APPENDIX K

Preliminary Settlement and Stability Assessment

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M E M O R A N D U M

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DATE: 12 August 2006

SUBJECT: **Preliminary Settlement and Stability Assessment, Conceptual Site Development Plan, ASCON Landfill Site, Huntington Beach, California**

INTRODUCTION

At your request, preliminary settlement and stability analyses in support of conceptual site development plans for the ASCON Landfill site in Huntington Beach, California have been conducted. These stability and settlement analyses were based upon "Alternative 4 – Partial Source Removal with Protective Cap" as portrayed in Figure 9.2-4 of the Second Feasibility Study by Project Navigator, Limited dated November 2005, and Figure 9.2-4 of the Second Feasibility Study Revision 1 by Project Navigator, Limited dated August 2006. However, results from these analyses were also used to draw conclusions regarding the feasibility of "Alternative 3 – Protective Cap" as portrayed in Figure 9.2-3 in the same documents. Based upon direction from Project Navigator, Limited it was assumed that the protective cap shown in Figures 9.2-3 and 9.2-4 would be a 5-ft thick soil cap.

The settlement and stability analyses were based primarily on soil profiles and soil properties as defined by Borings PNL-21, PNL-23, and PNL-28 and the laboratory tests performed on samples recovered from these borings. Data from laboratory testing results on samples recovered from Test Pits PNL-5A and PNL-4B and the GIS-EVS site profiles generated by Project Navigator, Limited were also considered in the analyses, but solely for evaluation of consistency with the boring data used in the analyses.

For purposes of the analyses presented herein, the soil profiles in borings PNL-21, PNL-23, and PNL-28 were considered to be representative soil profiles that may be found anywhere across the site. The soil profile at PNL-21 was idealized as 6 ft of relatively incompressible fill underlain by 18 ft of compressible waste overlying 2 ft of native organic silty clay. The soil profile at PNL-23 was idealized as 3 ft of relatively incompressible fill underlain by 19 ft of compressible waste overlying 7 ft of native organic silty clay. The soil profile at PNL-28 was idealized as 10 ft of relatively incompressible fill underlain by 6 ft of compressible waste overlying 1 ft of native organic silty clay.

For the settlement analysis, two site development cases were considered: 1) the 5 ft soil cover placed on existing grade; and 2) the 5 ft soil cover placed after excavation to street level. For each case, primary consolidation settlement under four different loading cases was evaluated: the cap load only and the cap plus surface loadings of 250, 500, and 750 psf. These surface loads correspond roughly to a lightly loaded one story building (e.g. a recreation center), a heavily loaded one story building (e.g. a warehouse), and a two- story office building. The stability evaluation was conducted based upon the cross sections shown in Figure 9.2-4 for Alternative 4 and Figure 9.2.3 for Alternative 3. The primary difference between these two cross sections is that for Alternative 4 a thin layer of Lagoon 4 waste material is left in place beneath a descending slope constructed out over the waste material while in Alternative 3 a substantial portion of Lagoon 4 waste is left in place beneath the descending slope. Analyses were conducted for slope heights of 10 ft and 25 ft for two slope inclinations; 3H:1V (3 horizontal: 1 vertical) and 4H:1V.

ALTERNATIVE 4: PARTIAL SOURCE REMOVAL WITH PROTECTIVE CAP

Settlement Analysis

In the settlement analysis, each boring was treated as an "independent" horizontally-layered (1-Dimensional) soil profile. One-dimensional consolidation settlement analyses were conducted for each profile (boring location) for the two site development cases (cap placed on existing grade and cap placed after excavation to street level) and the four post-capping loading scenarios (cap load only and the cap plus surface loadings of 250, 500, and 750 psf). Consolidation properties of the compressible waste for each "profile" (boring) were based upon the laboratory consolidation test results on samples recovered from the borings. Figures 1 and 2 show plots of the coefficient of consolidation, c_v, and the saturated hydraulic conductivity, k_{sat}, versus effective stress as calculated from the consolidation test data from the samples recovered from borings PNL-21, PNL-23, and PNL-28 and Test Pits PNL-4B and PNL-5A.

For the soil profile represented by boring PNL-21, the consolidation properties (compression index, coefficient of consolidation) used in the settlement analyses for the waste came from sample 22/24 from boring PNL21. For the soil profile represented by boring PNL-23, consolidation properties for the waste came from sample 17/19 from boring PNL-23. For the soil profile represented by boring PNL-28, the waste consolidation properties came from sample 15/17 from boring PNL-28.

For the soil profile represented by boring PNL-21, the consolidation properties used in the settlement analyses for the silty clay came from sample 22/24 from boring PNL-21. For the soil profile represented by boring PNL-23, the compressibility of the silty clay was based upon sample 24/26 from PNL-23. However, the coefficient of consolidation values from laboratory testing on the silty clay sample from PNL-23 were anomalous. The compressibility and index properties (Atterberg limits and grain size) measured on the silty clay sample from PNL-23 were similar to the values measured on the silty clay sample from PNL-21 while the coefficient of consolidation values were over two orders of magnitude higher for the PNL-23 silty clay sample compared to the PNL-21 silty clay sample. Because the coefficient of consolidation values measured on the silty clay sample from PNL-21 were consistent with the index properties and compressibility measured on the silty clay samples from both borings (PNL-21 and PNL-23), the coefficient of consolidation values from the PNL-21 sample were used to represent the coefficient of consolidation for the silty clay in both the PNL-21 and PNL-23 profiles and the anomalous coefficient of consolidation values measured in the silty clay sample from PNL-23 were disregarded. No laboratory testing was conducted on silty clay recovered from boring PNL-28. Therefore, for the silty clay in the PNL-28 soil profile the compressibility was taken as the average of the values for the PNL-21 and PNL-23 profile and the coefficient of consolidation was based upon the values from PNL-21. The overlying fill was assumed to be relatively incompressible and free draining at each location.

