PSEUDO-STATIC STABILITY ANALYSIS OF THE 1976 DASHIHE TAILINGS DAM

by

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Report 5

Report for:

Sino-Canadian Joint Investigation on
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The Dashihe tailings dam in North China was shaken by the Tangshan Earthquake, M=7.8 on July 28, 1976 and suffered minor cracking on the downstream shell. Liquefaction was widespread on the beach area with many large cracks, sand blows and some evidence of sliding of tailings materials into the pond.

Finn and Xin (1992) investigated the seismic response and triggering of liquefaction of Dashihe tailings dam using the two-dimensional nonlinear dynamic effective stress analysis program TARA-3 (Finn et al., 1986). The analyses predicted the zones of liquefaction under the pond and beach shown in Figs. 1, 2, and 3. These zones represent the liquefaction potential corresponding to different corrections for the fines content of the fine and silty sands. Field data (Seed et al., 1985) show that the resistance to liquefaction increases with fines content. This increase is taken into account by increasing the measured $(N_1)_{60}$ values. The field data suggest that for the fine and silty sands the $(N_1)_{60}$ values should be increased by 7 blows/ft. To explore more fully the risk to the structure, two other cases were considered: an intermediate increase of 4 blows/ft, and the most conservative option of no increase at all.

Finn and Xin (1992) also analyzed the post-liquefaction behaviour of the dam using the program TARA-3FL (Finn and Yogendrakumar, 1989). The largest horizontal post-liquefaction displacements, ranging between 4 to 6 metres, were computed on the beach near the pond and gradually decreased from there as the crest was approached. However, there were no significant displacements in the downstream slope of the dam or around the crest.

Pseudo-static analyses of the stability of both the upstream slope (or beach area) and the downstream slope of the dam before and after the earthquake were performed using the program XSTABL (Sharma, 1991). The analyses were originally conducted by Xin but all the data were lost from the computer system. The analyses were subsequently repeated by Amante (1993).

In the liquefied zones, the residual strengths of the soils were used. These strengths were obtained using the lower bound of the correlation between the residual strength and the normalized standard penetration resistance $(N_1)_{60}$ (Seed and Harder, 1990). In other zones, the seismic porewater pressures derived from the results by Finn and Xin (1992) were taken into account in the evaluation of shearing resistance.

Pseudo-static forces were incorporated in the analysis to represent the effects of earthquake loading. The seismic coefficients ranged from 0 to 0.12. The latter corresponds to the maximum acceleration of 0.12 g. The resulting factors of safety are summarized in Fig. 4 and the calculated critical slip surfaces are presented in Figs. 5 to 12. The results for the extreme loading condition when the seismic coefficient is 0.12 are discussed in detail. This condition gives a very conservative picture of stability.

Before the earthquake, the factor of safety of the upstream slope was high (F_s = 21.143). However, the earthquake-induced liquefaction and high porewater pressures under the beach and the pond greatly reduced the stability of the upstream slope. For the cases where the liquefaction resistances are evaluated with $(N_1)_{60}$ values of fine and silty sands increased by 7 and 4 blows/ft due to fines content, the factor of safety of the upstream slope dropped to 1.168 and 0.504, respectively. These results indicate that significant sliding or displacements of topmost materials in the upstream slope toward the pond could occur which is consistent with the predicted response by Finn and Xin (1992) and the observed damage features of the dam on the beach area during the 1976 Tangshan Earthquake. When no correction for fines content is taken into account in the analysis, very low values of factor of safety area calculated indicating that massive flow failure of the upstream slope could occur. Such a scenario, however, was not observed during the earthquake. Therefore, analyses taking into account correction of $(N_1)_{60}$ values due to fines content yield more realistic results.

The downstream slope had a factor of safety $F_S = 3.183$ before the earthquake. After the earthquake, the downstream slope remained stable with a factor of safety of 1.440 and 1.310 corresponding to fines corrections of 7 and 4 blows/ft, respectively. When no correction for the effect of fines is applied, the downstream slope had factor of safety of about 1.0 indicating potential instability. Therefore, for realistic estimation of the seismic response and post-liquefaction stability of the dam the effects of fines on liquefaction resistance should be taken into account. Although minor cracking occurred on the downstream shell, no evidence of sliding was observed there.

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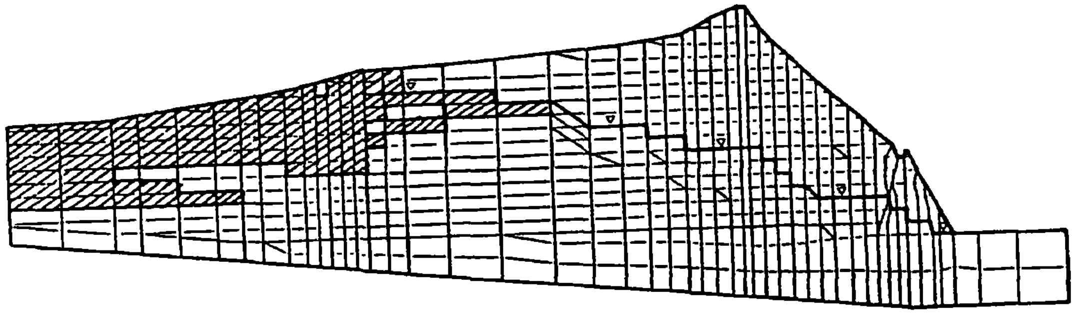


Fig. 1 - Computed liquefied zone shown in shaded area. $(N_1)_{60}$ values increased by 7 blows/ft to account for the effect of fines on liquefaction resistance. (After Finn and Xin, 1992)

