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DRAFT MEMORANDUM

DATE: May 14, 2001

TO: Pete Douglass, P.E. and Embankment Technical Review Board Members

FROM: Douglas Lindquist, E.I.T., and Michael Bailey, P.E., Hart Crowser, Inc.

RE: **Proposed Liquefaction Procedure under Walls and Slopes**
4978-30

CC: Jim Thomson, P.E., HNTB

This memorandum describes the proposed procedure that will be used to evaluate liquefaction potential for areas beneath walls and slopes of the Third Runway embankment. The liquefaction procedure takes into account (a) the anticipated level of ground shaking, (b) the existing soil conditions as reflected in measured standard penetration test resistances, (c) the irregular topography associated with the embankment (i.e., walls and slopes), (d) the high confining pressures that will exist beneath the embankment, and (e) the potentially high static shear stresses that may exist beneath walls and/or slopes.

The following procedure accounts for each of these factors in a practical and reasonably conservative manner. It uses conventional, one-dimensional, equivalent linear site response analyses to evaluate liquefaction potential in areas unaffected by the presence of walls or slopes and two-dimensional FLAC analyses to account for geometric/topographic effects in areas that are affected by walls and slopes. The procedure allows evaluation of liquefaction potential for a variety of improved zone geometries so that an optimum improved zone geometry can be identified.

A simplified representation of a typical profile through the edge of the embankment is shown on Figure 1; note that the transition from embankment grade to adjacent natural grade may occur through a slope or a retaining wall. A generalized soil profile consists of dense natural soils overlain by shallower natural soils that may contain zones of looser soils. Improvement of some region of the looser natural soils using stone columns or other methods is anticipated beneath the walls and some of the slopes; the geometric extent of the improved zone will be determined on the basis of stability considerations. The improved zone must extend far enough behind the toe of the wall/slope that stability of the embankment is not compromised, even if zones of the looser natural soils do liquefy during

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1910 Fairview Avenue East
Seattle, Washington 98102-3699
Fax 206.328.5581
Tel 206.324.9530

earthquake shaking. Stability of the embankment will be evaluated using limit equilibrium procedures.

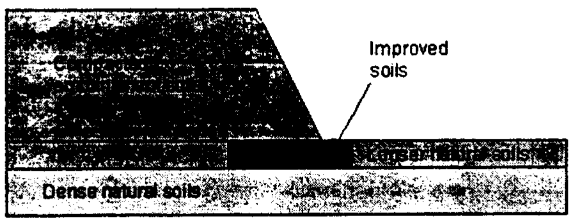

Figure 1 - Schematic Illustration of Wall/Slope Region

An important consideration in design of the embankment is the provision for a sufficient zone of improved soil to maintain stability in the event of liquefaction beneath some portion of the embankment. The depth of the improved zone is limited by the presence of the dense natural soils; therefore, the distance it extends behind the toe of the wall/slope will characterize the geometric extent of soil improvement. If the improved zone extends a great distance behind the toe of the wall/slope, the potentially liquefiable soil will experience ground motions that will be unaffected by the presence of the wall/slope; in this case, one-dimensional site response analyses will be sufficient to evaluate the liquefaction potential of these soils. If the improved zone is small compared to the height of the wall/slope the loading applied to potentially liquefiable soils may be influenced by the presence of the wall/slope. In such cases, the degree of ground motion amplification associated with slope geometry/topography should be considered.

The procedure used to evaluate liquefaction potential on a section by section basis is based on the assumptions that (a) an equivalent linear analysis that iterates to strain-compatible soil properties using well-established, continuous modulus reduction and damping curves provides the best indication of cyclic stresses in a one-dimensional soil profile, (b) a two-dimensional response analysis such as FLAC can provide a good estimate of the relative level of cyclic stress in areas influenced by two-dimensional geometry, and (c) the relative cyclic stress levels in the two-dimensional analysis (ratio of cyclic stresses beneath slope to cyclic stresses beneath main portion of embankment) are relatively insensitive to material properties. The procedure, which was designed to take advantage of the capabilities of both analysis techniques, will be implemented as follows:

1. Perform standard liquefaction potential evaluation using one-dimensional equivalent linear site response analyses using the average of Motions A and B that were created to match the spectra of our probabilistic seismic hazard analysis (Hart

Crowser 2001). For each section, perform two analyses - one for Profile 1 (Figure 2a), which represents conditions outboard of the toe of the wall/slope, and one for Profile 2, which represents conditions behind the slope/wall. Use shear stresses computed in analysis to determine cyclic stress ratio (our ProShake program calculates the cyclic stress ratios automatically). Let the cyclic stress ratios computed by this technique be called S_1 and S_2 (Figure 2b) for Profiles 1 and 2, respectively.