Table 1 summarizes the consolidation properties used in the settlement analysis for each profile. For the soil profile represented by boring PNL-21, the compressible waste was assigned a modified compression index, $C_{C\varepsilon}$ (= $C_c/[1+e_0]$), of 0.28 and a modified recompression index, $C_{R\varepsilon}$ (= $C_R/[1+e_0]$), of 0.03. The native organic silty clay in PNL-21 was assigned a modified compression index, $C_{C\varepsilon}$, of 0.4 and a modified recompression index, $C_{R\varepsilon}$, of 0.052. Values for the coefficient of consolidation, c_v, for "virgin" loading of the compressible waste and silty clay material of this profile were established based on the PNL-21 trend line in Figure 1 and the stress levels within the waste and clay layers. Figure 1 indicates that the coefficient of consolidation, c_v, varies between 2 ft^2/yr and 6 ft^2/yr (5.8 x 10^{-5} cm^2/s and 1.7 x 10^{-4} cm^2/s) for overburden pressures between 1,000 lb/ft^2 and 3,000 lb/ft^2 for "virgin" loading for both the clay and compressible waste. Note that these values are representative of a compressible low permeability material (e.g. high plasticity clay) with a saturated hydraulic conductivity on the order of 2 x 10^{-8} cm/s to 8 x 10^{-9} cm/s at an overburden pressure of between 1000 lb/ft^2 and 3000 lb/ft^2 (9 ft to 26 ft of fill overburden). Based upon comparison of c_v values in Figure 1 and the k_{sat} values in Figure 2 for samples recovered from Test Pits PNL-4b and PNL-4A with values for the waste material from boring PNL-21, the consolidation properties for PNL-21 waste material were assumed to also be representative of consolidated Lagoon 4 waste material.

For the soil profile represented by boring PNL-23, the compressible waste was assigned a modified compression index, $C_{C\varepsilon}$, of 0.16 and a modified recompression index, $C_{R\varepsilon}$, of 0.036. The silty clay layer in PNL-23 was assigned a modified compression index, $C_{C\varepsilon}$, of 0.31 and a modified recompression index, $C_{R\varepsilon}$, of 0.045. Based upon the trend line in Figure 1 for PNL-23, a coefficient of consolidation, c_v, of 200 ft^2/yr to 250 ft^2/yr (5.8 x 10^{-3} cm^2/s to 7.3 x 10^{-3} cm^2/s) was assigned to the waste material in this profile for virgin loading, approximately two orders of magnitude higher than the c_v values for the waste at boring PNL-21. As noted earlier, because the c_v values from laboratory testing of the silty clay from boring PNL-23 (i.e. the data in Figure 1 from sample PNL-23 24/26) falls well outside of the typical range of c_v values for clays and silty clays with similar Atterberg limits, c_v values from the clay recovered from PNL-21 (i.e. from sample PNL-21 24/26) were used to represent the coefficient of consolidation for virgin loading of the silty clay layer at PNL-23.

For the soil profile represented by boring PNL-28, the compressible waste was assigned a modified compression index, $C_{C\varepsilon}$, of 0.18, a modified recompression index, $C_{R\varepsilon}$, of 0.035, and c_v values for virgin loading based upon the trend line for PNL-28 in Figure 1. Because no consolidation tests were conducted on samples recovered from the native organic silty clay layer at this location, the native organic silty clay in PNL-28 was assigned compressibility indices ($C_{C\varepsilon}$ and $C_{R\varepsilon}$) equal to the average of the properties for the native clay material from PNL-21 andPNL-23, i.e. a modified compression index, $C_{C\varepsilon}$, of 0.36 and a modified recompression index, $C_{R\varepsilon}$, of 0.049, and c_v values based upon the PNL-21 trend line in Figure 1.

The effective stress used in establishing c_v values from the trend lines in Figure 1 was based upon the average of the initial and final vertical effective stress for each loading case. Effective stresses were based upon the assumption that the water table (phreatic surface) was in the middle of the compressible waste layer for each profile. For the cases where the cap was placed after excavation to grade, because of uncertainty on the duration of excavation and on the time that will elapse between the completion of excavation and cap placement, the initial effective stress was assumed to be the effective stress prior to excavation (as opposed to the initial effective stress after excavation). This assumption results in an upper bound (conservative) value for the time required for the completion of consolidation under cap and building loads subsequent to excavation, as this assumption yields the greatest possible value for the average effective stress and thus the lowest possible value for c_v. Furthermore, for both the compressible waste and silty clay in all profiles, unload/reload c_v values were assumed to be 5 times the values for virgin loading. This is also a conservative assumption with respect to the time required for consolidation under loads subsequent to excavation as 5 is a lower bound value for typical values of the ratio of the unload/reload coefficient of consolidation to the virgin compression coefficient of consolidation. The assumption of one-dimensional conditions is likewise a conservative assumption leading to upper bound values on the time required for the completion of consolidation.

Based upon the relative saturated hydraulic conductivity values presented in Figure 2, when estimating the rate of consolidation settlement the compressible waste layer and silty clay layer in the profile represented by PNL-21 were modeled as one single layer (due to a minimal difference between k_{sat} and c_v values for the waste material and the native clay at this location). This single" waste/clay" layer was assumed to drain from both the top and bottom of the layer. In the profiles represented by borings PNL-23 and PNL-28, the compressible waste layer was assumed to drain only from top and the silty clay layer was assumed to drain from both top and bottom due to the higher saturated hydraulic conductivity of the waste material compared to underlying organic silty clay material.

