslides are documented for this area. The west side of Icehouse Hill has had some noted rock fall which the City has addressed through placement of concrete k-rail barriers to prevent falling rocks and soil from reaching Bayshore Boulevard travel lanes.

As described above, soils within much of the Project Site consist of fill that is generally underlain by a layer of very soft to soft, compressible marine clay, known as Bay Mud. The thickness of the Bay Mud layer ranges from zero to about 50 feet, and generally increases in thickness toward the southern portion of the site.

Settlement and Differential Settlement

Settlement and differential settlement can be caused by several factors, including primary settlement, settlement related to liquefaction and lateral spreading, and cyclic densification related to strong groundshaking (Treadwell & Rollo, 2008). Fill subsidence and settlement can affect long-term durability and maintenance requirements of constructed slopes, built structures, roadways, and underground utilities.

For the former landfill, there is ongoing decomposition of the waste material which causes settlement as well as consolidation of the underlying Bay Mud. As a result, the landfill surface is expected to continue to undergo some degree of differential settlement.

Expansive Soils

Expansive soils exhibit "shrink-swell" behavior, also called linear extensibility. Shrink-swell is the cyclic change in volume (expansion and contraction) that occurs in fine-grained sediments from the process of wetting and drying. An expansive soil hazard is considered to exist where soils with an expansion index greater than 20 are present. Typically, the expansion index of a soil is directly correlative to the amount of clay in the soil, with a high clay percentage resulting in a high expansion index (Edwards et al., 1970).

If it is not currently saturated, Young Bay Mud has a fairly high shrink-swell potential, due to the presence of expansive clay derived from upland areas (Helley and LaJolie, 1979). Since the Bay Mud on the Project Site is buried by landfill material and is beneath the groundwater level, it remains wet and the corresponding potential for shrink-swell is relatively low. The surface fill has not been identified as having a significant expansive clay component (Geosyntec, 2006).

Bay Mud and associated estuarine deposits are present beneath the former landfill area at depths greater than 71 feet (Geosyntec, 2008) and beneath the former railyard area starting from seven feet to 50 feet (Treadwell & Rollo, 2008). For the majority of the former railyard area, Bay Mud and other clay-rich deposits are located primarily beneath the groundwater level, and therefore have a relatively low corresponding potential for shrink-swell (Geosyntec, 2008). However, the depth of these deposits in the former railyard area is somewhat poorly constrained, and in one boring near Icehouse Hill, Bay Mud is located above the groundwater table, suggesting a possible higher shrink-swell potential.

Soil Corrosivity

Corrosivity of soils is dependent on soil texture, soil pH, moisture content, and geochemical composition of fluids within the soil. These factors, in turn, are influenced by the physical and mineralogical composition of soils. Soil composition often is directly derived from the characteristics of the underlying geologic deposits on which they develop. Silty, loamy, and clayey soils tend to be among the more potentially corrosive soils, in contrast to granular soils (sands and gravels). In addition, the topography of the land, depth to groundwater, and native vegetation all influence the soil corrosivity potential.

Although soil corrosivity can exist within a broad range of soil conditions, the extent of acidity or alkalinity of a soil, as expressed by pH, directly influences corrosion susceptibility. Soils with a pH less than 4.0 have been found to indicate the highest risk of corrosion (Muckel, 2004). Typically soils with a pH of 0.0 to 4.0 are acidic and, where saturated, can serve as a corrosive electrolyte. Soils with a more neutral pH of 6.5 to 7.5 and low redox, or oxidizing, conditions are optimal for sulfate reduction by bacteria, which can cause localized corrosion. Soil resistivity, the measure of a soils' ability to retard the conduction of an electrical current, also has a strong influence on the corrosion rate. As a general rule, higher resistivity values correlate to lower corrosion potential. Soil resistivity arises from a number of factors, but fine-grained soils (silts, loams, clays) typically have the lowest resistivities and thus the greatest corrosion susceptibility.

Corrosive subsurface soils may exist in places within the Project Site and are especially likely along Bayshore Boulevard, where Bay Mud is present beneath the fill. The landfill waste can also have corrosive properties depending on the chemistry of the leachate. Corrosive soils could have a detrimental effect on concrete and metals. Corrosion is typically a result of contact with soluble chloride salts found in the soil or water, which requires moisture to form solutions of these salts. Several key factors that influence the severity and rate of corrosion include: the amount of moisture in the soil, the conductivity of the solution, the pH of the solution, and the oxygen concentration within the soil (aeration). The organic content of the soil, soil porosity, and soil texture indirectly affect corrosion of metals in soil by influencing the key factors listed above. Depending on the degree of corrosivity of subsurface soils, concrete and reinforcing steel in concrete structures and bare-metal structures exposed to these soils could deteriorate, eventually leading to structural failures.

Soil Erosion

Erosion is the wearing away of soil and rock by processes such as mechanical or chemical weathering, mass wasting, and the action of waves, wind and underground water. Surficial and near-surface materials are prone to erosion, with increased potential for deposits on steep slopes. Erosion of materials can lead to the destabilization of ground surfaces and exposure of buried materials.

The Project Site is mainly covered with undocumented fill materials, and thus fill is the most likely deposit at risk of soil erosion. Icehouse Hill is the only portion of the Project Site with native soils that overlie bedrock. Typically, the soil erosion potential is reduced once the soil is graded and covered with concrete, structures, asphalt, or vegetated with landscaping. The area with an increased risk of soil erosion includes the former landfill area, where steeper slopes, exposed/unvegetated soil, and low-lying areas that direct runoff (e.g., unlined drainage ditches, swales, and channels) may increase the potential for soil erosion.

Seismic Hazards

Surface Fault Rupture

Ground surface displacement, or surface rupture, caused by an earthquake is a major consideration in the siting of buildings in areas that are traversed by active faults. Surface rupture occurs when movement on a fault deep within the earth breaks through to the surface. Most surface faulting is confined to a relatively narrow zone several feet to tens of feet wide, making avoidance (i.e., building setbacks) the common mitigation method. Fault rupture typically follows preexisting faults, which are zones of weakness. Specific geomorphic features commonly coincide with the locations of repeated fault rupture. Thus, identification of active faults that might produce surface rupture requires (1) knowing the location of existing faults, and (2) evaluating recent fault activity. The most useful and direct method of evaluating fault activity is to document the youngest geologic unit faulted and the oldest unit that is not faulted to constrain the timing of the most recent surface offset on the fault.

As defined in Alquist-Priolo Earthquake Fault Zoning Act of 1972 (see Subsection 4.E.3, *Regulatory Setting*, below), a fault or fault zone is considered active under the provisions of the act if there is evidence of surface displacement within the last 11,000 years (Holocene time). A fault is thought to be "sufficiently active" if one or more of its segments or strands show evidence of surface displacement during Holocene time. A fault is considered "well-defined" if its trace can be clearly identified by a trained geologist at the ground surface or in the shallow subsurface, using standard professional techniques, criteria, and judgment (Hart and Bryant, 1997).

The northwest-trending City College fault is mapped crossing the center of the Project Site, but is considered not active (see Figure 4.E-6) (Bonilla et al., 1998). This fault is defined as a pre-Quaternary fault (older than 1.8 million years), with no associated seismicity, and therefore lacks recognized Quaternary displacement or shows evidence of no displacement during Quaternary time. Faults older than 10,000 years are not considered active. Observations of historic ground failures in Northern California triggered by major earthquakes from 1800 to 1970 indicate that no movement on the City College fault at the Project Site was observed and/or recorded during that time. Based on the lack of evidence for active faulting along the City College fault, the potential for surface rupture at the Project Site is judged to be low.

Groundshaking

An earthquake produces seismic waves that emanate in all directions from the fault rupture surface. The seismic waves cause strong groundshaking, which is typically strongest near the fault and diminishes (attenuates) as the waves move through the earth away from the fault. The magnitude of an earthquake is a measure of the seismic waves or energy released by the earthquake. The severity of groundshaking at any particular point is referred to as "intensity" and is a subjective measure of the effects of groundshaking on people, structures, and earth materials. Groundshaking intensity commonly is measured using the Modified Mercalli scale, which provides a means of correlating felt effects of an earthquake to the size (magnitude) of an earthquake (see Table 4.E-4).

