Appendix E Geotechnical Evaluation

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November 11, 2008

Mr. Rick King
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Subject: Review of the Geo-Logic Associates June 2008 Geotechnical Evaluation for Proposed Vertical Expansion – Newby Island Sanitary Landfill, San Jose, California

Dear Mr. King:

This letter summarizes our review of the geotechnical evaluation that was recently completed to support a proposed vertical expansion at the Newby Island Sanitary Landfill, located in San Jose, California. As you know, Allied Waste Industries, Inc. (Allied) authorized our committee of Professor Jonathan D. Bray, Professor James K. Mitchell, Richard A. Mitchell, and Professor Timothy D. Stark to collaborate with Allied and Geo-Logic Associates (GLA) during GLA's work to complete the referenced evaluation. The scope of the committee's work included:

- Review of GLA's initial site characterization studies, laboratory testing, engineering evaluations, and conclusions;
- Development of recommendations for additional GLA site investigations, laboratory testing, and engineering evaluations to address uncertainties associated with the initial studies;
- Meetings and teleconferences to review the data and evaluations as the GLA work progressed; and
- Review of the GLA June 2008 report that incorporated the review committee recommendations and documented the geotechnical work completed to support the vertical expansion project.[1]

[1] Geo-Logic Associates, 2008, *Geotechnical Evaluation for Proposed Vertical Expansion, Newby Island Sanitary Landfill, San Jose, California*, unpublished report prepared for International Disposal Corporation of California, Inc., 2 vols., June.

Our comments on the June 2008 report are presented below and include general comments regarding the overall procedures and conclusions presented in the report, specific comments that pertain to selected sections portions of the report, and several minor corrections that should be addressed if the report is amended or revised in the future.

Findings

Based on our review, the GLA June 2008 report has incorporated the recommendations put forth by the review committee. The geotechnical characterization and analyses employed by GLA are consistent with the available data and with the state-of-practice in geotechnical earthquake engineering. The conclusions and recommendations in the report are supported by the results of the field investigations, the laboratory testing, and the analyses. There are, however, two areas of the report where it is our opinion that GLA's interpretations may be overly conservative. In particular:

- For the purposes of its evaluations and report, GLA assumed that a post-earthquake liquefied strength was applicable to sand horizons with a safety factor against liquefaction of 1.3 or less. Although this is consistent with California guidance, the approach is conservative because the liquefaction case histories used by various investigators to back-calculate values of liquefied strength all exhibited static factors of safety less than unity. Moreover, it is reasonable to assume that the liquefied strength would be mobilized only if the triggering safety factor is near unity. As a result of these considerations, deformation analyses that assume liquefied strengths for all horizons with a triggering safety factor less than 1.3 will tend to overestimate the potential seismic deformation and may also lead to overly conservative liquefaction mitigation requirements.

- Consistent with the review committee recommendations, GLA makes a sound argument on page 39 of the report for using a 6 inch running average of cone penetration test (CPT) data to identify potentially liquefiable strata in a particular boring.[2] On page 14 of the report, however, GLA states that a 2 inch layer of sand is sufficient to classify a 5 foot thick zone as containing sand strata as shown in Figure 5-1. Later on page 41, when describing Figures 7-3A to 7-3T, GLA states again that if only one 2 inch liquefiable

[2] This interpretation is conservative compared to the Boulanger et al. (1997) recommendation that the identification of potentially liquefiable strata should be based on 2 foot thick intervals. However, it was judged that such a large interval could miss potentially liquefiable strata at this site and it was agreed to use a minimum layer thickness of 6 inches to identify potentially liquefiable sand layers.

layer was identified in a 5 foot thick zone, the entire zone would be marked as liquefiable if the seismic demand exceeded its seismic resistance. Therefore, there is an inconsistency in using a minimum thickness of 2 inches to display results such as those shown in Figures 5-1 and Figures 7-3A to 7-3T when it would be more reasonable to focus only on sand layers that are at least 6 inches thick when evaluating liquefaction triggering, and more importantly, its consequences. In our opinion, although GLA's approach is conservative, it likely unnecessarily overemphasizes the importance of potentially liquefiable sand strata that are only 2 inches to 6 inches thick.

The preceding assumptions were appropriate to evaluate the technical feasibility of the project and to provide an upper-bound estimate of the magnitude of liquefaction mitigation that could ultimately be required. However, it is our opinion that final evaluation and design of liquefaction mitigation measures for future phases of expansion should be based on cell- or phase-specific investigations that consider only sand layers that are least 6 inches thick, that are relatively continuous, and that have safety factors of 1.1 or lower in the liquefaction triggering analyses.

Specific Comments

There are some anomalies and inconsistencies in the June 2008 report that we believe GLA should address or that should be considered as part of the final design analyses because they may raise questions and/or require additional discussion. These include:

1. The undrained shear strength (s_u) characterizations of the Young Bay Mud (YBM) and Old Bay Alluvium (OBA) are reasonable and consistent with the site data. However, there are several apparent anomalies with the strength evaluations and associated presentations. For example, Figure 5-18c shows a noticeable shift in the red line representing the modeled strength of the OBA compared to the other plots shown in Figures 5-18a, 5-18b, and 5-18d (i.e. Figure 5-18c suggests a higher strength model than the other plots). Additionally, the CPT interpreted shear strength in the YBM in Figure 5-18d appears to be relatively low compared to the plots shown in 5-18a, 5-18b, and 5-18c.

2. Figure 5-22 shows the variation in s_u with failure plane orientation and indicates that s_u in simple shear is about 0.78 times the s_u value obtained in triaxial compression. The vane shear test (VST) and CPT strength characterizations represent a simple shear mechanism and the strength models that GLA are comparing these data to in Figures 5-18(a-d) do not consider this relationship. This is noted on page 25 of the report and the report further

notes that the strength reduction due to anisotropy was incorporated into the stability analyses. For the purposes of clarity, a note to this effect could be added to Figures 5-18(a-d) to emphasize that the stability analyses incorporated appropriate strength reductions to account for anisotropy along the failure surface.

3. Figure 6-2 of the GLA report does not match Figure 2 of the Bray seismicity evaluation report in Appendix I-1 in the period range of 1 second to 3 seconds (i.e., the average spectrum of the suite of seven ground motions does not match the upper target as well in Figure 6-2 as in Figure 2 of the Bray report). Although this difference is not particularly significant, the GLA figure suggests the average spectrum is slightly below the target spectrum when it is actually above or very close to the target. We recommend that Figure 6-2 of the GLA report be revised to match Figure 2 of the Bray report.

4. The legend in Figure 5-1 indicates that all silt layers within the OBA are not susceptible liquefaction. Strictly speaking, this is an overstatement as there are a few silt layers within the OBA that appear susceptible to liquefaction according to the criteria proposed by Bray and Sancio (2006).[3] However, these silts appear to be isolated and deep so they are not likely a liquefaction-induced slope deformation problem. A minor note could be added to Figure 5-1 that states that a few low plasticity silt layers or lenses that are potentially liquefiable were identified by index property testing and Bray and Sancio (2006), but these soil layers are isolated and deep and they are not a concern with respect to liquefaction-induced slope movements.

5. The calculated seismic displacement of 0.8 cm for the geogrid-reinforced HDPE prescriptive cover was based on the Bray and Rathje (1998) 50 percent normalized maximum horizontal acceleration (MHA) values. This displacement appears to be relatively low given a static safety factor of 1.55 for the slope. It is our opinion that the cover displacement calculations should also consider the more conservative 16 percent exceedence values to capture some of the uncertainty in the procedure (i.e., use of the 16 percent exceedence values provides an upper, but still reasonable, estimate of the potential seismic displacement). Therefore, we recommend that the 50 percent and 16 percent exceedence normalized MHA values and the 50 percent and 16 percent exceedence seismic cover displacement values be used to calculate a range of seismic displacements. Under this approach, k_{max} = 0.315 to 0.389, k_y/k_{max} = 0.733 to 0.905, and

[3] For example, in Table 4-1 of the GLA Report, the (ML) soil at Elev. -45.5 ft in RB-2 has w_c/LL = 0.99 and PI = 3, the (CL-ML) soil at Elev. -52 ft in RB-12V has w_c/LL = 0.98 and PI= 4, and the (ML) soil at -70 ft in RB-12V has w_c/LL = 0.98 and PI = 1.

the cover displacement = 0.8 cm to 3 cm. Because this range is still somewhat lower than we would expect for cover sliding on a 3H:1V slope under sever strong shaking, we recommend that GLA confirm their k_y calculation.

Minor Corrections

1. On page 18, OCR is more properly defined as the ratio of the maximum past vertical effective stress to the current vertical effective stress.

2. Page 19 of the report states that the average OCR of the OBA is about 2.4. However, on page 24, the statement is made that the OBA OCR ranges from 1.5 to 2.0. This inconsistency should be corrected.

3. Page 19 of the report suggests that there is no discernable correlation between the OBA OCR and elevation across the site although the variation in OCR is greater above elevation -40 than below (this variation is shown in Figure 5-5). We note that the variation in OCR within the OBA can be explained by the lowering and rising of sea level which at times allowed the upper part of a layer within the OBA unit to be exposed to sun and air and become desiccated. With additional deposition of OBA materials, previously desiccated surface layers of OBA were buried. Therefore, it should be expected that there are several buried desiccated layers of soil within the OBA unit that have higher OCR values. These layers are more frequently found in the upper part of the OBA unit, so it is not surprising that higher OCR values are often found in the upper part of the unit.

4. The field VST gives peak and large displacement strengths, both of which are important parameters. In the text of the report (on pages 9 and 22 for example) only the measured peak strength values are discussed. For the purposes of context, we recommend that the sensitivity of the YBM and OBA be mentioned in future updates or amendments to the report.

5. The calibration curves and data for the CPT used on this project are not provided in the report. Because this information can be used to confirm some of the input parameters used in the liquefaction analyses, we recommend that these data be included in future reports or otherwise be made available.[4]

[4] Without this information, one cannot confirm, for example, that a = 0.85 for the CPT used on the project.

6. Figure 5-6 has black diamonds in its legend where it should have the symbol "σ."

7. On page 20, the laterally continuous sub-drainage layer beneath the landfill cell is mentioned. However, no information is provided regarding the relative density of this cohesionless material. This is important to mention so that its liquefaction susceptibility can be evaluated.

8. A note should be added at the bottom of Table 5-4 that indicates that the s_u/p' values are for the triaxial compression mode of shear and that these values were adjusted for strength anisotropy according to the relationship shown in Figure 5-22.

9. On Figure 5-12, the legend of "s_u/p' YBM/OBA shear strength model" should be removed.

10. The last paragraph on page 36 and continuing on page 37 indicates that the average PGA for the free field and dike profiles at the site is neither attenuated nor amplified by the soil column that overlies bedrock. However, the last sentence of this paragraph implies there is no attenuation or amplification of the shaking *intensity*. This statement is overly broad because PGA and its amplification or attenuation represents the spectral ordinate at zero period and the shaking intensity of a motion is not just characterized by PGA. Rather, spectral ordinates across the full range of periods better represents the shaking intensity of a motion and the long period energy in the ground motion was likely amplified for the design ground motions at this site. Accordingly, it would be more correct to revise the last sentence of this paragraph to state: "…suggesting neither attenuation nor amplification of the ~~shaking intensity~~ PGA by the soil column overlying bedrock, on average."

11. Page 40 of the report states that 10 years is a reasonable average age for the liquefaction case history database. This statement is overly broad with respect to the "average" age of the liquefiable layer in the liquefaction case histories that probably range in age from around 10 years to 100 years. We agree, however, that 10 years is a reasonable normalization age and suggest the sentence on page 40 be revised to state: "…the liquefaction triggering evaluation at the NISL used the Leon et al. (2006) methodology normalized to 10 years~~, a reasonable average age for the liquefaction case history database~~."

12. On page 44, the Abramson et al. (2002) reference should be Rathje and Bray (2001). There is no Abramson et al. (2002) paper in the reference section of the report and this

concept was explained in the Rathje and Bray (2001) paper that is referenced in Appendix I.

13. Page 48 of the report states that "catastrophic failure" is not anticipated if the -45 ft elevation sands liquefy. We agree that some minor cracking or ground disturbance may occur but believe that reference to catastrophic failure is unnecessary. Therefore, we suggest that the last sentence of this paragraph be deleted.

14. The figures associated with Table 6-5 occasionally show the profile of maximum shear strain vs. depth. However, this plot is not identified in the legend on the horizontal axis of the figure and the units are uncertain (i.e., is it shear strain in percent?).

15. On Figure 7-3A and many other similar figures, the legend states "Liquefiable… if Wet." To be correct, the legend should state "Liquefiable… if Saturated."

16. In the legend of Figure 8-2, it is stated that the "… surfaces are based on the average seismic displacement calculated using site-specific SHAKE …" It is important for the reader to know that the curves are based on the *average* of the seven seismic displacements calculated for a particular slope and k_y value.

<u>Summary</u>

Our review shows the GLA June 2008 report presents conclusions and recommendations that are consistent and supported by the results of the field investigations, the laboratory testing, and the analyses. As a result, we concur with these conclusions and recommendations. There are several areas where GLA incorporated conservative assumptions regarding the thickness and continuity of potentially liquefiable deposits and the liquefaction triggering safety factor that were appropriate to evaluate the technical feasibility of the project and to provide an upper-bound estimate of the magnitude of liquefaction mitigation that could ultimately be required. Therefore, it is our opinion that final evaluation and design of liquefaction mitigation measures for future phases of the NISL expansion should be based on cell- or phase-specific investigations that only consider sand layers that are least 6 inches thick, appear to be continuous, and that have safety factors of 1.1 or lower.

We appreciate the opportunity to support Allied on this challenging and interesting project. Please contact us if you have any questions or need additional information.

Very best regards,

Richard A. Mitchell
Review Committee Chairman

Dr. Jonathan D. Bray

Dr. James K. Mitchell

Dr. Timothy D. Stark

cc: Mr. Tony Pelletier/Allied
Mr. Gary Lass/GLA

GEOTECHNICAL EVALUATION FOR PROPOSED VERTICAL EXPANSION

VOLUME I

NEWBY ISLAND SANITARY LANDFILL
SAN JOSE, CALIFORNIA

JUNE 2008

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GEOTECHNICAL EVALUATION FOR PROPOSED VERTICAL EXPANSION NEWBY ISLAND LANDFILL SAN JOSE, CALIFORNIA

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1.0 INTRODUCTION

1.1 SCOPE OF EVALUATION

At the request of the International Disposal Corporation of California (IDC), **GeoLogic Associates (GLA)** has completed a geotechnical evaluation to assess the potential for extending the vertical height of the Newby Island Sanitary Landfill (NISL) in San Jose, California. In order to assess this proposed expansion, **GLA** has completed a review of pertinent site reports and geologic/geotechnical literature, a subsurface investigation, surface geophysics, laboratory testing, a site-specific seismic hazard analysis, an assessment of the potential for liquefaction at the site, static and dynamic slope stability analyses, static consolidation analyses, dynamic settlement analyses, evaluation of and recommendations for mitigation of potentially problematic conditions, and preparation of this report.

1.2 SITE DESCRIPTION

The Newby Island Sanitary Landfill (NISL) is located within an irregularly shaped, 342-acre property, bounded by a perimeter levee which is largely abutted by water. Approximately 29 acres of this property consist of sloughs and marshlands beyond the perimeter levee. Coyote Creek abuts the NISL perimeter levee on the east, northeast, and northwest, and an unnamed slough abuts this levee on the south and southwest (Figures 1-1 and 1-2).

The island perimeter levee was constructed in the late 1800's. The top of the levee varies from approximately elevation +14 feet (relative to mean sea level) along the south, to elevation +24 feet in the north. The pre-development site elevation was at or near mean sea level. Within the perimeter levee, elevations currently range from about -40 feet in un-constructed landfill cells areas up to about +140 feet in areas that contain municipal solid waste (MSW).

1.3 SITE HISTORY

Newby Island was reclaimed from tidal marshland by the construction of a perimeter levee system which was completed in the late 1800's. The island was used for agricultural production including orchards and pastureland until 1932.

In 1932, the Newby Island Improvement Company began using the island as an unlined solid waste disposal facility. Between 1931 and 1956, the disposal and incineration of solid waste took place in select northern and eastern portions of the island. After 1956, burning was discontinued and subsequent waste disposal practices were more conventional. The NISL is currently classified as a Class III solid waste management facility and accepts nonhazardous solid wastes, including residential, commercial, industrial, agricultural and inert wastes.

Prior to implementation of California Code of Regulations Title 23, Chapter 15 (Chapter 15), landfill cells were apparently constructed on an exposed subgrade that was excavated to about elevation -10 feet. Subsequent to approval of Chapter 15 regulations, a low permeability liner consisting of compacted low permeability subgrade was placed above the floor of the excavation and a leachate collection and removal system (LCRS) was also constructed (see Subareas 1 to 8 in Figure 1-3). In response to subsequent revisions to the California Code of Regulation (Title 27) and implementation of the Code of Federal Regulations (Subtitle D), more recent expansion areas in the south central portion of the landfill (i.e. Subareas 9A and greater) have incorporated a base and floor liner system that consists of compacted subgrade, overlain by a subdrain gravel layer, a low permeability soil barrier layer, an 80-mil high density polyethylene (HDPE) flexible membrane liner and an LCRS (Details 1 and 2 on Figure 1-4).

2.0 GEOLOGIC/SEISMIC SETTING

The NISL lies in a geologically complex area of central California. Descriptions of the regional and site geology, hydrogeology, faulting and seismicity are presented below in this section. More detailed descriptions of the site stratigraphy and site-specific seismicity are presented later in the text in Section 5.1.1 – Site Stratigraphy and Section 6. 0 – Site-Specific Ground Motion Evaluation, respectively.