Tables 2 and 3 summarize the results of the consolidation settlement analysis. For the site development case where the 5 ft cap is placed on existing grade, the estimated settlement under the cap load was 1.7 in. for PNL-28, 8.9 in. for PNL-23, and 10.8 in. for PNL-21. The estimated time for this settlement to occur was on the order of 2 years for the PNL-28 profile but was approximately 11 years and 60 years for the PNL-21 and the PNL-23 profiles, respectively. Incremental settlement under the 250 psf building load was only 0.4 in. for the PNL-28 profile but was 2.8 in. for the PNL-23 profile and 3.4 in. for the PNL-21 profile. The time for these incremental settlements to occur is only slightly greater than the time for settlement under the cap, as the increment in effective overburden stress at the middle of the compressible layers due to these building loads is minimal for the “existing grade” profiles. PNL-28 is the boring closest to the boundary of the site and is also the boring where the compressible waste layer is the thinnest and consolidates the fastest. This boring may therefore be representative of the site margin areas. For the 500 psf building load, the estimated incremental settlement for the PNL-28 profile was 1.1 in. (marginal, but generally considered acceptable for a slab on grade wood framed structure) and the estimated time for this settlement to occur was approximately 3 years. However, the compressible waste settlement at this location occurs in approximately 4 months. For the 750 psf building load, the estimated incremental settlement for the PNL-28 profile was 1.7 in., somewhat more than generally acceptable. However, the estimated time for completion of the compressible waste settlement of approximately 5 months at this location suggests that surcharging could be used to mitigate the settlement under the building load if it was desired to construct a building with a 750 psf foundation load on top of a 5 ft cap placed at existing grade at locations underlain by the PNL-28 profile, i.e. at areas close to the edge of the site.

For the site development case where the area is excavated to street grade prior to placing a 5 ft soil cap, presented in Table 3, estimated total settlement under the cap load varied from 1.3 in. for PNL-28 to 2.2 in. for PNL-23. The estimated time from the end of construction for this settlement to occur was less than 1 month for the PNL-28 profile and on the order of 1.5 years for the PNL-21 and PNL-23 profiles. The total settlement under a building load of 250 psf varied from 1.6 to 2.6 inches (incremental settlement due to a 250 psf building load of 0.3 to 0.4 in.), while the total settlement under a building load of 750 psf varied from 2 to 3.3 in. (incremental settlement due to a 750 psf building load from 0.7 to 1.1 in.) for this case. The incremental building load settlements all indicate acceptable settlement (settlement of 1 in. or less) if the buildings are constructed after the end of primary settlement due to cap placement. The time for primary settlement under the weight of the cap was a maximum of 2 years for this case. The estimated time for this building settlement to occur may be up to 25 percent longer than the time required for cap settlement due to the greater effective overburden stress during consolidation.

The settlement values presented in Table 2 and 3 are for primary consolidation settlement only. No laboratory data on the coefficient of secondary compression was available for either the waste or silty clay. Therefore, estimates of secondary compression settlement were based on the assumption that the ratio of the coefficient of secondary compression to the modified virgin compression index was 0.02, a typical value for natural soils. Based upon this assumption, secondary compression settlements over a period of 30 years following the end of primary consolidation were on the order of 1 inch or less for all cases.

Stability Analyses

Stability analyses were complicated by relatively poor definition of the shear strength of the compressible waste and silty clay materials. Unconsolidated Undrained triaxial test results from Cooper Testing Laboratory indicate low shear strengths but also appear to have been subject to excessive sample disturbance. Consolidated Undrained triaxial test results from Ninyo and Moore also appear to be representative of disturbed samples and may have been conducted without allowing enough time for complete consolidation of the specimens prior to shear. Based upon typical values for cohesive soils, the undrained shear strength of the unconsolidated Lagoon 4 waste, assumed to consist mostly of drilling mud, was based upon an assumed "c/p ratio" (the ratio of the undrained shear strength, S_u, to the effective overburden stress, σ'_v) of 0.25, with a minimum undrained shear strength of 250 psf for the existing conditions. This assumed c/p ratio and minimum strength should be verified by laboratory and/or field testing as part of final design. The consolidated waste material in the soil profiles represented by PNL-21, PNL-23, and PNL-28, assumed to consist of a mixture of soil and drilling mud, was assigned a friction angle of 30 degrees and a unit weight of 115 lb/ft^3. The silty clay was assigned an undrained strength of 500 lb/ft^2 in all analyses based upon typical values for this type of material. Both the silty clay and the Lagoon 4 waste material, assumed to consist primarily of drilling mud, were assumed to have a unit weight of 90 lb/ft^3 in the stability analysis. The soil cap was assigned a unit weight of 120 lb/ft^3 and a friction angle of 33 degrees. The native sand underlying the silty clay was assigned a friction angle of 30 degrees and a unit weight of 115 lb/ft^3 for the stability analyses. The strength value assigned to the underlying native sand is excessively conservative but of no consequence with respect to the results of the stability analyses.

Stability analyses were conducted using the computer program PCSTABL5. Analyses were conducted using the short term undrained strength, i.e. the strength anticipated at the start of construction, and the long term undrained strength, i.e. the strength after consolidation under the toe of the slope (or embankment) and the cap loads. The long term stability analyses for Alternative 4, "Partial Source Removal with Protective Cap," conducted using fully consolidated strengths for the Lagoon 4 waste material, indicated acceptable static stability (i.e. a static factor of safety greater than 1.5) for all cases and a yield acceleration of between 0.1 and 0.14 g. The results of these analyses are illustrated in Figure 3.

