Seismic risk at Vancouver International Airport

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ABSTRACT: The probability of triggering liquefaction beneath the surrounding dykes and the runways at Vancouver International Airport due to earthquake activity is examined. The damage caused by such triggering and the likely cost of repair is assessed together with the cost of remedial work which would prevent liquefaction from occurring. These costs and probabilities are incorporated in an economic study which indicates that it is better to repair the dykes and runways when and if they should be damaged by an earthquake rather than expend money now to prevent such damage from occurring at a later date.

1. INTRODUCTION

Vancouver International Airport (VIA) is located on Sea Island in the delta of the Fraser river as shown in Fig. 1. The island is flat, low lying and is ringed by dykes to prevent flooding during high tides. It is underlain by significant depths of loose saturated sandy soils that are susceptible to complete strength loss or liquefaction if subjected to earthquake shaking of sufficient intensity. Such liquefaction has led to very severe earthquake induced damage to buildings, dykes, airports and lifeline facilities in other areas underlain by similar soil conditions.

The coastal area of British Columbia is a region of high seismic activity. This paper examines the probability of triggering liquefaction, and the likely consequences to the dykes and runways if liquefaction should occur. In addition, the feasibilty of expending money now to prevent liquefaction from occurring as compared to repairing the damage if and when it should occur is examined.

Because of the deep deltaic deposits at the site, a critical factor in the liquefaction assessment is the magnitude of the surface accelerations. These are computed from rock motions using a one-dimensional dynamic analysis with an equivalent linear soil stress-strain relation. The computed surface amplified motion is compared with measured motions at the site and with measurements at other sites with similar soil conditions. The possibility of long period base rock motion such as occurred at Mexico City is also examined.

2. METHODS OF ASSESSING LIQUEFACTION RESISTANCE

In dealing with granular soils under earthquake loading there are basically three questions to address:

1) what level of shaking will trigger initial liquefaction, and if triggered,
2) what strains are likely to occur, and
3) what is the residual strength?

The answer to all three of these questions depends largely upon the state or density of the soil and has been addressed by Seed et al., (1984), and Seed (1986).

A measure of density or liquefaction resistance can be obtained from a penetration test such as the standard penetration test (SPT) or the cone penetration test (CPT). Because the SPT is currently the standard field test for evaluating liquefaction potential and because much of the field data at the site comprised of SPT values, this test was used to evaluate the likely earthquake response of the soil. In this test the number of blows to drive the sampling tube 0.3 metres is termed the standard penetration value, N. This value was corrected for both energy and confining

Fig. 1. Location of Vancouver International Airport.