2.1 GEOLOGY

2.1.1 REGIONAL GEOLOGIC SETTING

The NISL is located on former agricultural property surrounded by marshlands that border the southern end of the San Francisco Bay in the California Coast Range geomorphic province (Figure 2-1). More specifically, the site is located within the northwest-trending Santa Clara Valley structural trough, which is bounded on the southwest by the Santa Cruz Mountains and on the northeast by the Diablo Range (Figure 2-2).

Historically, the area of the present San Francisco Bay exhibited significant relief with deposition of fluvial sands, silts and interbedded clays being largely controlled by changes in sea level. Most of the older Plio-Pleistocene sediments consist of interbedded sands with silts and clays, and these soils were deposited at a time when worldwide sea levels were much lower with the ocean shoreline located near the present-day Farallon Islands. At that time, the combined flow of the Sacramento and San Joaquin Rivers probably flowed through the gap presently occupied by the Golden Gate Bridge with fluvial and estuary sediments deposited to the east [Atwater et al (1977) refer to sediments deposited during the Wisconsin Glacial Epoch as estuarine deposits]. Sea levels began rising about 15,000 to 18,000 years ago, after the end of the Wisconsin Glacial Epoch, and encroachment of the ocean into the present day San Francisco Bay began about 10,000 year ago. Over the last 5,000 years, sea levels have remained relatively static.

2.1.2 FAULTING

The geologic structures of the San Francisco Bay region, one of the most seismically active areas in the United States, are controlled by the major northwest-trending San Andreas fault system located within the Santa Cruz Mountains west of the bay (Figure 2-3). The Hayward and Calaveras faults, which are splays of the San Andreas fault, are located in the Diablo Range on the east side of the San Francisco Bay.

The NISL site is approximately 3.3 and 10.8 km (2.1 and 6.1 miles) southwest of the Hayward and Calaveras faults, respectively, and 24.9 km (15.5 miles) northeast of the San Andreas fault (Figure 1-1). No known active or potentially active faults transect the property.

While the likelihood of surface rupture and associated displacements within the site is relatively low, the potential for strong earthquake-induced ground motion necessitates further evaluation of ground motion parameters, including liquefaction potential. A further discussion of seismicity and faulting, including maximum credible earthquake (MCE) magnitudes and liquefaction potential, is presented below in Section 6.0 – Site-Specific Ground Motion Evaluation and in Section 7.0 – Liquefaction.

2.1.3 REGIONAL STRATIGRAPHY

Based on gravity surveys, the Santa Clara Valley is estimated to be filled with up to 3,000 feet of Plio-Pleistocene through Holocene-age fluvial and estuary deposits (Emcon, 1972). Bedrock beneath the alluvium of the Santa Clara Valley consists of highly sheared sandstone, shale, chert, conglomerate, and melánge of the Jurassic-Cretaceous Franciscan Formation (Purcell, Rhoades & Associates, 1985).

The Santa Clara Formation directly overlies the Franciscan Formation in many areas and is composed of poorly sorted pebbly sandstones, siltstones and clay which generally become finer grained and thicker from southeast to northwest. Overlying the Santa Clara Formation, are an interfingering sequence of clay, silt and sand deposits resulting form on-lapping and regression of the ocean throughout the Quaternary. Regionally, these sediments have been referred to by a variety of names based on hydrogeologic, stratigraphic and engineering properties. Treasher (1963) cites two sources that refer to these sediments as the Bay Mud while the Army Corps of Engineers refers to them as Younger Bay Mud. Sediments deposited in San Francisco Bay during the last 10,000 years are regarded to be estuarine in origin while sediments deposited from about 40,000 years to 10,000 years ago are generally regarded to be alluvial in origin.

For a discussion of site stratigraphy as interpreted from findings of recent subsurface investigations, see Section 5.1.1 – Site Stratigraphy

2.2 HYDROGEOLOGY

2.2.1 REGIONAL GROUNDWATER

The NISL is located within the Santa Clara Valley Groundwater Basin at the boundary of the Niles Cone and Santa Clara Subbasins (Figure 2-4). This groundwater basin is composed of coalescing distal alluvial fans that flank the San Francisco Bay and is composed of clayey material (aquicludes) and isolated sandy aquifers (Iwamura, 1995). The extensive nature of the regional aquifers in the Niles Cone subarea east of the site has made it possible to delineate the major water bearing units.

The major water-bearing units at and underlying the site is briefly described as follows (Figure 2-5):

Perched Water Zone: - Throughout the site, perched water conditions are common in the unconsolidated Young Bay Mud unit at depths typically less than +20 feet. A design report for Subarea 9 states that water from surface infiltration into about the upper 100 feet of bay mud (including the Young Bay Mud and Older Bay Alluvium, collectively termed the Newark Aquiclude) tends to be isolated or perched within pockets of relatively pervious, but discontinuous and sporadic, sandy material (PRA Group, 1993). Data collected from the current investigation support this interpretation and indicate that the occasional sandy units within the deeper Older Bay Alluvium unit do provide confined storage for small volumes of groundwater.

Newark Aquifer: - It has been reported that the regional Newark Aquifer is located within Older Bay Alluvium deposits at depths of 90 to 110 feet below ground level (PRA Group, 1993). Wells screened in this confined aquifer have been reported to have low yields (less than 5 gpm) of water which is typically saline (PRA Group, 1993). The aquifer is reported to consist generally of sandy clay with interbeds of sand and gravel. Two deep wells were drilled in separate locations at or near the NISL. One 246 foot-deep boring, drilled by Caltrans just beyond the eastern portion of the site in 1998, shows an approximately 4 foot-thick layer of sand with abundant silt at a depth of 108 feet below the ground surface (State of California, 2001; see Appendix A). The other deep boring, MW-DE, located in the northern portion of the site along the perimeter levee, shows 9 feet of fine sand at a depth of 80 feet below the ground surface (PRA Group, 1989). Both sand layers roughly correspond to the reported depth of the Newark Aquifer.

Centerville Aquifer: - The regional Centerville Aquifer is separated from the overlying Newark Aquifer by a thick clay aquiclude of Older Bay Alluvium. The top of the Centerville Aquifer is located approximately 200 to 250 feet below the ground surface and consists of sandy clay with interbeds of sand and gravel (PRA Group, 1993). This unit locally exhibits artesian conditions and is potable (Emcon, 1972). Boring MW-DE on the northern perimeter levee encountered a dense sand layer with some gravel from elevation -238 to -260, which corresponds to the Centerville Aquifer.

2.2.2 SITE GROUNDWATER

Groundwater at the NISL is locally influenced by both tidally-affected surface water (i.e. Coyote Creek and the unnamed slough) and on-site pumping for surface and ground water control. This pumping has resulted in an inward-directed groundwater gradient toward a temporary sump on the southern side of the landfill, where groundwater is about at elevation -40 (Figure 2-6).

2.2.3 SURFACE HYDROLOGY

Newby Island is bounded to the northwest, northeast, and east by Coyote Creek, which along with the Guadalupe River and Los Gatos Creek, drain 240 square miles of the Santa Clara Valley (State of California, Dept. of Water Resources, 2003). An unnamed slough located along the south and west boundaries of the site is a tidal channel; older maps, however, suggest that this slough has been a tributary of Coyote Creek. In the

vicinity of the island, both Coyote Creek and the unnamed slough are influenced by tidal fluctuations.

Within the Newby Island Landfill site, borrow and subgrade excavations have often extended into and below the adjacent shallow aquifer system creating seeps within the exposed temporary slopes. These seeps have typically been directed into temporary drainage swales which drain to holding ponds that are monitored and ultimately pumped into the surface water drainage systems outside of the perimeter levee. Runoff from the site itself is controlled by the placement of interim cover and through the construction of a series of temporary bench collectors and down drains which ultimately discharge into the holding ponds noted above.

2.2.4 LEACHATE

Mounded Leachate in the Northern Portions of the Landfill: Liquids derived from consolidation of underlying foundation soils, precipitation events, and/or from the overlying waste itself has resulted in development of a liquid mound within the older MSW cells in the northern portion of the NISL (Figure 2-7). This leachate mound currently rises to about elevation 40. In response to directives from the RWQCB, IDC is implementing a leachate extraction program intended to reduce leachate elevations to near sea level.

3.0 SUBSURFACE INVESTIGATIONS

3.1 PREVIOUS INVESTIGATIONS

Numerous geotechnical investigations have been completed at the Newby Island Landfill since Emcon's original 1972 studies. A listing of these historic investigations is included in the References section of this report and boring logs from known previous investigations are presented in Appendix A. The approximate locations of these historic borings are shown on Figure 1-2.

Relevant portions of previous investigations have been included herein and are referred to in the evaluations summarized in subsequent sections of this report.

3.2 RECENT FIELD INVESTIGATIONS

Although a number of subsurface investigations had previously been performed at the Newby Island site, a new round of exploration was completed for the proposed vertical expansion in order to supplement the existing stratigraphic record, to obtain samples for laboratory testing, and to perform additional, more sophisticated testing than had previously been completed. To facilitate boring location and other aspects of this investigation, a station line was established along the rough centerline of the perimeter levee (Figure 1-2). Table 3-1 presents a summary of all borings completed for the current investigation and the locations of these borings by approximate northing and easting coordinates and/or perimeter levee Stationing. Included on this table are also the ground surface elevation at the boring location, total boring depth, and comments, if applicable.

The current geotechnical field investigation at the NISL was performed in phases between 2004 and 2007 and consisted of surface mapping of landfill cell excavations, excavation of subsurface borings, installation of monitoring systems, surface and downhole in-situ testing. Borings included Cone Penetration Test (CPT) borings and mud-rotary borings. In-situ tests performed "down-hole" consisted of Standard Penetration Tests (SPT), Vane Shear Tests (VST), piezometric measurements, including CPT pore pressure dissipation tests, and Shear Wave Velocity (V_s) tests. In-situ V_s testing was also performed from the ground surface. A more complete description of the recent field investigations is presented below.

3.2.1 CONE PENETRATION TESTS

A total of one hundred and nine (109) CPTs were advanced at the NISL as part of the current investigation (the approximate locations of these CPTs are presented in Figure 1-2). These CPT borings are designated as CPT-1 through CPT-109. CPTs represent the primary method by which the geologic stratigraphy was interpreted in this investigation.

CPT borings were advanced to depths of between 56 to 100 feet below the ground surface by Gregg Drilling, Inc. using a truck-mounted electronic cone apparatus. CPTs

were completed under the supervision of a **GLA** geologist or engineer and were advanced in two phases during mid- to late-September 2004 and between April and May 2007. Borings CPT-1 to CPT-41 were performed in 2004 and were spaced approximately 250 to 500 feet apart along the southern perimeter of the site and somewhat further apart along the northern perimeter. Borings CPT-42 to CPT-109 were advanced in 2007 to provide more detailed stratigraphic information in critical areas and to assess the effectiveness of a compaction grout test program which was being considered for mitigation of liquefaction in relatively shallow sands.

Pore Pressure Dissipation Tests (PPDT) were conducted in distinct sand layers or lenses in roughly half of the CPT borings. The PPDTs, which measure the hydrostatic water pressure near the tip of the cone, were conducted during a halt in the advance of the CPT cone.

Details of the CPT test procedures and CPT boring logs are included in Appendix A. A summary of the CPT borings is presented in Table 3-1.

3.2.2 MUD-ROTARY BORINGS

A total of twenty-one 4-7/8-inch diameter mud-rotary borings were completed at the NISL during the current investigation. These borings are designated as RB-1 through RB-13. In most cases, rotary borings were located adjacent to previously-advanced CPT borings in order to provide validation of the CPT results, to allow for collection of discrete samples and/or to allow for performance of in-situ testing from various stratigraphic intervals. As such, sampling intervals varied from continuous to widely-spaced depending on the objectives of a particular boring. The depth of rotary borings ranged from 10 to 127 feet, with the majority on the boring depths in the 75- to 100-foot range. Rotary borings were advanced by Pitcher Drilling and were supervised and logged by a **GLA** geologist. Boring logs are presented in Appendix A and summarized in Table 3-1.

The mud-rotary borings completed for the current investigation were excavated in three phases in September 2004, July 2005, and April/May 2007. In the first phase of drilling, continuous SPT sampling was generally performed across sandy lenses or layers, while in clayey soils samples were obtained on 5- to 10-foot intervals with Shelby tube and/or modified California samplers. In the second phase of drilling, the sampling was much more widely spaced and was performed only with Shelby tube samplers. Some of the second phase borings were advanced for the purpose of in-situ testing (i.e. VST or V_s testing) or piezometer installation, and, therefore, no samples were recovered. In the third phase of drilling, four rotary wash borings were excavated in order to install piezometers and one sampled-CPT boring (Boring NB-5) was completed to facilitate collection of soil samples for geologic age dating.

3.2.3 VANE SHEAR TESTING

The vane shear device is useful for making in-situ shear strength measurements in very soft (cohesive) soil deposits. The device consists of two orthogonal blades on a rod which is inserted into the soil deposit and rotated at a constant angular rotation of one to six degrees per minute. Procedures for vane shear testing were in general conformance with ASTM D-2573 – Field Vane Shear Test in Cohesive Soils, and the results were corrected for plasticity using the *average* Young Bay Mud and Older Bay Alluvium plasticity values. Vane shear strength data are considered undrained (i.e. S_u). The vane shear test results are included in Appendix B.

3.2.4 PIEZOMETERS

As part of a related investigation of leachate conditions within areas of the landfill that are not underlain by an engineered liner system, **GLA** advanced five borings into waste (LPZ-1 to LPZ-5) on the north side of Newby Island (Figure 1-2). A total of 6 piezometers were installed in these borings and one of these borings (LPZ-3) was advanced completely through the waste in order to install a piezometer within underlying subgrade soils.

In addition to the piezometers installed in the unlined areas of the Newby Island Landfill, triple-stacked vibrating wire piezometers were installed in each of Borings PZ-1 and PZ-2 which were advanced below the Subarea 10 subgrade prior to landfill liner construction in July 2005 (Figure 1-2). These piezometers were installed within Older Bay Alluvium at depths of about 25, 45, and 65 feet below the excavated subgrade at each location and they have been monitored through July 2007. These piezometers were installed in order to monitor the development and dissipation of pore pressures in subgrade soils in response to subsequent waste placement activities.

3.2.5 DOWNHOLE SHEAR WAVE VELOCITY TESTING

Shear wave velocity (V_s) measurements, particularly in the top 100 feet of soil, are useful in characterizing the response of site soils to seismic vibrations. In July, 2005 downhole shear wave velocity testing was performed by GeoVision Geophysical Services to depths of 110 and 118 feet below ground surface in Borings RB-10VA and RB-12V, respectively (Figure 1-2). Shear wave velocity testing was performed using a Model 170 Suspension Logging system manufactured by the OYO Corporation. Measurements were made in uncased boreholes from the base of the perimeter levee fill to the bottom of each boring at 0.5 m (1.6 foot) intervals. A complete report by GeoVision, including shear wave velocity data and an explanation of the testing method, is contained in Appendix C.

3.2.6 2-D REFRACTION MICROTREMOR SHEAR WAVE VELOCITY TESTING

Two-dimensional (2-D) refraction microtremor (ReMi) shear wave velocity surveys were performed by **GLA** at several locations in the southern portion of the NISL site during July 2006 and April-May 2007 (Figure 3-1). The ReMi method uses a passive source,

such as ambient noise from vehicles, equipment, or even a person walking, to construct a one-dimensional (1-D) average shear wave velocity model from data collected along an array of geophones. By sequentially analyzing data subsets along the array, a series of 1-D models can be aggregated to create a 2-D shear wave velocity model. The ReMi data was collected with a DAQ Link II 24 channel-seismograph connected to an array of 4.5 Hz geophones spaced 5- to 10-feet apart. The acquired data was processed for **GLA** by Optim LLC using their SeisOpt ReMi software (Optim, 2004).

ReMi surveys were performed along the levee road between Stations 49+00 and 56+20 (Lines 1-4; Figure 3-2), perpendicular to the levee road at about Station 50+70 (Lines 5-6; Figure 3-3), perpendicular to Lines 5-6 and about 30 to 40 feet inboard of the levee road (Line 7; Figure 3-4), and along the levee road near Station 54+00 then turning north over landfill waste at about the same elevation (Line 8; Figure 3-5).

The results of the ReMi 2-D shear wave velocity surveys are presented in Figures 3-2 to 3-5 and are compared to other site-specific and regional data in Section 5.4 below. In Figure 3-2, the ReMi V_s data for Line 1-4 are superimposed on a portion of the geologic cross section from Plate 2. As indicated thereon, there is good correspondence between the CPT-based geologic profile and the 2-D shear wave velocity survey for the levee (a.k.a. dike) material and underlying Young Bay Mud.

Correspondence of higher velocity zones (likely representing sand/silty sand strata) beneath the Young Bay Mud is less distinct, though the presence of high velocity contrasts is unmistakable. The fact that the CPT borings were conducted on the inboard shoulder of the levee road, while the 2-D shear wave velocity survey was conducted on the outboard shoulder may be responsible for this apparent contrast since the presumed meandering stream mode of sand deposition could have resulted in significant differences in the location of sand deposits over relatively small distances.

4.0 LABORATORY TESTING

In addition to the in-situ evaluation of foundation materials in the vicinity of the NISL, a substantial laboratory testing program was implemented in order to validate and supplement the field studies. All of the relatively undisturbed, disturbed, and bulk samples obtained during the exploration program were taken to the geotechnical laboratory where they were examined to verify field classification. Representative samples of different soil materials types were selected for laboratory testing. The laboratory test program was designed to classify the predominant material types encountered at the site and to measure their basic engineering properties, including: organic content, consolidation, moisture/density, grain size, specific gravity, triaxial shear strength, and the other physical and chemical properties required to characterize these materials.