Seismic stability analyses were based upon both the earthquake with a 10% probability of not being exceeded in 50 years specified by the building code, i.e. the "500-year event" (actually, the earthquake with a return period of 475 years), and the Maximum Credible Earthquake (MCE) specified by California regulations for hazardous waste landfills. The 500-year event was established an earthquake with a free field peak horizontal ground acceleration (PHGA) at the site of approximately 0.4 g and a moment magnitude of approximately 7. The MCE was also a moment magnitude 7 event and was assumed to occur directly beneath the site on the Newport Inglewood fault, generating a free field PHGA at the site of 0.8 g. For 500-year design event, simplified analyses yielded calculated permanent seismic displacements generally less than 6 in. for the 10 ft high slopes (with a yield acceleration of approximately 0.14 g), a value generally considered acceptable for all cases. For the 25 ft high slopes, with a yield acceleration of approximately 0.1 g, the simplified analyses yield calculated permanent seismic displacements of as much as 12 in. This value is considered acceptable for general earthwork but somewhat marginal for buildings. For the MCE, calculated seismic displacements for the 10 ft high slope were on the order of 12 in. while for the 25 ft high slopes the calculated seismic displacement was on the order of 20 in. Past experience indicates that calculated permanent seismic displacements on the order of 6 to 12 in. in simplified analyses, as found for the 10 ft high slope, are generally indicative of satisfactory performance in the design earthquake, i.e. no significant damage to waste containment structures. Calculated seismic displacements on the order of 20 in., as calculated for the 25 ft high slope subject to the MCE, may indicate limited cracking and soil displacement in the design earthquake. This is not surprising, considering the magnitude of the ME and its intensity at the site. However, more sophisticated analyses are required to confirm these preliminary conclusions. In the unlikely event the more sophisticated analyses indicate unacceptable performance, setbacks from the crest and toe of the slope may be required for buildings constructed at the site.

The short term stability analysis for Alternative 4, i.e. the stability at the end of construction, was based upon the use of a friction angle of 30 degrees for the consolidated waste material in the profiles and for the embankment constructed out over the Lagoon 4 waste (Figure 4). However, analyses were also conducted using a layer of waste with a shear strength of 250 psf to represent the strength of an unconsolidated layer of waste left in place beneath the embankment constructed out over the excavated portion of Lagoon 4 (Figure 5). Results of the analysis using the consolidated waste strength, illustrated in Figure 4, indicated the minimum static factor of safety at the end of construction was 1.4, a value generally considered to be acceptable. However, results of the analysis using the unconsolidated waste strength of 250 psf for the layer of waste beneath the toe of the embankment, illustrated in Figure 5, yielded static factors of safety for the 4H:1V and 3H:1V 25 ft high slopes of 1.1 and 1.0, respectively. While acceptable factors of safety were calculated for the 10 ft high embankments and the use of a uniform layer with 250 psf shear strength everywhere beneath the slope is conservative, the results of these analyses indicate that caution should be exercised in building the embankment out over the Lagoon 4 waste that is left in place in Alternative 4. More sophisticated analyses are required during final design to assess the allowable slope height for this embankment. If these more sophisticated analyses still indicate unacceptable factors of safety for the end of construction, some type of ground improvement measure may be required for the areas where the Lagoon 4 waste is left in place under an embankment greater than 10-15 ft in height. Ground improvement measures could include removing waste and keying the descending slope (i.e. embankment) into the underlying native sands or improving the shear strength of the Lagoon 4 waste material beneath the slope by either solidification, i.e. mixing it with sand or other suitable soil, or stabilization, i.e. mixing the waste with some type of reactive chemical such as quicklime, fly ash, cement kiln dust, or portland cement. Bench scale testing to evaluate the shear strength of the solidified or stabilized material would be required if this option was chosen. As the long term stability analyses indicate acceptable stability for all slope heights, staged construction of the embankment is a possible solution. However, considering the very low values of c_v for the compressible waste, staged construction would likely require several years for completion.

ALTERNATIVE 3: PROTECTIVE CAP

Additional stability analyses were conducted to evaluate the feasibility of Alternative 3, "Protective Cap," in which the Lagoon 4 waste material is essentially left in place beneath the descending slope from the areas left at existing grade and the areas excavated to street level. For this alternative, the short term "end of construction:" stability was modeled by assigning the unconsolidated Lagoon 4 waste shear strength of 250 psf to the waste left in place beneath the descending slope. Both 3H:1V and 4H:1V slopes were evaluated. Results of these analyses, illustrated in Figure 6, yielded static factors of safety of 0.7 and 0.8 for the 25 ft high slope. While acceptable safety factors where obtained for the 10 ft high slopes, it is not clear how this alternative could be constructed, given the low shear strength and high mobility of the Lagoon 4 wastes. At a minimum, substantial shoring would be required, with this shoring abandoned in place after cap construction in all likelihood. Furthermore, for the taller slopes, ground improvement measures such as those discussed above for Alternative 4 are likely to be required, but to a much greater extent than required in even the worst case for Alternative 4.

The long term stability for Alternative 3 is similar to the long term stability of Alternative 4, as in the long term the Lagoon 4 waste will consolidate under the weight of the cap and embankment fill. However, the potential for limited earthquake-induced soil displacement in the design earthquake has more serious consequences for Alternative 3 than for Alternative 4, as even limited earthquake-induced displacements could potentially result in the long term in exposure of the Lagoon 4 waste left in place beneath the descending slope if the displacements are not remediated.