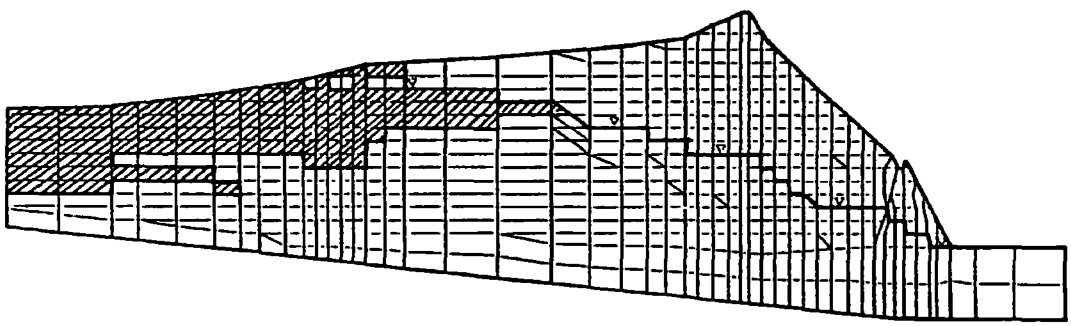


Fig. 2 - Computed liquefied zone shown in shaded area. $(N_1)_{60}$ values increased by 4 blows/ft to account for the effect of fines on liquefaction resistance. (After Finn and Xin, 1992)

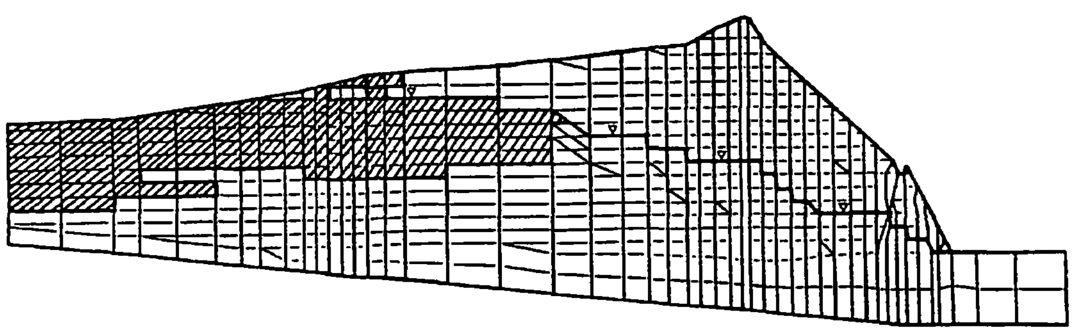


Fig 3 - Computed liquefaction zone shown in shaded area. No correction is made for fines content. (After Finn and Xin, 1992)

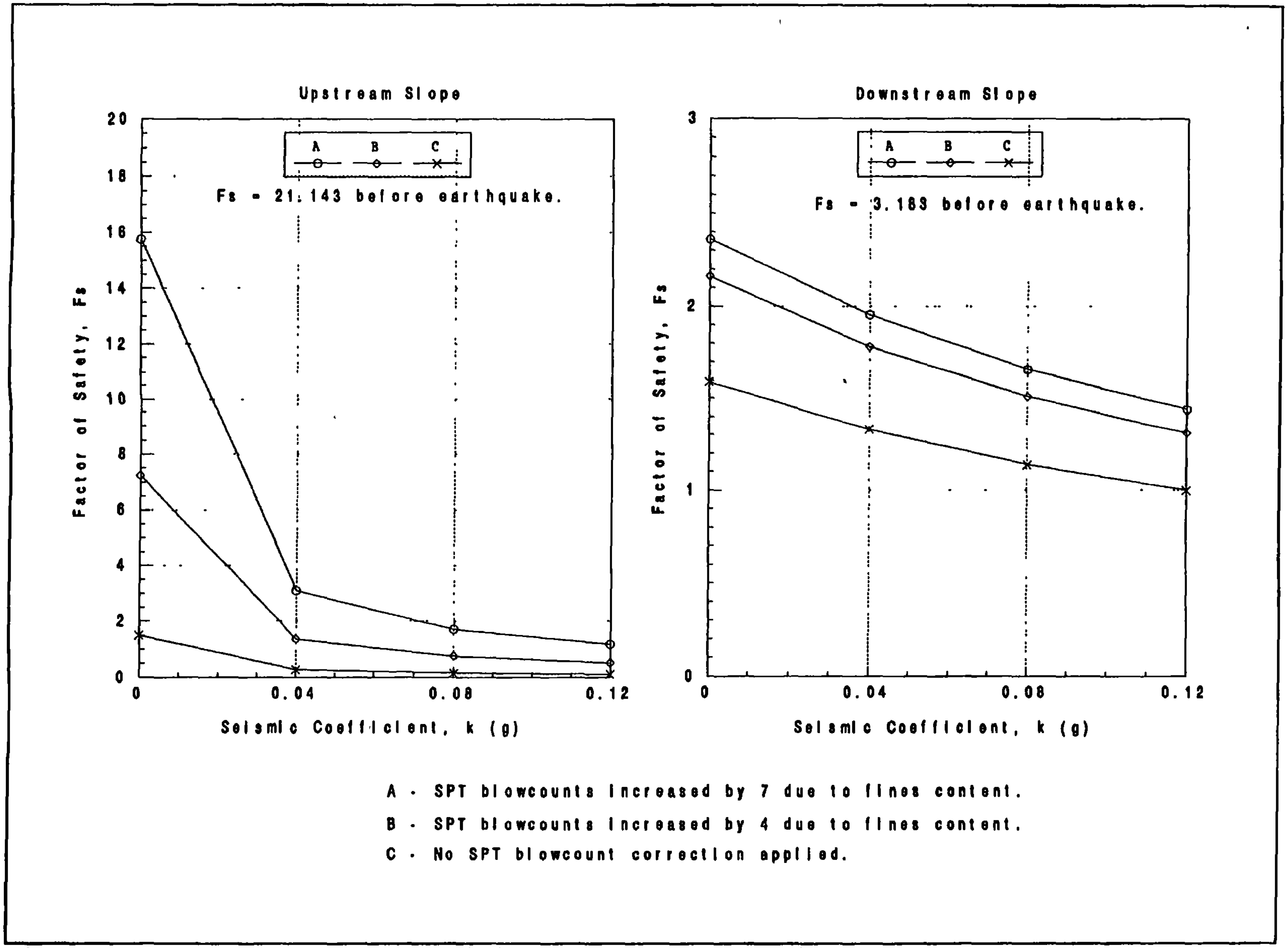


Fig. 4 - Results of slope stability analyses of Dashihe Dam (1976).