The severity of groundshaking at a particular site is controlled by the interaction of several factors, including the distance from the earthquake source, earthquake magnitude, and the type, thickness, and condition of underlying geologic materials such as bedrock, sediment, soils, and man-made fill. Recent research has shown that areas underlain by unconsolidated, recent alluvium and/or man-made fill may amplify the strength and duration of strong ground motions during major earthquakes, increasing the risk of damage. During the Loma Prieta earthquake in 1989, ground motion locally was amplified up to four times.

The distribution of near-surface geologic materials for the Project Site is shown in Figure 4.E-6**,** and the accompanying Table 4.E-1 summarizes the characteristics of these geologic materials. The characteristics of these materials suggest that they describe the NEHRP Classifications D and E (see Table 4.E-3). These soil classifications are expected to amplify strong groundshaking (see Figure 4.E-10).

Strong groundshaking caused by fault movement during an earthquake has the potential to result in significant life and safety hazards and property damage throughout the City of Brisbane.

Maximum groundshaking for the Project Site would be expected to result from a large earthquake on the nearby San Andreas fault, although strong groundshaking may also occur as a result of moderate or large earthquakes on other faults in the San Francisco Bay region (see Table 4.E-2 and Figure 4.E-9).

The predicted maximum earthquake intensity for the site is characterized as “very violent” by the USGS (Borcherdt, 1975) and “very strong” (Modified Mercalli Intensity VIII) by the Association of Bay Area Governments (ABAG) (ABAG, 2007). The estimate of the range of peak ground accelerations (PGA) expected in the vicinity of the Project Site is between 0.5 to 0.6 g PGA within a 475-year period (10 percent in 50 years; see Table 4.E-5) (USGS/CGS, 2002).

Moving ground accelerates during earthquakes and imposes forces on buildings. Structural engineers use the horizontal acceleration to design buildings. Peak ground acceleration generated in the vicinity of the Project Site by the 1989 Loma Prieta earthquake was about 0.1g (Geosyntec, 2006) (see Figure 4.E-8). The epicenter of the Loma Prieta event was about 50 miles southeast of the Project Site, whereas the San Andreas fault is five miles west of the site (see Figure 4.E-8). The proximity of the site to the San Andreas fault and other nearby faults increases the probability of very strong ground motion on the site during a major earthquake (Geosyntec, 2006).

Liquefaction and Lateral Spreading

Liquefaction

The potential for liquefaction depends on both the susceptibility of a deposit to liquefy and the opportunity for ground motions to exceed a specified threshold level. Liquefaction susceptibility is the relative resistance of a deposit to loss of strength when subjected to groundshaking. Loss of soil strength can result in ground failures at the earth’s surface. These failures, including localized ground settlement and lateral spreading, can cause significant property damage.

Physical properties of surficial deposits govern the degree of resistance to liquefaction during an earthquake. These properties include sediment grain-size distribution, density, cementation, saturation, and depth. Sediments that lack resistance to liquefaction (susceptible deposits) commonly include saturated young sediments that are sandy and loose. Sediments resistant to liquefaction include older surficial deposits that are dry or sufficiently dense.

Unconsolidated, water-saturated sand is most likely to liquefy under seismic stress. Water in pores between sand grains is compressed again and again during groundshaking until the water moves the grains apart and the soil loses its strength. If the grains are cemented together or well packed with silt- or clay-sized grains, or if water does not fill all the available pore space between grains, liquefaction is not as likely to occur.

The sandy alluvial saturated sediment underlying the Young Bay Mud at the Project Site is relatively dense and cohesive and has the potential to resist liquefaction. Saturated artificial fill and younger sandy deposits within and overlying Young Bay Mud, on the other hand, may be susceptible to liquefaction.

Figure 4.E-11 shows that the liquefaction hazard at the Project Site is very high according to maps of Quaternary (less than 1.8 million years old) deposits and liquefaction susceptibility prepared by the USGS (USGS, 2006b). Various geotechnical investigations at the Project Site have confirmed the presence of potentially liquefiable deposits in subsurface materials (Treadwell & Rollo, 2008). Geosyntec (2006) stated that the potential for surface manifestations of liquefaction of underlying material beneath the former landfill area "are expected to be somewhat limited" due to the depths of the sandy materials underlying the landfill area (Geosyntec, 2006). Conversely, Treadwell & Rollo, Inc. (2008) conducted preliminary liquefaction susceptibility analysis of the former railyard area and concluded that sandy layers in historic fill and sand within native deposits beneath the area are susceptible to liquefaction and capable of producing from 0 to 4 inches generally, and up to 8 inches of liquefaction-related settlement.

Lateral Spreading

Lateral spreading is a ground-failure condition induced by liquefaction where a slide plane develops within the liquefied sediment layer, causing the overlying soil to move. Lateral spreading generally occurs toward a free-face (e.g., a slope along a creek) or down a gentle ground slope. According to the Treadwell & Rollo investigation of the former railyard site, constructed slopes created through grading can be considered to be an unsupported face that could potentially be susceptible to lateral spreading and would require site specific evaluation (Treadwell & Rollo, 2008).

Earthquake-Induced Settlement

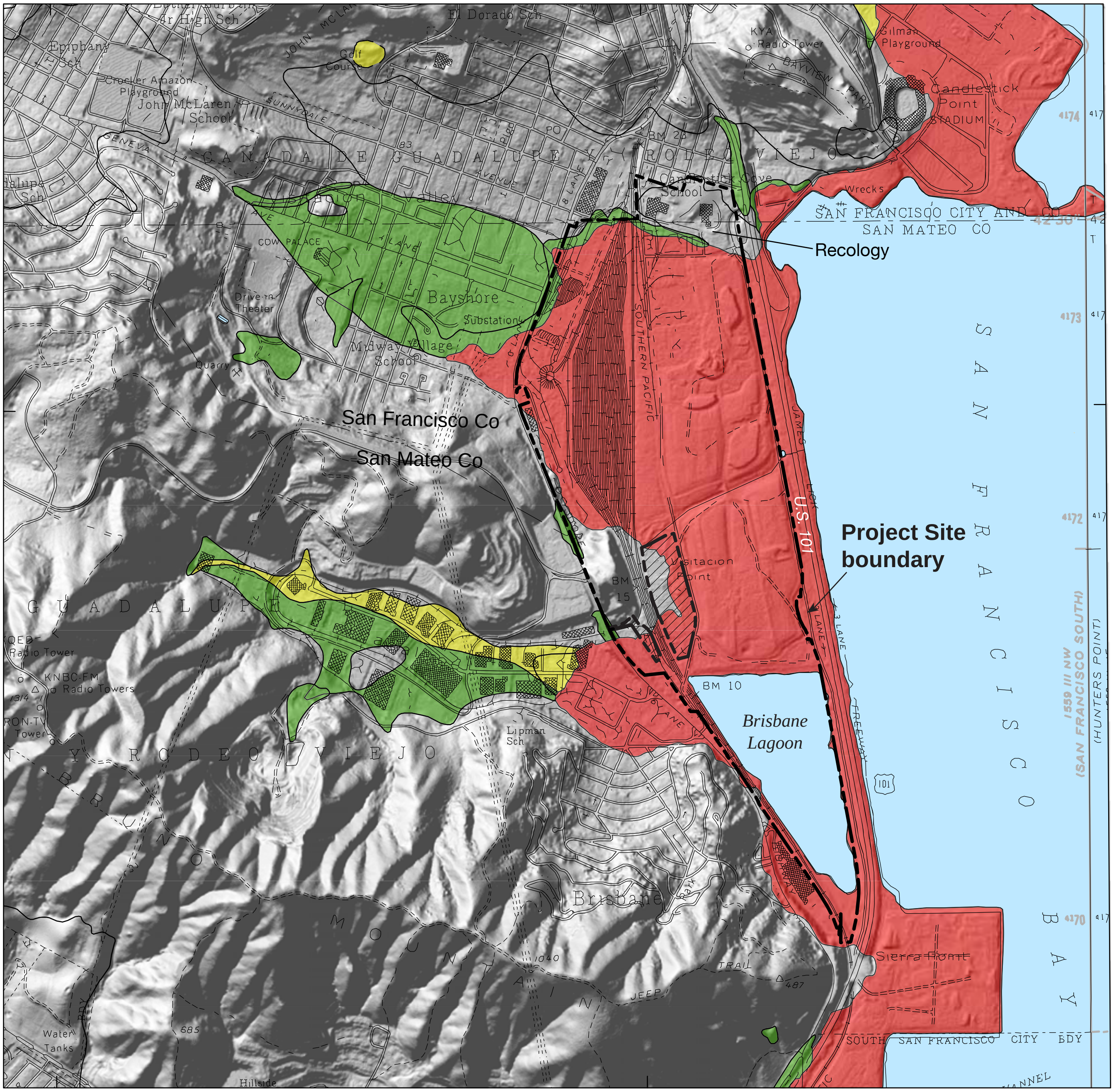
Strong ground motion can cause seismic settlement of dry, mostly cohesionless soils that make up the upper part of the landfill. Seismic settlement is typically induced in sandy deposits due to liquefaction.