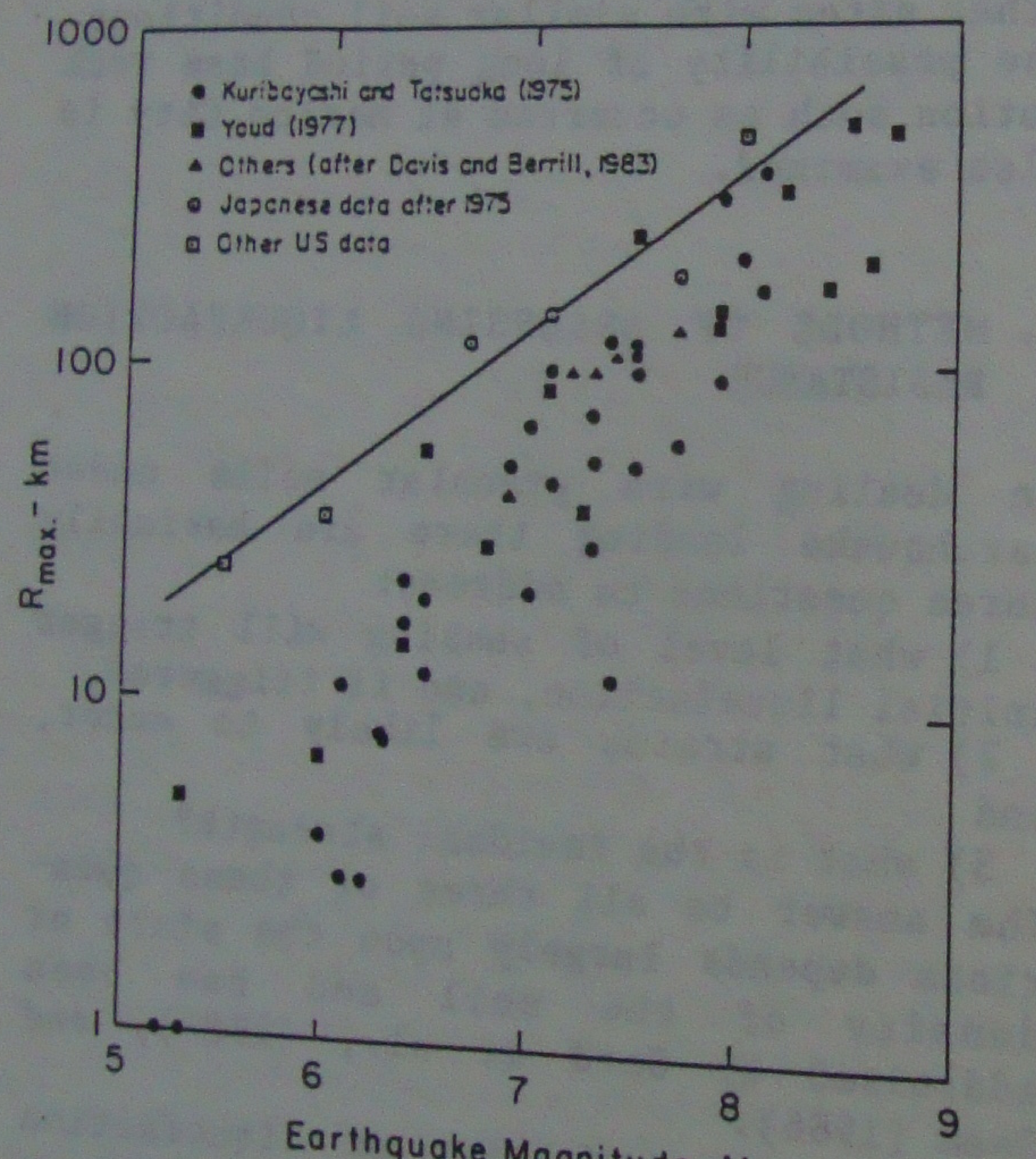

Fig. 2. Maximum Epicentral Distance to a Site of Liquefaction vs. Earthquake Magnitude (Seed et al., 1984).

stress as suggested by Seed et al. (1984), and the resulting value denoted as $(N_1)_{60}$.

The likelihood of triggering liquefaction is best assessed from field data based on experience during past earthquakes. Seed et al. (1984) have presented two charts, Figs. 2 and 3,

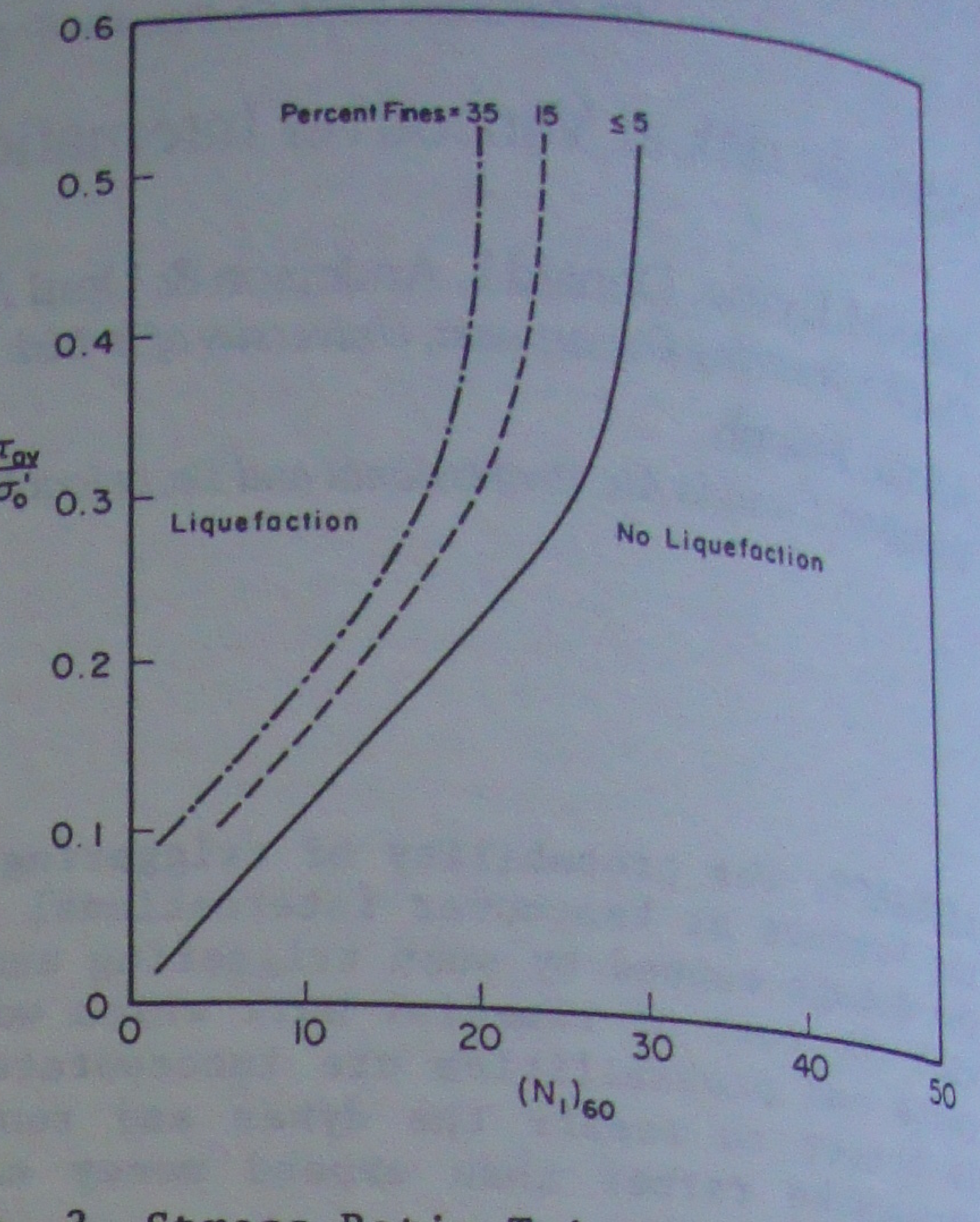

Fig. 3. Stress Ratio Triggering Liquefaction vs. $(N_1)_{60}$ Values for M = 7.5 Earthquakes (Seed et al., 1984).

which represent the current state of the art in liquefaction assessment.