LIQUEFACTION

A preliminary liquefaction assessment was conducted using the blow counts reported in the various borings and soundings conducted at the site. For the most part, these data indicate the native sand underlying the silty clay is a dense poorly graded sand not susceptible to earthquake-induced liquefaction. Occasionally, the first blow count beneath the silty clay layer was lower than subsequent blow counts and indicated the soil at that particular location was susceptible to liquefaction. However, these samples also usually were classified as silty sands indicating a relatively high fines content, a mitigating factor with respect to liquefaction potential. Furthermore, as the borings were hollow stem auger borings, the low blow counts recorded for the first SPT beneath the silty clay layer could be due to heaving of soil in the bottom of the bore hole during sample recovery. Based upon the blow counts and soil descriptions in the borings, the native sand underlying the silty clay layer appears to be a marine terrace deposit that is not susceptible to liquefaction. While there may be isolated small areas of silty sand directly beneath the silty clay that are susceptible to earthquake-induced liquefaction, the consequences of these isolated areas liquefying should be minimal, i.e. there should not be any liquefaction-induced instability and liquefaction-induced settlements should be minimal.

SUMMARY

Based upon the assumption that the soil profiles from borings PNL-21, PNL-23, and PNL-28 represent the range of profiles that will be encountered across the site, preliminary settlement analyses indicate primary consolidation settlements due to cap placement of up to a little more than 2 in. in areas where the site is excavated to street level prior to placement of a 5 ft cap. The estimated duration for this cap settlement to occur is a maximum of 1.5 years. However, both the magnitude and duration of settlement will be less than the estimated values if the cap is placed soon after excavation. Note that if the excavation and cap construction occur in the same construction season the reduction in both time and magnitude of settlement will be significant. Detailed analyses are required to estimate the relationship between the magnitude and duration of settlement and the time required for excavation and cap construction. These analyses can be conducted in final design. Analyses reported herein also indicate acceptable incremental settlements for building loads up to 750 psf as long as the buildings are constructed after cap settlement is complete. These building-induced settlements could take up to 2 years to be complete. Secondary compression settlement was estimated to be less than 1 inch over the 30 year period following the completion of primary consolidation for all cases.

For the areas represented by the soil profiles at borings PNL-21 and PNL-23, presumably representative of the areas in the middle of the site with thick layers of compressible waste, primary consolidation settlement under a 5 ft cap placed on existing grade is significantly higher than for areas represented by the soil profile at boring PNL-28. Primary settlement for the areas represented by the soil profiles from borings PNL-2 and PNL-23 were estimated to be on the order of 11 to 9 inches, respectively. Based upon the consolidation properties of the waste and clay tested from boring PNL-21, primary consolidation settlement could take as much as 60 years to be complete. For the profile based upon boring PNL-23, the estimated time for primary consolidation was estimated to be about 11 years. However, cap settlement in areas represented by the soil profile from boring PNL-28, presumably representative of areas around the perimeter of the site, is significantly less and will occur significantly faster, with estimated cap settlements on the order of 1.7 inches over a period of approximately 2.1 years. One again, secondary compression settlements over a period of 30 years following the end of primary consolidation were estimated to be less than or equal to 1 inch in all cases.

Even 250 psf building loads would induce incremental settlements after cap construction of greater than 1 in. in areas represented by borings PNL-21 and PNL-23. However, incremental settlements due to building loads of 250 and 500 psf are acceptable, i.e. 1 in. or less, in the areas represented by PNL-28. Surcharge loading could be employed to reduce the incremental settlement for building construction to acceptable values in areas represented by PNL-21 and PNL-23 where the cap is placed on existing grade. However, the time for surcharge settlement may be excessive (i.e. 10

years or more) unless either measures are taken to accelerate settlement, e.g. wick drains, or reduce compressibility, e.g. deep soil mixing, or deep foundations are used.

Preliminary stability analyses for Alternative 4, Partial Removal and Protective Cap," indicate acceptable static and seismic stability for 10 ft high, 3H:1V and 4H:1V slopes. However, analyses indicate marginal short term (end of construction) stability for 25 ft high slopes where a relatively thin layer of unconsolidated Lagoon 4 waste is left in place beneath the descending slope, suggesting that more sophisticated analyses are required in final to make a proper assessment. Seismic stability analyses for the 25 ft high slopes also indicate somewhat marginal performance, possibly requiring setbacks or ground improvement subject to the results of more sophisticated analyses. If the more sophisticated analyses still indicate marginal stability, ground improvement measures may be required. Potential ground improvement measures include solidification of the waste left in place beneath the slope by mixing with soil or stabilization by mixing with reactive agents. Stage construction is also possible, but the time required for stage construction may be excessive.

Analyses of short term, end of construction stability for Alternative 3, "Protective Cap," indicate unsatisfactory stability for the higher slopes. Substantial shoring and relatively extensive ground improvement is likely to be required for implementation of this alternative. The 25 ft high slopes in Alternative 3 may also be problematic with respect to the potential for limited earthquake-induced ground displacements in the Maximum Credible Earthquake leading to exposure of the Lagoon 4 waste left in place beneath the slope.

Soil profiles and soil properties should be confirmed by additional exploration and testing for final design. In particular, assumptions regarding the undrained shear strength of the compressible waste must be confirmed and the shear strength associated with any proposed stabilization or solidification measures must be evaluated. Furthermore, final design for any structures constructed upon the cap should include borings at the locations of the structures to confirm the assumed typical profiles and refine the estimates of magnitude and duration of settlement.