The laboratory testing completed for this project and the data derived from that testing are presented in detail in Appendix D. A summary of the material classification test results is presented in Table 4-1.

A list of the tests performed and the applicable laboratory standards follows:

Consolidation Tests: Consolidation tests were performed on selected, relatively undisturbed shelby tube samples in accordance with ASTM D2435. Samples were placed in a consolidometer and loads were applied in geometric progression. The percent consolidation for each load cycle was recorded as the ratio of the amount of vertical compression to the original 1-inch height. The consolidation pressure curves are presented in the test data. On the consolidation test data sheet, where applicable, interpreted data from the Casagrande/Schmertmann construction are presented including: the compression index (C_c), recompression index (C_r), overconsolidaiton ratio (OCR), and the preconsolidation (P_p). Where applicable, time-rates of consolidation were recorded at specific overburden pressures. The coefficient of consolidation (C_v) values for this investigation were determined from laboratory time-rate consolidation curves using the Casagrande Log-Time method, but are also shown by the use of the Taylor Square-Root Method.

Moisture and Density Determination Tests: Moisture content and dry density determinations were performed in accordance with ASTM D2216 and D2937 on relatively undisturbed samples obtained from the test borings. The results of these tests are presented in the test data.

Classification or Grain Size Tests: Typical materials were subjected to mechanical grain-size analysis by sieving from U.S. Standard brass screens (ASTM Test Method D422). Hydrometer analyses were performed where appreciable quantities of fines were encountered. The data was evaluated in determining the classification of the materials.

Atterberg Limits: The Atterberg Limits (Liquid Limit, Plastic Limit, and Plasticity Index) were determined in accordance with ASTM Test Method D423 for engineering classification of the fine-grained materials and used in correlation with other engineering

parameters. Figures 4-1 to 4-3 show plasticity charts for fine-grained materials at the NISL site.

Specific Gravity: The specific gravity of selected samples was determined in accordance with ASTM D854M by Pycnometer.

Organic Contest Test: The organic content of selected samples was determined in accordance with ASTM D 2974-00 (Method C-440°C).

Triaxial Shear Strength Testing: Triaxial shear testing was performed on selected samples in accordance with ASTM D2850 for unconsolidated-undrained conditions and ASTM D4767 for consolidated-undrained conditions to model the different anticipated field conditions. Test data provided the undrained shear strength of various fine-grained soil materials.

5.0 DATA INTERPRETATION

5.1 MATERIAL CLASSIFICATION AND STRATIGRAPHY

5.1.1 SITE STRATIGRAPHY

As noted previously, sediments deposited in San Francisco Bay during the last 10,000 years are regarded to be estuarine in origin while sediments deposited from about 40,000 years to 10,000 years ago are generally regarded to be alluvial in origin. Local stratigraphic convention used previously at the NISL includes, from oldest to youngest, Pleistocene-age Older Bay Alluvium and Holocene-age Young Bay Mud.

Discrete sand and silty sand bodies occur throughout the Older Bay Alluvium and the Young Bay Mud. Sand bodies encountered at or near the base of the Young Bay Mud are not continuous on a site-wide basis; however in areas of the southern perimeter levee, especially, and also along portions of the northern perimeter levee, sand bodies near this Holocene/Pleistocene transition appear to be somewhat more continuous than other sandy units encountered at the NISL.

The nature of these and other sand bodies in the section require interpretation in the context of their probable sedimentary environments. It seems likely that the sand present in the Older Bay Alluvium represents mainly discrete, coarse-grained deposits associated with meandering stream courses. Similarly, most sands within the Young Bay Mud are likely related to tidal and distributary channels within the estuarine deposits. With the possible exception of highly localized dune or beach sands, environments of deposition conducive to the formation of continuous sand blankets or tabular sand bodies are not characteristic of the depositional margins of the southern San Francisco Bay. Further, the transition from Older Bay Alluvium to Young Bay Mud is probably characterized by mixed sedimentary environments marking the onset of sea level rise and giving rise to localized accumulations of sand (e.g., deltaic and flood deposits). Even more likely, ancestral channels of Coyote Creek and the un-named slough to the south have migrated across the site throughout the late Pleistocene and Holocene depositing overlapping channeled sands. Thus, the probable environments of deposition argue against these sand bodies as site-wide, continuous layers, which in any case, if present would appear systematically in the CPT records regardless of borehole location or spacing.

Rather, the CPT-based stratigraphy presented on Plates 1 through 6 suggests that the sand bodies, while relatively common, are restricted in stratigraphic continuity. The main question related to engineering analysis is the mainly horizontal extent over which these deposits can be correlated. One limiting dimension would be channel width. Assuming Coyote Creek and the un-named slough on the south to be representative of major channels in this depositional environment, widths of about 100 to 150 feet could be considered a maximum cross sectional dimension. Sand bodies correlated over larger distances could represent sampling along channel lengths, or sampling of more than one channel at or near the same elevation, as suggested in the above paragraph. It is also true that widely spaced CPT boreholes would allow interpretive correlations over longer

distances than would be the case with more closely spaced data. Interpretive problems are compounded if the distribution of channels is neither random nor uniform across the site, which we believe to be the case. For example, it seems likely that the concentration of sands encountered in CPTs between Stations 45+00 to 60+00 represent ancestral channels or tributaries of the unnamed slough.

Both geology and sample spacing may affect how the continuity of sand bodies is viewed. Note, for example, that the widely spaced data of Plate 1 is shown at a scale of 1" = 200', whereas the closely spaced data of Plate 6 is shown at a scale of 1" = 3'. Plate 6, however, represents a maximum horizontal dimension of only 600' over which the correlation of sand intervals between CPTs is nearly one-hundred percent. In contrast, on Plate 1, CPT spacing is 500 to greater than 1,000 feet, and the correlation of sand intervals between borehole locations is arguable. Plates 2 through 5, at a scale of 1" = 40', provide an intermediate representation of similar data.

In order to introduce a degree of conservatism into the engineering analysis, sand intervals on Plates 1 through 5 are extrapolated or interpolated over large horizontal intervals where they are not constrained by neighboring data. However, if the maximum cross sectional width of a channel in the depositional environments presumed here is taken at 150', then a sampling interval <150' is necessary to demonstrate greater horizontal continuity. Figure 5-1 is a diagrammatic summary of the CPT data from Plates 1 through 5 illustrating the pattern of sampling incorporated in this investigation. The upper diagram includes all CPTs located on the perimeter levee encircling the landfill (see Plates 1 and 2), and the lower diagram includes CPTs along an arcuate section through the southern landfill interior area (see Plate 3). CPT results on Figure 5-1 are shown as colored bars whose width represents 50 feet horizontally. Each CPT center point is shown in its true horizontal spacing within the profile to allow the assessment of stratigraphic continuity. Vertically, each diagram groups the CPT-based soil profile into 5-vertical-foot *packages*. Each five foot *package* is colored orange if the presence of sand is indicated by the CPT interpretation anywhere within the package. For example, even if only one two-inch sandy stratum was identified in a five-foot elevation zone, the entire zone was colored-coded orange for sand. Conversely, if no sandy strata were encountered in a given five-foot zone, this package would be color-coded blue to represent clayey silt, silty clay, and/or clay. Note that two inches (50 mm) is the minimum sounding interval of the CPT.

In Figure 5-1 and Plates 1 through 6, correlation between neighboring CPTs, regardless of horizontal spacing, was assumed for any two sand intervals within a 5-foot vertical package, therefore the diagrams strongly favor the interpretation of continuity (i.e., the diagrams are conservative). Note that less continuity would be apparent if the vertical interval were taken as, say, 2.5 feet or any other value less than 5 feet. Careful review of the figure shows that where the CPTs are closely spaced (less than 100 to 200 feet), as between station 20+00 and 40+00, correlations up to 900 feet are evident using these criteria. It is also the case that sand bodies whose cross sections can be no greater than about 150 feet are evident. Both possibilities are accounted for in our geologic interpretation.

For the widely-spaced CPTs between about stations 70+00 and 165+00, interpreting continuity is more problematic. Correlation over 2,000 or 2,500 feet is almost certainly not real, but is allowed by our treatment of the data. Where intervals of sand are seen in a single boring, but not correlative with neighboring CPTs, they might represent individual channels of perhaps 150 foot width, or less. Alternatively, they may be part of a laterally more continuous series whose horizontal extent is any distance allowed by the spacing interval. For purposes of engineering analyses, we have assumed the sand-bearing intervals extend roughly to the mid-points between CPTs, as shown in Figure 7-4. This figure is similar to Figure 5-1, except that each five-foot vertical *package* containing sand is colored in accordance with the worst-case liquefaction potential identified anywhere within the package (see Section 7.0 for a further discussion of liquefaction). In plan view, Figures 7-3A through 7-3T similarly present the distribution of potentially liquefiable sand deposits within these same 5-foot vertical packages. Note that the detail area of Figure 7-3G shows the likely presence of a north-south trending sand deposit bounded by clay layers at the same depth interval (i.e. elevation -25 to -30). Subsequent geologic mapping of the Subarea 12 excavation in this area identified this sand deposit and determined that it pinched out within the excavation consistent with a channel mode of deposition.

5.1.2 DEPOSITIONAL AGES

Samples of sediments obtained from Boring NB-5 at the NISL were age dated using Carbon-14 (^{14}C) radiocarbon dating techniques. Boring NB-5 is located adjacent to the perimeter levee on the south side of NISL approximately at Station 50+80. Radiocarbon age dates were measured by Beta Analytic, Inc. located in Miami, Florida using Accelerator Mass Spectrometry methods (Appendix D). Average calibrated ages for sediments collected from elevations between -8.75 and -16.75 (Young Bay Mud) ranged from 3,155 years before present (ybp) to 3,855 ybp, and ages generally increase with depth (Figure 5-2 and Table 5-1). One notable exception to the expected depth related increases in age is age reported for the sand sample collected at elevation -13.75. While this sand layer was identified in the middle of the Young Bay Mud, its age was measured to be 12,865 ybp. The fact that the two clay samples collected at a lower elevation were measured at 3,585 ybp and 3,855 ybp suggest that this sample represents reworked organic debris from older Pleistocene sediments. It is likely that this organic debris was transported into the area along with the surrounding sandy sediments in a high energy depositional environment.

There is a significant age gap between the oldest portion of the Young Bay Mud sampled at elevation -16.75 (3,855 ybp) and the shallowest Older Bay Alluvium sampled at elevation -22.75 (19,530 ybp). The boundary between these two materials is also demonstrated by a change in both the physical appearance and engineering characteristics of the two materials. The oldest material sampled and age-dated at the NISL for this project was a sand at elevation -58.25 (27,730 ybp).

5.1.3 MATERIAL CLASSIFICATION

The soil materials encountered during the current investigation were generally categorized as compacted subgrade and berm fill (silty to sandy clay), Young Bay Mud (sandy to silty clay), Older Bay Alluvium (sandy to silty clay), sand layers (clean to silty to clayey). CPTs performed for this investigation were the primary means for identifying stratigraphy at the NISL. CPT-based soil classification was performed per Robertson (1990), and laboratory testing of samples retrieved in adjacent rotary borings were used to confirm the CPT-based classification. In addition, an extensive laboratory testing program was performed to further characterize these materials and to provide index properties used to correlate with other engineering properties. The results of the laboratory testing are presented in Appendix D and summarized in Table 4-1.

Fill soils associated with levee construction was found around the perimeter of the site from a berm crest elevation ranging from about elevation +14 to +23 and extending down to about elevation +5 to –5 (Plates 1 and 2). These materials are generally associated with levee construction and are sandy silt to silty clay; note that desiccation of these materials can lead to an erroneous CPT-based classification of sand in the upper several feet of the profile. These materials were generally constructed directly above the Young Bay Mud deposits.

The Young Bay Mud is a relatively young Holocene-aged deposit typically composed of a silty to clayey material with non-continuous bodies of clean to silty sand. According to laboratory test results, this material typically classifies as a CH according to the Unified Soil Classification System (USCS), with a plasticity index ranging from 17 to 42 (average: 30). This material exists beneath the perimeter levee and at the existing ground surface beyond the levee. This unit was reportedly removed beneath the unlined landfill cells on the north and west portions of the NISL (PRA Group, 1993). The subgrade beneath all lined landfill cells constructed under Chapter 15 (Subareas 1 to 8) or Title 27 (Subareas 9A and greater) consists of Older Bay Alluvium and, as such, is considerably below the Young Bay Mud stratigraphic horizon (Figure 1-3). Young Bay Mud encountered at the NISL is anticipated to be normally consolidated to slightly overconsolidated. These materials were encountered at the existing ground surface near the toe of the levee and extend down to an elevation of -5 to -22 (Plates 1 and 2). Strength criteria used for these materials are discussed in Section 5.3.5.1.

The Older Bay Alluvium underlies the Young Bay Mud on the site and forms the foundation unit for all lined cells of the landfill. While relatively similar in composition to the Young Bay Mud, this material is differentiated by age, deposition, and competency. The Older Bay Alluvium is composed of overconsolidated silty clay and clay with scattered interbeds of silt and sand. Laboratory test typically classify this material as a CL according to the USCS, with plasticity indices ranging from 1 to 33 (average: 19). Based on borings in and near the site that extend to depths of about 250 feet, or deeper, this unit is known to extend to depths considerably deeper than any CPT advanced for this investigation (i.e., in excess of 100 feet). This unit is much stiffer than the overlying unit as reflected by both an increase in the tip resistance and sleeve friction

in the CPT test data. Geologic sections depicting **GLA**'s interpretation of the occurrence of these two units at the NISL is presented on Plates 1 and 2. Strength criteria used for these materials are discussed in Section 5.3.5.1. Based on laboratory classification for this project, the perimeter levee soils more closely resemble Older Bay Alluvium than Young Bay Mud.

Intermittent sand/silty sand strata in both the Young Bay Mud and Older Bay Alluvium were noted in the CPT borings (by an increase in the tip resistance of the CPT probe) and in the adjacent rotary wash borings (by both visual interpretation and laboratory test data). Based on laboratory and CPT data and confirmed by geologic mapping of Subarea 12, the sandy strata vary from relatively clean to silty or clayey over very short horizontal and vertical distances. Sand fines contents based on laboratory testing of samples retrieved in SPT borings varied from 5 to 47 percent. CPT-based estimates of sand fines content based on Robertson and Wride (1998) ranged from zero to 35 percent, though the latter represents the upper-bound value for a material classified as a silty sand using the CPT soil behavior index, I_c. Since CPT-based estimates of sand fines contents were used extensively for liquefaction assessment on this project, fines contents determined by laboratory analyses of SPT samples were compared with CPT fines estimates for CPTs adjacent to the respective sampled borings. As shown in Figures 5-3A to 5-3G, CPT estimation of fines contents at NISL appears to be relatively accurate.

5.2 CLAY CONSOLIDATION

The results of consolidation testing of Young Bay Mud and Older Bay Alluvium are discussed below. The discussion is divided into subsections on consolidation indices, overconsolidation, and time rate of consolidation.

5.2.1 CONSOLIDATION INDICES

The compression index, C_c, is an index of the compressibility of a soil, with higher values indicating more compressibility. Bonaparte and Mitchell (1979) report Young Bay Mud C_c values ranging from 0.8 to 1.8. Table 5-2 presents a summary of consolidation parameters identified in this investigation and indicates that the values of C_c for Young Bay Mud at the site range from about 0.4 to 1.0, with an average of 0.6, making the Young Bay Mud at the NISL somewhat stiffer by comparison. It should be noted that C_c values on the south side of the NISL tended to be nearer the low end of the range. As expected, C_c values for the stiffer Older Bay Alluvium were lower, ranging from about 0.2 to 0.4, with an average of 0.28.

Both the values of C_c for Young Bay Mud where it was encountered on the south side of the site and Older Bay Alluvium overall are in good agreement with a modified Cam-

Clay model correlation with the plasticity index:

$C_c \approx 0.5\ G_s\ (PI/100)$ (Kulhawy and Mayne, 1990)

where:

G_s = specific gravity, and
PI = plasticity index.

The C_c for Young Bay Mud encountered on the north side of the site was higher and indicates the presence of a somewhat softer and more compressible material at that location.

The recompression index, C_r, is an index of the compressibility of the soil which has experienced a greater maximum pressure in the past than what is currently applied to the soil, and ranges from about 0.06 to 0.16 for the Young Bay Mud and 0.02 to 0.09 for the Older Bay Alluvium, with averages of 0.10 and 0.05, respectively.

5.2.2 OVERCONSOLIDATION

An important parameter for both compressibility and strength of clay soils is the degree to which they are currently loaded compared to the maximum past loading. Reasons for a greater past loading can include erosion of overlying soil and/or a rise in groundwater or sea levels. The ratio of maximum past effective stress to current effective stress is the overconsolidation ratio (OCR). A clay with an OCR of unity is said to be normally consolidated, while a clay with a larger OCR is overconsolidated. Estimates of the maximum past effective stress for this investigation were determined by the Casagrande method using the Schmertmann correction for disturbance. Consolidation curves with the Casagrande/Schmertmann constructions are presented in Appendix D.

Calculation of OCR requires an estimation not only of maximum past effective stress, but also the current effective stress. Since the perimeter levee, through which these many of rotary borings were advanced, lies above the surrounding terrain, an adjustment was made in order to account for the diminishing stress due to the weight of the levee at progressively deeper depths beneath the base of levee. The adjustment consisted of manually adjusting the unit weight of the levee (dike) materials in a spreadsheet according to the method shown in Figure 5-4. For example, immediately beneath the dike, the full unit weight of 120 pcf is used, while at a depth of 50 feet (or about 38 feet beneath the base of the dike), the unit weight is adjusted to 76 pcf (i.e. 63% of 120 pcf). Depending on the location of the boring, one of three stress conditions was used to calculate the current effective stress: *full* (the terrain surrounding boring is at the same elevation as the boring); *dike* (the boring is on the levee and is higher than surrounding terrain; and *terrace* (the boring is on the levee, but the landfill inboard of the levee is higher than the levee).