Fig. 2 shows the maximum distance from the epicentre at which liquefaction has been observed to occur for various magnitude earthquakes. The upper bound line represents very loose material that could have an $(N_1)_{60}$ value of about 4. Sites with denser materials with higher $(N_1)_{60}$ values would not liquefy, but cannot be accounted for in this plot.

In Fig. 3 the average cyclic stress ratio τ_{av}/σ'_o required to trigger liquefaction is shown as a function of the standard penetration value $(N_1)_{60}$ for sands containing various percentages of fines. τ_{av} is taken as 0.65 the maximum shear stress caused by the earthquake and σ'_o is the effective overburden pressure. Fig. 3 is appropriate for earthquakes of magnitude 7.5 on the Richter scale. For other magnitude earthquakes the resistance ratio should be corrected as shown in Table 1. This correction is necessary because both field and laboratory observations indicate that liquefaction depends not only on the stress ratio but also on the number of cycles of strong motion, which in turn depends on the magnitude of the earthquake as shown in Table 1.

Both the charts shown in Figs. 2 and 3 will be used in the assessment of liquefaction at the airport.

Table 1. Correction Factors for Influence of Earthquake Magnitude on Liquefaction Resistance

Earthquake Mag., M	Number of Equiv. Cycles	$\frac{\tau_{av}}{\sigma'_o}$ for M=M / $\frac{\tau_{av}}{\sigma'_o}$ for M=7.5
$8^{1/2}$	26	0.89
$7^{1/2}$	15	1.0
$6^{3/4}$	10	1.13
6	5-6	1.32
$5^{1/4}$	2-3	1.5

3. SOIL CONDITIONS

The soil conditions underlying Sea Island have been described in some detail by Johnson (1981). The soils comprise of about 180 m of recent deltaic deposits underlain by pleistocine glacial till and bedrock. A typical soil profile is shown in Figure 4.