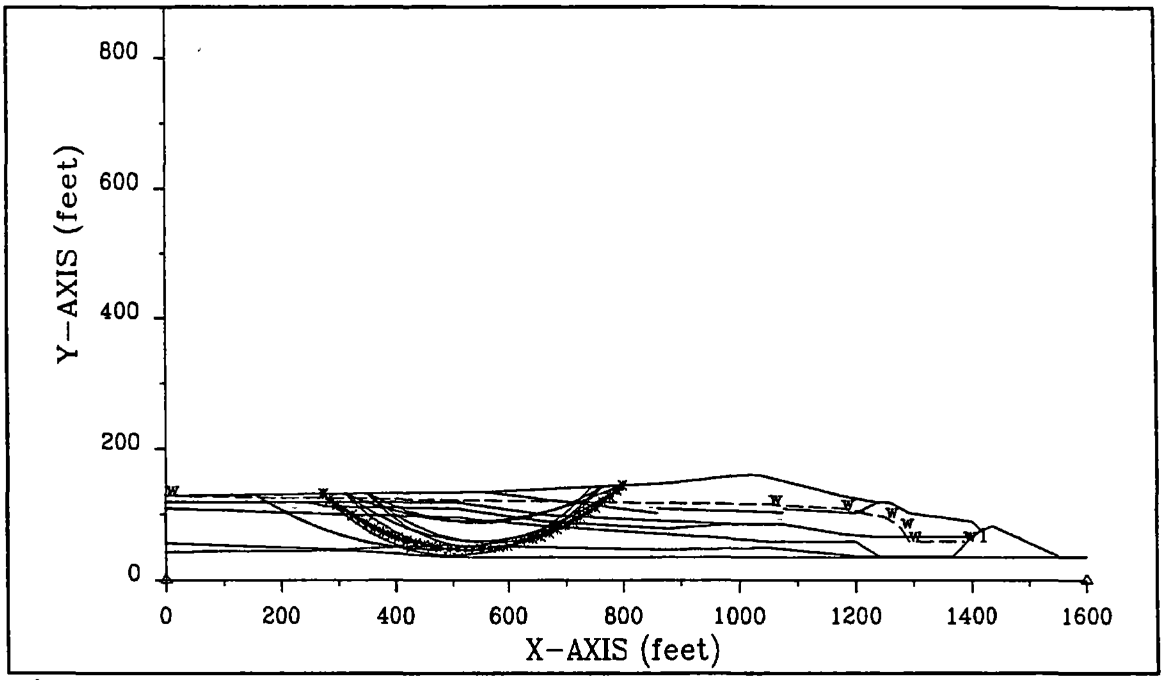


Fig. 5 - Upstream slope Fs = 21.143 before earthquake.