Since the Holocene-age Young Bay Mud is geologically recent, and the relatively thin Young Bay Mud strata encountered in this investigation was all beneath the long-since

constructed perimeter levee, it was expected to be normally consolidated to slightly overconsolidated. In fact, the Young Bay Mud OCR's identified in this study ranged from 1.0 to 1.5, generally validating this interpretation.

While the Older Bay Alluvium at the site is likely to be overconsolidated as a result of the documented rise of seawater levels within the San Francisco Bay, it may not be as overconsolidated as the well-known Older Bay Mud that is found further to the north (beneath and along the margins of the San Francisco Bay). While Older Bay Mud can be highly overconsolidated, with OCR's on the order of 2 to 8, laboratory-calculated OCR values for the Older Bay Alluvium at the site ranged from 1.2 to 5.2, with an average of about 2.4. As shown on Figure 5-5, there does not seem to be any discernable correlation between Older Bay Alluvium OCR and elevation across the site, although the OCR variability is somewhat greater above elevation -40 than below.

Figure 5-6 presents Young Bay Mud and Older Bay Alluvium OCR's calculated from CPT data paired with laboratory OCR values from samples obtained from adjacent rotary borings. Since the CPT data is continuous, yet displays similar wide variations in OCR values, the variability in laboratory OCR values is believed to be a real phenomenon. Despite the variability, it is apparent that CPT-based Older Bay Alluvium OCR values above elevation -40 are somewhat higher (average OCR = 2.8) than below this level (average OCR = 2.0), as illustrated by the 1.2-foot running average OCR for the five CPT borings shown.

Using the clay strength model that will be discussed in Section 5.3.5.1, a lower value of OCR will result in lower estimated shear strength values. Accordingly, Older Bay Alluvium OCR values of 2.0 down to elevation -40, transitioning from 2.0 to 1.5 between elevation -40 and -50, and 1.5 below elevation -50 were conservatively selected for use in subsequent analyses. Young Bay Mud OCR values were generally assumed to be 1.0 (i.e. normally consolidated), except for the overconsolidated crust that exists outside the perimeter levee. The OCR of this crust was assumed to vary from 3.0 at the surface to 1.0 at a depth of about 19 feet, which is consistent with the characterization of Young Bay Mud overconsolidation by Bonaparte and Mitchell (1979).

5.2.3 TIME RATE OF CONSOLIDATION

Of particular interest for this investigation is the rate at which consolidation of clay strata occurs when loaded by the weight of MSW. Since the shear strength of clay increases with consolidation, estimates of the rate and degree of consolidation can have a large impact on slope stability analyses and can influence the timing of waste placement. The coefficient of consolidation, C_v, is an index parameter for the rate of consolidation, with higher values indicating faster consolidation. C_v values for this investigation were determined from laboratory time-rate consolidation curves using the Casagrande Log-Time method. Laboratory curves showing the time to 50 percent consolidation, t_{50}, are presented in Appendix D for both Young Bay Mud and Older Bay Alluvium. Table 5-3 presents calculated C_v values for load ranges representative of future MSW loading at the NISL site for both Young Bay Mud and Older Bay Alluvium.

It should be noted that because clay consolidation is controlled by the length of the drainage path for dissipation of induced pore pressures, it can be significantly influenced by local sand layers which would not be present in a laboratory sample. In addition, due to natural sedimentary stratification, the horizontal permeability of a soil can be an order of magnitude greater than vertical permeability. Since drainage in a laboratory sample is vertical and would not benefit from drainage provided by sand lenses or anisotropic soil conditions in the field, laboratory-determined values of C_v are typically found to be low relative to actual field performance.

Since historic MSW placement reportedly occurred after complete excavation of Young Bay Mud soils at NISL (PRA Group, 1995), and the base of all future landfill cells will be excavated down to an elevation well below the base of the Young Bay Mud, consolidation of the Young Bay Mud is not particularly relevant to the proposed vertical expansion. The following discussion, therefore, pertains only to consolidation characteristics of Older Bay Alluvium.

A plot of all laboratory-determined Older Bay Alluvium C_v values is presented in Figure 5-7. Since the four highest curves on this plot represent much faster consolidation than the other samples tested, they were not considered to be typical of Older Bay Alluvium strata and have been excluded from further analyses. A summary of the Older Bay Alluvium C_v data excluding these tests is shown on Figure 5-8. Based on an interpolated load of about 14,000 psf (approximately 160 feet of solid waste), C_v values of 10, 44, 78, and 112 ft^2/yr were calculated for use in time-consolidation analyses from the data presented in Figure 5-8. [The 160 feet of future MSW loading is approximately the height from the landfill liner to just below the crest of the front-face slope.] These four calculated C_v values conservatively represent the spread of Figure 5-8 by including:

(u-SD),
(u-SD/2),
(u), and
(u+SD/2) , respectively,

where u = average C_v and SD = standard deviation of C_v.

In addition to the coefficient of consolidation, understanding drainage path length is a critical parameter in estimating the time-rate of consolidation. A significant potential for a reduced drainage path length has been reasonably assumed to exist for sediments in the vicinity of the NISL for the following reasons:

- Numerous sand lenses and layers are found throughout the Young Bay Mud and Older Bay Alluvium strata;
- Landfill cells constructed since implementation of Subtitle D have a laterally continuous sub-drainage layer beneath the entire cell; and
- Older Chapter 15 cells have continuous drainage above the compacted subgrade.

Using the above C_v values and assuming four idealized doubly-drained Older Bay Alluvium clay strata ranging from 25 to 150 feet thick, families of curves have been constructed to estimate the time to a given percent consolidation (Figures 5-9 and 5-10).

5.2.4 PORE PRESSURE DISSIPATION DATA

In order to assess the validity of the theoretical consolidation rates calculated above, actual pore pressure dissipation due to MSW loading was monitored in triple-stacked vibrating wire piezometers installed in Borings PZ-1 and PZ-2 which were advanced below the Subarea 10 subgrade prior to landfill liner construction in July 2005 (Figure 1-2). These piezometers were installed within Older Bay Alluvium at depths of about 25, 45, and 65 feet below the excavated subgrade at each location and have been monitored through July 2007. Pressure data from Piezometers PZ-1 and PZ-2 are presented in Figure 5-11 in units equivalent to feet of water.

Waste placement in Subarea 10 began in November 2005 and as of July 2007 had reached approximately the level of the perimeter levee (a total thickness of over 50 feet). During normal landfilling operations, waste was placed in Subarea 10 for several months before being diverted to other areas of the landfill. Waste placement was then reinitiated in Subarea 10 for a period before being diverted again. These alternating periods of active filling and static loading are clearly reflected in the piezometric record as rising pore pressure followed by leveling or falling pressures.

Most noteworthy from the perspective of consolidation is that the maximum rise in pore pressures in PZ-1 and PZ-2 is much less than the pressure applied by the weight of the overlying MSW. In fact, in roughly a year-and-a-half, the average *maximum* rise in all six piezometers is about 19 feet of water (or 1,186 psf), while the pressure due to of 54 of MSW would be 4,320 psf at a waste density of 80 pcf. This piezometric data suggests that the excess pore pressure due to the MSW loading is only about 27 % of the load itself (i.e. 1,186/4,320 x 100%) and that the Older Bay Alluvium clay strata underlying Subarea 10 has drained about 73% of the excess pore pressure in a year-and-a-half. Data from Subarea 10 would suggest that after about 5 years, about 98% of the primary consolidation due to waste placement would have occurred.

This field measured pore pressure dissipation data suggests that actual Older Bay Alluvium consolidation performance in Subarea 10 is on the faster end of the range of theoretical consolidation in Figures 5-9 and 5-10. As discussed above, this situation is neither uncommon nor unexpected.

5.3 SHEAR STRENGTH

The shear strength of natural soil and fill materials at the site was evaluated by several methods, including unconsolidated-undrained (UU) and consolidated-undrained (CU) triaxial tests, in-situ vane shear testing, and correlations with CPT tip resistance. A discussion of the results of these tests is presented below followed by a discussion of the shear strength model used for Young Bay Mud and Older Bay Alluvium at the site.

Table 5-4 presents a summary of shear strength and other material properties used in subsequent stability analyses.

5.3.1 UNCONSOLIDATED-UNDRAINED TRIAXIAL TESTS

UU triaxial tests were performed on selected Young Bay Mud and Older Bay Alluvium samples. Results of these tests are presented numerically in Table 5-5 and graphically on Figure 5-12. Young Bay Mud and Older Bay Alluvium UU undrained shear strengths (S_u) ranged from about 0.5 to 0.7 ksf and 0.7 to 3.3 ksf, respectively. In Section 5.3.5.1 below, UU shear strength data is compared with vane shear and CPT shear strength data at specific locations.

5.3.2 CONSOLIDATED-UNDRAINED TRIAXIAL TESTS

CU triaxial tests with pore pressure measurements were also performed on selected Young Bay Mud, Older Bay Alluvium, and perimeter levee (a.k.a. "dike") fill samples. The results of these tests are presented in Figures 5-13 through 5-15. A regression of all the test data resulted in effective stress friction angles for Young Bay Mud and Older Bay Alluvium of 32.0 and 31.2 degrees, respectively; with the cohesion intercept set to zero. Similarly, the relatively limited data set for levee (dike) fill resulted in friction angle and cohesion values of 27.6 degrees and 0.24 ksf, respectively. Note that of these materials, Mohr-Coulomb strength parameters (i.e. friction angle and cohesion intercept) were only used in analyses for levee fill. Since the range of stresses experienced by the levee fill falls within a relatively narrow range, the linear Mohr-Coulomb shear strength model is appropriate. As will be discussed below, the shear strength of Young Bay Mud and Older Bay Alluvium were characterized by an undrained strength model, which doesn't use Mohr-Coulomb parameters. CU tests were performed on these latter materials as part of the undrained strength model characterization.

5.3.3 VANE SHEAR TESTS

Results of in-situ vane shear testing on Young Bay Mud and Older Bay Alluvium are presented in Figure 5-16 and were adjusted for *average* values of Young Bay Mud and Older Bay Alluvium plasticity using Bjerrum's (1972) VST correction, as shown in Figure 5-17. Note that peak shear strengths are presented in this figure. In Section 5.3.5 below, vane shear data is compared with UU and CPT shear strength data at specific locations.

5.3.4 CONE PENETRATION TESTS

Undrained clay shear strength (S_u) can be correlated with CPT tip resistance by the following relation:

$$S_u = (q_t - \sigma'_v)/N_{kt} \qquad \text{(Lunne et al., 1997; Eq. 5.17)}$$

where:

q_t = CPT tip resistance
N_k = CPT cone bearing factor.

Based on several *theoretical* models, the N_k value should range from about 9 to 18. Lunne et al. (1997) suggests that for normally to lightly-consolidated clays (i.e. OCR's of about 1 to 2), N_k may be as low as 10, while for more heavily overconsolidated, stiff, fissured clays, N_k may be as high as 30. In light of the fact that for the analyses presented herein, the OCR for Young Bay Mud and Older Bay Alluvium have been conservatively assumed to be 1.0 and 1.5 to 2.0, respectively, a range of N_k values of between 12 and 15 appears to be appropriate. [It is noted that studies for the seismic replacement of the east span of the San Francisco-Oakland Bay Bridge in similar clayey soils used the same N_k range of 12 to15 (Fugro-Earth Mechanics, 1999).] The estimated range of shear strengths for N_k values of between 12 and 15 are presented on Figures 5-18A through 5-18D. For comparison, UU triaxial and vane shear strength data are also shown on these figures. Note that the CPT-based shear strengths shown in these figures consider the diminishing stress influence of the levee at depth as discussed in Section 5.2.2 above.

When taken as a whole, there is reasonable agreement between the CPT-based S_u and those of the UU and vane shear tests, although owing to the vastly greater number of data points, the CPT data tends to exhibit more scatter. Note that UU-based S_u values are known to exhibit a degree of uncertainty due to sampling disturbance and unknown effective confining stress, and the vane shear S_u data also exhibits some uncertainty owing to corrections that are made on the basis of sample plasticity. As a result of these uncertainties, the variability of UU and vane shear S_u data shown in Figures 5-18A through 5-18D is not unexpected.

5.3.5 SHEAR STRENGTHS USED IN ANALYSES

A discussion of shear strength parameters and models used in slope stability analyses is presented below and summarized in Table 5-4.

5.3.5.1 Young Bay Mud/Older Bay Alluvium Shear Strength Model

Derivation of the shear strengths of the Young Bay Mud and Older Bay Alluvium strata was a primary goal of the testing and interpretation completed for this investigation. These efforts focused on evaluating the undrained shear strength, S_u, of these materials since loading, such as by filling of a landfill cell, typically takes place faster than generated excess pore pressures can be *fully* dissipated. For the purposes of shear strength assessment in subsequent analyses, 75% dissipation of excess pore pressures (i.e. 75% consolidation) due to the imposition of MSW loading will be assumed for Young Bay Mud and Older Bay Alluvium strata throughout the site. On the south side of the landfill where there appears to be somewhat more sand strata, the 75% consolidation was shown to have occurred in about a year-and-a-half in piezometers positioned under the Subarea 10 development area. On the north side of the landfill, while sandy strata may be somewhat less common, the waste has been in place for decades, so the assumption of 75% consolidation is also considered conservative in this area. Regardless, since the Young Bay Mud strata is much thinner than the minimum 25-foot thick layer considered in theoretical time-rate-of-consolidation analyses conducted for the Older Bay Alluvium

and is drained by both the elevation -15 sandy strata and the landfill slope drain on the south side of the landfill, the assumption of 75% consolidation is likely to be conservative.

S_u has been found to be stress dependent, i.e. the greater the effective overburden pressure, σ'_v, the greater the strength. Because of this dependency, the Young Bay Mud S_u within the San Francisco Bay area is typically evaluated in terms of the undrained strength ratio, S_u/σ'_v, which is also expressed as S_u/P'. Typical normally consolidated S_u/P' values for Young Bay Mud are 0.31 to 0.34 (Bonaparte and Mitchell, 1979), with 0.32 being the most commonly used undrained strength ratio. It has been shown that the Su/P' ratio for an overconsolidated clay increases with increasing OCR and that the relationship between the overconsolidated $[S_u/P']_{OC}$ and OCR follows a predictable trend for a given material (see Figure 5-19, Ladd et al., 1977). In fact, using concepts from the modified Cam-Clay model, it has been shown that the relationship between the normally consolidated $[S_u/P']_{NC}$ and the overconsolidated $[S_u/P']_{OC}$ can be expressed as:

$$[S_u/P']_{OC}/[S_u/P']_{NC} = OCR^{\Lambda} \qquad \text{(Kulhawy and Mayne, 1990; Eq. 4-39)}$$

where:

$[S_u/P']_{NC}$	= normally consolidated undrained strength ratio
$[S_u/P']_{OC}$	= overconsolidated undrained strength ratio
OCR	= overconsolidation ratio
Λ	= modified Cam-Clay parameter = $1 - (C_r/C_c)$
C_c	= consolidation compression index
C_r	= consolidation recompression index

Since the OCR and Λ parameters can be determined from consolidation data (Table 5-2), the remaining parameter necessary to complete the strength model used for the Young Bay Mud and Older Bay Alluvium in this investigation is the determination of an appropriate $[S_u/P']_{NC}$ value. Figures 5-20 and 5-21 present plots of S_u vs. P' for both Young Bay Mud and Older Bay Alluvium [Note that in the laboratory, P' represents the cell consolidation pressure prior to initiating shearing]. For most samples tested (i.e. those shown with the same symbol), the right-most one or two points represents samples that were consolidated to a high enough load to become normally consolidated. Also shown on these figures are S_u/P' lines representing 0.25, 0.30, and 0.32. As can be seen, most data points lie above the 0.32 line.

While the left-most points for a given sample represent more overconsolidated states and would be expected to lie above the $[S_u/P']_{NC}$ line, normally consolidated data points should approach a common lower-bound line. Since most points for both Young Bay Mud and Older Bay Alluvium lie above the 0.30 $[S_u/P']_{NC}$ line, this value was conservatively selected as the $[S_u/P']_{NC}$ ratio for subsequent analyses.

Based on Older Bay Alluvium OCR's of 1.5 to 2.0 and a Λ value of 0.805, the calculated $[S_u/P']_{OC}$ will range from 0.416 to 0.524. Young Bay Mud strengths will typically be based on $[S_u/P']_{NC} = 0.30$, except for the overconsolidated crust (OCR = 3.0) outside the

perimeter levee, in which case $[S_u/P']_{OC}$ will range from 0.751 near the surface down to 0.30 at a depth of about 19 feet below the ground surface. Note that S_u values derived from this model will be further reduced for anisotropy, as discussed below.

The fact that landfill cells at the NISL are excavated below the original ground surface means that underlying clay strata might otherwise be weaker than in their original state. In practice, however, clays retain most of their original strength on unloading. In order to prevent this artificial weakening of clay strata after MSW is placed but before significant drainage can occur, minimum strengths equal to the original condition have been applied to soils using the S_u/P' model, as shown in Table 5-4.

Figures 5-18A through 5-18D compare the above-described S_u model with CPT, VST, and UU results at five CPT locations at the NISL site. As discussed above in Section 5.2.2, effective stresses beneath the perimeter levee were calculated based on a diminishing stress influence of the perimeter levee at progressively deeper depths. The S_u model strengths shown in these figures were calculated based on consideration of the diminishing stress influence of the levee at depth; they do not, however, reflect the reduction in strength due to anisotropy.