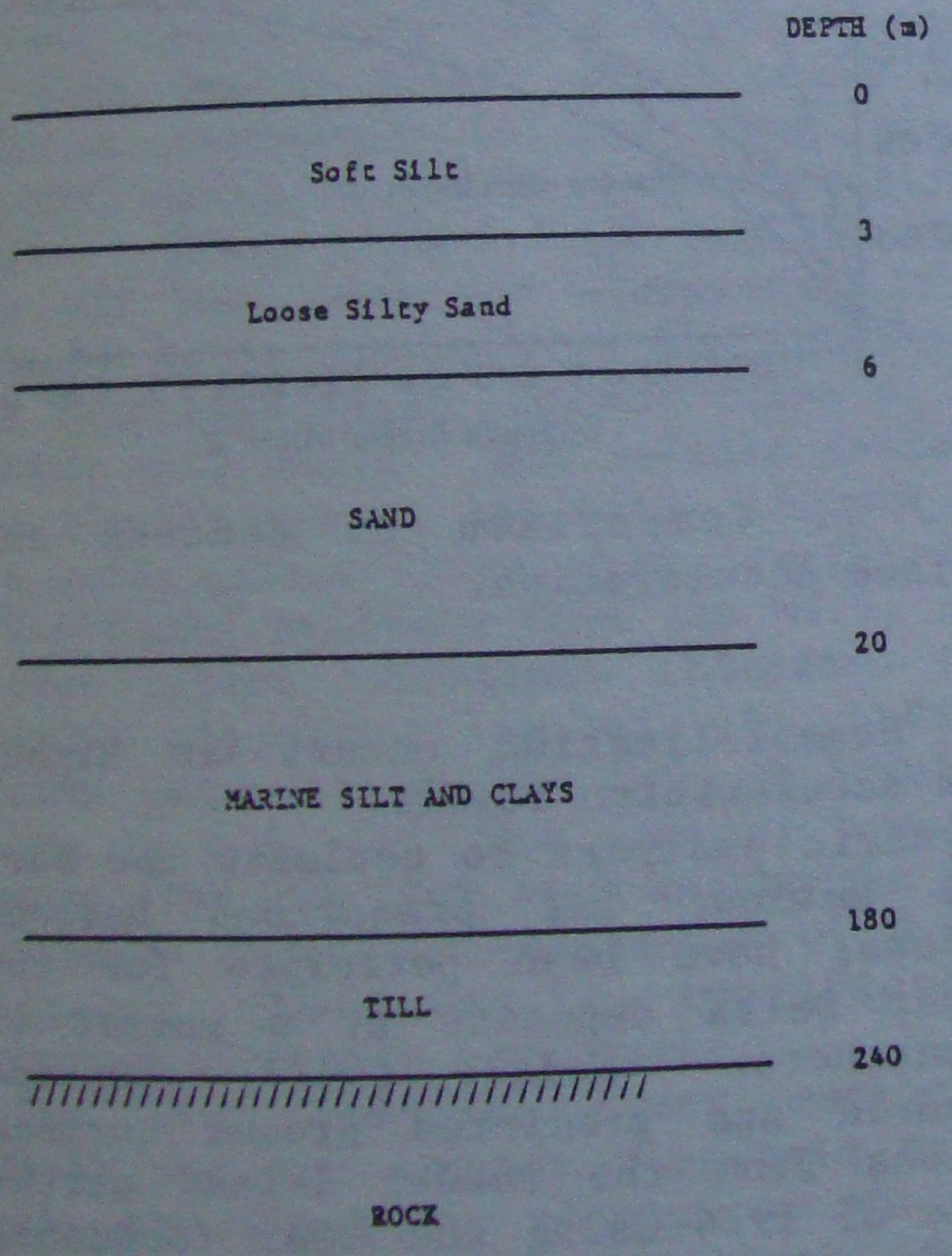

Fig. 4. Typical Soil Profile, Sea Island

The dyke is comprised mainly of loose silty sand material 2 to 3 m in thickness overlying the natural deposits described above. The standard penetration values are less than 10, reflecting loose materials, to depths of 6 to 8 m below the crest of the dyke.

The resistance ratio of the soils versus depth is shown in Fig. 5. This ratio was computed using measured N values and the liquefaction chart of Fig. 3. It may be seen that the computed liquefaction resistance ratios in the top 8 m range mainly between 0.05 and 0.20 with an average value of about 0.10. The values increase with depth below 8 m. The N values were obtained from 22 boreholes which were drilled at approximately equal spacing along the crest of the dyke.

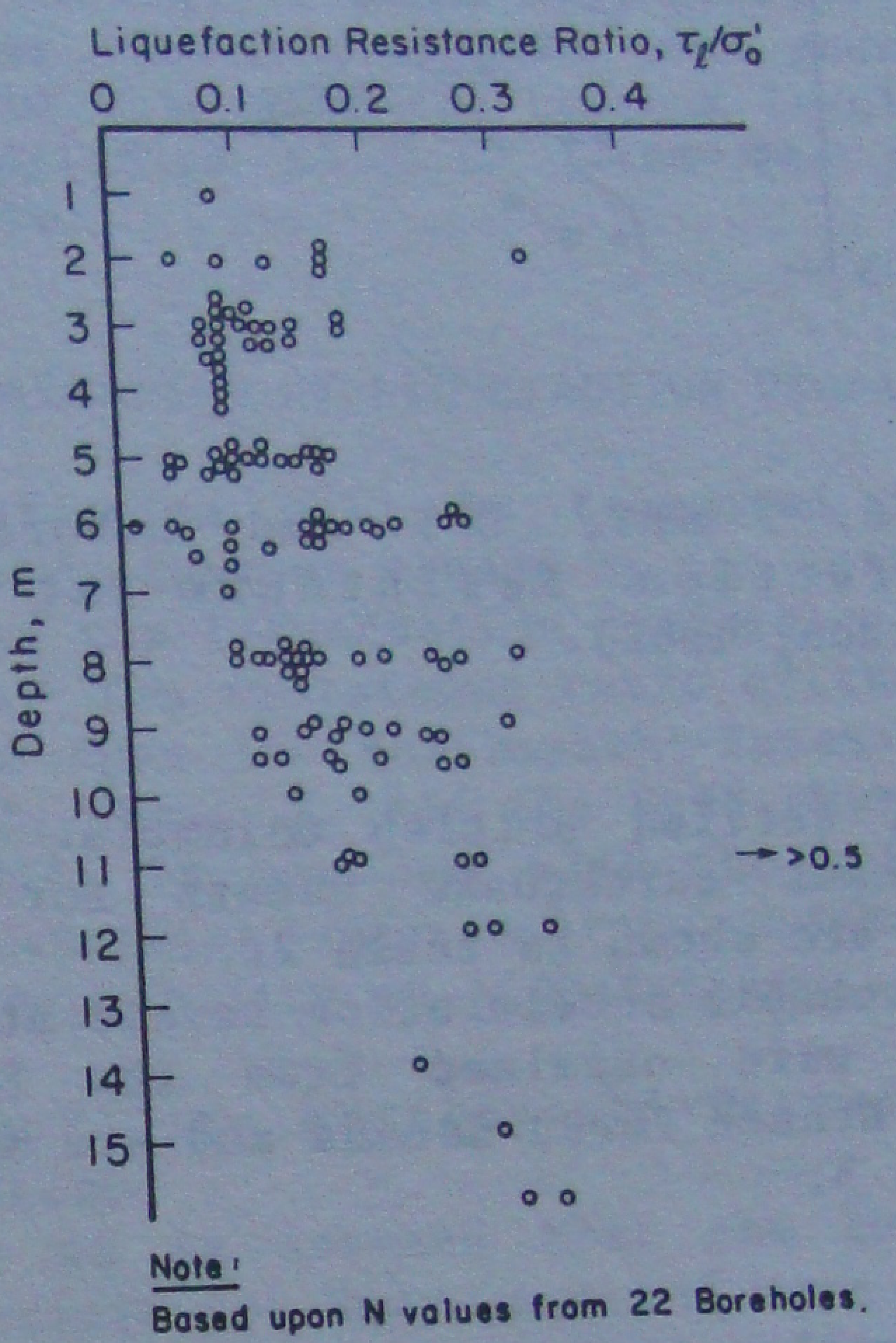

Fig. 5. Liquefaction Resistance Ratio of VIA Soils.