A preliminary evaluation of earthquake induced settlement was completed for the former railyard area by Treadwell and Rollo, Inc. (2008). These results suggested that up to eight inches of earthquake-induced liquefaction-related settlement is possible. As mentioned above, the geotechnical evaluation for the landfill area of the Project Site concluded that seismic settlement was expected to be somewhat limited or relatively minor due to the depths of the sandy layers (Geosyntec, 2006).

4.E.3 Regulatory Setting

Development within the Project Site boundaries must comply with federal, state, regional, and local regulations. This section discusses these requirements to the extent that they affect the way Project development will occur.

Geologic, soil, and seismic conditions at the Project Site are subject to a variety of federal, state, regional, and local regulations, as discussed below.



Area not a part of Proposed Project

Liquefaction Susceptibility
(USGS, 2006B)

- Very High
- High
- Moderate
- Low

0 500 1000 m
0 2000 4000 ft

Brisbane Baylands . 206069

SOURCE: Fugro Consultants, Inc., 2011

Figure 4.E-11
Liquefaction Susceptibility Map
of the Project Site Vicinity

Federal Regulations

Earthquake Hazards Reduction Act

The Earthquake Hazards Reduction Act was enacted in 1997 to "reduce the risks to life and property from future earthquakes in the United States through the establishment and maintenance of an effective earthquake hazards and reduction program." To accomplish this, the act established NEHRP, amended in November 1990, which refined the description of agency responsibilities, program goals, and objectives. NEHRP's mission includes improved understanding, characterization, and prediction of hazards and vulnerabilities; improvement of building codes and land use practices; risk reduction through post-earthquake investigations and education; development and improvement of design and construction techniques; improvement of mitigation capacity; and accelerated application of research results. NEHRP designates the Federal Emergency Management Agency as the lead agency of the program and assigns it several planning, coordinating, and reporting responsibilities. Programs under NEHRP help inform and guide planning and building code requirements such as emergency evacuation responsibilities and seismic code standards such as those to which the proposed project would be required to adhere.

State Regulations

The California Department of Conservation, CGS compiles, updates, and maintains information regarding regional and local geologic conditions. This task includes mapping potentially active and known active faults and seismic evaluations under the Alquist-Priolo Earthquake Fault Zoning Act (Public Resources Code Sections 2621–2630). CGS also defines Seismic Hazard Zones where amplified groundshaking, liquefaction, and earthquake-induced landsliding may occur and that will require site-specific geologic study under the Seismic Hazards Mapping Act of 1990 (Public Resources Code Sections 2690–2699.6). CGS makes this information available to other state and local agencies.

Alquist-Priolo Earthquake Fault Zoning Act

The Alquist-Priolo Special Studies Act of 1972 (recently renamed the Alquist-Priolo Earthquake Fault Zoning Act), Public Resources Code Sections 2621 through 2630, mandates the identification and disclosure of areas of potential surface fault rupture and designates a Special Studies Zone (now called Earthquake Fault Zone) around each potentially active fault within which a geologic investigation must be completed prior to development. The purpose of the Alquist-Priolo Earthquake Fault Zoning Act is to prevent development of structures directly on top of active faults in order to mitigate the effects of surface fault rupture. Review of the maps indicates that the Project Site is not located within an Earthquake Fault Zone.

Seismic Hazards Mapping Act

Following the 1989 Loma Prieta earthquake, the California legislature enacted the Seismic Hazards Mapping Act (Public Resources Code Sections 2690–2699.6), which requires the State Geologist to create Alquist-Priolo-type zones where amplified groundshaking, liquefaction, and earthquake-induced slope failures are likely to occur. Unlike surface fault rupture, which typically is confined to a relatively narrow zone and requires a correspondingly narrow Earthquake Fault Zone, the effects of amplified strong ground motion, liquefaction, and earthquake-induced slope failure are more widespread in extent.

The maps currently are in the process of being prepared and will be completed for many parts of the Bay Area within the next decade. Mapping has not been officially released for the Brisbane Baylands area.

California Building Code

The California Building Code (CBC) has been codified in the California Code of Regulations (CCR) as Title 24, Part 2. Title 24 is administered by the California Building Standards Commission, which, by law, is responsible for coordinating all building standards. These regulations were adopted by the City, as provided under Brisbane Municipal Code Section 15.04.040, and are applicable to all Project Site development. Under state law, all building standards must be centralized in Title 24 or they are not enforceable. The purpose of the CBC is to establish minimum standards to safeguard the public health, safety, and general welfare through structural strength, means of egress facilities, and general stability by regulating and controlling the design, construction, quality of materials, use and occupancy, location, and maintenance of all building and structures within its jurisdiction.

The 2010 CBC is based on the 2009 International Building Code published by the International Code Conference. In addition, the CBC contains necessary California amendments, which are based on reference standards obtained from various technical committees and organizations such as the American Society of Civil Engineers (ASCE), the American Institute of Steel Construction, and the American Concrete Institute. ASCE Minimum Design Standards 7-05 provides requirements for general structural design and includes means for determining earthquake loads as well as other loads (flood, snow, wind, etc.) for inclusion into building codes. The provisions of the CBC apply to the construction, alteration, movement, replacement, and demolition of every building or structure or any appurtenances connected or attached to such buildings or structures throughout California.

The earthquake design requirements take into account the occupancy category of the structure, site class, soil classifications, and various seismic coefficients which are used to determine a Seismic Design Category (SDC) for a project as described in Chapter 16 of the CBC. The SDC is a classification system that combines the occupancy categories with the level of expected ground motions at the site and ranges from SDC A (very small seismic vulnerability) to SDC E (very high seismic vulnerability and near a major fault). Design specifications are then determined according to the SDC and in accordance with Chapter 16 of the CBC. Chapter 16, Section 1613 provides earthquake loading specifications for every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, which shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7-05. Chapter 18 of the CBC covers the requirements of geotechnical investigations (Section 1803), excavation, grading, and fills (Section 1804), load-bearing of soils (1805), as well as foundations (Section 1808), shallow foundations (Section 1809), and deep foundations (Section 1810). Chapter 18 also describes analysis of expansive soils and the determination of the depth to groundwater table. For SDCs D, E, and F, Chapter 18 requires analysis of slope instability, liquefaction, and surface rupture attributable to faulting or lateral spreading, plus an evaluation of lateral pressures on basement and retaining walls, liquefaction and soil strength loss, and lateral movement or reduction in foundation soil-bearing capacity. It also addresses mitigation measures to be considered in structural design, which may include ground stabilization, selecting appropriate foundation type and depths, selecting appropriate structural systems to accommodate anticipated displacements, or any combination of these measures. The potential for liquefaction and soil strength loss must be evaluated for site-specific peak ground acceleration magnitudes and source characteristics consistent with the design earthquake ground motions.

CCR Title 24 also includes the California Residential Code and the California Green Building Code, which have been adopted as separate documents (CCR Title 24, Part 2.5 and 11, respectively). The California Residential Code includes structural design standards for residential one- and two-family dwellings and covers all structural requirements for conventional construction. This part incorporates by adoption the 2009 International Residential Code of the International Code Council with necessary California amendments for seismic design. All other structures including multi-family residential projects are found in the other parts of the CBC as discussed above.

California Code of Regulations, Title 27, Section 21190

CCR, Title 27, Section 21190 pertains to development in or within 1,000 feet of active, inactive, and abandoned solid waste landfills. It requires that all proposed post-closure land uses be designed and maintained to:

(1) Protect public health and safety and prevent damage to structures, roads, utilities and gas monitoring and control systems;

(2) Prevent public contact with waste, landfill gas and leachate; and

(3) Prevent landfill gas explosions.

This regulation dictates various construction requirements for buildings including measures to mitigate the effect of differential settlement through use of flexible connections and utility collars for the placement of utilities.