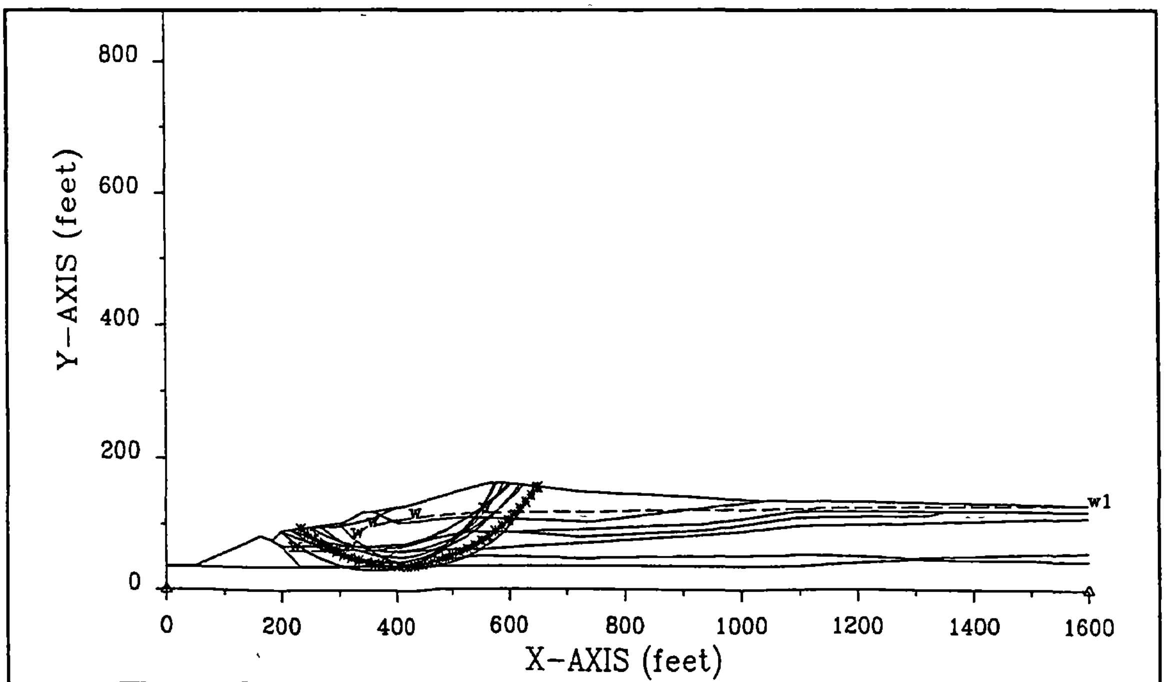
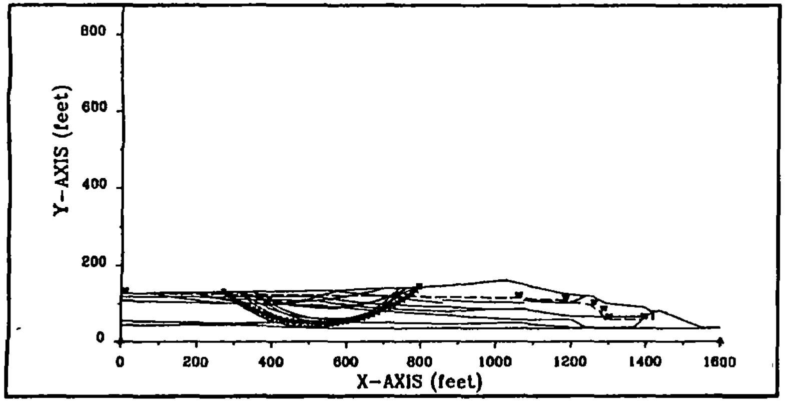
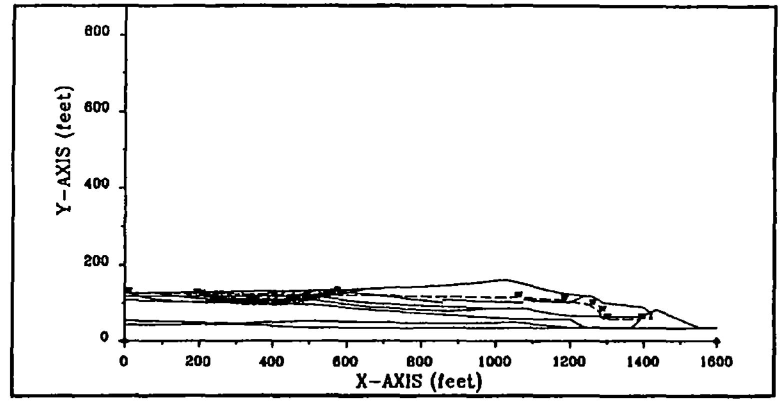


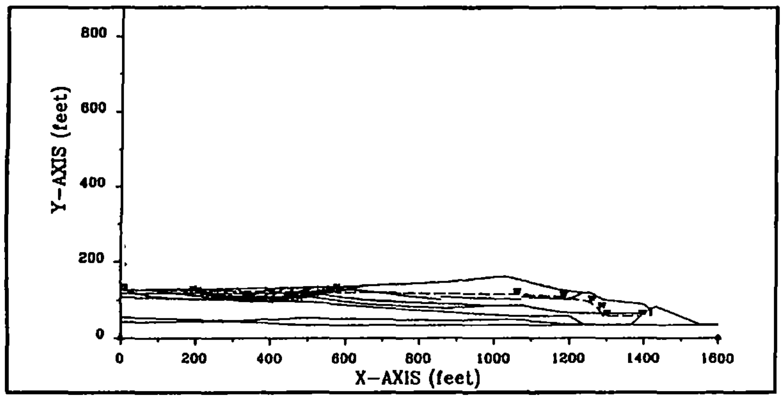
Fig. 6 - Downstream slope Fs = 3.183 before earthquake.



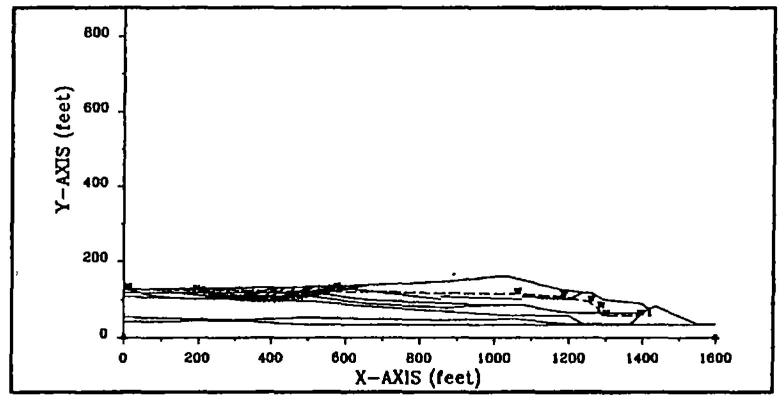
(a) Computed Fs = 15.759 for k = 0.



(b) Computed Fs = 3.088 for k = 0.04.

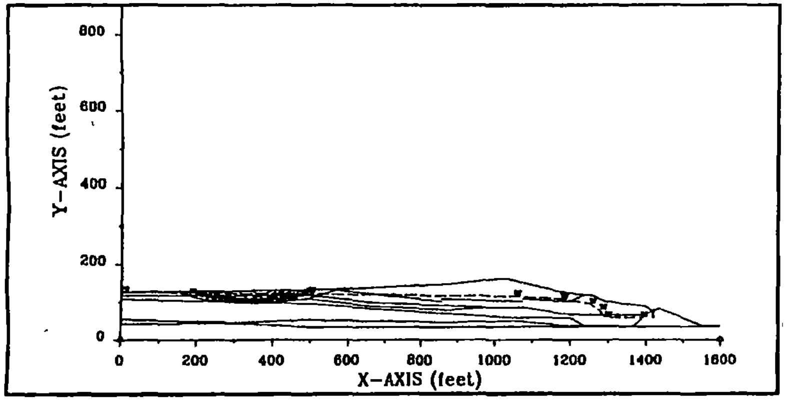


(c) Computed Fs = 1.698 for k = 0.08.