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TABLE 1
CONSOLIDATION PROPERTIES USED IN THE SETTLEMENT ANALYSIS

Boring (Profile)	**Strata**	**Compression Index $C_{\varepsilon C}$**	**Coefficient of Consolidation***	**Unit Weight γ**
PNL-21	**Compressible Waste**	0.28	PNL-21	115 lb/ft^3
	Native Organic Clay	0.4	PNL-21	90 lb/ft^3
PNL-23	**Compressible Waste**	0.16	PNL-23	115 lb/ft^3
	Native Organic Clay	0.31	PNL-21	90 lb/ft^3
PNL-28	**Compressible Waste**	0.18	PNL-28	115 lb/ft^3
	Native Organic Clay	0.36	PNL-21	90 lb/ft^3

*** Boring designations refers to c_v values from trend lines in Figure 1.**

TABLE 2
SETTLEMENT ANALYSIS RESULTS: CAP PLACED ON EXISTING GRADE

Assumed Structural Load (psf)	Soil Profile	Thickness of Waste (ft)	Thickness of Clay (ft)	Settlement due to Compression of Waste (in.)	Time for 99% Consolidation of Waste	Settlement due to Compression of Native Clay (in.)	Time for 99% Consolidation of Clay	Total Settlement (in.)
0 (5-ft soil cover only)	PNL-21	18	2	9.8	60 yrs	1	60 yrs(1)	10.8
	PNL-23	19	7	6.7	3.2 yrs	2.2	11 yrs	8.9
	PNL-28	6	1	1.7	2.1 yrs	0	> 1 mo.s	1.7
250	PNL-21	18	2	12.8	60 yrs	1.4	60 yrs(1)	14.2
	PNL-23	19	7	8.6	3.2 yrs	3.1	11 yrs	11.7
	PNL-28	6	1	2	2.1 yrs	0.1	2 mo.s	2.1
500	PNL-21	18	2	15.6	65 yrs	1.6	65 yrs(1)	17.2
	PNL-23	19	7	10.4	3.2 yrs	3.7	12 yrs	14.1
	PNL-28	6	1	2.5	2.6 yrs	0.3	2 mo.s	2.8
750	PNL-21	18	2	18	71 yrs	2	71 yrs(1)	20
	PNL-23	19	7	12	3.2 yrs	4.4	12 yrs	16.4
	PNL-28	6	1	3	2.6 yrs	0.4	3 mo.s	3.4

(1) Clay and waste layers were modeled as one single layer.

TABLE 3
SETTLEMENT ANALYSIS RESULTS: CAP PLACED AFTER EXCAVATION TO STREET GRADE

Assumed Structural Load (psf)	Soil Profile	Thickness of Waste (ft)	Thickness of Clay (ft)	Settlement due to Compression of Waste (in.)	Time for 99% Consolidation of Waste	Settlement due to Compression of Native Clay (in.)	Time for 99% Consolidation of Clay	Total Settlement (in.)
0 (5-ft soil cover only)	PNL-21	5	2	1.2	1.5 yrs	0.4	1.5 yrs(1)	1.6
	PNL-23	2	7	0.8	< 1 mo.s	1.4	1.5 yrs	2.2
	PNL-28	4	1	1.1	2 mo.s	0.2	< 1 mo.s	1.3
250	PNL-21	5	2	1.3	1.5 yrs	0.6	1.5 yrs(1)	1.9
	PNL-23	2	7	0.8	< 1 mo.s	1.8	1.8 yrs	2.6
	PNL-28	4	1	1.3	2 mo.s	0.3	< 1 mo.s	1.6
500	PNL-21	5	2	1.6	1.5 yrs	0.6	1.5 yrs(1)	2.2
	PNL-23	2	7	1	< 1 mo.s	2	1.8 yrs	3
	PNL-28	4	1	1.4	2 mo.s	0.4	< 1 mo.s	1.8
750	PNL-21	5	2	1.7	1.6 yrs	0.7	1.6 yrs(1)	2.4
	PNL-23	2	7	1.1	< 1 mo.s	2.2	2 yrs	3.3
	PNL-28	4	1	1.6	2 mo.s	0.4	< 1 mo.s	2

(1) Clay and waste layers were modeled as one single layer.