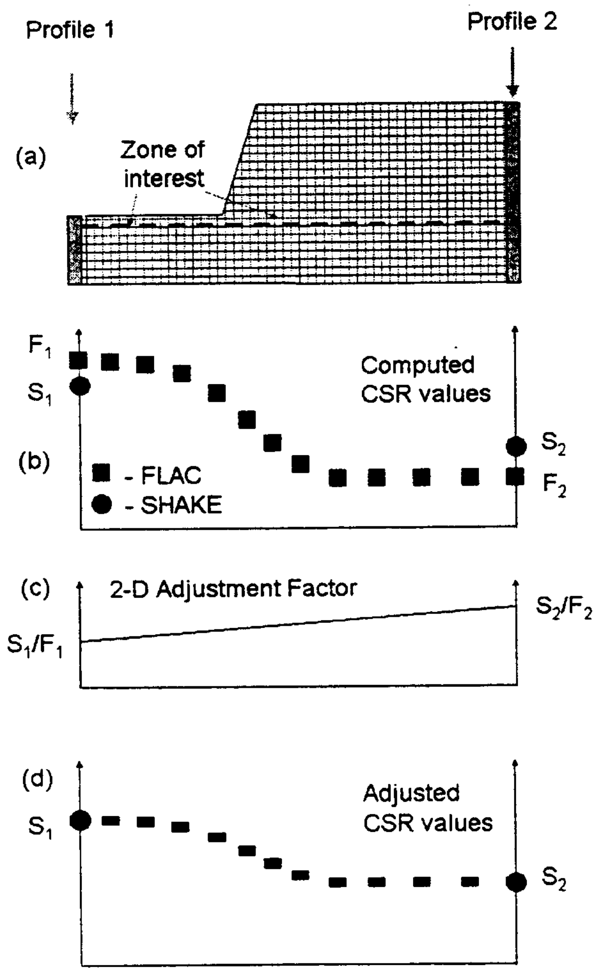

Figure 2 - Illustration of Scaling Procedure

2. Create an elastic FLAC model of the embankment and underlying natural soils. We propose to create three wall models and three 2H:1V slope models with heights of 50, 100, and 150 feet. Liquefaction potential of actual wall and slope geometries will be interpolated from the results of these six geometries.

3. Perform static analysis of FLAC model. Compute maximum shear stresses and major principal effective stresses within zone of looser natural soils.

4. Use stresses computed in previous step to compute K_σ within zone of looser natural soils.

5. Perform dynamic FLAC analysis and compute dynamic τ_{max} values within zone of looser natural soils.

6. Use τ_{max} and σ'_1 values to compute CSR values within the zone of looser natural soils.

7. Plot variation of CSR within the zone of looser natural soils. Let values of CSR computed at locations of Profiles 1 and 2 be known as F_1 and F_2 (Figure 2b), respectively.

8. Define a 2-D adjustment factor as a function varying between S_1/F_1 at the location of Profile 1 and S_2/F_2 at the location of Profile 2 (Figure 2c).

9. Multiply the FLAC CSR values by the corresponding 2-D adjustment factor values. The result will be a set of CSR values (Figure 2d) that are consistent with the equivalent linear CSR values and that reflect the effects of two-dimensional response.

10. Use the *CSR*, K_σ value, along a *MSF* value consistent with the results of the deaggregation analysis to evaluate the required CRR based on a factor of safety against trigger liquefaction of 1.25 in accordance with Youd and Idriss (2001).

11. Use the required CRR to calculated $(N_1)_{60CS}$ blow counts required within the looser natural soils to have a factor of safety against trigger liquefaction of 1.25.

12. Compare the required $(N_1)_{60CS}$ blow counts to the actual blow counts within the looser natural soils to evaluate whether liquefaction will be assumed for each location.

In the zone of looser natural soils, the post-liquefaction shear strength will be taken as the original (pre-earthquake) strength for zones with FS > 1.25 and the residual strength (after Idriss 1998) for zones with FS < 1.25. The value of 1.25 is understood to account for the effects of (a) partial pore pressure generation in elements of soil that do not reach initial liquefaction and (b) uncertainty in measured standard penetration resistance.

The residual strengths will be calculated for each sample with FS < 1.25. The residual strength data will then be grouped according to location (North Wall, West Wall, South Wall, and 2H:1V slope between the North and West Walls) to obtain a data set for stability analysis.

REFERENCES

Hart Crowser, 2001. Memorandum: Additional Information on the Seismic Design, Sea-Tac International Airport, SeaTac, Washington, January 25, 2001.

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Youd, T.L. and I.M. Idriss, 2001. "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 127, No. 4, pp. 297-313.

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