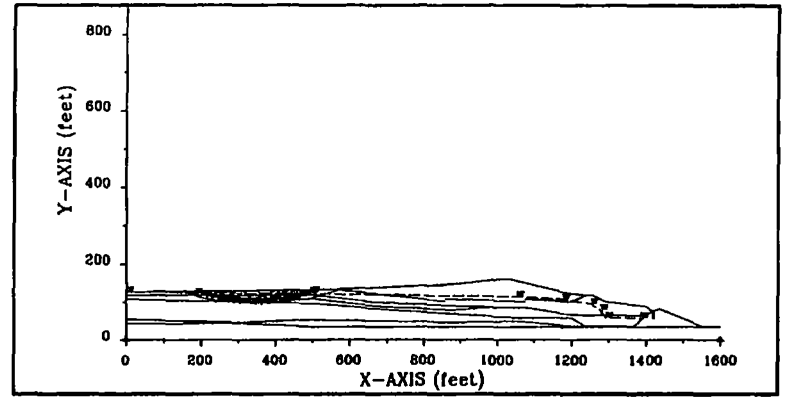


(d) Computed Fs = 1.168 for k = 0.12.

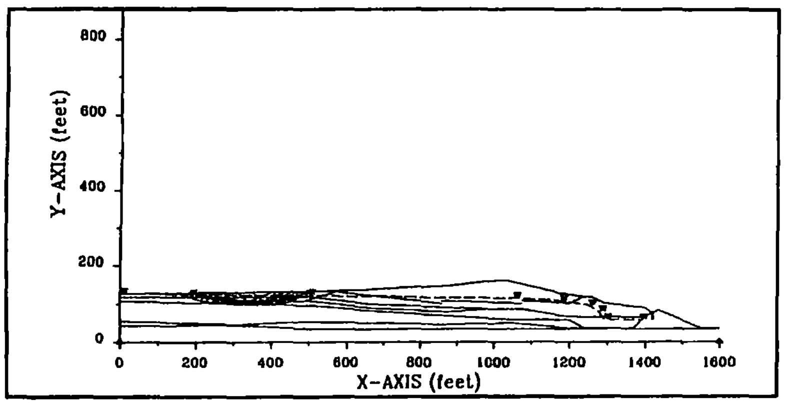
Fig. 7 - Critical slip surfaces for upstream slope where effect of fines on liquefaction resistance is considered. $(N_1)_{60}$ values of both fine and silty sands are increased by 7 blows/ft.



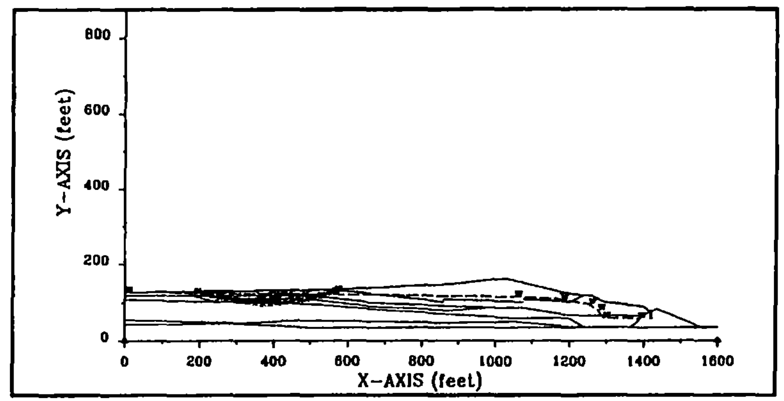
(a) Computed Fs = 7.235 for k = 0.



(b) Computed Fs = 1.353 for k = 0.04.

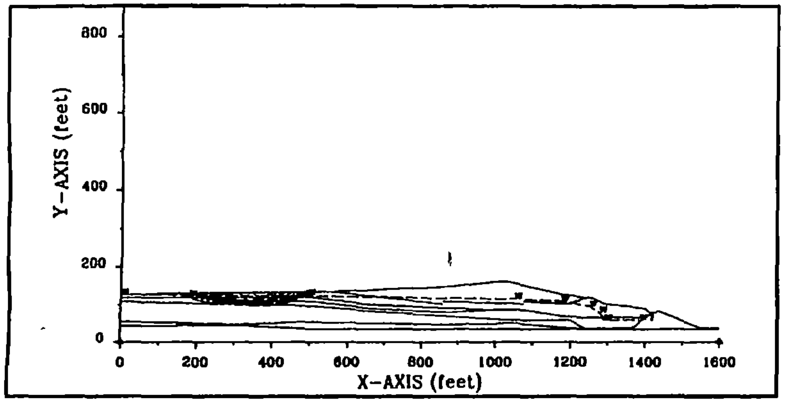


(c) Computed Fs = 0.746 for k = 0.08.

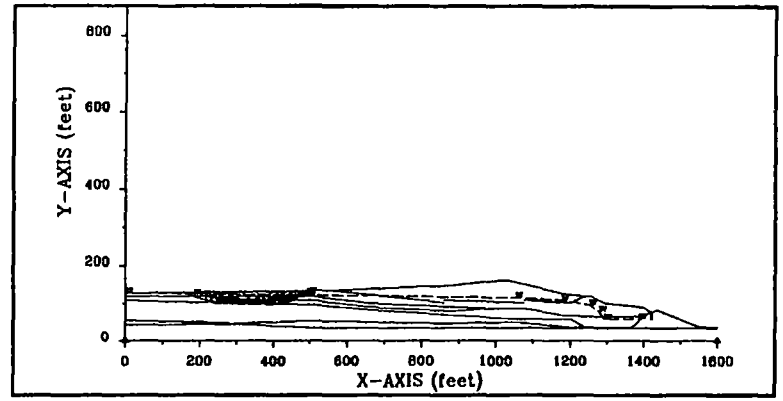


(d) Computed Fs = 0.504 for k = 0.12.