McAteer-Petris Act

The McAteer-Petris Act (California Government Code Sections 66600–66694) is the California state law that established the San Francisco Bay Conservation and Development Commission (BCDC) as a state agency (see discussion under “Regional Regulations” below). The act prescribes BCDC’s powers, responsibilities, and structure and describes the broad policies to regulate development within 100 feet of the shoreline of San Francisco Bay.

Regional Regulations

Bay Conservation and Development Commission

The BCDC Administrative Regulations (14 California Code of Regulations Division 5, Sections 10110–11990) supplement and interpret the McAteer-Petris Act, the Suisun Marsh Preservation Act, the San Francisco Bay Plan, the Suisun Marsh Protection Plan, the federal Coastal Zone Management Act, the California Environmental Quality Act, and the State Coastal Conservancy Act as they apply to the regulation, planning, and management of the area within BCDC's authority and jurisdiction. BCDC is empowered to issue or deny permits, after public hearings, for any proposed project that involves placing fill, extracting materials, or making any substantial change in use of any water, land, or structure within the area of BCDC's jurisdiction.

Regional Water Quality Control Board

The RWQCB regulates the development and enforcement of water quality objectives and implementation of plans to protect the area's waters while "recognizing our local differences in climate, topography, geology and hydrology."

Under Order 01-041 from the RWQCB (2001), clay cap material must be maintained over landfill materials and undeveloped or open space areas. If the cap should be breached by any means (differential settlement, construction, plantings, etc.), adequate measures must be taken to keep the cap sealed.

San Mateo County Health System – Solid Waste Program

The San Mateo County Environmental Health Division's Solid Waste Program is responsible for ensuring that businesses, garbage collection and disposal companies, and individual residents follow federal, state and local standards and permitting requirements for proper handling and disposal of solid waste. The solid waste facilities and landfill sites are monitored for compliance with state minimum standards for the proper handling and disposal of solid waste to prevent the creation of public health and safety and environmental concerns. The program includes regulatory oversight of 16 closed landfills and oversight of post closure development (building construction on top of landfill sites).

Local Regulations

The Community Health and Safety Element of the City of Brisbane General Plan contains policies and programs pertaining to geology, soils, and seismicity that are relevant to the Project Site development. The following policy and programs relate to the seismic safety of structural improvements:

> ***Policy 149:*** Construct new buildings and retrofit existing ones to withstand seismic forces.
>
> > *Program 149a:* Require that all new construction meet current codes for seismic stability.

Program 149e: Require soils reports and engineering recommendations for structural stability in conjunction with building permit applications in areas which have been identified as prone to seismically-induced landslides or subsidence in seismic events.

The following policy and programs relate to the slope stability requirements:

Policy 152: Consider issues of slope stability in conjunction with development applications.

Program 152a: Require soil and geologic investigations in areas identified as prone to slope instability. Consider both on-site and off-site impacts.

Program 152b: Unless adequate mitigating measures are undertaken, prohibit land alteration, including any grading and structural development, in identified areas of slope instability.

Program 152e: Encourage placement of structures away from areas identified as prone to slope failure or erosion unless effective mitigation measures are proposed as a part of the project design.

4.E.4 Impacts and Mitigation Measures

Significance Criteria

Based on Appendix G of the CEQA Guidelines the project would have a significant effect on the environment if it were to:

- Expose people or structures to potential substantial adverse effects, including the risk of loss, injury, or death involving:
  - Rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area or based on other substantial evidence of a known fault;
  - Strong seismic groundshaking;
  - Seismic-related ground failure (including liquefaction); and/or
  - Landslides.
- Result in substantial soil erosion or loss of topsoil;
- Be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction or collapse;
- Be located on expansive soil, as defined in Table 18-1-B of the Uniform Building Code, creating substantial risks to life or property; or
- Have soils incapable of adequately supporting the use of septic tanks or alternative wastewater disposal systems where sewers are not available for the disposal of wastewater.

Because the Project Site contains corrosive soils, this document analyzes the suitability of the Project Site for placement of underground utilities. The project would have a significant effect on the environment if it would:

- Place concrete or steel elements, including piles that could be damaged by corrosive soils present within the Project Site.

Impact Assessment Methodology

General Approach

This section identifies the geologic and seismic impacts associated with the proposed Project Site development. Project Site development scenarios were evaluated against existing (2010) conditions to determine whether geologic or seismic effects would trigger any significant impacts based on the identified significance criteria. Feasible mitigation measures are identified to mitigate significant impacts. This section also identifies impacts that are considered to be less than significant.

Construction-related impacts including site grading (and site remediation to the extent it relates to geologic impact criteria) associated with implementation of the Project Site development infrastructure improvements described in the Chapter 3, *Project Description*, are included in the analysis below[5].

The geotechnical and seismic hazards associated with the Project Site development scenarios would present similar challenges regardless of the specific land use or density configurations. Therefore, the analysis of impacts associated with each of the four proposed development scenarios generally is grouped together in this section.

Project Site development scenarios would include the construction of an integrated sewer system across the Project Site. Therefore, the Project Site development would have no impacts related to soils being incapable of supporting septic systems or other alternative wastewater disposal systems. Furthermore, no known mineral resources are located within or near the Project Site. Mineral resource extraction activities have not taken place within or around the Project Site during recent history. The Project Site is mapped by CGS as MRZ-1, a zone where no significant mineral or aggregate deposits are present. Therefore, these issues are not addressed further in this analysis, consistent with CEQA Guidelines Section 15128.

Use of Previous Geotechnical Investigations

The geologic hazards present within the Project Site have been well studied and documented in numerous geotechnical investigations that have been performed at various areas of the Project Site by several different reputable geotechnical firms. The following analysis is based on available local geologic data and seismic hazards, as well as a review of the findings presented in geotechnical evaluations completed in 1990, 2006 and 2008 (Kleinfelder, 1990; Geosyntec, 2006; Treadwell & Rollo, 2008; and Geosyntec, 2008). As a result of these previous geotechnical studies, much is known about the underlying conditions including thicknesses of fills, Bay Mud and landfill waste. While there have been changes to some of the amount of fills on the surface in

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5 For a more detailed discussion of Project Site remediation, see Section 4.H, *Hazards and Hazardous Materials* of this EIR.

various areas, the underlying conditions of subsurface materials in terms of thicknesses and composition have not substantially changed from 1990 to the baseline of this analysis (2010) even if some consolidation and settlement may have occurred over that time period.

The presence of fills, soft compressible Bay Mud deposits, and landfill waste material at the Project Site presents significant hazards for un-engineered structures during ground shaking and/or conditions that would likely induce liquefaction. However, as is commonplace for construction in such an environment, use of established geotechnical design measures (discussed further below) have been successfully employed at many areas of similar geologic conditions. A sound geotechnical approach typically includes improvements to the foundation soils, such as compaction or densification, combined with a building foundation design that takes into account underlying soil properties. Individual foundation designs vary depending on the size and height of the structure proposed.

The geotechnical investigations completed to date provide a sound understanding of the geotechnical hazards present across the site for purposes of analyzing programmatic impacts under CEQA, but are not intended for site-specific construction design. The foundation system for each building site within the Baylands must be designed in accordance with the site specific engineering properties of the materials beneath the proposed structure, combined with the intended loading (weight) of the proposed structure. These design criteria can only be developed with information obtained from a site-specific geotechnical investigation that is conducted according to the requirements of the relevant regulations. For example, site-specific investigations would more accurately determine the depth of the fill materials and Bay Mud at each building site. The identified depths would influence whether shallow foundations or deep foundation pilings are appropriate, the number and dimensions of each deep foundation piling (a primary consideration for each building site), and the seismic design coefficients used by structural engineers to determine the type and sizing of structural building materials. Once appropriately designed and subsequently constructed in accordance with local and state building code requirements, the structures would have the structural fortitude to withstand anticipated seismic hazards without significant damage.

Project Impacts and Mitigation Measures

Impact 4.E-1: Would the Project expose people or structures to potential substantial adverse effects, including the risk of loss, injury, or death involving rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area or based on other substantial evidence of a known fault?

Impact Significance by Scenario (before Mitigation)			
DSP	**DSP-V**	**CPP**	**CPP-V**
LTS	LTS	LTS	LTS

SU = Significant Unavoidable
SM = Significant but Mitigable
LTS = Less than Significant
- = no impact

DSP, DSP-V, CPP, and CPP-V

As discussed in the preceding sections, no known active fault traces across the Project Site, and the site is not located in an Alquist-Priolo Earthquake Fault Zone.