5.3.5.2 Anisotropy

The horizontal layering of clays due to sedimentary stratification can give rise to different shear strengths depending on the orientation of the shear plane. Based on series of UU triaxial tests, Duncan (1965) reports approximately a 20 percent variation in Young Bay Mud shear strength depending on this orientation. For this investigation, we have assumed a similar anisotropic condition applies to Young Bay Mud and Older Bay Alluvium at the NISL site. This shear strength anisotropy has been implemented in slope stability analyses by adopting Duncan's (1965) Young Bay Mud strength variation with failure plane orientation as reported in Bonaparte and Mitchell (1979). The slope stability software reduces the undrained shear strength, calculated as described above, at the base of each slice of the sliding mass based on its orientation as shown in Figure 5-22.

5.3.5.3 Liquefied Sand Residual Strength

Some of the sand/silty sand layers encountered in borings completed for this investigation were sufficiently loose and displayed enough lateral continuity that they have been assumed to present a liquefaction risk (see Section 7.0 for a further discussion of liquefaction, including a fuller discussion of terms used in this section). As discussed below, residual shear strengths for such materials that have liquefied were estimated based on correlations with normalized CPT tip resistance data for CPTs performed around the NISL site. In addition, CPT-based results were compared with a commonly used SPT-based approach for estimation of liquefied residual shear strength.

Beginning about in the 1960s, investigators developing estimates of the liquefied residual shear strength (S_r) of soils tackled the problem from two different approaches: laboratory testing and empirical correlation with penetration resistances (i.e. SPT blowcount or N

values). Due to difficulties in retrieving representative undisturbed samples of sandy soils, however, the latter approach has gained prominence over the years. This approach is based on case histories of sites that experienced liquefaction during an earthquake, their associated penetration resistances, and back-calculated strengths using stability analyses.

Refinements to this approach have recognized both that the CPT tip resistance (q_c) is a fundamentally better method for estimating S_r, as well as the dependence of S_r on not only penetration resistance, but also on initial effective overburden stress, σ'_v. Accordingly, estimates of S_r for this project utilized the normalized S_r/σ'_v ratio concept that has been adopted in more recent studies (e.g. Stark and Mesri 1992, Ishihara 1993, Wride et al. 1999, Olson and Stark 2002, Idriss and Boulanger 2007).

Specifically, the normalized liquefied residual shear strength ratio of Olson and Stark (2002) as modified by Stark (2008) has been used on this project to estimate S_r, as shown on Figure 5-23. Note that the tip resistance normalized to an effective overburden stress of one atmosphere, q_{c1}, is used in this method (Olson and Stark, 2002; Kayen et al., 1992). The original 2002 method is based on case histories which tend to be limited to relatively loose (i.e. low tip resistance) soils. While not as useful for estimating S_r directly, experimental data have been useful to show that a pronounced increase in S_r can be expected as relative density, and therefore q_{c1}, increase. This increase is reflected in Stark's 2008 modification, as shown by the red line in Figure 5-23. As will be shown below, however, the range of the S_r/σ'_v ratios utilized for this project is restricted to that of the original 2002 relationship.

Figure 5-24 shows the predominant range and 1-foot running average of S_r/σ'_v ratios for potentially liquefiable soils (i.e. with a liquefaction factor of safety less than 1.3) encountered in recent CPTs at the NISL site (except post-grout CPTs used to evaluate the compaction grout test program). Figure 5-25 shows the running average of this data alone. Although there is a degree of scatter, the data appear to cluster around a ratio of 0.08 at about elevation -10, increasing to about 0.10 to 0.12 at elevation -35 and below. Isolated subsets of this data present a similar picture. Figures 5-26 and 5-27 show S_r/σ'_v data for the southern portion of the NISL west and east of Station 37+50, respectively; Figure 5-28 shows the remaining S_r/σ'_v data for more northerly portions of the site.

Considering the CPT-based S_r/σ'_v data discussed above, the values of this ratio used to compute S_r for this project were: 0.08 above elevation -20, 0.10 below elevation -35, and 0.09 in between. Given the meandering channel mode of deposition and the demonstrated variability of fines content over short distances, using somewhat-less-than-average values such as these, rather than a lower bound, for example, is considered appropriate. Using these ratios and typical levels of effective stress beneath the perimeter levee, S_r values of 220 psf, 320 psf, and 485 psf would be calculated for potentially liquefiable soils at elevation -12, -25, and -45, respectively.

Although the CPT-based estimates of S_r are now considered more reliable than SPT-based methods, and many more CPT borings were performed at the NISL than SPT borings, the CPT results were compared with SPT-based methods to provide a check on the former. SPT values of normalized clean-sand blowcount, $(N_1)_{60cs}$, for samples determined to be liquefiable (i.e. with a liquefaction factor of safety less than 1.3) per Youd et al. (2001) are shown on Figure 5-29, both individually and as elevation interval averages. Using the highest, lowest, and elevation-interval average $(N_1)_{60cs}$ values shown in this figure, a range of S_r values was estimated with the well-known Seed and Harder (1990) relationship shown in upper portion of Figure 5-30. Based on the recommendations of Seed (1991) and the State of California Marine Oil Terminal Engineering and Maintenance Standards (Chapter 31F, Division 6, Title 24, Part 2, Volume 1, California Code of Regulations), S_r was conservatively estimated as the lower one-third of the range of the curves shown on this figure. The lower portion of this figure estimates a range of S_r/σ'_v values with this same data using Stark and Mesri (1992). Since the range shown in the original Seed and Harder (1989) and Stark and Mesri (1992) relationships don't extend beyond about $(N_1)_{60cs}$ = 16 and 20, respectively, the lower- and upper-bound curves of both relationships have been extended to higher $(N_1)_{60cs}$ values in general accordance with the recommendations of Seed et al. (2003). With the exception of a single low point (Boring RB-1 @ elevation -14), the SPT-based estimates of S_r are generally higher than the CPT-based values used for design on this project, which were discussed in the preceding paragraph. Given the limited number of SPT samples performed at the NISL, the discontinuous nature of this method, and the inherent variability of this "standard" method, this result is not unexpected.

Because S_r has been estimated using a strength ratio approach, the value of S_r would approach zero as the depth of analysis approaches the ground surface. Both case histories and principles of soil mechanics[1] suggest that there may be some limiting liquefied residual strength at low confining stress. Based on the recommendations of Dr. Timothy Stark of the University of Illinois at Urbana-Champaign (see Appendix I), minimum liquefied residual strengths of 150 and 300 psf were assigned to liquefied sand/silty sand layers approximately at elevation -10 to -15 and -25 to -30, respectively. Due to the higher confining stress at greater depths, no minimum was applied to potentially liquefiable sands below these levels.

5.3.5.4 Municipal Solid Waste

The shear strength of municipal solid waste (MSW) was estimated based on Zekkos et al. (2007). This relationship is shown in Figure 5-31 along with some other well-known MSW shear strength relationships. Note that waste used in the Zekkos study came from the Tri-Cities landfill just a few miles north of the NISL and is believed to represent a similar waste stream and disposal history. Per the recommendations of Zekkos, et al. (2007), the MSW dynamic strengths were increased 20% for pseudo-static slope stability analyses.

[1] i.e. the tendency for less contractive behavior and higher static friction angles at low confining stresses.

5.3.5.5 Landfill Liner

From a shear strength perspective, the critical components of the landfill liner system are the geotextile/textured HDPE liner and geotextile/smooth HDPE liner interfaces on the base and slope, respectively. In accordance with common practice, residual (a.k.a. large deformation) shear strengths will be used for these interfaces. An authoritative database of geosynthetic interface shear strengths (Koerner and Narejo, 2005) was consulted for estimates of these interface shear strength values. Linear least-squares regression on a large set of data resulted in residual shear strengths of 17 and 9 degrees for the geotextile/textured HDPE and geotextile/smooth HDPE interfaces, respectively. Based on **GLA**'s extensive experience with landfill liner strength properties, however, shear strengths for these critical liner interfaces at the NISL were estimated at 14 and 8 degrees, respectively, with no cohesion. For slope stability considerations, a textured HDPE liner will be used on both the base and slopes for future cells adjacent to perimeter levee Station 30+00 and below.

5.3.5.6 Perimeter Levee and Miscellaneous Fill

The shear strength for the perimeter levee and other site fill soils was based on consolidated-drained triaxial data from samples of levee soils (see Figure 5-15). For these materials, a friction angle of 27.6 degrees and a cohesion of 240 psf were used in stability analyses.

5.3.5.7 Unliquefied Sand and Silty Sand

The shear strength of sand and silty sand strata that are not expected to liquefy (i.e. with a liquefaction factor of safety greater than 1.3) was estimated from CPT data using the following correlation by Kulhawy and Mayne (1990):

$$\phi' = 17.6 + 11.0 \log\left[(q_c / P_a)/(\sigma'_v / P_a)^{0.5}\right]$$

where:

ϕ' = effective stress friction angle
q_c = CPT tip resistance
P_a = atmospheric pressure
σ'_v = overburden effective stress

For the NISL CPT data, the above formula results in friction angles ranging from about 34 to 45 degrees. Considering the fines contents for these materials as discussed in Section 5.1.3, the shear strength of sand and silty sand strata at the site was conservatively characterized with a friction angle of 32 degrees and a nominal cohesion of 50 psf.

5.4 SHEAR WAVE VELOCITY FOR SITE-RESPONSE MODELING

Shear wave velocity (V_s) data used for site response modeling came from five sources: downhole measurements in Borings RB-10VA and RB-12V at Stations 30+00 and

125+00, respectively, to a depth of about 120 feet (see Appendix C); 2-D refraction microtremor (ReMi) surveys conducted in the southern portion of the site; seismic CPT measurements conducted in three CPT borings on or near the southern levee in the vicinity of Station 51+00; previous downhole measurements in Caltrans Boring 98-3 at the nearby Interstate 880/Dixon Landing Road interchange to a depth of about 225 feet; and a shear wave velocity profile to a depth of 500 feet in undifferentiated San Francisco Bay Mud developed by Schneider et al. (2000) for their report "Earthquake Scenario Ground Motion Hazard Maps for the San Francisco Bay Region."

Shear wave velocity data from various sources to depths of about 100 feet are presented in Figure 5-32 along with the V_s model used in site response analyses as part of this investigation. The data presented in this figure have all been adjusted to a common Mean Sea Level elevation datum. Comparison of the V_s measured in Borings RB-10VA and RB-12V shows a relatively consistent V_s vertical profile across the site. It is noted however, that the borings were advanced through the perimeter levee (about 10 to 15 feet higher than the ground surface beyond the levee) in areas where the Young Bay Mud appeared to be relatively softer based on CPT data. As a result, Young Bay Mud V_s data from these borings indicate velocities that are a bit slower than other site data. The Young Bay Mud V_s model profile utilized in site response analyses more closely approximates the seismic CPT data.

The V_s model for shallow sand/silty sand strata was based on seismic CPT data. Note that based on this data, the sand/silty sand strata at about elevation -15 was assigned the same V_s value as Young Bay Mud at that depth. The site specific Young Bay Mud and Older Bay Alluvium V_s model profile was used down to a depth of about 100 feet, after which the Schneider et al. (2000) Bay Mud profile was used to a depth of about 500 feet. From a depth of 500 feet down to about 1,000 feet, the Schneider et al. (2000) profile has been extrapolated as shown in Figure 5-33.

The Caltrans V_s profile is superimposed on both the Schneider et al. (2000) profile and the profile used in this investigation for Young Bay Mud and Older Bay Alluvium in Figure 5-33. Also shown is the Schneider et al. (2000) younger alluvium profile, which was used for the sand layer at about elevation -250. Although not shown on Figure 5-33, bedrock was modeled with a shear wave velocity of 2,950 fps (900 m/s).

Due to anticipated consolidation of Young Bay Mud and Older Bay Alluvium in response to the load imposed by the overlying MSW, the shear wave velocity of these materials beneath MSW was increased from the model described above. Since shear wave velocity is proportional to overburden stress raised to the 0.25 power, the shear wave of these materials was increased in accordance with the following relationship:

$$V_{s-MSW} = V_{s-o}\left[\frac{\sigma'_{v-MSW}}{\sigma'_{vo}}\right]^{0.25}$$

where:

$V_{s\text{-}MSW}$	=	increased shear wave velocity beneath MSW
$V_{s\text{-}o}$	=	shear wave velocity in free field
$\sigma'_{v\text{-}MSW}$	=	effective vertical stress in beneath MSW
$\sigma'_{v\text{-}o}$	=	effective vertical stress in free field.

For example, the shear wave velocity of sandy strata at elevation -20 and beneath 34 feet of MSW was increased from 600 fps to 650 fps. Likewise, the shear wave velocity of sandy strata at elevation -55 was increased from 850 fps to 930 fps in response to 220 feet of MSW loading. These calculations took into account the unloading of the natural soil materials prior to the reloading by MSW.

6.0 SITE-SPECIFIC GROUND MOTION EVALUATION

Due to the presence of soft, clayey soils at the NISL, a site-specific evaluation was performed to estimate the response of site soils and the overlying landfill to cyclic loading from the maximum credible earthquake on the nearby Hayward fault. Development of a target ground motion for design and selection of representative earthquake acceleration time histories, which are discussed in the first two subsections below, generally follow the recommendations of Dr. Jonathan Bray of the University of California at Berkeley, whose report is presented in Appendix I-1 (Bray, 2008).

The development and results of time-domain ground motion modeling are discussed below; the results of this modeling were used as input for the liquefaction triggering evaluation discussed in Section 7.0.

6.1 SEISMIC HAZARD ASSESSMENT BASIS

Seismic hazard assessments are performed to determine seismic parameters for a project, such as earthquake magnitude and peak ground acceleration (PGA). A seismic hazard assessment for a given site can be conducted by two different methods: deterministic or probabilistic. A deterministic seismic hazard analysis (DSHA) looks at ground motions generated by earthquakes on individual faults, and the PGA is the largest acceleration value at the site based on all known fault sources. The deterministic PGA at a site is a function of the site-to-source distance, the earthquake magnitude, the rupture scenario, and the selected attenuation relationship used to model the diminution of shaking intensity with distance from all possible fault sources within a reasonable distance *considered individually*.

A probabilistic seismic hazard assessment (PSHA), on the other hand, determines the PGA by assigning probability distributions at every step in the process, including recurrence interval, moment magnitude (M_W), distance, rupture dimensions, and attenuation. Rather than reflecting a particular earthquake scenario, the seismic hazard at a given location (represented by PGA, M, etc.) is an aggregate from all potential sources.

Although PSHA's are gaining wider acceptance and form the basis for current building codes, DSHA's are more commonly used for solid waste landfills and dams in areas of high seismicity, such as the San Francisco Bay Area specifically. In fact, the California Division of Safety of Dams utilizes a deterministic methodology to develop ground motion parameters for all dam safety evaluations. Accordingly, the seismic hazard assessment for the NISL, which is described herein, is deterministically based.

California Code of Regulations Title 27 (Title 27) requires that stability analyses performed for a Class III landfill be based on the expected PGA at the site associated with the maximum probable earthquake (MPE). There has been a recent trend by Regional Water Quality Control Boards (RWQCBs), however, to require the use of the maximum credible earthquake (MCE)[2] in such analyses. The MCE has been defined as

[2] Also known as maximum earthquake. For this report, these terms are considered interchangeable.

the "maximum earthquake that appears capable of occurring under the presently known tectonic framework" (CDMG, 1975). The analyses for the NISL presented below have been performed using DSHA-based ground motion parameters at the site associated with an MCE-level event.

6.2 REGIONAL SEISMICITY

Geologic structures in the seismically active San Francisco Bay region are controlled by the major northwest-trending San Andreas fault system. The Hayward and Calaveras faults, which are splays of the San Andreas fault, are located in the Diablo Range on the east side of the San Francisco Bay (Figure 2-3). The San Andreas fault is located within the Santa Cruz Mountains west of the San Francisco Bay. The project vicinity has experienced strong shaking from earthquakes during historic times, notably the 1906 event on the San Andreas fault and the 1868 event on the Hayward fault. Table 6-1 presents a list of historic earthquakes greater than M_w 5.0 within 100 km of the site and the estimated PGA at the NISL site associated with these events.

6.3 PRELIMINARY SITE GROUND MOTION ESTIMATES

Table 6-2 presents the estimated MCE moment magnitude and site PGA for all faults within 100 km of the NISL site. These estimates were made using software (Blake, 2007) that employs DSHA techniques. In order to estimate ground motion parameters for the MCE scenario, such as PGA, this software uses the 2002 US Geological Survey/California Geological Survey (USGS/CGS) fault model and a user-specified attenuation relationship; for preliminary estimates, the relationship by Abrahamson and Silva (1997) was used. Attenuation relationships are mathematical equations that model the decay of seismic waves as they travel from the source fault to the project site.

The Abrahamson and Silva (1997) attenuation relationship used for preliminary estimates of the NISL site PGA is one of the most commonly-used and, along with three to four 1997 relationships by other authors, represents the current state-of-practice for ground motion estimation. Due to the importance of the project and the relatively high estimates of PGA at the site, however, ground motion estimates used for project design employed leading-edge Next Generation Attenuation relationships, as discussed in Section 6.5 below.