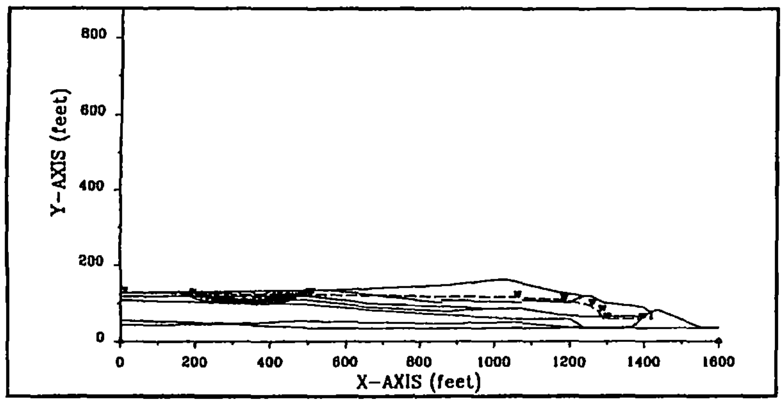
Fig. 8 - Critical slip surfaces for upstream slope where effect of fines on liquefaction resistance is considered. $(N_1)_{60}$ values of both fine and silty sands are increased by 4 blows/ft.



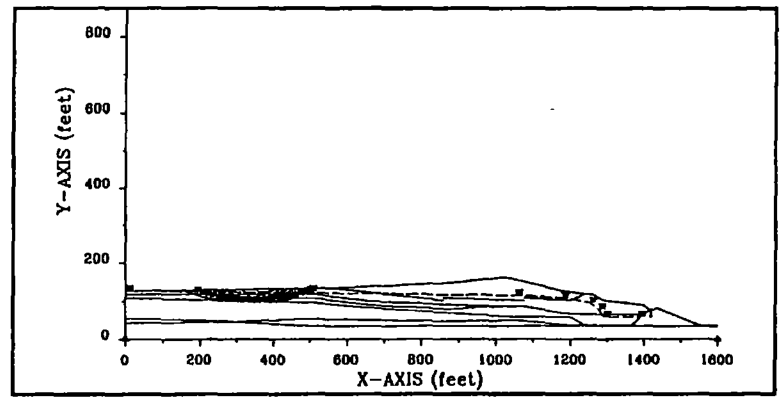
(a) Computed Fs = 1.497 for k = 0.



(b) Computed Fs = 0.280 for k = 0.04.

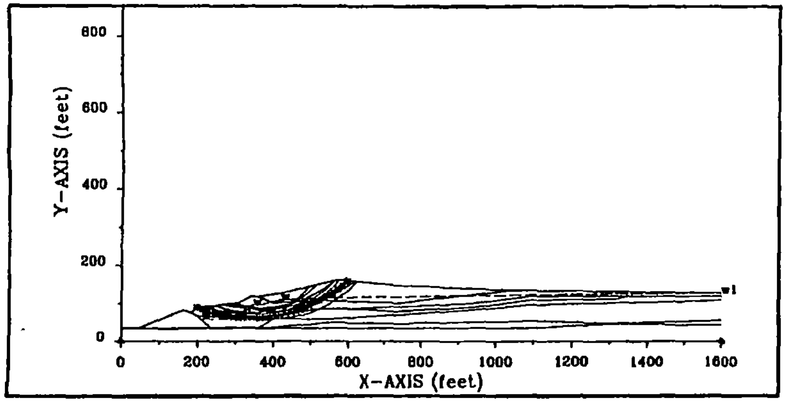


(c) Computed Fs = 0.154 for k = 0.08.

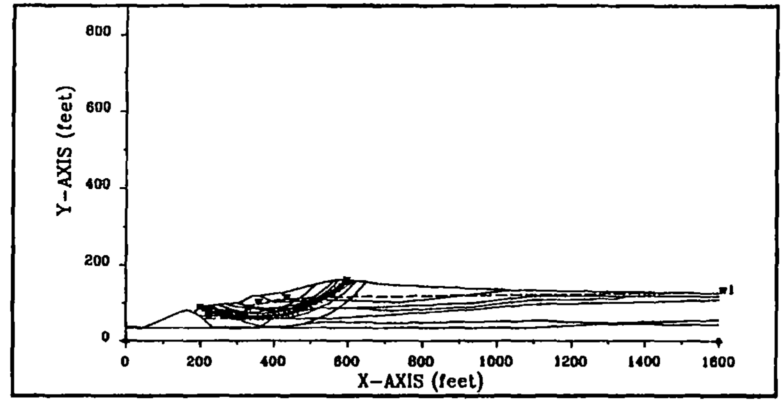


(d) Computed Fs = 0.107 for k = 0.12.