Dynamic resistance ratios at McDonald's farm on Sea Island (Fig. 1) have been determined by Robertson (1982) and are shown in Fig. 6. These resistance ratios are based on standard penetration tests, cone penetration tests, and laboratory tests on undisturbed samples, and range between 0.08 and 0.2. It may be seen that these values are in reasonable agreement with those determined from N values beneath the dykes.

4. SEISMICITY

The coastal area of British Columbia lies within a high seismic activity belt. Reports of seismic activity in the lower mainland area in which Sea Island and the Airport is located date back to 1872 when a large earthquake was felt throughout

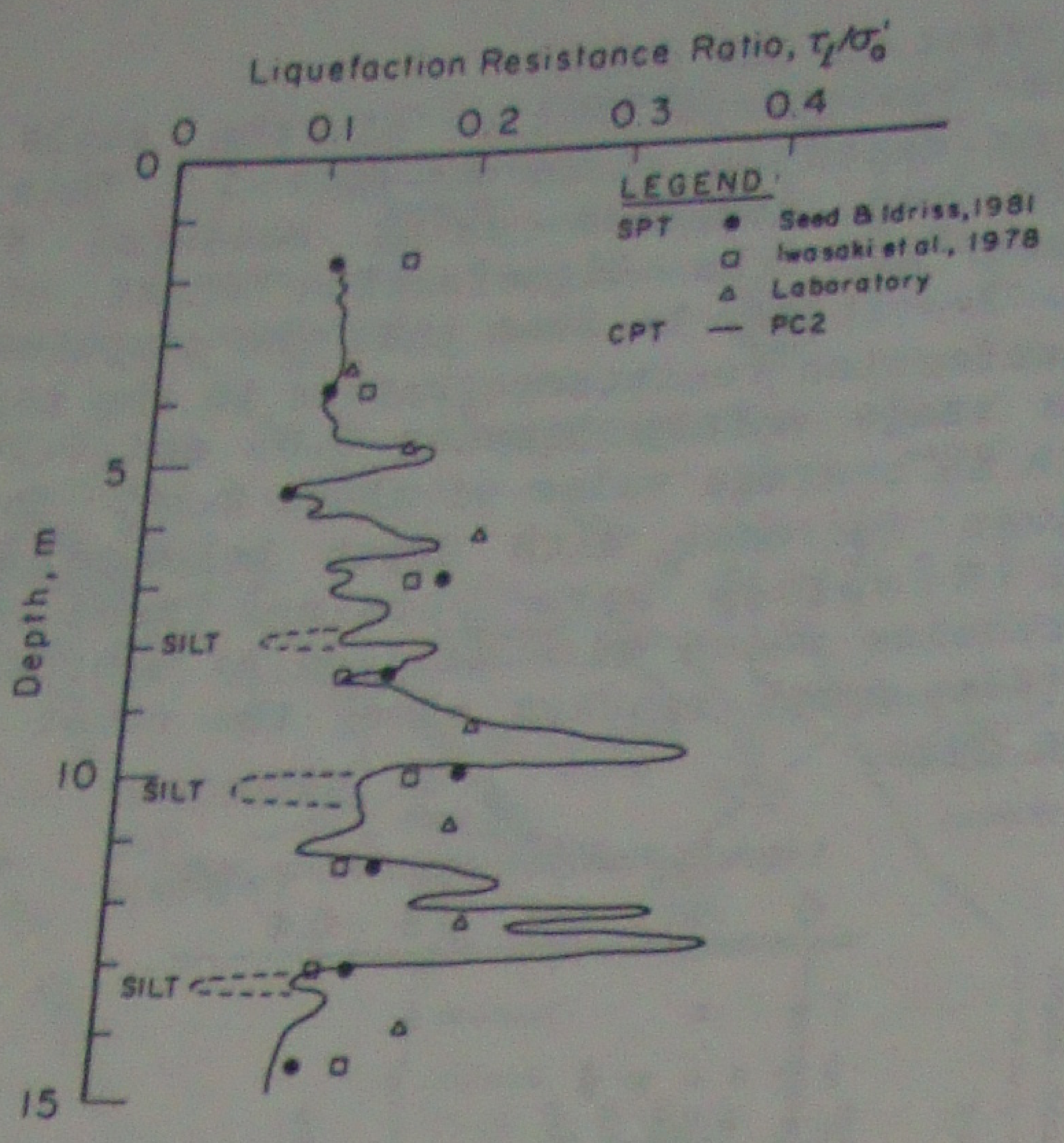

Fig. 6. SPT, CPT and Laboratory Liquefaction Resistance at VIA (Robertson, 1982).

most of settled British Columbia. The significant earthquake events for the Airport are shown in Table 2.

The probable acceleration levels at the Airport were obtained from the Earth Physics Branch (EPB) Canada and are shown in Table 3.