Conclusion: The Project Site development would have a less-than-significant impact in relation to this criterion. No mitigation would be required.

Significance Criterion – Seismic Groundshaking

Impact 4.E-2: Would the Project expose people or structures to potential substantial adverse effects from strong seismic groundshaking?

Impact Significance by Scenario (before Mitigation)			
DSP	**DSP-V**	**CPP**	**CPP-V**
SM	SM	SM	SM

SU = Significant Unavoidable
SM = Significant but Mitigable
LTS = Less than Significant
- = no impact

DSP, DSP-V, CPP, and CPP-V

The Project Site would likely experience at least one major earthquake (M 6.7 or higher) within the next 20 years. The intensity of such an event would depend on the causative fault and the distance to the epicenter, the depth of the rupture below ground surface, the moment magnitude, and the duration of shaking. A seismic event in the Bay Area could produce considerable ground accelerations within the Project Site. Earthquake hazard mapping for the Project Site indicates that violent to very violent (MM IX to MM X) groundshaking (ABAG, 2007) and peak ground accelerations of 0.565(g) (CGS, 2012) would potentially occur at the Project Site. The 1989 Loma Prieta earthquake caused damage within the area with an epicenter located approximately 50 miles away. A larger earthquake with a closer epicenter could cause even greater groundshaking and damage. The geotechnical studies prepared for the Project Site development (and described in Subsection 4.E.2, *Environmental Setting*, above) provide recommendations to minimize adverse effects from seismic groundshaking on the Project Site.

Geotechnical engineering methods for building design, underground utilities, and roadways (including bridge crossings) in accordance with CBC requirements have been used throughout the Bay Area in areas where similar challenges of development on thick deposits of Bay Mud and imported fills have been encountered. In addition, impacts from a major seismic event would be further reduced by carrying out the site-specific analyses required by the CBC and the City Engineer. Chapter 16, Section 1613 of the CBC provides earthquake loading specifications for every structure and associated attachments that must also meet ASCE 07-05.This approach of preparing site-specific investigations is standard practice within the geotechnical engineering industry and required by Chapter 18, Section 1803 of the CBC. Site-specific investigations are used to obtain site-specific data, such as the depths of artificial fill and Bay Mud, to be considered along with the proposed loading (size of building) that would overlie that area. Engineers would use this information to identify the appropriate design parameters for the spacing and dimensions of the foundation systems for each specific structure within the Project Site.

The geotechnical report required by **Mitigation Measure 4.E-2a** would provide site-specific construction methods regarding grading activities, fill placement, soil corrosivity/expansion/ erosion potential, compaction, foundation construction, drainage control (both surface and subsurface), and avoidance of settlement, liquefaction, differential settlement, and seismic hazards. The report would also include stability analyses of final design cut and fill slopes, including recommendations for avoidance of slope failure(s). The final grading plan and associated development elements would be designed and constructed in accordance with requirements of the final design-level geotechnical investigation, and would be submitted to the City Engineer prior to the issuance of building permits. Designers and contractors would comply with recommendations of the design-level geotechnical investigation during Project construction. Additionally, a licensed geotechnical or soil engineer would monitor earthwork and construction activities to ensure that site-specific construction methods are followed during Project construction. The recommendations would be incorporated into all development plans submitted for site-specific development projects within the Baylands.

In general, deep foundation systems would be required for most Project Site structures to ensure that the proposed structures are founded on dense competent materials found at depth. The site-specific investigations would be used to determine the specific design of the foundation systems required for the specific design of each structure. The results of the site-specific investigations would include specifications to ensure that anticipated seismic groundshaking risk hazards at the Project Site are minimized.

In addition, strong groundshaking could potentially compromise the stability of the final landfill cap that is required as part of the landfill closure requirements and constructed as part of the Project Site development. In general, the final clay cap cover would be designed for potential groundshaking hazards in accordance with geotechnical standards such that protection of human health is maintained even in the event of a seismic event. Under Order 01-041 from the RWQCB (2001) the final clay cap material over the former landfill must be maintained over the landfill materials and undeveloped or open space areas. If the cap should be breached by any means (differential settlement, construction, plantings, etc.), adequate restorative measures are required by Order 01-041 to maintain the integrity of the cap.

The landfill owner is required to comply with California Code of Regulations, Title 27, Section 21130(c), which requires the operator to amend emergency response plans in the event that post-closure land use and/or structures on the site change and these changes are not addressed in existing plans.

Conclusion: This impact would be significant. **Mitigation Measures 4.E-2a** and **4.E-2b**, along with adherence to building code requirements and landfill closure requirements, are recommended to minimize impacts from strong seismic groundshaking for Project Site development.

Mitigation

Mitigation Measure Applicability by Scenario			
DSP	DSP-V	CPP	CPP-V
✓	✓	✓	✓

✓ = measure applies
- = measure does not apply

Mitigation Measure 4.E-2a: Prior to the issuance of a grading permit, applicants for all site-specific development and infrastructure projects within the Project Site, including structures, utilities, and roadways shall submit to the City Engineer a final design-level geotechnical report prepared by a licensed geotechnical or soil engineer experienced in construction methods on fill materials in an active seismic area. The report shall provide site-specific construction methods and recommendations regarding grading activities, fill placement, soil corrosivity/expansion/erosion potential, compaction, foundation construction, drainage control (both surface and subsurface), and avoidance of settlement, liquefaction, differential settlement, and seismic hazards in accordance with current California Building Code requirements including Chapter 16, Section 1613. The report shall also require that all subsurface improvements such as utilities that include any materials susceptible to corrosive effects would be engineered in conformance with the most recently adopted California Building Code requirements including the use of engineered backfill. The report shall also include stability analyses of final design cut and fill slopes, including recommendations for avoidance of slope failure(s). The final grading plan and associated development elements including the landfill cap layer shall be designed and constructed in accordance with requirements of the final design-level geotechnical investigation as approved by the City Engineer prior to the issuance of any building permits. Designers and contractors shall comply with recommendations of the design-level geotechnical investigation during project construction including any modifications required by the City Engineer. A licensed geotechnical or soil engineer shall monitor earthwork and construction activities to ensure that recommended site-specific construction methods are followed during Project construction. These recommendations shall be incorporated into all development plans submitted and approved for the Project Site development as conditions of approval.

Mitigation Measure Applicability by Scenario			
DSP	DSP-V	CPP	CPP-V
✓	✓	✓	✓

✓ = measure applies
- = measure does not apply

Mitigation Measure 4.E-2b: To address recovery from damage to future structures and to the landfill itself that may be caused by future earthquakes[6], a Post-Earthquake Inspection and Corrective Action Plan (Plan) for the site-specific development projects within the former landfill portion of the Project Site shall be prepared and implemented by all Project applicants in accordance with Title 27 landfill closure requirements as approved by the RWQCB and the San Mateo County Department of Environmental Health prior to issuance of a building permit. The plan shall be implemented in the event of a magnitude 7.0 or greater earthquake centered within 30 miles of the former Brisbane Landfill. Results of the inspection of containment features and groundwater and leachate control facilities potentially affected by any static or seismic deformations of the landfill shall be reported to the RWQCB within 72 hours of the event. Immediately following an earthquake event causing damage to the landfill structures, the Plan shall be implemented and the RWQCB notified of any damage. Plan activities following a triggering event shall include assessing perimeter dikes and shoreline erosion

6 Because the required plan addresses specific structures that will be located and designed as part of subsequent actions, and also addresses specific yet to be approved by the RWQCB measures related to landfill closure, it cannot be prepared until after specific structures have been designed and a landfill closure plan has been approved.

protection measures, the surface locations of underground utilities, landfill cover including roads and parking areas, groundwater monitoring systems, leachate monitoring systems, and surface-water drainage and outlet facilities. Any restorative measures as required under Order 01-041 shall be implemented in accordance with RWQCB requirements.

Conclusion with Mitigation: With the inclusion of **Mitigation Measures 4.E-2a** and **4.E-2b**, impacts related to strong seismic groundshaking associated with Project Site development would be reduced to less-than-significant levels.

Impact 4.E-3: Would the Project expose people or structures to potential substantial adverse effects from seismic-related ground failure including liquefaction?