6.4 DESIGN EARTHQUAKES

6.4.1 SOURCE-TO-SITE DISTANCE

According to the State of California Alquist-Priolo Special Studies Zones (A-P SSZ) maps (CDMG, 1974, 1982), the NISL site is approximately 3.3 and 10.8 km (2.1 and 6.1 miles) southwest of the Hayward fault and Calaveras fault, respectively, and 24.9 km (15.5 miles) northeast of the San Andreas fault (see Figure 1-1). Note that the above source-to-site distance for the Hayward fault, the controlling fault for design purposes, is different from that of the US Geological Survey/California Geological Survey

(USGS/CGS) fault model, for which the source-to-site distance would be 6.0 km (Table 6-2). [Since the USGS/CGS 2002 Fault Model is widely used in computer codes, the locations of faults in that model are necessarily simplified.] The closer (and more conservative) 3.3 km value based on the more accurate A-P SSZ maps will be used for this deterministic NISL seismicity evaluation.

6.4.2 DESIGN EARTHQUAKE SOURCE PARAMETERS

The Working Group On California Earthquake Probabilities (USGS, 2003) has estimated MCE moment magnitudes for individual- and multi-segment ruptures of the Hayward, Calaveras, and San Andreas faults, which are the faults that would likely produce the largest ground motions at the NISL site. For a Hayward fault event that would rupture the Hayward South and Hayward North fault segments, the mean MCE magnitude (M_w) has been estimated as M_w 6.9. Similarly, mean MCE magnitudes of M_w 6.9 and M_w 7.9 have been estimated for multi-segment events on the Calaveras and San Andreas faults, respectively (USGS, 2003). Because the Hayward and Calaveras faults are estimated to have the same MCE magnitude, yet the latter is more than twice as far away, the Calaveras fault has been dropped from further consideration for the NISL project.

Although the MCE for rupture of Hayward South and Hayward North fault segments has been estimated at M_w 6.9, a slightly larger M_w 7.1 event has been conservatively assumed for this event in order to maintain consistency with earlier estimates for this rupture scenario.

In summary, the following MCE-level events will be used for further analyses at the NISL site:

Fault	MCE Magnitude, M_w	Source-to-Site Distance, R
Hayward fault	7.1	3.3 km (2.1 miles)
San Andreas fault	7.9	24.9 km (15.5 miles)

6.5 DESIGN GROUND MOTION PARAMETERS

6.5.1 NEXT GENERATION ATTENUATION RELATIONSHIPS

The preliminary deterministic ground motion estimates discussed in Section 6.3 above were made using one of four commonly-used, *state-of-practice* attenuation relationships dating to 1997. Due to the importance of the project and the relatively high estimates of PGA at the site, however, final ground motion estimates for the NISL project will be based on the Next Generation Attenuation relationships.

The seismologists and engineers who were authors of the 1997 attenuation relationships have participated in a program sponsored by the Pacific Earthquake Engineering Research Center, National Science Foundation, the US Geological Survey (USGS),

Caltrans, PG&E, and other organizations to update their previously published attenuation relationships based on a consistent, up-to-date earthquake ground motion database. The results of this project, known as the Next Generation Attenuation Relationships (NGA), are considered the *best available science* and will be used by the USGS to update the National Seismic Hazard Maps. The NGA relationships will not be formally completed until mid-2008, but preliminary versions are currently available.

The estimation of ground motion parameters for the NISL project, including development of NGA-based response spectra and selection of representative earthquake acceleration time histories, has been performed by Professor Jonathan Bray, PhD of the University of California at Berkeley. A discussion of the response spectra development and time history selection is presented in the following sections. Dr. Bray's ground motion report for the NISL project is provided in Appendix I.

6.5.2 TARGET ACCELERATION RESPONSE SPECTRA

As earthquake vibrations radiate out from a causative fault, their intensity and frequency content are altered as they pass through geologic materials. Acceleration response spectra (ARS), which graphically present the intensity of vibrations across a range of periods (*period = 1/frequency*), can be used to characterize earthquake shaking (see Figure 6-1). ARS curves can be calculated directly from an earthquake time history record (i.e. acceleration vs. time), or the ARS curve for a hypothetical earthquake can be estimated given an earthquake magnitude, style of faulting, source-to-site distance, and site ground conditions. Such estimates are made using attenuation relationships; as discussed above, NGA-based relationships were used by Dr. Bray to estimate the target ARS curves for the design earthquakes discussed in Section 6.5.1 above.

All five preliminary NGA-based attenuation relationships[3] were used to estimate the 5 percent damped bedrock motion ARS curves for the two MCE-level design events discussed above, i.e. a M7.1 event 3.3 km from the site on the Hayward fault and a M7.9 event 24.9 km from the site on the San Andreas fault (Figure 6-1). Based on the above attenuation relations, estimated PGA (i.e. the spectral acceleration at the zero period) for the MCE on the Hayward and San Andreas faults ranged from 0.35g to 0.50g (avg. 0.42g) and 0.15g to 0.20g (avg. 0.18g), respectively. Since the estimated site-response to the San Andreas event is considerably lower than that of the Hayward event for all periods, including the most critical period range of engineering interest (i.e. under about two seconds), the San Andreas event is considered non-critical and will not be discussed further.

At a distance of 3.3 km (2.1 miles), the NISL is relatively close to the Hayward fault. Research has shown that the amplitudes of long period motion (i.e. greater than about 0.6 seconds) are increased for sites close to fault rupture due to phenomenon known as near-fault forward-directivity. As a result, the Hayward fault average ARS was modified per Abrahamson (2000) using a modification to the Somerville et al. (1997) procedure to

[3] Abrahamson and Silva (2007), Boore and Atkinson (2007), Campbell and Bozorgnia (2007), Chiou and Youngs (2006), and Idriss (2007)

account for this phenomenon. The modified ARS shown in Figure 6-1 is the target ARS for an MCE event at the NISL.

6.6 REPRESENTATIVE ACCELERATION TIME HISTORY RECORDS

A suite of seven bedrock "outcropping" acceleration time history records were selected by Dr. Bray as a basis for modeling the ground response at the NISL due to an MCE-level earthquake (see Table 6-3). By considering style of faulting, source-to-site distance, duration of strong shaking, magnitude, site conditions, forward-directivity, and other conditions, the suite of seven earthquake records was selected to be collectively representative of the Hayward fault MCE. Four of the records (1992 Landers/Lucerne, Coronado Bridge Synthetic, 1994 Northridge/Pacoima Dam, and 1999 Kocaeli/Gebeze) possess forward-directivity effects. Although actual earthquake records are generally preferable to synthetic records, the Coronado Bridge record, which was developed by Dr. Norm Abrahamson of Pacific Gas & Electric to simulate magnitude 7 near-fault motions, was included in the suite of earthquakes due to the scarcity of large, near-fault empirical records.

Each of the seven selected earthquake records was individually scaled such that the average ARS of all records matched the target ARS as closely as possible, particularly for periods below two seconds (Figure 6-2). In addition, the average PGA for these seven scaled records (0.41g) is quite close to the target PGA of 0.42g.

6.7 SHAKE GROUND MOTION ANALYSES

The computer program SHAKE2000 (Ordóñez, 2007) was used to model the effects of local site conditions on ground response during an earthquake. SHAKE2000 is a pre- and post-processor for the widely used one-dimensional, equivalent-linear, frequency domain ground response program SHAKE (latest version: SHAKE91, Idriss and Sun, 1992). SHAKE2000 prepares input files and then runs a slightly modified version of SHAKE91 (modified to fix the incorrect output of total stress in Option 2), so the analysis results are essentially from SHAKE91. Although SHAKE is the most widely used ground response program, studies comparing it to fully-linear programs and case histories have found that it can over-predict the ground response, particularly at higher levels of shaking (e.g., Bardet, et al., 1992; Ching and Glaser, 2001). The results of the analyses discussed below are, therefore, considered conservative.

Since SHAKE models ground response of wave propagation through a one-dimensional column of soil, nine soil profiles were used to model different conditions at the site (Tables 6-4A to 6-4I and Figure 6-3). These included a *free field* (away from the landfill) a perimeter levee (a.k.a. the *dike* that surrounds Newby Island) and 7 locations inboard from the levee (each with a different waste configuration – varying waste height from *35 feet to 275 feet of MSW*). These soil profile locations were selected in the context of both liquefaction triggering analysis and deeper foundation failures, so they incorporate a light to heavy overburden stress.

The *free field* profile models the site soil conditions outboard of the perimeter levee with Young Bay Mud underlain by sandy strata and Older Bay Alluvium (Figure 6-3). As discussed above in Section 5.1, the presence of sand layers is non-uniform and sporadic, often occurring at different elevations in adjacent borings and often not appearing at all. The *free field* profile has sand layers at about elevations -15, -25, and -50 (approximating the most common depths at which potentially liquefiable soils were encountered along the southern portion of the perimeter levee) and at about elevation -250. The remaining profiles reflect the same stratigraphic conditions, except where they have been modified by excavation or placement of overlying landfill strata (e.g., levee fill, compacted clay subgrade, and MSW). In Profiles 5 to 9, Young Bay Mud is not modeled because these materials were reportedly removed prior to the placement of MSW. Because the depth to bedrock at the site is greater than 625 m (2,000 feet)[4], Older Bay Alluvium was carried down to the base of the model at about 1,000 feet. Trial runs performed for this investigation have shown that the change in ground response by lowering the base of the model from 500 feet to 1,000 feet was not significant.

Five sets of modulus reduction and damping curves were used to model the materials at the site. For clayey materials (Young Bay Mud, Older Bay Alluvium, and levee fill), Vucetic and Dobry (1991) curves were used based on the average plasticity indices determined in this investigation (Figure 6-4). Since the shallowest sandy strata at about elevation -15 often contained significant fines and was closely interfingered with clayey soils, this material was modeled with modulus reduction and damping curves between those of Seed and Idriss (1970) for sand and Vucetic and Dobry (1991) for a low plasticity clay (Figure 6-5). For other sandy strata and Older Bay Alluvium soils deeper than about 100 feet, EPRI (1993) cohesionless soil curves, which represent gravels, sands, and low plasticity-index clays and are based on depth to the strata, were used (Figure 6-6). MSW was modeled using Matasovic and Kavazanjian (1998) curves (Figure 6-7) and bedrock was modeled using the rock curves of Schnabel (1973) (Figure 6-8).

The shear wave velocity profiles used for the SHAKE analyses are shown in Figures 5-32 and 5-33. One profile was used for Young Bay Mud and Older Bay Alluvium, and a second profile was used for the sand layer at elevation -250. The development of these profiles is discussed above in Section 5.4.

As discussed in the previous section, seven earthquake time histories were used in the site-response modeling: six recorded time histories and one synthetic. The earthquake records used are summarized in Table 6-3. Acceleration response spectra for the earthquake input motions are presented in Figure 6-2.

The maximum ground acceleration (i.e. PGA) from the SHAKE analyses for all earthquake records and all soil profiles ranged from 0.24 g to 0.63 g (see Appendix H). The average PGA for *free field* and *dike* profiles, which would be the most critical profiles in terms of liquefaction triggering, ranged from 0.23 g to 0.61 g, with an average of 0.41 g. Note that the average PGA is about the same as the average peak acceleration

[4] Schneider et al. (2000)

of the input records, which was 0.42 g, suggesting neither attenuation nor amplification of the shaking intensity by the soil column overlying bedrock, on average.

A summary of average parameters for the SHAKE runs for seven earthquake records and the free field and dike profiles described above is presented in Table 6-5. Complete analysis summaries of each of the sixty-three earthquake-soil profile pairs are presented in Appendix H.

7.0 LIQUEFACTION

Liquefaction is a phenomenon whereby loose, sandy soils below the water table loose strength in response to the cyclic build up of earthquake-induced groundwater pore pressures. In severe cases, liquefied soils can lose nearly all strength, causing slope failures, ground distortion, and damage to overlying structures.

7.1 HISTORIC LIQUEFACTION

Liquefaction-induced cracking, lateral spreading, and sand boils were reported to have occurred south of Newby Island along Coyote Creek during the 1906 earthquake (Emcon, 1972). In addition, Plate 1.2 of the "Seismic Hazard Zone Report for the Milpitas 7.5-Minute Quadrangle" (California Geological Survey, 2004) shows Coyote Creek in the vicinity of Newby Island as a "reach of river along which multiple failures were recorded." Further, the US Geological Survey has recently published maps for the San Francisco Bay Area that rate Newby Island has having "Very High" liquefaction susceptibility (Figure 7-1). No specific information has been found to substantiate the occurrence of past liquefaction at Newby Island, but it cannot be ruled out.

7.2 LIQUEFACTION ASSESSMENT

An extensive assessment of liquefaction triggering at the site using CPT-based procedures was performed using three different methods. Spreadsheets showing the results of these calculations at two CPT locations are presented in Appendix E. Spreadsheets showing the results of alternative SPT-based liquefaction assessment calculations are presented in Appendix G. The average liquefaction factor of safety results for the three CPT-based methods is graphically presented on Plates 1 to 6 superimposed on geologic cross sections.

7.2.1 LIQUEFACTON TRIGGERING ASSESSMENT METHODS

The liquefaction potential at the NISL site was evaluated in general accordance with the recommendations of the 1996 National Center for Earthquake Engineering Research (NCEER) and 1998 NCEER/National Science Foundation Workshops on Evaluation of Liquefaction Resistance of Soils (Youd, et al., 2001). This procedure has been verified with case history data down to depths of about 15 m (49 ft.), which encompasses the vast majority of reported liquefaction cases. In addition to Youd, et al. (2001), liquefaction triggering assessments were carried out by two other, more recent methods, and the final liquefaction factors of safety, FS_{LIQ}, were averaged for all three methods. The first of these two additional methods is Moss, et al. (2006), which is essentially the method outlined by Professor Raymond Seed, PhD of the University of California at Berkeley in his "Queen Mary" lecture (Seed, et al., 2003). For the last method, Professor Ross Boulanger, PhD of the University of California at Davis (UC Davis) kindly provided a draft copy of a soon to be published Earthquake Engineering Research Institute monograph that he has co-authored with Dr. Izzat Idriss, Professor Emeritus at UC Davis (Idriss and Boulanger, 2007 draft). This latter document is a refinement of an earlier

paper (Idriss and Boulanger, 2004) to account for the presence of fines in potentially liquefiable soils.

Though the liquefaction assessment procedures discussed above have been extrapolated for use with CPT data at depths as great as 100 ft., the sand/silty sand strata encountered higher up in the profile at the site are the main focus of this evaluation. Liquefaction triggering of deeper sandy strata (i.e. below about elevation -60) was less widespread than in the shallower strata. Reports in the literature of liquefaction at these depths are uncommon, either because liquefaction at such depths is, in fact, uncommon, or because if it has happened, surface manifestations, such as sand boils or damage to facilities, are rare. Since it is expected that the applied cyclic stress will be lower for these deeper sandy strata, should it occur at greater depth, liquefaction is likely to be relatively isolated and thus capable of being bridged by overlying and/or adjacent unliquefied soils.

Due to the large number of CPTs at the site and the advantage of a continuous profile that this technique provides, CPT-based procedures are becoming the more preferred method of liquefaction triggering assessment within the profession. As with a SPT-based approach, CPT-based procedures divide the liquefaction resistance by demand to arrive at a factor of safety against liquefaction, FS_{LIQ}. The demand is calculated from the site PGA and is corrected for earthquake magnitude. Based on the site-specific ground motion analyses from this investigation (Section 6.0), a PGA of 0.41 g and an earthquake magnitude of M_w 7.1 were used as inputs to the liquefaction assessment.

CPT data are generally acquired on every 5 cm (2 inches) of penetration. Boulanger, et al. (1997) recommend 0.6 m (2 ft.) as a reasonable interval over which to average CPT data for development of CPT-based liquefaction correlations. This thickness corresponded to the thickness of liquefiable layers experiencing significant deformations in that study, and it also corresponds approximately to the measurement interval for the SPT, which is widely used in liquefaction assessment. While using data from every 5 cm of CPT penetration might overstate the risk of liquefaction, we felt that averaging data over an interval of 2 feet might miss potentially liquefiable strata. Accordingly, a 0.5-foot running average of CPT data was used in this investigation for all three liquefaction assessment methods.

Research has shown that a thin layer of sand surrounded by soft clay cannot develop the full tip resistance due to the yielding of material above and below. Corrections were thus made for thin layers of sand in all three methods. For the Youd, et al. (2001) and Moss, et al. (2006) analyses, the thin layer corrections recommended in these respective papers were used. Since the Idriss & Boulanger (2007) method was silent on the thin layer issue, the correction of Moss, et al. (2006) was used. Likewise, since the Idriss & Boulanger (2007) method requires the input of soil fines (rather than estimating it from CPT data), fines content for this method was estimated from CPT data using the recommendation of Robertson and Wride (1998). As was discussed above in Section 5.1.3 and presented in Figures 5-3A to 5-3G, this method appears to adequately estimate the fines content of soils at the NISL.

7.2.2 AGE EFFECTS ON LIQUEFACTION TRIGGERING

As shown by numerous researchers, the resistance of sand to liquefaction triggering increases with the age of the deposit (Arango, et al., 2000; Leon et al., 2006). This effect is apparently not entirely reflected by CPT or SPT penetration resistances, so liquefaction evaluations for older sands (i.e. pre-Holocene) using these data and based on the commonly-used correlations may be overly conservative. The works cited above present curves of for assessing the increase in the cyclic strength of sands with time (i.e. a decrease in liquefaction triggering susceptibility) on a logarithmic scale, and these curves span 10^{-2} years (a few days) to 10^{8} years (Figure 7-2). The curves are based on both laboratory studies (short term) and case histories (long term).