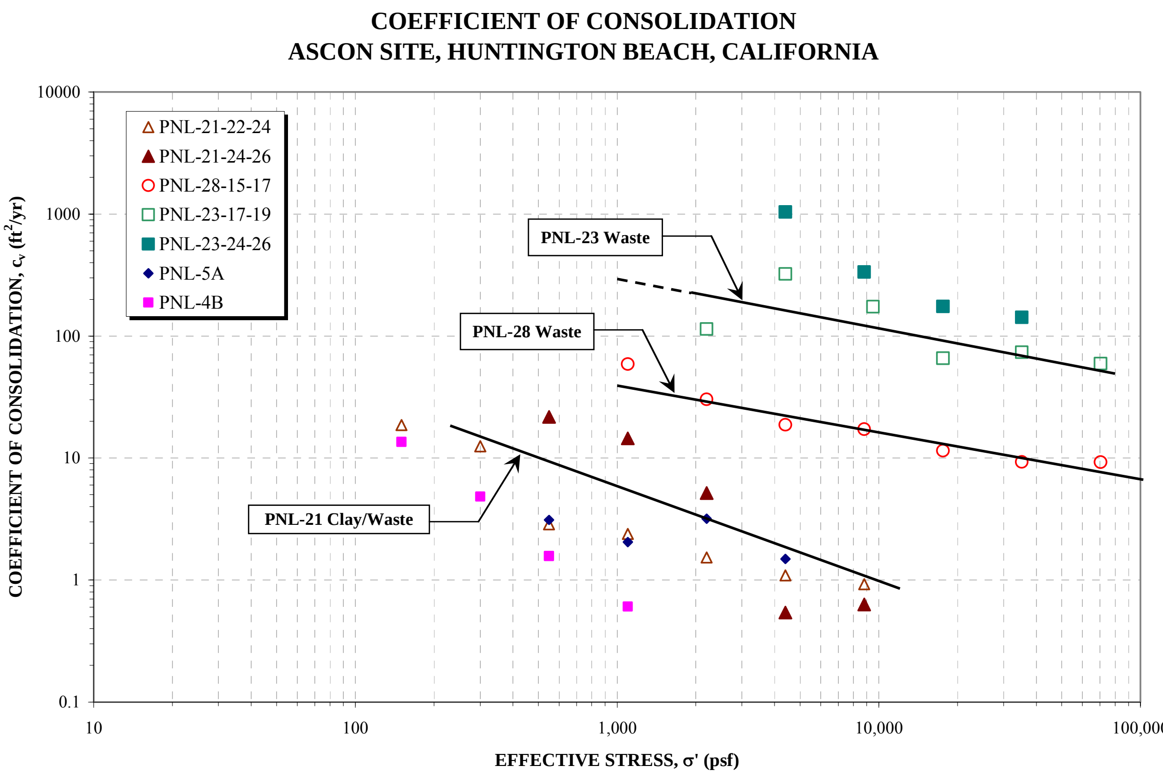

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Figure 1 Coefficient of Consolidation versus Vertical Effective Stress

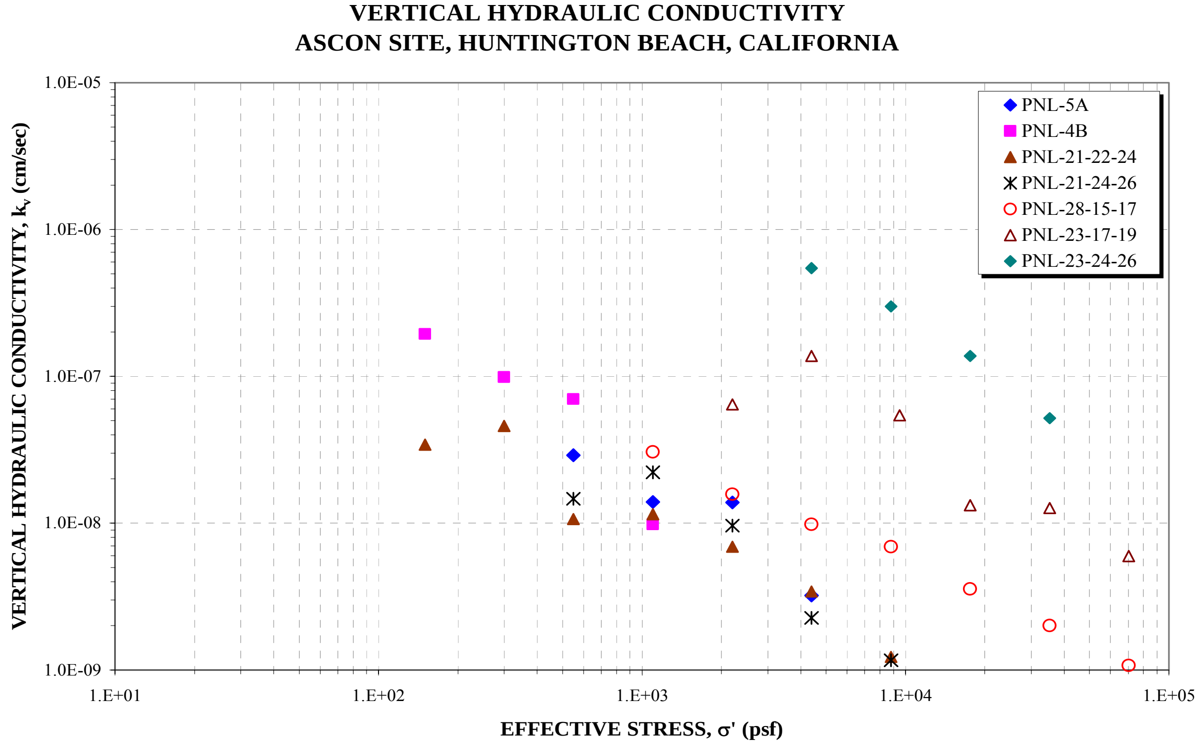
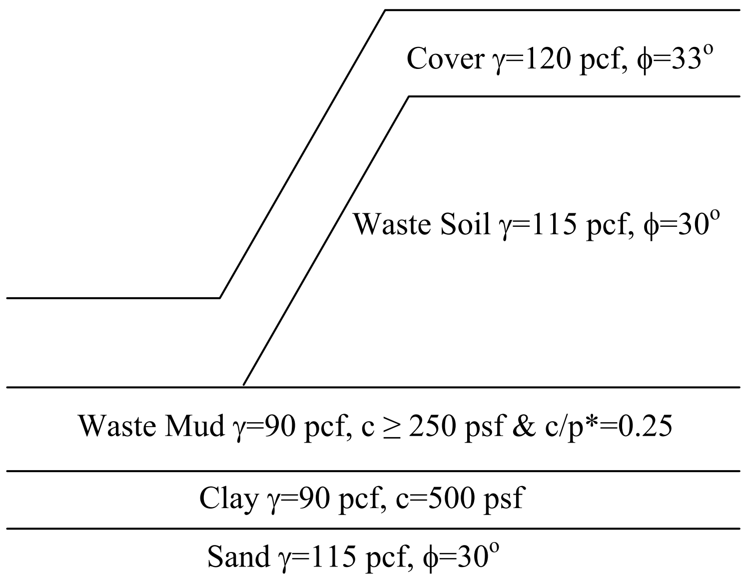