Impact Significance by Scenario (before Mitigation)			
DSP	**DSP-V**	**CPP**	**CPP-V**
SM	SM	SM	SM

SU = Significant Unavoidable
SM = Significant but Mitigable
LTS = Less than Significant
- = no impact

DSP, DSP-V, CPP, and CPP-V

According to general maps compiled by the USGS and preliminary geotechnical investigations at the Project Site, there is a potential risk from liquefaction of saturated sand layers within existing fill, Young Bay Mud, and below Young Bay Mud beneath the Project Site. Liquefaction at the site could result in loss of bearing pressure, lateral spreading, sand boils (liquefied soil exiting at the ground surface), and other potentially damaging effects if not addressed in geotechnical engineering design. Analysis of site-specific soils data determined that liquefaction susceptibility at the former railyard area was relatively high (Treadwell & Rollo, 2008). In contrast, the Geosyntec (2008) report suggests that the liquefaction risk for the Project site is low because of the depth to the sand and the type of subsurface material (i.e., clayey soils); however, further investigation is necessary to pinpoint the site-specific liquefaction risk (Geosyntec, 2008). As discussed above, site-specific investigations would be required for all Project Site development to determine the site specific risk and appropriate foundation system design.

The site is underlain by Bay Mud that has a high potential to amplify ground shaking and can contain saturated sand lenses that are susceptible to liquefaction. The landfill portion of the Project Site may be more susceptible to liquefaction because it contains unknown buried materials that may be prone to liquefaction during strong ground shaking. The final design-level geotechnical report as required by the City Engineer and the California Building Code (Chapters 16 and 18) would be prepared by a licensed professional and submitted to the City for review and approval. As is standard for the geotechnical industry, the final design-level geotechnical report would address liquefaction and lateral spreading potential at each development site and provide site-specific recommendations to reduce the potential damage in accordance with building code requirements. The report recommendations would be incorporated into all site-specific development plans submitted within the Baylands.

Conclusion: Because the potential for liquefaction may be present at the site and would require site-specific analysis to determine the amount of potential settlement that could occur, this impact would be significant. **Mitigation Measure 4.E-3** is recommended to minimize impacts under all of the proposed development scenarios.

Mitigation

Mitigation Measure 4.E-3: The final design-level geotechnical investigation recommended in Mitigation Measure 4.E-2a above, to be prepared by a licensed professional and submitted to the City for review and approval, shall address liquefaction potential. The geotechnical investigation shall include recommendations for foundation design to address site specific potential liquefaction issues. The recommendations of the investigation shall be in accordance with the most recent California Building Code requirements for building design and incorporated into all development plans submitted for the Project Site development. All final design and engineering plans submitted by the applicant shall be subject to review and approval by the City of Brisbane Building Official.

Mitigation Measure Applicability by Scenario			
DSP	DSP-V	CPP	CPP-V
✓	✓	✓	✓
✓ = measure applies - = measure does not apply			

Conclusion with Mitigation: With the inclusion of **Mitigation Measure 4.E-3**, implementation of the Project Site development would result in a less-than-significant impact related to liquefaction.

Impact 4.E-4: Would the Project expose people or structures to potential substantial adverse effects from landslides?

Impact Significance by Scenario (before Mitigation)			
DSP	DSP-V	CPP	CPP-V
SM	SM	SM	SM
SU = Significant Unavoidable SM = Significant but Mitigable LTS = Less than Significant - = no impact			

DSP, DSP-V, CPP, and CPP-V

Project Site development would require substantial re-grading activities that would include creating constructed slopes of fill materials. If not engineered appropriately, these constructed slopes could be subject to slope failure which could damage proposed improvements or potentially adversely affect local visitors, residents, or workers. Based on the proposed conceptual grading plan for the Project Site, both Geosyntec (2008) and Treadwell & Rollo, Inc. (2008) concluded the potential placement of engineered fill may cause underlying Bay Mud to fail. The underlying, or in some areas, exposed weak Bay Mud layer has the potential to fail under the proposed fills, which represent substantial additional loading (Treadwell & Rollo, Inc (2008). Geosyntec conducted slope stability analysis on a cross-section extending from the edge of Brisbane Lagoon approximately 1,000 feet toward the north for the former landfill area. The analysis showed that the extent of possible slope instability is estimated to extend from the edge of Brisbane Lagoon toward the site approximately 600 feet, increased from the 480 feet for the existing conditions. Geosyntec (2006, 2008) recommended that fill not be placed within 600 feet of the edge of Brisbane Lagoon and that the stability of the area be re-evaluated once final designs are available. Likewise, Treadwell & Rollo, Inc. (2008) concluded that placement of engineered fill may cause underlying Bay Mud to fail and recommended that additional subsurface exploration and static/seismic stability of the proposed slopes be analyzed prior to final design and construction.

Quantitative slope stability analyses (e.g., slope stability modeling) is a geotechnical modeling of slope stability to determine what is known as the slope factor of safety. Expressed as a numeric figure, the factor of safety represents a comparison of shearing forces (e.g. gravitational forces and internal pressures) versus resisting forces of the soil or bedrock. Therefore, the higher the factor of safety, the more stable the slope because it represents a determination of greater resisting forces present. Generally accepted geotechnical practices for the San Francisco Bay Area regard a slope safety factor of 1.5 as suitable for development under static or non-earthquake conditions. For pseudo-static or non-earthquake conditions a lower safety factor is typically used because a higher factor cannot be practically achieved. Therefore, a safety factor of 1.2 for pseudo-static conditions is generally accepted practice in the Bay Area.

Neither the Geosyntec nor Treadwell & Rollo, Inc. reports addressed the potential for dynamic (seismically induced) slope instability in a quantitative analysis at the Project Site in their preliminary geotechnical evaluations of the Project Site. Given that the soils are potentially unstable under static conditions, the soil beneath the Project Site is also likely unstable under dynamic conditions. Therefore, this impact would be significant.

Conclusion: Impacts related to slope stability risk would be significant. **Mitigation Measures 4.E-4a** and **4.E-4b** are recommended to reduce this impact to a less-than-significant level.

Mitigation

Mitigation Measure 4.E-4a: Site-specific development projects within the Project Site shall not place new fill materials within 600 feet of Brisbane Lagoon. All manufactured slopes shall require certification by a licensed geotechnical engineer to the satisfaction of the City Engineer that a factor of safety[7] of at least 1.5 for static conditions and 1.2 under dynamic conditions will be achieved.

Mitigation Measure Applicability by Scenario			
DSP	DSP-V	CPP	CPP-V
✓	✓	✓	✓
✓ = measure applies - = measure does not apply			

Mitigation Measure 4.E-4b: Site-specific development projects within the Project Site shall comply with Brisbane General Plan policy requirements and the most recent California Building Code requirements for slope stability, including Chapters 16 and 18 that require geotechnical investigations. The recommendations of the investigation shall be in accordance with the most recent California Building Code requirements for building design and incorporated into all development plans submitted for Project Site development. All final design and engineering plans submitted

Mitigation Measure Applicability by Scenario			
DSP	DSP-V	CPP	CPP-V
✓	✓	✓	✓
✓ = measure applies - = measure does not apply			

7 As noted above, the factor of safety represents a comparison of shearing forces (e.g. gravitational forces and internal pressures) versus resisting forces of the soil or bedrock. The higher the factor of safety, the more stable the slope because it represents a determination of greater resisting forces present.

by the Project applicant shall be subject to review and approval by the City of Brisbane Building Official prior to issuance of a building permit.

Conclusion with Mitigation: With the inclusion of **Mitigation Measures 4.E-4a** and **4.E-4b**, implementation of the Project Site development would not result in a significant environmental effect, because mitigation would include minimum standards for slope stability to reduce the risk from static and dynamic slope instability. This impact would be reduced to a less-than-significant level.

Impact 4.E-5: Would the Project result in substantial soil erosion or loss of topsoil?

Impact Significance by Scenario (before Mitigation)			
DSP	**DSP-V**	**CPP**	**CPP-V**
SM	SM	SM	SM

SU = Significant Unavoidable
SM = Significant but Mitigable
LTS = Less than Significant
- = no impact

DSP, DSP-V, CPP, and CPP-V

Construction Activities

Construction and remediation activities required for Project Site development, such as excavation, backfilling, grading, and placement of fill material for surcharging purposes can expose areas of loose soil. Grading activities alone would require movement of large quantities of soils with preliminary estimates of 4,475,000 cubic yards of cut, and approximately 3,397,000 cubic yards of fill.[8] In general, the grading would primarily consist of cuts from the landfill area, approximately 3,730,000 cubic yards that would be placed on the westerly, former railyard portion of the Project Site. If not properly stabilized or protected, these soils and fills could be subjected to soil loss and erosion by wind and storm water runoff. Concentrated water erosion, if not managed or controlled, can eventually result in significant soil loss. Excessive soil erosion can also eventually lead to damage of building foundations and roadways. At the Project Site, areas that are susceptible to erosion are those that would be exposed during the construction phase and along the shoreline where soil is subjected to wave action. However, construction contractors for the Project Site development would be required by law to obtain a National Pollutant Discharge Elimination System (NPDES) Permit for Discharges of Stormwater Associated with Construction Activities from the RWQCB-San Francisco Bay Region for all proposed construction as part of the proposed Project. Conditions of this permit would include preparation and implementation of a Storm Water Pollution Prevention Plan (SWPPP).