Table 2. Peak Accelerations (Firm Ground) at VIA For Events of M>5

Year	M	Dist(km)	PGA(g)
1872	7.4	146	0.087
1909	6.0	35	0.120
1920	5.0	57	0.016
1926	5.5	64	0.025
1943	5.0	91	0.008
1946	7.3	168	0.062
1976	5.4	46	0.036

Note: PGA = Probable maximum ground acceleration based on attenuation relation of Hasegawa et al. (1981)

Table 3. Probable Acceleration Levels for VIA

Probability Per Annum	Return Period	PGA
P	1/P	
0.01	100	0.09
0.005	200	0.14
0.0021	475	0.21
0.001	1000	0.26

5. AMPLIFICATION OF ACCELERATION DUE TO PRESENCE OF DEEP SOFT DEPOSITS

Predicted maximum accelerations from the EPB are for rock or firm ground conditions. When a site is overlain by unconsolidated sediments as is the case here, there is a possibility of amplification as the earthquake motion passes upward through the softer sediments. In addition, there will be a shift to longer period motion. A comparison of recorded base rock and surface motions was made by Seed (1976), and is shown in Fig. 7, and indicates that amplification generally occurs when the peak rock acceleration levels are less than 0.13 g

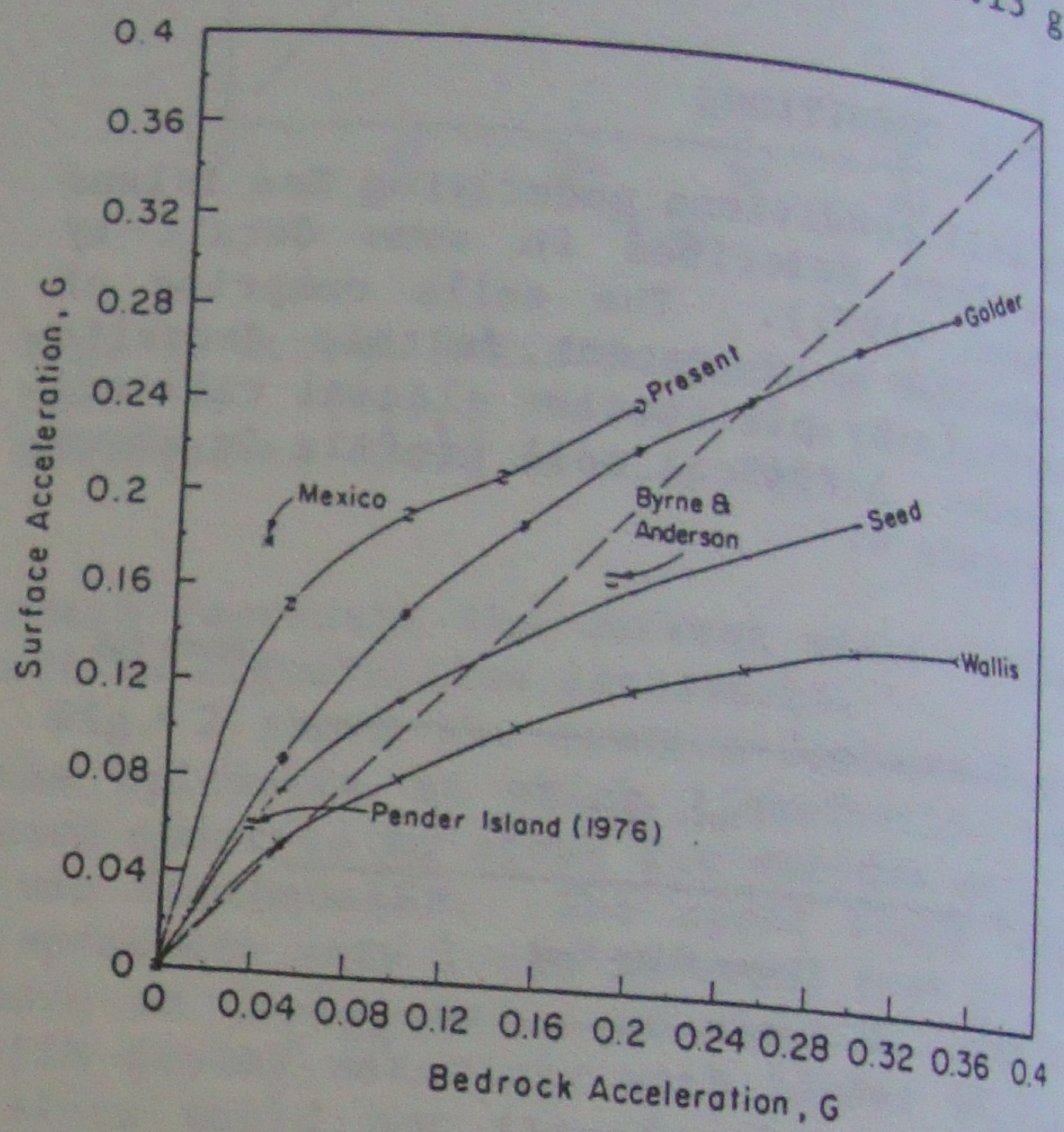

Fig. 7. Comparison of Bedrock and Surface Acceleration.

and deamplification occurs at higher rock acceleration levels.