Figure 2 Saturated Hydraulic Conductivity versus Vertical Effective Stress

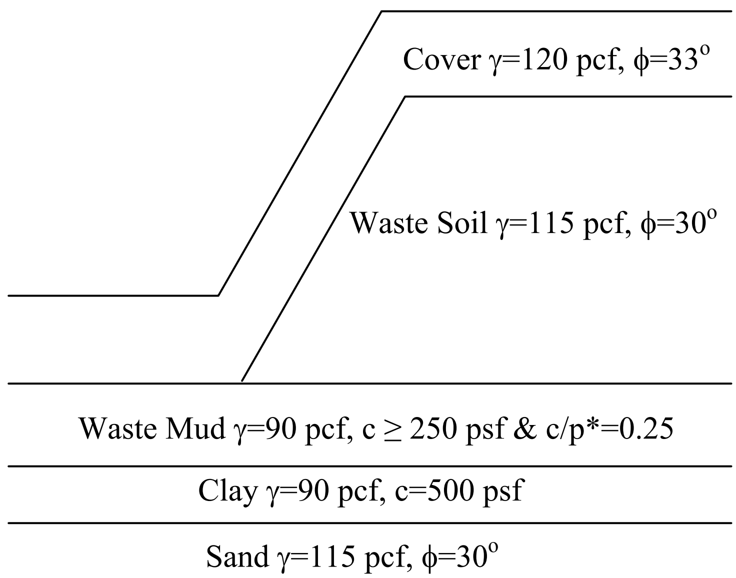


***p includes overburden due to cover (long-term stability)**

Slope	Slope Height	Static FS(1)	ky
3H:1V	**10 ft**	**> 1.9**	**0.14 g**
	25 ft	**1.5**	**0.1 g**
4H:1V	**10 ft**	**> 1.9**	**0.14 g**
	25 ft	**1.7**	**0.1 g**

(1)Spencer

Figure 3 Long Term Stability Analysis Results, Alterative 4

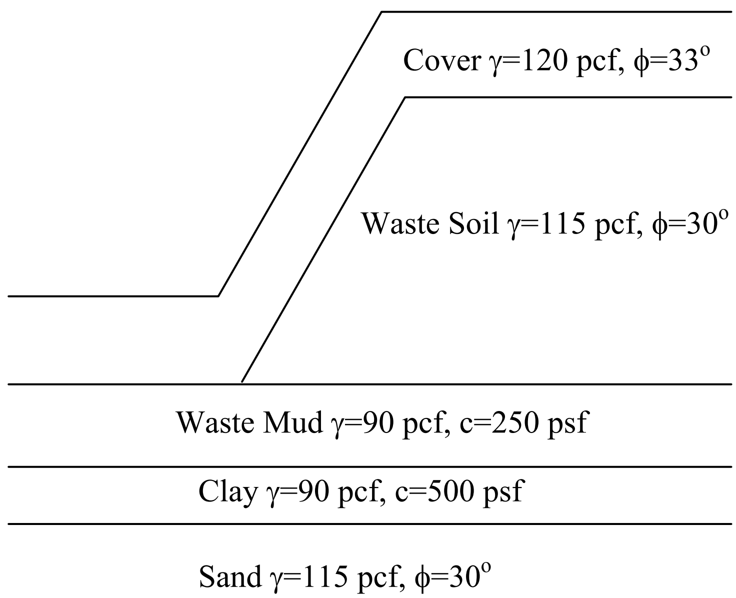


***p does not include overburden due to cover (short term stability)**

Slope	Slope Height	Static FS(1)	ky
3H:1V	**10 ft**	**1.9**	**N/A**
	25 ft	**1.4**	**N/A**
4H:1V	**10 ft**	**1.9**	**N/A**
	25 ft	**1.6**	**N/A**

(1) Spencer

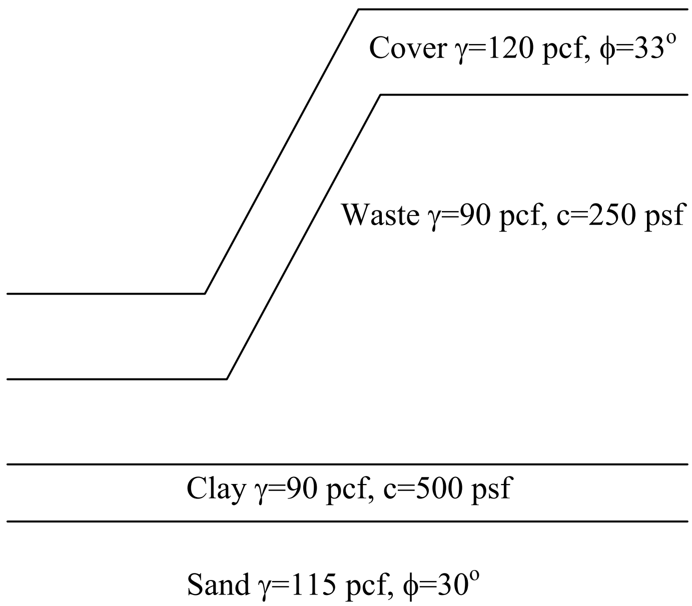
Figure 4 Short Term Stability Analysis Results, Alternative 4
Consolidated Waste Strength



Slope	**Slope Height**	**Static FS(1)**	**ky**
3H:1V	**10 ft**	**1.5**	**N/A**
	25 ft	**1.0**	**N/A**
4H:1V	**10 ft**	**1.6**	**N/A**
	25 ft	**1.1**	**N/A**

(1) Janbu

Figure 5 Short Term Stability Analysis Results, Alternative 4 Unconsolidated Lagoon 4 Waste



Slope	Slope Height	Static FS(1)	ky
3H:1V	**10 ft**	**1.4**	**N/A**
	25 ft	**0.7**	**N/A**
4H:1V	**10 ft**	**1.5**	**N/A**
	25 ft	**0.8**	**N/A**

(1) Janbu

Figure 6 Short Term Stability Analysis Results, Alternative 3