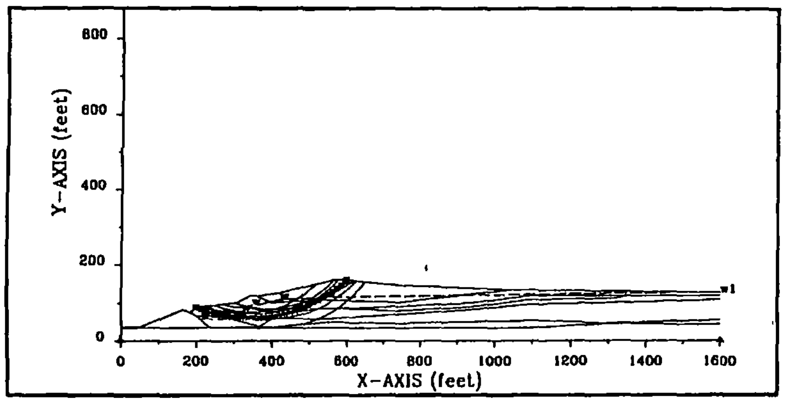
Fig. 9 - Critical slip surfaces for upstream slope where effect of fines on liquefaction resistance is not considered.



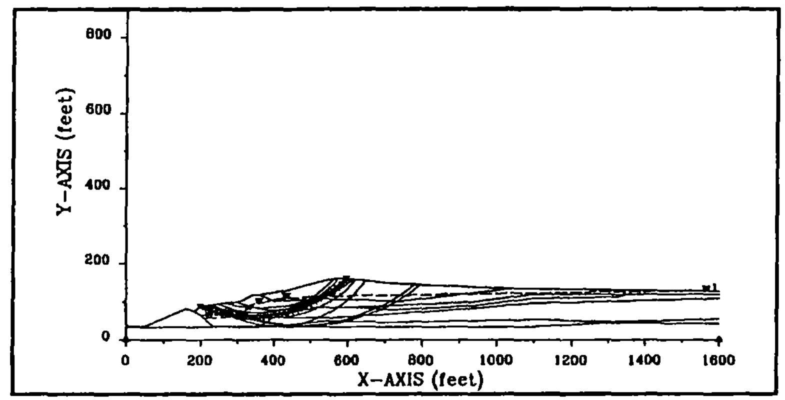
(a) Computed Fs = 2.360 for k = 0.



(b) Computed Fs = 1.953 for k = 0.04.

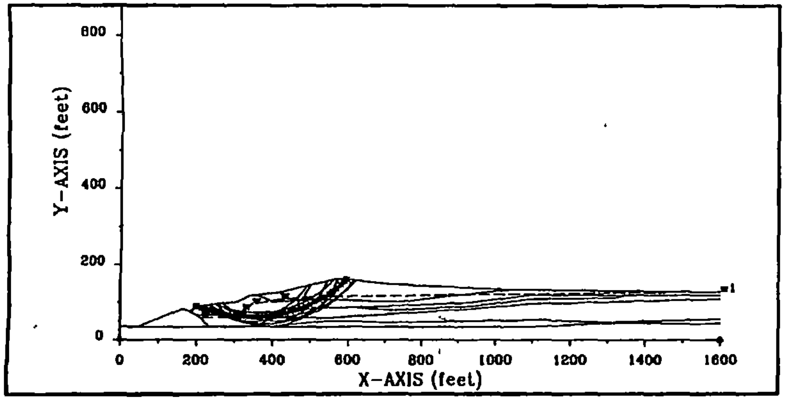


(c) Computed Fs = 1.659 for k = 0.08.

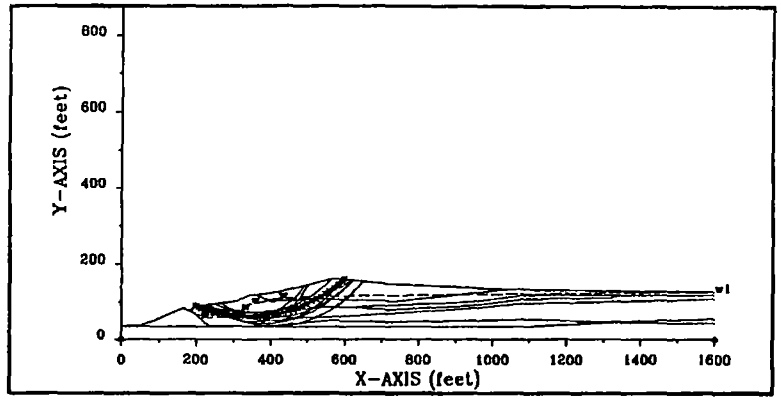


(d) Computed Fs = 1.440 for k = 0.12.

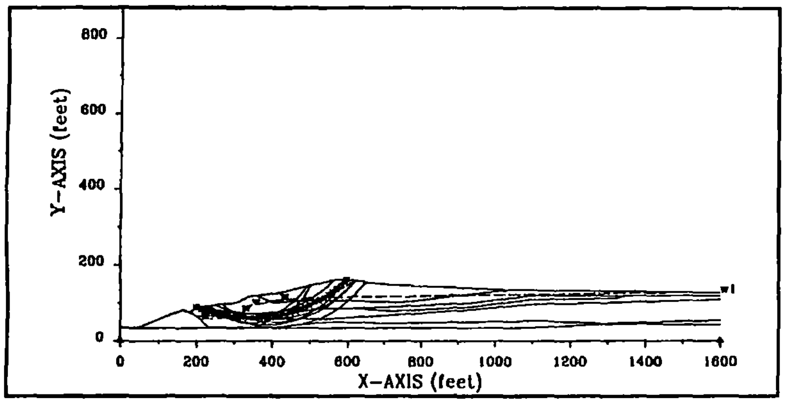
Fig. 10 - Critical slip surfaces for downstream slope where effect of fines on liquefaction resistance is considered. $(N_1)_{60}$ values of both fine and silty sands are increased by 7 blows/ft.