Dynamic analyses to estimate the surface motion, for prescribed bedrock motions, have been performed for the Fraser Delta deposits by a number of researchers. Wallis (1979) compared measured and predicted ground surface motions from the Pender Island earthquake of 1976 using the SHAKE (Schnabel et al., 1972) one-dimensional vertically propagating shear wave analysis. His predictions for surface motions as compared to bedrock motions are shown in Fig. 7, where it may be seen that he generally predicted a deamplification of the motion as it rises through the soil. Byrne and Anderson (1983) in a study of the seismic hazard in Richmond which includes Sea Island, also performed

SHAKE analyses and predicted somewhat higher surface acceleration levels, also shown in Fig. 7. Golder Associates, in a study of the nearby Annacis Island Bridge, performed a similar study and predicted higher acceleration as shown. SHAKE analyses were also performed for this study for the section shown in Fig. 4 and the predicted surface accelerations which are also shown in Fig. 7 lie above all the previous predictions. The difference in predicted response between the various researchers is mainly due to amplification effects in the top 3m. Our predicted acceleration at a depth of 3m lies well below that predicted at the surface and essentially coincides with Seed's curve shown in Fig. 7.

In the September 1985 Mexico earthquake, very high amplification of base motion occurred in the overlying, very soft lake bed sediments of Mexico City. Base motions of 0.04 g were amplified to about 0.18 g on the surface. This point is also shown in Fig. 7 and is higher than all preceeding data. The motion involved here was from a very large magnitude earthquake, M8.1, at a distance of some 400 km. Such a motion, even on firm ground, would have had a high component of long period waves, which could have induced resonance in the long period natural modes of the very soft overlying soils. Since such a high amplification has not been observed elsewhere, it is thought to be due to very low damping in the very high water content clays of the Mexico City basin. The deltaic soils at the Airport are of low water content and are quite unlike those of Mexico City.

The field evidence from the M5.4, 1976 Pender Island earthquake indicates that little amplification of the bedrock motion occurred at the 0.04 g level as shown in Fig. 7. The lack of reported damage from the 1909 M6.0 earthquake with estimated bedrock acceleration of 0.12 g suggests that little amplification occurred with this earthquake.

The 1946 Campbell River earthquake (M7.5) was the largest magnitude earthquake to affect the airport in the past 100 years. While this earthquake would, because of its long distance from the site, cause a predicted acceleration on bedrock of only 0.06 g, there would have been the possibility of a high level of amplification due to a resonance effect. Hodgson (1946) reporting on this earthquake makes no mention of damage in the Fraser Delta. The newspaper reports of the earthquake state that the runways at Vancouver Airport "rolled in waves" but that no significant damage occurred. Thus the field evidence suggests that the soft delta deposits should not give rise to excessive amplification of bedrock motions such as occurred at Mexico City.

Bedrock accelerations in the range of 0.10 g will be of most concern for liquefaction considerations. Because the field evidence suggests that excessive amplification has not occurred at the Airport during past earthquakes, an amplification factor of 1.3, which is based upon the average of the curves shown in Fig. 7, seems appropriate. This implies that a predicted peak acceleration of 0.10 g at the bedrock level will be amplified to 0.13 g at the ground surface.

6. EVALUATION OF LIQUEFACTION PROBABILITY

Liquefaction will be triggered if the dynamic stresses caused by the earthquake exceed the liquefaction resistance of the soil. The resistance ratio of the soils based upon the Koribayashi-Tatsuoka-Seed chart, Fig. 2, and the Seed chart, Fig. 3, will be combined with the seismicity to determine two separate estimates of the probability of liquefaction.

Method 1: Koribayashi-Tatsuoka-Seed Approach

If it is assumed that the foundation materials are very weak and have a liquefaction resistance corresponding to the weakest materials that have been observed during past earthquakes, then the Koribayashi-Tatsuoka-Seed chart can be used to assess the probability of liquefaction. A modified version of the computer program PROLIQ (Atkinson et al., 1984) was used for this purpose and gave a probability of 0.011 per annum (91 year return period) that liquefaction would occur.

The modified PROLIQ program computes the probability as follows:

1) A magnitude of earthquake is selected.

2) The maximum distance, D, from the site at which such a magnitude of earthquake would cause liquefaction is obtained from Figure 2. This determines the area over which such a magnitude earthquake could occur and cause liquefaction at the site.

3) The number of occurrences per unit area of seismogenic zone is computed from the magnitude recurrence relationship.

4) The total number of occurrences for this magnitude of earthquake is then the

product of the occurrences per unit area multiplied by the contributing area.

5) By considering the complete range of earthquake magnitudes, the contribution of all magnitudes is accounted for and leads to the probability of liquefaction.

Method 2: Seed N Value Method

The shear stress ratio level caused by the earthquake can be estimated from the maximum ground surface acceleration as follows:

$$\frac{\tau_{av}}{\sigma_o'} = 0.65 \frac{\sigma_o}{\sigma_o'} A \, r_d \qquad (1)$$

in which

τ_{av} = the average cyclic shear stress caused by the earthquake

σ_o = the total vertical stress

σ_o' = the effective vertical stress

A = the maximum surface acceleration in gravity units

r_d = a reduction factor with depth which varies from 1 at the surface to 0.9 at a depth of 10 m.