As also discussed in Section 4.H, *Hydrology and Water Quality*, of this EIR, a SWPPP includes specific construction-related best management practices (BMPs) to prevent soil erosion and loss of topsoil. BMPs implemented could include, but would not be limited to, physical barriers to prevent erosion and sedimentation, construction of sedimentation basins, limitations on work periods during storm events, use of swales, protection of stockpiled materials, and a variety of other measures that would substantially reduce or prevent erosion from occurring during

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8 The 4,475,000 cubic yards figure is inclusive of the 700,000 cubic yards that has been placed on the site since 2007 as part of the surcharging activities (BKF, 2011).

construction (see also the discussion in Section 4.H, *Hydrology and Water Quality*). Project Site development would not otherwise change drainage patterns to the extent that it would cause significant erosion resulting in damage to existing or proposed improvements. Construction impacts associated with earth movement also are discussed in Section 4.B, *Air Quality*, and Section 4.N, *Traffic and Circulation*, of this EIR.

Post-Construction Development

Once construction is completed, the interior areas of the Project Site would be largely developed, with the exception of open spaces which would be landscaped. As a result, few locations would be created that would be exposed to the forces that cause erosion. With implementation of the requirements of the NPDES permit and the associated SWPPP, the impact of erosion and loss of topsoil would be less than significant.

New development within the Baylands must adhere to Policy 152 of the Community Health and Safety Element of the Brisbane General Plan, which requires the consideration of slope stability in conjunction with development applications. Policy 152 requires, among other things, that soil and geologic investigations be done in areas identified as prone to slope instability. Therefore, in complying with the directives of Policy 152, erosion or loss of soil would be prevented.

Conclusion: With implementation of a SWPPP, which is required to be prepared and implemented under the NPDES General Construction Permit, and compliance with Brisbane General Plan Policy 152, impacts related erosion or loss of topsoil would be reduced to less-than-significant levels for Project Site development. **Mitigation Measures 4.H-1a** and **4.H1b** incorporate requirements for preparation and implementation of a SWPPP in relation to hydrology impacts of proposed site development (see Section 4.H *Hydrology and Water Quality*, in this EIR).

Impact 4.E-6: Would the Project be located on a geologic unit or soil that is unstable or that would become unstable as a result of the Project including landslide, lateral spreading, subsidence, liquefaction or collapse?

Impact Significance by Scenario (before Mitigation)			
DSP	**DSP-V**	**CPP**	**CPP-V**
SM	SM	SM	SM
SU = Significant Unavoidable SM = Significant but Mitigable LTS = Less than Significant - = no impact			

DSP, DSP-V, CPP, and CPP-V

Settlement would occur in the former landfill, as well as in the overlying non-engineered fill and in natural deposits (Young Bay Mud [YBM], Old Bay Mud [OBM], etc.). Settlement within the Project Site (in both the short and long term) is expected to be differential due to variances in deposit thickness and material properties. Additional fill placed within the Project Site as part of site development would increase total surface settlement (Geosyntec, 2008; Treadwell & Rollo, 2008). Consolidation of Bay Mud and tidal flat deposits and non-engineered artificial fill beneath proposed engineered fills may also be associated with differential settlement across the Project Site (Geosyntec, 2008; Treadwell & Rollo, 2008). Fill subsidence and settlement can affect long-term durability and maintenance requirements of roadways and underground utilities. Detailed geotechnical characterization and engineering analysis would be required to determine the composition and thicknesses of undocumented, non-engineered fills and underlying tidal deposits and to evaluate the settlement potential across the entire Project Site using similar assumptions and analytical methods from preliminary studies. Treadwell & Rollo (2008) indicated that consolidation settlement will occur between one and 30 years after fill placement and presented several mitigation concepts to reduce post-construction settlement.

Based on geotechnical data collected for the Project Site, it is estimated that 6 to 30 inches of settlement may occur in the former landfill area (Geosyntec, 2008) and 12 to 38 inches of settlement may occur in the former railyard area (Treadwell & Rollo, 2008). However, because these studies have different assumptions and methods for calculating the potential primary settlement, direct comparisons between settlement of the former landfill and railyard areas should not be made. Below is a discussion of each study.

Geosyntec (2008) performed settlement calculations for waste fill, Young Bay Mud, and portions of Old Bay Mud beneath the former landfill area. Based on laboratory data and settlement analyses, Geosyntec (2008) reported the following:

- Young and Old Bay Mud are generally to slightly over-consolidated in the northern portion of the former landfill area;
- Settlement is expected for waste fill, Young Bay Mud, and portions of Old Bay Mud; and
- Placement of fill over non-engineered fill and refuse would cause differential settlements.

Assuming site elevations of 21 feet or 26 feet, the total settlement estimates presented by Geosyntec (2008) assume use of wick drains to facilitate primary settlement in Young and Old Bay Mud and secondary settlement of municipal waste after use of deep dynamic compaction.[9] Geosyntec (2008) did not perform a site-specific liquefaction evaluation and concluded that the "surface manifestations [of liquefaction] are expected to be somewhat limited" (Geosyntec, 2008, p. 25). Geosyntec (2008) suggested that liquefaction-related settlement can be mitigated using features such as stone columns and compaction grouting as part of the final design concept.

Treadwell & Rollo (2008) considered consolidation of Bay Mud, post-liquefaction reconsolidation, cyclic densification, and compression of new/existing fills using laboratory data from Michelucci (2004) and Kleinfelder (1990). These settlement analyses indicated that primary consolidation of Bay Mud beneath the former railyard area is essentially complete, but that placement of up to 26 feet of new engineered fill would cause additional settlement in Bay Mud. Placement of additional fill may cause from one to six inches of settlement of existing fills and settlement associated with compression of proposed engineered fill will be from 0 to 1.6 inches (Treadwell and Rollo, 2008). Liquefaction and cyclic densification related to strong

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9 Wick drains are prefabricated vertical drains installed through soft soils that are designed to help remove moisture and allow compressible soils to consolidate rapidly prior to construction. Deep dynamic compaction also called "heavy tamping" is a geotechnical technique of consolidating soft soils through repeated systematic application of a heavy weight.

groundshaking may cause up to four inches (and up to eight inches in Kleinfelder boring resulting from liquefaction) of settlement.

With the ongoing decomposition of the in-place refuse and consolidation of the underlying Bay Mud, the landfill surface is expected to continue to undergo differential settlement. Considering future development on the former landfill area, differential settlement of the landfill surface will require detailed engineering analysis and design as future site-specific projects are proposed for development within the Project Site. As part of a design-level geotechnical report, as also described above, analyses of the depth, thickness, and liquefaction potential of saturated deposits would provide necessary information on possible surface effects associated with earthquake-induced settlement. These effects, if calculated to be a potential hazard, could be mitigated as part of the final site design through widely accepted geotechnical engineering practices. These practices include surcharging the area with fill prior to construction and site development, installing wick drains to increase rate of consolidation of Bay Mud, limiting thickness of new engineered fill, using light-weight fills, and using deep dynamic compaction to densify the near-surface fill. Treadwell & Rollo also suggested pile foundations for structures to reduce differential settlement.

The geotechnical design recommendation for construction of heavily loaded structures at the Project Site includes the use of pile foundations in order to minimize impacts of surface settlement on the structures (Treadwell & Rollo, 2008). Such piles may be up to 110 feet deep. The surface of the site, which includes landscaping, roads, structures, and utilities, would continue to settle as the soil compacts. Such settlement could damage improvements and/or change drainage on the Project Site if not engineered appropriately.