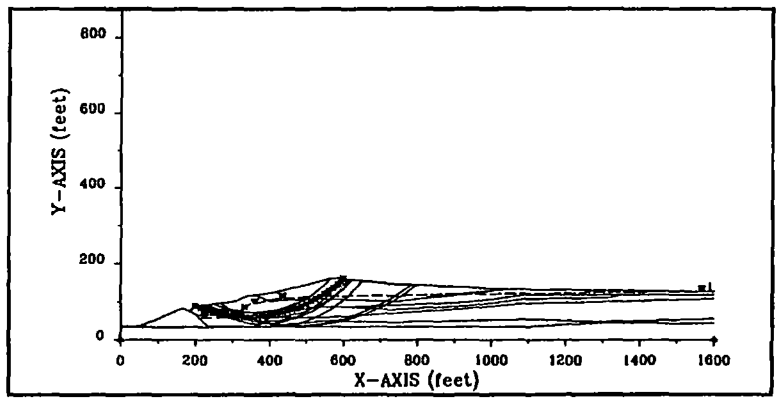
(a) Computed Fs = 2.161 for k = 0.



(b) Computed Fs = 1.781 for k = 0.04.

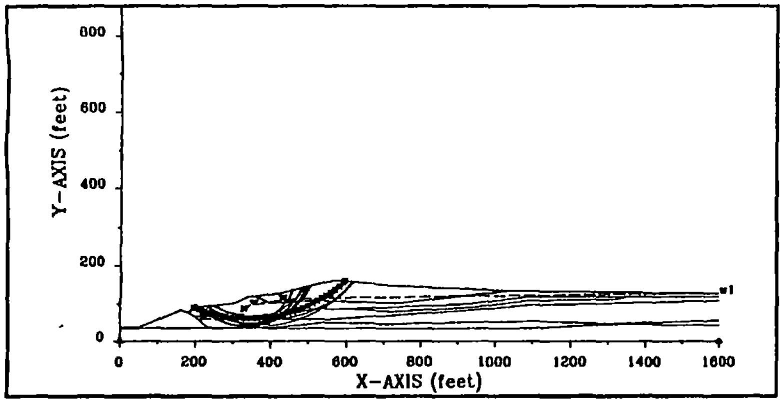


(c) Computed Fs = 1.511 for k = 0.08.

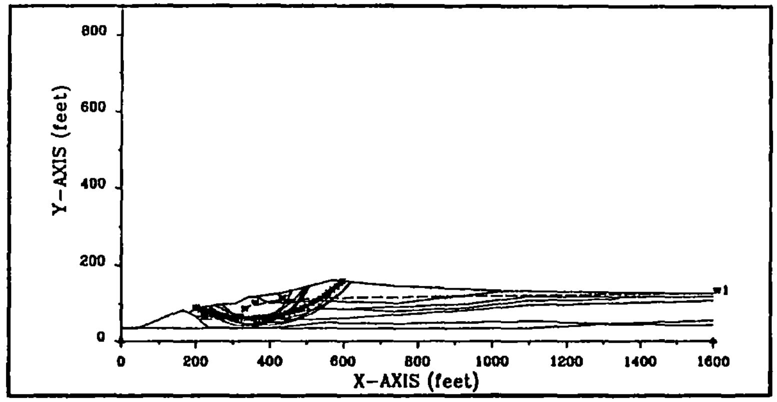


(d) Computed Fs = 1.310 for k = 0.12.

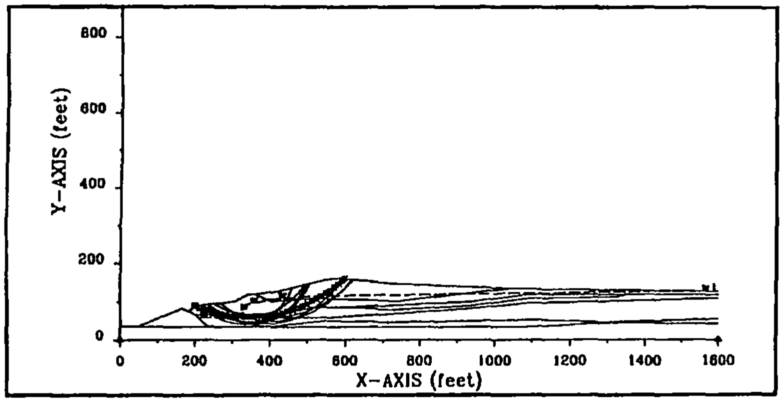
Fig. 11 - Critical slip surfaces for downstream slope where effect of fines on liquefaction resistance is considered. $(N_1)_{60}$ values of both fine and silty sands are increased by 4 blows/ft.



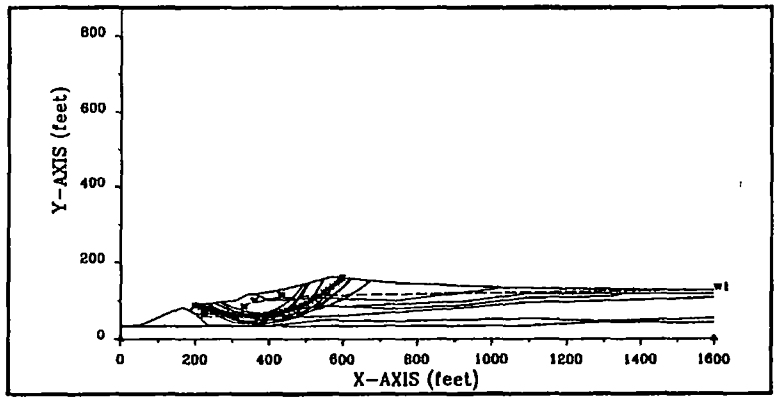
(a) Computed Fs = 1.592 for k = 0.



(b) Computed Fs = 1.326 for k = 0.04.



(c) Computed Fs = 1.138 for k = 0.08.



(d) Computed Fs = 0.998 for k = 0.12.

Fig. 12 - Critical slip surfaces for downstream slope where effect of fines on liquefaction resistance is not considered.