Using the methodology suggested by Leon et al. (2006), the evaluation of liquefaction resistance is normalized to that of a "freshly deposited" soil; both the penetration resistance and the liquefaction triggering resistance are normalized by factors (c_{CRR} and c_{aging}, respectively), and the evaluation proceeds using the desired triggering correlation (e.g. Youd, et al., 2001; Moss et al., 2006). With this methodology, "freshly deposited" is at the left side of the curves (Figure 7-2), which equates to a few days old. As was pointed out by Professor Bray during discussions on the NISL project, however, liquefaction triggering correlations, such as the three methods used in this study, are based on a case history database of predominantly constructed fill and Holocene soils which, though "young," are older than a few days. The result of using the Leon et al. (2006) methodology normalized to 10^{-2} years is to unconservatively bias the liquefaction triggering assessment. On the recommendation of Professor Bray, the liquefaction triggering evaluation at the NISL used the Leon et al. (2006) methodology normalized to 10 years, a reasonable average age for the liquefaction case history database. The result of this normalization, which is graphically illustrated in Figure 7-2, was to conservatively reduce the estimated liquefaction resistance of NISL soils by about 10 to 15 percent compared with the Leon et al. (2006) methodology as published. This age correction procedure for liquefaction triggering was only applied to Pleistocene sandy strata, which were typically below about elevation -5 to -20 (see Plates 1 to 6). The range of laboratory-determined ages for these soils is about 12,000 to 28,000 years old, as shown in Figure 5-2. The age-depth regression line shown in this figure was used to estimate the age of Pleistocene soils at any depth for which the liquefaction triggering evaluation was performed.

7.2.3 LIQUEFACTON TRIGGERING ASSESSMENT RESULTS

The results of the liquefaction assessment are presented in Plates 1 to 6 in the form of plotted average FS_{LIQ} values for each CPT boring. As shown thereon, only calculated FS_{LIQ} values less than 1.3 are presented. A guidance document for implementing California seismic regulations suggests that a FS_{LIQ} value of 1.3 is appropriate for "assessing hazards related to flow failure potential for large magnitude earthquake events...." (Martin and Lew, 1999). The calculated FS_{LIQ} values were graphically broken down by FS_{LIQ} range: blue triangles represent $1 < FS_{LIQ} \leq 1.3$ and red dots represent $FS_{LIQ} \leq 1.0$. For strata above groundwater or where future landfill development will

result in removal or dewatering of sandy soils, Plates 1 to 6 do not show FS_{LIQ} data. While CPT soil behavior (e.g. "Silty Sand to Sandy Silt") was determined based on the estimated groundwater level at the time of the CPT, liquefaction triggering calculations were based on the post-development groundwater level.

Figures 7-3A to 7-3T present a plan view summary of the liquefaction status of each of the 109 CPTs performed at the NISL site for this investigation. This summary groups the CPT-based liquefaction potential into 5-vertical-foot *packages* by elevation. Each five foot *package* is colored in accordance with the worst-case liquefaction potential identified anywhere within each *package*. For example, even if only one, two-inch liquefiable layer was identified in a five-foot zone, the entire zone was colored in accordance with the calculated liquefaction susceptibility for that one zone. Zones with no sandy strata are colored blue, and where the existing ground surface is below the indicated elevation zone[5], no color is assigned. Since these figures were used to assess the continuity of potentially liquefiable strata, as might be reflected by the creek channel mode of deposition, the packages are color coded regardless of the post-development groundwater level, unlike Plates 1 to 6. As such, they may show individual *packages* as potentially liquefiable, even though the soil therein represented may later be removed during landfill cell construction.

Figure 7-4 presents a simplified cross-sectional liquefaction summary cut along the same section lines as Plates 1 to 3. As with Figures 7-3A to 7-3T, this figure groups the CPT-based liquefaction potential into 5-vertical-foot *packages* by elevation in accordance with the worst-case liquefaction potential identified anywhere therein. Again, zones are colored regardless of the post-development groundwater level, which is shown as a dashed line in this figure.

As indicated on Plates 1 to 6, clayey soils are generally non-liquefiable and as a result, plotted FS_{LIQ} are confined to sand or silty-sand strata. As can be seen, many FS_{LIQ} values are considerably less than unity. Because of the relatively loose state of many of the sand/silty sand strata at the NISL, particularly in the upper horizons, the calculated FS_{LIQ} is relatively insensitive to modest changes in the PGA value (for example, lowering the PGA from 0.41 to 0.3 would still leave many instances of FS_{LIQ} below unity). The low calculated FS_{LIQ} values, combined with the fact that liquefaction is known to have occurred in the general vicinity of Newby Island in the historic past, leads us to conclude that potential liquefaction of loose sand zones in native soils and fills is critical design consideration for the NISL. Though these soils are thought to be relatively discontinuous channel and deltaic deposits, they exist with sufficient continuity in some areas so as to require further investigation during detailed landfill cell design and, in some areas, will likely require mitigation to prevent liquefaction-related stability failures.

[5] CPTs performed inside the perimeter levee on the south side of the landfill were generally at or below sea level, but the highest elevation zone for Figures 7-3A to 7-3T is from elevation +5 to 0.

8.0 SLOPE STABILITY

In order to assess the static and seismic stability of the proposed vertical expansion of the Newby Island Sanitary Landfill, a number of typical cross-sections were constructed around the perimeter of the site (Figure 8-1). Slope stability cross sections are identified by perimeter levee station number. Cross sections were selected both to be representative of conditions around the site, as well as to capture the most critical slopes at the NISL based on height, steepness, and the geometry of foundation soils and/or landfill components, such as the liner.

Due to the critical nature of potential liquefaction-related slope stability failures, more detailed slope stability stratigraphic models, which are referred to as liquefaction *Type Sections*, were constructed at four of the cross sections shown in Figure 8-1. These Type Sections are designated as TS-1, TS-2A, TS-2B, and TS-3 and are located at perimeter levee Stations 50+46 25+93, 150+00, and 45+00, respectively (Plate 7). The selection of these locations for Type Sections followed the same rationale as above, i.e. to be both representative of site conditions and to capture the most critical slopes.

In addition to several liquefaction cases, which are discussed in more detail below, four slope stability potential failure modes were analyzed for each of the cross sections shown in Figure 8-1: deep foundation failures through Older Bay Alluvium; failures through MSW, along the liner, and out into Older Bay Alluvium/Young Bay Mud outboard of the perimeter levee; outer-slope failures through MSW and into Young Bay Mud, and failures through MSW and then completely along the liner. For unlined and Chapter 15 lined sections of the NISL (sections at Station 96+00 and higher), the latter failure mode was not applicable.

Finally, stability analyses for both prescriptive and alternative final covers were performed.

Table 8-1 presents a complete summary of all slope stability analyses results, including seismic deformation analyses as discussed below. Output from the slope stability computer runs is presented in Appendix F.

8.1 SLOPE STABILITY METHODOLOGY

After the cross sections were prepared and the soil and groundwater parameters selected, the cross sections were transferred into a computer program for analysis. Two-dimensional static and pseudo-static stability analyses were performed by **GLA** using the Morgenstern-Price/constant side force option within the limit equilibrium slope stability software, SLOPE/W (GEO-SLOPE International, 2007, release 7.11, May 2008). A variety of search procedures were utilized to determine the critical potential failure surface for a given cross section. Generally, the entry and exit method was selected since control is maintained with both the number and location of entry points of the potential failure surface and the exit points. In addition, a specific number of radius points of the failure surface can also be selected. Many of the individual analyses took advantage of a relatively new slip surface "optimization procedure" within SLOPE/W, wherein the lowest factor of safety potential slip surface at the end of standard limit equilibrium iterations is further iterated on a segment-wise basis to find potentially lower factor of

safety (and often non-circular) slip surfaces. Use of this procedure will always result in a factor of safety that is as low or lower than if it had not been used (i.e. it is conservative). In certain instances, generally with the pseudo-static stability calculations, when the entry and exit method failed to converge on a factor of safety of unity, the most critical failure surface from the entry and exit solution was specified as a "trial failure surface". The factor of safety along this failure surface was then calculated and then "optimized" to yield the most critical failure surface and the lowest (most conservative) factor of safety. Even in some of these cases, this fully-specified surface wouldn't converge, so the next lowest yield acceleration is reported (e.g., rather than FS=1.0, FS = 1.029 and k_y=0.259 for Station 150+00 MSW&YBM potential failure is conservatively reported).

In some stability analyses, the materials below a given depth were made impenetrable (i.e. "bedrock") to prevent analyses extending below, forcing the failure into overlying materials. When this technique was used, the "bedrock" was shaded blue in each individual cross section. Material labels were not changed, however.

8.2 TREATMENT OF PORE PRESSURES

Groundwater and leachate pore pressures were modeled in their anticipated post-development condition, which is generally affected by on-site pumping for surface and groundwater control. This pumping has resulted in an inward-directed groundwater gradient (Figure 2-6). The post-development condition also considers on-going program to reduce the hydraulic head to mean sea level within the existing unlined waste prism.

Based a correlation of local tide tables with actual tide and survey measurements made in the unnamed slough on the south side of the NISL in 2007, groundwater outside the perimeter levee was modeled at elevation +0.7, which represents the average tidally-influenced water elevation at the NISL site. Note that this elevation should not be confused with the project datum, which is *mean sea level* in the North American Datum of 1927. For the existing and future Title 27 lined cells, the groundwater elevation beneath the perimeter levee was determined by CPT pore pressure dissipation tests *in areas of existing developed landfill cells* to be generally about elevation -6, though in some places it was found to be somewhat higher. Based on the above information and considering the subgrade slope drain from which water will be pumped continuously, the post-development groundwater level was estimated for the slope stability cross sections. The groundwater as modeled is reflected in Plates 1 to 6 and in the slope stability output in Appendix F.

As MSW is initially placed in a landfill cell, the relatively slow draining Young Bay Mud and Older Bay Alluvium strata will cause excess pore pressures to build up which will tend to negate the beneficial effect of increased MSW overburden. With time, however, these excess pore pressures will drain away, a process which is enhanced by the presence of sandy strata within the clay matrix. As discussed in Section 5.2.4, piezometric measurements beneath the subgrade of Subarea 10 would suggest that after about 5 years, about 98% of the primary consolidation (i.e. pore pressure build-up) due to waste

placement would have occurred. Based on this data and on the fact that waste in the unlined northern cells of the NISL have been in place for many years, 75% pore pressure dissipation in clayey soils was assumed for all stability analyses.

As discussed in Section 5.3.5.1, the strength of Older Bay Alluvium and Young Bay Mud was characterized using an S_u/P' model, where P' is the effective stress (i.e. total stress minus pore pressure). The MSW loading contributes to the total stress *and* increases the pore pressure, but the latter by only 25% of the MSW load, which equates to 75% pore pressure dissipation. The SLOPE/W slope stability computer program handles this situation with a parameter called B-bar. A B-bar of 1.0 means that 100% of the overburden stress gets translated into pore pressure (i.e. no drainage), while a B-bar of zero means that none of the overburden is translated into pore pressure (i.e. complete drainage = full beneficial effect of overburden stress). B-bar was set to 0.25 for Older Bay Alluvium and Young Bay Mud.

8.3 POTENTIAL SEISMICALLY-INDUCED DISPLACEMENTS

8.3.1 DISPLACEMENT ESTIMATION METHODOLOGY

For Class III landfills, Title 27 requires that further analysis be done to demonstrate that a proposed design will be functional during the MPE if the pseudo-static slope stability analysis indicates a factor of safety less than 1.5. As discussed in Section 6.1, analyses for this project are based on an MCE-level event.

Since in most cases, pseudo-static factors of safety less than 1.5 would be realized for the site PGA of 0.41, deformation analyses were performed by the Newmark (1965) method using the results of the SHAKE site-specific site response analyses discussed in Section 6.7. Using the SHAKE2000 program, displacements were calculated via direct integration of the pseudo-static yield acceleration (k_y) versus horizontal equivalent acceleration (HEA) time history representing the weighted-average motion for a given potential failure surface. The results of these analyses are presented in Table 8-2 and graphically in Figure 8-2 as a series of displacement-k_y curves for various potential failure surfaces.

Four to five HEA time histories along the base of a representative sliding surface (Figure 6-3) for each failure mode were averaged per Abramson, et al., (2002) to create the representative HEA time history for integration against k_y. This process was repeated for each of the seven earthquake time histories discussed in Section 6.6, and the results of each were averaged to create the curves in Figure 8-2. Using these curves and the results of the pseudo-static slope stability analyses (i.e. the k_y and the failure surface geometry), potential seismically-induced deformations were estimated for each cross section and potential mode of failure as shown in Table 8-1. In some cases, interpolation between curves was required where a given failure geometry fell between those shown on Figure 6-3.

In a survey of industry practice regarding the seismic design of solid waste landfills lined with geosynthetic materials, Seed and Bonaparte (1992) report maximum calculated seismic displacements on the order of six to twelve inches as being acceptable for most conditions. Since that time, the industry and regulators have been moving toward the lower-bound of this range, i.e. that acceptable displacements of lined landfills should be less than about 6 inches. Accordingly, estimated seismically-induced permanent displacements on the order of 6 inches or less have been adopted as the criteria for acceptability at the NISL. For the northerly, unlined cells, however, where critical systems are less sensitive to movement, larger seismic displacements, say on the order of one to two feet, may be acceptable.

8.3.2 SEISMIC WAVE INCOHERENCE ON LARGE SLIDING MASS

Unpublished research (Rathje and Bray, 2008; Appendix I-4) suggests that as length of a sliding mass increases, the destabilizing effects of seismic waves are lessened due to spatial incoherence (i.e. the waves don't shake the whole mass in the same direction simultaneously). This work suggests accounting for this effect by applying a sliding block length reduction factor C_L that varies from 1.0 to 0.8 as the length of a slide mass increases from about 200 feet to 1,000 feet. The reduction can be applied to an HEA time history developed with one-dimensional site response analysis (i.e. SHAKE) prior to integration by the Newmark method.

Generally speaking, the most critical potential slope stability failure surfaces reported herein were on the order of 200 feet long, so this reduction factor was not used in the analyses in this report. Use of this reduction factor should be considered future analyses for design-level analyses of landfill cells and liquefaction mitigation, however.

8.4 POTENTIAL WASTE MASS FAILURE MODES ANALYZED

Various potential modes of failure for the proposed NISL vertical expansion are discussed below. Summaries of these analyses are presented on Figure 8-1 and Table 8-1. Note that potential failures discussed in Sections 8.4.1 to 8.4.3 do not consider liquefaction, which is considered separately in Section 8.4.4.

8.4.1 POTENTIAL DEEP FOUNDATION SOIL FAILURES

The static factor of safety for deep foundation stability (where the potential failure surface is well into the Older Bay Alluvium) for the NISL vertical expansion varies from 1.68 to 2.53. Seismically-induced deformations of between 0.5 to 2 inches were estimated. Given the conservative nature of the strength assumptions used for the deep Older Bay Alluvium, these factors of safety and displacements are considered acceptable.

8.4.2 POTENTIAL LINER FAILURES

Potential failures through MSW and then completely along the base and slope liner were analyzed for existing and future Title 27 landfill cells (sections at Station 65+00 and

below). Calculated static factors of safety for this condition varied from about 1.85 to about 2.48 and seismically-induced deformations of between 0 to 4 inches were estimated. As discussed above, these values are considered within the normal standards of practice for solid waste system design.

8.4.3 POTENTIAL COMPOSITE FAILURES: LINER/DEEP FOUNDATION SOIL

In addition potential failures completely along the liner, composite failures through MSW, along the liner, and out into Older Bay Alluvium/Young Bay Mud outboard of the perimeter levee or just through MSW and into Young Bay Mud were analyzed. For these potential failures into Older Bay Alluvium, static factors of safety ranged from 1.96 to 3.65, and seismically-induced deformations of between 0 to 3.5 inches were estimated. For these potential failures into Young Bay Mud, static factors of safety for this case ranged from 1.99 to 3.36, and seismically-induced deformations of between 2 to 6 inches were estimated. Due to the proposed sedimentation basin in the areas of Station 22+00, a slight alteration of the liner configuration from that shown in Figure 1-3 was made to raise the base of the liner and daylight it inboard of the basin. Also, as noted in Section 5.3.5.5, a textured HDPE liner will be used on both the base and slopes for future cells adjacent to perimeter levee Station 30+00 and below.

8.4.4 POTENTIAL LIQUEFACITON-RELATED FAILURES

As discussed in Section 7.2.3, potentially liquefiable sandy native soils and levee fills exist with sufficient continuity in some areas of the NISL as to present a critical stability issue. The occurrence of these materials is summarized in Figures 7-3A to 7-3T, Figure 7-4, and Plates 1 to 6, both spatially and with reference to FS_{LIQ}. In particular, inspection of the plates reveals that while potentially liquefiable materials may be more prevalent in certain zones than others, they are also sporadically discontinuous over short horizontal and vertical distances. This observation was also confirmed by detailed geologic mapping of the subgrade excavation for Subarea 12, as reported in Section 5.1.1.

In order to better characterize the potential risk due to liquefaction-related slope stability failures, detailed *Type Sections* were constructed in areas with more CPT data. In particular, Type Section TS-1 was constructed at Station 50+46 through a dense cluster of CPTs that were performed for a compaction grout test program (Plate 7). Type Section TS-1 represents the most detailed view available at the NISL of the distribution of potentially liquefiable materials. As with Plates 1 to 6, red dots to the left of a CPT on Plate 7 indicate $FS_{LIQ}<1.0$, and blue triangles indicate $1.0 < FS_{LIQ}$ 1.3; these absence of these symbols indicates a nonliquefiable material. As can be seen in the Detail TS-1A on Plate 7, even the more prevalent liquefiable strata at about elevation -10 is discontinuous over short distances, such as at CPT-54.