California Code of Regulations, Title 27, Section 21190 contains specific requirements for development on former solid waste landfills including***:***

- Enclosed basement construction shall be prohibited;
- Buildings and utilities shall be constructed to mitigate the effects of differential settlement. All utility connections shall be designed with flexible connections and utility collars;
- Utilities shall not be installed in or below any low permeability layer of final cover;
- Pilings shall not be installed in or through any bottom liner unless approved by the RWQCB; and
- If pilings are installed in or through the low permeability layer of final cover, then the low permeability layer must be replaced or repaired.[10]

The requirements of Title 27, Section 21190 are mandatory. However, there are a variety of alternative measures that could be imposed to meet the Title 27, Section 21190 standards. The potential for consolidation settlements can be addressed prior to development through several

10 Considering that there may be potential resistance to achieving desired pile depth due to encountering large waste materials or debris, there are a number of different methods such as pre-drilling, placement of a concrete plug at the bottom of the pile, using a larger capacity hammer, or other measures.

different methods including: (1) surcharging[11] the site with fill prior to the construction of the proposed improvements; (2) installing wick drains to increase the rate of consolidation-related Bay Mud settlements where approved by the regulatory agencies; (3) limiting thicknesses of new fill; (4) using light-weight fills; and (5) using soil improvement techniques, such as deep dynamic compaction to increase the density of the near-surface fill or grouting techniques to reduce the potential for settlements associated with liquefaction and cyclic densification.

In addition, proposed structures may be supported on pile foundations to limit total and differential settlements. Surcharging accelerates the amount of settlement that would normally occur with development so that the majority of anticipated settlement occurs prior to initial construction. Prefabricated vertical drains, also known as wick drains, can be used to significantly decrease surcharge durations from years to months and would be specified as part of the surcharging process for specific development sites where appropriate. Wick drains allow pore waters that are being dissipated by the new loads to drain away more quickly, allowing settlement to occur faster. Any of the aforementioned geotechnical approaches to reducing the potential for settlement would be in accordance with building code requirements and subject to review and approval by the City Engineer prior to issuance of a building permit as discussed above in Impact 4.E-2.

Potential impacts related to landslides are addressed above in Impact 4.E-4 and lateral spreading/ liquefaction in Impact 4.E-3. The potential for Project Site development to cause or be subject to collapse is considered to be very low at the site based on the known characteristics of underlying materials at the site. Collapse is considered to have a greater potential in soils with high porosities, low densities such as windblown silt deposits known as Loess which are often found in more arid climates. The materials at the Project Site are denser and not considered susceptible to collapse.

Conclusion: While preliminary ground settlement estimates have been made, final parcel-specific ground settlement calculations are not available at this time, and cannot be determined until more detailed grading plans for site-specific development are available. Because it is known that some degree of ground settlement would occur, this impact is considered significant. Implementation of **Mitigation Measure 4.E-2a**, which requires that all structures be designed and constructed in conformance with the most recently adopted California Building Code requirements, which set performance standards for building design in areas undergoing compaction, and that all final design and engineering plans be submitted by the licensed geotechnical engineer and subject to review and approval by the City Engineer to confirm that Project Site development meets those performance standards, is recommended for each of the proposed development scenarios.

Conclusion with Mitigation: Implementation of **Mitigation Measure 4.E-2a** would reduce ground settlement impacts to a less-than-significant level under all development scenarios.

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[11] Surcharging is the placement of temporary loads on areas susceptible to settlement prior to development.

Impact 4.E-7: Would the Project place concrete or steel elements including piles that could be damaged by corrosive soils present on the Project Site?

Impact Significance by Scenario (before Mitigation)			
DSP	**DSP-V**	**CPP**	**CPP-V**
SM	SM	SM	SM
SU = Significant Unavoidable SM = Significant but Mitigable LTS = Less than Significant - = no impact			

DSP, DSP-V, CPP, and CPP-V

Corrosive subsurface soils may exist in places within the Project Site and are especially likely wherever Bay Mud is encountered. As such, corrosivity of future engineered fill at the Project Site would require evaluation as part of site specific analysis of geotechnical hazards for buildings within the Project Site. Typically, use of imported engineered fill or reuse of suitable onsite materials, as determined by building code requirements, are resistant to corrosion. As described above, and in compliance with the CBC, final design-level site specific geotechnical evaluations would be submitted to the City for final approval which would include an assessment of potentially corrosive soils on the Project Site. Development elements would be designed and constructed in accordance with requirements of the final design-level geotechnical report and would be verified prior to the issuance of building permits. Based on that report, all concrete in contact with the soil would be designed in accordance with local building code requirements. All metals in contact with corrosive soil would be designed based on the results of the soil corrosivity testing and subsequent recommendations of the manufacturer or a corrosion engineer. The City Engineer would approve all final design and engineering plans prior to any construction.

Corrosivity is a geotechnical hazard that is assessed as part of standard geotechnical investigations required to conform to the most recently adopted CBC requirements for building design. All final design and engineering plans as submitted by the licensed geotechnical engineer would be subject to review and approval by the City Engineer, as required and discussed above in Impact 4.E-2. Therefore, with application of engineered fill and use of corrosion-resistant materials, that are part of widely accepted geotechnical practices, the potential for adverse effects from corrosion would be minimized.

Conclusion: Without final design and engineering plans for site-specific development within the Project Site which provide parcel-specific evaluation of the corrosion potential of native soils and the waste layer and since it is known that corrosive soils are present with the Project Site, this impact is considered to be significant. Implementation of **Mitigation Measure 4.E-2a** is recommended.

Conclusion with Mitigation: With implementation of **Mitigation Measure 4.E-2a,** impacts related to corrosive soils would be reduced to less-than-significant levels under all of the proposed development scenarios.

Impact 4.E-8: Would the Project locate structures on expansive soils as defined in Table 18-1B of the Uniform Building Code, potentially creating substantial risks to life or property?

Impact Significance by Scenario (before Mitigation)			
DSP	DSP-V	CPP	CPP-V
SM	SM	SM	SM

SU = Significant Unavoidable
SM = Significant but Mitigable
LTS = Less than Significant
- = no impact

DSP, DSP-V, CPP, and CPP-V

Soil conditions within the Project Site vary considerably, and expansive soils may exist in certain places, especially along Bayshore Boulevard, where Bay Mud is present beneath the fill. As recommended in previous geotechnical investigations for the Project Site, engineered fill or reused onsite materials would be used for placement beneath foundations and in utility trenches, provided they meet the non-expansive criteria found in the CBC. As required by **Mitigation Measure 4.E-2a**, a final design-level geotechnical report would be required to address the potential for expansive soils on each site-specific development within the Project Site, and to ensure that the performance standards set forth in the CBC are met. Development elements would be designed and constructed in accordance with requirements of the final design-level geotechnical report which include moisture content requirements along with minimum standards for expansion potential and would be submitted to the City for review and approval prior to the issuance of building permits. Characterization of the potential for expansive soil at the Project Site in accordance with standard geotechnical practices and building code requirements is required prior to issuance of building permits.

Although site conditions vary across the Project Site, there is a potential for expansive soils in areas of proposed improvements. Proposed development would include substantial earthwork activities including the placement of engineered fill materials. Evaluation of the potential for expansive soils and prevention of the placement of expansive fill materials is part of standard geotechnical investigations that are required to conform to the most recently adopted CBC requirements for building design.

Conclusion: Without final design and engineering plans based on parcel-specific evaluation of the expansion potential and since it is known that expansive soils are present with the Project Site, this impact is considered to be significant. Implementation of **Mitigation Measure 4.E-2a** is recommended for Project Site development.

Conclusion with Mitigation: With implementation of **Mitigation Measure 4.E-2a**, impacts related to expansive soils would be reduced to less-than-significant levels under all development scenarios.

Impact 4.E-9: Would the Project have soils incapable of adequately supporting the use of septic tanks or alternative wastewater disposal systems where sewers are not available for the disposal of wastewater?

Impact Significance by Scenario (before Mitigation)			
DSP	DSP-V	CPP	CPP-V
-	-	-	-
SU = Significant Unavoidable SM = Significant but Mitigable LTS = Less than Significant - = no impact			

DSP, DSP-V, CPP, and CPP-V

Wastewater services within the Project Site are currently provided by the Bayshore Sanitary District (BSD) in the area north of the Lagoon. BSD maintains wastewater collection facilities and contracts with the San Francisco Public Utilities Commission for wastewater treatment. None of the development scenarios would include the use of septic tanks or alternative wastewater disposal systems.

Conclusion: Project Site development would have no impact in relation to this criterion. No mitigation would be required.

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