Liquefiable and nonliqueifable sands were modeled in the slope stability analyses with a high level of detail as shown on Plate 7. Where specific CPT data were unavailable, strata information was filled in guided by their occurrence in areas with more detailed CPT data. In this manner, Type Sections TS-1, TS-2A, TS-2B, and TS-3 located at

perimeter levee Stations 50+46 25+93, 150+00, and 45+00, respectively, were generated for use in stability analyses (Plate 7). Pseudo-static stability analyses were generally performed on these sections for shallow, intermediate, and deeper potential liquefaction failures at about elevations -15, -25, and -45, respectively. Note that at Type Section TS-2B, the shallow potentially liquefiable layer is at about elevation -1, and the deeper liquefiable strata were not present.

Due to the higher level of detail in the Type Sections, liquefaction was only modeled on these sections, and only for the applicable strata. Also, since Type Section TS-1 represents a worst-case liquefaction scenario, post-mitigation seismic displacements were only estimated for this Type Section. As with the pseudo-static stability analyses discussed above, seismic deformations were estimated with potential failure surfaces and displacement-k_y curves in Figures 6-3 and 8-2, respectively. For results of static and non-liquefaction pseudo-static stability analyses on the Type Sections, including seismic deformation analyses, see Sections 8.4.1 to 8.4.3 above.

8.4.4.1 Potential Elevation -15 Liquefaction Failures

Estimated liquefaction-related seismically-induced displacements of sandy strata at about elevation -15 on the southern side of the landfill (i.e. Title 27 lined cells) ranged from 11 to 23 inches (Table 8-1). These seismic displacements are unacceptably large, and mitigation will be required where future investigations demonstrate sufficient continuity of these strata to permit the failure mode described herein. Mitigation is discussed below in Section 8.4.5.

For the somewhat shallower sandy material within the perimeter levee (TS-2B; Station 150+00), the seismic displacement was estimated at 12 inches. Since this portion of the landfill is unlined, seismic displacements of this magnitude are likely tolerable. While some cracking and minor ground disturbance might be anticipated during the design earthquake event, catastrophic failure is not anticipated.

8.4.4.2 Potential Elevation -25 Liquefaction Failures

Estimated liquefaction-related seismically-induced displacements of sandy strata at about elevation -25 ranged from 1-½ to 18 inches. These seismic displacements span the range from acceptable to unacceptably large. Mitigation of this sandy strata will be required where future investigations demonstrate not only sufficient stratigraphic continuity, *but also* a high potential for liquefaction triggering.

On the northern, unlined side of the NISL, very little continuity of potentially liquefiable strata below about elevation -5 to -10 (i.e. within the Older Bay Alluvium) was found. Accordingly, this failure mode was not analyzed for Type Section TS-2B (Station 150+00).

8.4.4.3 Potential Elevation -45 Liquefaction Failures

Estimated liquefaction-related seismically-induced displacements of sandy strata at about elevation -45 ranged from 4-½ to 5 inches. Even under the conservative assumptions used for these analyses (e.g. that strata with $FS_{LIQ} < 1.3$ would liquefy), which resulted in a model with intermittent liquefaction at this depth, the estimated liquefaction-related seismically-induced displacements of sandy strata at this depth result are acceptable (i.e. less than 6 inches). Some cracking and minor ground disturbance might be anticipated during the design earthquake event, but catastrophic failure is not anticipated.

Due to lack of continuity of potentially liquefiable strata, this failure mode was not analyzed for Type Section TS-2B (Station 150+00).

8.4.5 MITIGATION OF POTENTIAL LIQUEFACITON-RELATED FAILURES

Based on the analyses described above, mitigation of the potential liquefiable sandy strata down to about elevation -25 to -30 likely will be required for Title 27 lined landfill cells at selected locations as determined by final design of liquefaction mitigation. A number of techniques are available to accomplish this mitigation, including densification, drainage, in-situ strengthening, replacement and recompaction, or a combination of the above. A memo by Richard Mitchell of RMC Geosciences, Inc. with more complete survey of liquefaction mitigation techniques applicable at the NISL, including recommendations, is presented in Appendix I-3.

For analyses in this report, a cement stabilization mitigation approach resulting in panels of soil within and beneath the perimeter levee with dramatically improved strength has been assumed (Figure 8-3). This type of in-situ treatment could be readily achieved with deep soil mixing or Transverse Shear Walls, among other techniques. Both involve mixing of in-situ soils with cement along walls or panels reaching down to and below the depth of potentially liquefiable soils. Though panels, walls, or interlocking columns could be constructed in various arrangements, for these analyses, the panels were assumed to be oriented perpendicular to the centerline of the perimeter levee.

Analyses of liquefaction mitigation were performed on Type Section TS-1, which represents the worst-case liquefaction section. Soil treatment was assumed to achieve a soil/cement slurry unconfined compressive strength (UCS) of 100 psi, and the soil replacement ratio[6] was varied until an acceptable seismic deformation was achieved based on the k_y-displacement curves in Figure 8-2.

Based on a slurry UCS of 100 psi and a replacement ratio of 12.5 percent, liquefaction-related seismically-induced displacements of sandy strata at about elevation -15 were estimated at 3-½ inches. Since slurry UCS values in the 300 psi range are routinely achieved, actual seismic deformations calculated during final mitigation design are likely

[6] The ratio of the volume of soil treated versus untreated, e.g. four-foot wide panels spaced 32 feet on-center perpendicular to the perimeter levee would represent a replacement ratio of 12.5 percent.

to be negligible. With this same slurry UCS and replacement ratio, seismic displacements of sandy strata at about elevation -25 were estimated as being negligible.

8.5 FINAL COVER STABILITY

At the end of active landfilling at the NISL final closure of the site will require the placement of a final cover intended to minimize the potential for moisture infiltration into the waste and potential for landfill gas escape from the landfill. California Code of Regulations (CCR) Title 27 § 21090 (a) require that landfill final covers be constructed according to identified minimum standards. For unlined Class III landfills, such as the northern portions of the NISL, the minimum regulatory requirements include a two-foot thick foundation layer, a one-foot thick low-hydraulic conductivity layer, and a one-foot thick vegetative layer. The prescriptive final cover for lined portions of the NISL would include the placement of a flexible membrane liner, so the cover section would include a one-foot thick foundation layer, a one-foot thick low-hydraulic conductivity layer, flexible membrane liner (FML) barrier layer, and a one-foot thick vegetative layer.

Analysis of the stability of prescriptive final cover systems placed over lined areas of the NISL after the end of active landfilling are included in Appendix F-3 and indicate a static factor of safety of 1.55 with a calculated displacement of less than an inch under an MCE earthquake event. In order to achieve this performance, a high-strength geogrid was included in the cover section. This small magnitude of displacement is considered acceptable for a final cover, since repairs can be made relatively quickly as a part of the post-closure maintenance of the site.

Regardless of the acceptable performance of a prescriptive final cover at the NISL, CCR Title 27, Section 21090 (a) allows for alternatives to the prescriptive standard if equivalent performance can be demonstrated. In response to recent developments in the standards of practice for the closure of solid waste facilities, alternative final covers that take advantage of the protection provided by deep rooting vegetation supported by soils with moderate permeability characteristics are becoming more common. Since these alternative final cover systems often yield a performance that is superior to the prescriptive standard, while being easier to construct and maintain, it is likely that some form of alternative final cover will ultimately constructed over the NISL. In recognition of this likelihood, the stability of an alternative final cover was also addressed (Appendix F-3). These calculations were completed using an assumption that the alternative final cover would consist of a monolithic section of on-site compacted soils and indicate a static factor of safety of 3.20 and negligible potential dynamic displacement; these conditions are generally consistent with those anticipated for prescriptive closure.

In summary, both the prescriptive and alternative cover designs analyzed herein provide acceptable static factors of safety in excess of 1.5, as well as acceptable dynamic performance under MCE scenario, i.e. displacements under an inch.

9.0 SETTLEMENT/CONSOLIDATION

Calculations were performed to estimate the magnitude of landfill foundation settlement in response to the static load of the landfill itself (i.e. consolidation) and due to seismically-induced dynamic consolidation of sandy strata. Calculations are presented in Appendix J.

9.1 CONSOLIDATION SETTLEMENT OF THE LANDFILL FOUNDATION

Analyses were conducted to estimate the amount of consolidation settlement that the landfill foundation would experience under the weight of the overlying MSW. Since the existing MSW on the north side of the landfill has been in place for decades, and thus the foundation beneath this waste has largely consolidated, the analyses were performed assuming only the proposed new MSW loading at the south side of the landfill on Cross Section 45+00, which reaches up to elevation +245 at the proposed top deck of the landfill (see Figure 9-1). As shown in this figure, the new MSW loading was approximated by a trapezoid 165 feet high and 1,800 feet wide at the base. The base of this idealized loading was carried down approximately to the pre-development ground surface so as to take credit for the soil removed during cell excavation. Note that the height of 165 feet includes a 10-foot reduction to account for the lighter weight of MSW as compared to soil below the pre-development grade.

Since there was no apparent trend with depth for coefficients of consolidation, recompression, and initial void ratio, (C_c, C_r, and e_o, respectively; see Table 5-2)), the average values of these parameters was assumed for calculations. Since these parameters would be expected to improve with depth, the calculations performed to an elevation of approximately -1,250 feet using these average values should be very conservative.

Modeled as described above, consolidation settlement due to MSW loading up to about elevation +245 is anticipated to result in about 13 feet of settlement at the maximum height of refuse and 4 feet of settlement at the LCRS sump at the toe of the north-facing slope (Points A and B, respectively, in Figure 9-1). The calculated differential settlement of 0.013 feet per foot (i.e. 1.3%) is well within the allowable strain capacity for the liner system. Anticipating settlement, current cells designs incorporate an average liner slope of about 1.8%, so a post-settlement of at least 0.5% will be maintained. Considering that the base of waste elevation in two borings on the north side of the landfill (LPZ-1 and LPZ-2) was approximately at the same elevation as it was reported to have been placed (i.e. elevation -10), the differential settlement is likely to be much less than the 0.013 feet per foot reported above.

9.2 DYNAMIC SETTLEMENT OF THE LANDFILL FOUNDATION

Analyses were performed to estimate the dynamic settlement based on the SHAKE-calculated site response from the design earthquake, i.e. a M_w 7.1 event at a distance of 3.3 km (2.1 miles) with a PGA of 0.41g. Three CPT records were chosen for this

analysis to represent the range of expected MSW loading: CPT-18, which is located near Station 45+00 in an area where very little sand was observed; CPT-31, which is located near 52+50 and was chosen since intermittent sand layers were observed from elevation -8 feet to -17 and from -43 feet to -53; and CPT-79, which is near Station 50+00 and also has intermittent sand layers from -18 to -28.

The data from the three CPT soundings were used as input into a dynamic settlement subroutine of the SHAKE 2000 (Ordonez, 2007) a computer program suite. The program uses raw data from the CPTs as input into the evaluation of the dynamic settlement of sands from the applied load of the design earthquake event. Analysis of settlement is performed by two methods: Ishihara and Yoshimine (1992), and Zhang, Robertson, and Brachman (2002). The results are presented in the table below and the output data is presented in Appendix J.

Analysis Method	**CPT-18**	**CPT-31**	**CPT-79**
Zhang, Robertson & Brachman, 2002	0.5 inches	4.4 inches	2.6 inches
Ishihara and Yoshimine, 1992	0.5 inches	4.5 inches	2.6 inches
Likely range of settlement:	**0.5 to 5 inches**		

The above values for CPT-18 and CPT-31 are in the area of the existing dike so the results will be similar to the above-modeled condition. The area of CPT-79 is in an area of a future lined cell so the future refuse overburden will further reduce the anticipated dynamic settlement.

The results of the analysis indicate that the dynamic settlement due to the design earthquake event may range from ½ inch and possibly up to 2 to 4 inches with a differential dynamic settlement of 1 to 2 inches in a horizontal distance of a few hundred feet. Given the typical geometry of site channel sand deposits, differential dynamic settlement is anticipated to be relatively small and the integrity of the landfill should not be significantly impaired.

10.0 TSUNAMIS AND SEICHES

A tsunami (sometimes incorrectly called a tidal wave) is a sea wave generated by large-scale sea floor displacements associated with large earthquakes, submarine landslides or volcanic activity which displace a relatively large volume of water in a very short period of time. Seiches are defined as oscillations in a semi-confined body of water due to earthquake shaking or fault rupture.

The California Office of Emergency Services has estimated that a tsunami entering the Golden Gate could be as large as 33 feet, but that such a wave would only be about 10% as high (i.e. about 3 feet) when it reached the southern San Francisco Bay[7]. Since the NISL site itself is about 5 miles east of the main portion of the southern San Francisco Bay, the height of any tsunami reaching site would certainly be less than 3 feet. As such, the threat of tsunami-induced damage to NISL facilities is insignificant.

Due to the limited stretch of open water adjacent to the NISL site, any seismically-induced seiche would be expected to be smaller than the worst-case tsunami discussed above. As such, the threat of seiche-induced damage to NISL facilities is insignificant.

[7] Quake06.org, Tsunami Fact Sheet, 2005.

11.0 CONCLUSIONS

On the basis of the field investigations, laboratory testing and analysis completed for this investigation, it is concluded that the proposed vertical expansion of the Newby Island Sanitary Landfill is feasible from a geologic and geotechnical perspective if the significant liquefaction mitigation measures summarized herein are implemented.

It should be noted that the potential liquefaction-related slope stability failures are generally limited to the front slope of the landfill adjacent to the perimeter levee, which is within the existing permitted footprint. As such, the proposed vertical expansion, *per se*, is not significantly impacted by the potential for liquefaction. Rather, the mitigation proposed herein would be recommended even if the vertical expansion were not being considered.

12.0 RECOMMENDATIONS

In order to mitigate the potential for liquefaction related slope instability and distress to the existing and expanded landfill improvements, a number of mitigation measures will be required. While the final selection of a mitigation plan and detailed design of mitigation are most appropriately deferred until the final construction level design drawings are prepared for the individual cells, **GLA** considers it important to demonstrate that mitigation technologies already exist and can be reasonably implemented for the geotechnical conditions identified during this study. More specifically:

- Where they exist with sufficient continuity, mitigation of loose, potentially liquefiable shallow to intermediate depth sandy strata at the NISL will be required. As mentioned in Section 8.4.5 and discussed more fully in Appendix I-3, the liquefaction mitigation methods which appear to be most applicable are those which strengthen the soil in place by the introduction of cement with some type of in-situ mixing. Theses methods include Transverse Shear Walls, Cement Deep Soil Mixing, Trench Cutting Remixing Deep Wall Method, and jet grouting. In undeveloped areas of the landfill where the shallowest sandy strata are determined to be the only potentially liquefiable layers, Dynamic Deep Compaction and Removal and Replacement may also be applicable. While an approach resulting in a block of soil within and beneath the perimeter levee with dramatically improved strength has been assumed for analyses of the mitigated condition in this report, any method which can stabilize potentially liquefiable soils, even one which may not yet have bee developed, should not be precluded from consideration for future mitigation.

- In order to mitigate potentially liquefiable conditions in a cost-effective manner, it will be important to more precisely delineate potentially problematic areas. As a result, it is recommended that additional, more closely spaced CPTs be completed prior to final selection and design of a mitigation system. CPT data should be analyzed both for stratigraphic information as well as for liquefaction triggering potential, the latter with the most reliable method available at the time. For this report, soils whose factor of safety against liquefaction triggering was less than 1.3 were considered to be potentially liquefiable per regulatory guidance. In line with the observation of Professor Stark (Appendix I-2) that using this 1.3 criterion may "tend to overestimate the potential seismic deformation and may also lead to overly conservative liquefaction mitigation requirements...," consideration should be given to lowering this criterion in future analyses. Lowering this criterion, say to 1.1 or 1.2, should be done in consultation with regulators and in keeping with the current standards of practice for liquefaction evaluation and mitigation.

- Continue the on-going program to reduce the hydraulic head within the existing unlined waste prism is recommended in order to improve stability conditions within this area of the landfill.

- Owing to the sensitivity of slope stability to groundwater elevations, it is recommended that lined cell subdrain systems be maintained until surface grades or subsurface conditions improve due to refuse settlement and/or subgrade consolidation such that

additional pore pressures can be accommodated without adversely affecting slope stability.

- The slope stability of the existing and proposed landfill is also sensitive to drainage conditions within the Older Bay Alluvium. As a result, it is recommended that piezometers be installed below at least two of the future waste cells in order to demonstrate that adequate drainage is taking place as the waste fills are placed.

- For slope stability considerations, both the base and slope membrane liners for future cells adjacent to perimeter levee Station 30+00 and below should be specified as textured HDPE.

- For slope stability considerations, geogrid reinforcement will be required in the final cover section for acceptable seismic performance if a prescriptive cover is selected for use. If an alternative monolithic soil cover system is selected, acceptable seismic performance can be achieved without geogrid reinforcement.

- For slope stability considerations, the landfill liner should "daylight" inboard of the proposed sedimentation basin near Stations 15+00 to 25+00 and be inclined no shallower than about 1- ¾ : 1 (H:V) .

- Foundation subdrains and the LCRS drainage systems should be designed to maintain an acceptable drainage slope after 0.013 feet per foot of differential settlement between the inboard "heel" of waste and the drainage sumps.

- Areas of toe scour from Coyote Creek along the northern perimeter levee should be repaired and the slope face armored to prevent future scour events.

13.0 CLOSURE

This report is based on the data and analyses described herein. Geo-Logic Associates should be notified of any conditions that differ from those described herein since this may require a re-evaluation of the data, conclusions and recommendations presented. This report has been prepared in accordance with generally accepted geotechnical practices, and makes no other warranties, either expressed or implied, as to the professional data presented in it.

This report has not been prepared for use by other parties and projects other than those named or described above. It may not contain sufficient information for other parties or other purposes.

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Note: The tables, figures, and technical appendices that accompany the geotechnical evaluation are on file at the City's Planning, Building, and Code Enforcement Department and can be viewed during normal business hours.