Liquefaction will be triggered if the shear stress ratio caused by the earthquake, τ_{av}/σ_o', exceeds the liquefaction resistance ratio of the soil, τ_ℓ/σ_o'.

The value of the surface acceleration A_{crit} that will just cause liquefaction to occur can be computed from the known τ_ℓ/σ'_o values of Fig. 5 and Eq. 1. The computed values are shown in Fig. 8 for the depth range of concern, where it may be seen that A_{crit} ranges between about 0.05 g and 0.20 g with an average value of about 0.10 g.

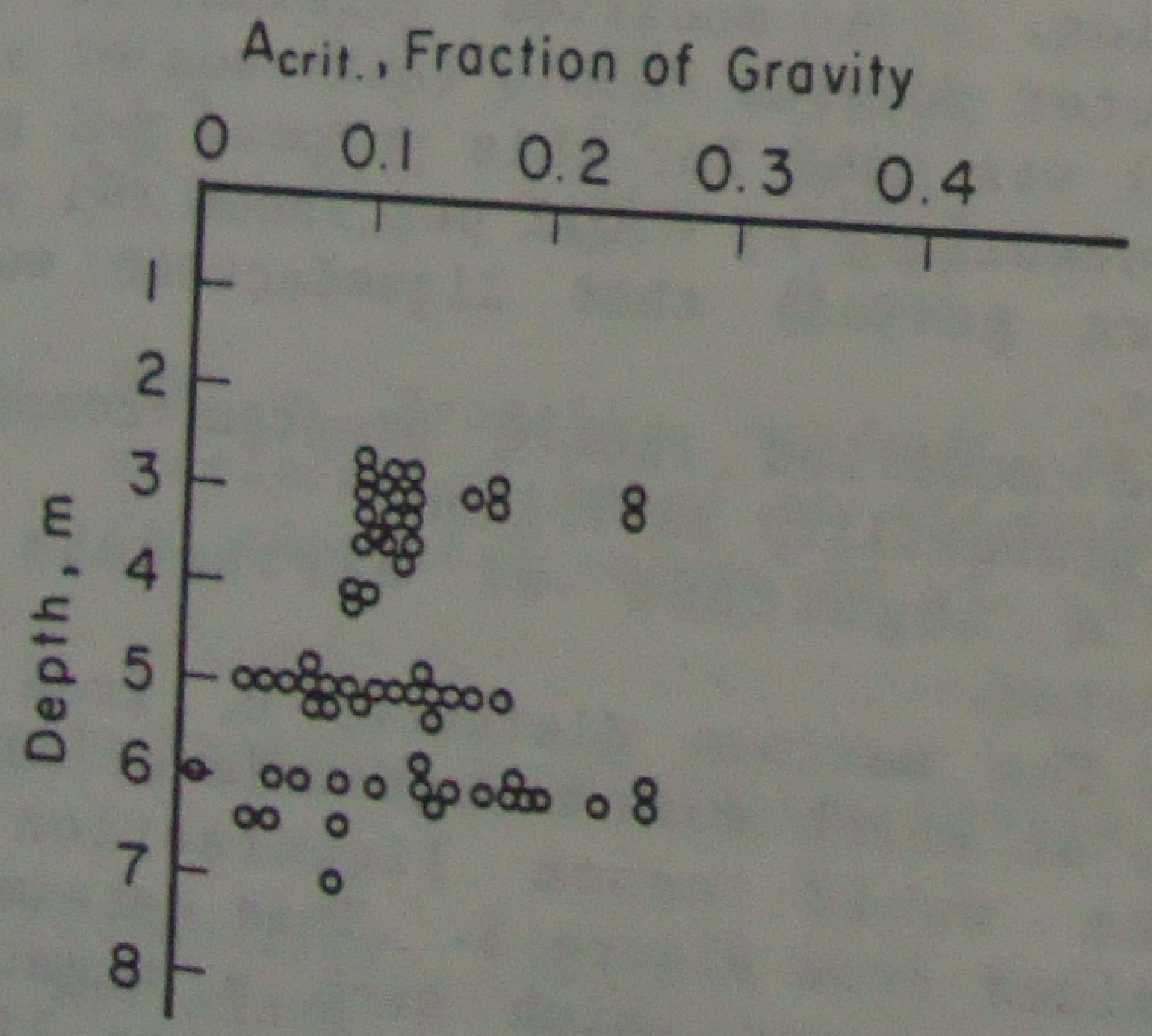

Fig. 8. Critical Surface Acceleration to Trigger Liquefation for M=7.5 Earthquake.

These A_{crit} values are appropriate for a magnitude 7.5 earthquake. For smaller magnitude earthquakes corresponding to a lesser number of cycles, the liquefaction resistance and critical acceleration would be higher as was shown on Table 1. Fitting a straight line through the data in Table 1 results in

$$A_{crit(surface)} = (0.262-0.0216\ M)f \qquad (2)$$

where the factor f has been inserted to account for the range in liquefaction resistance of the soil. A value of f = 1 corresponds with the average critical acceleration value of 0.1 g while f equal to 0.5 and 2 correspond to the lower and upper assessments of critical acceleration.

While it is the accelerations at the ground surface that give rise to liquefaction, it is the accelerations on rock that are known from the seismic risk analysis. Based upon an amplification factor of 1.3 the critical acceleration on rock is given by:

$$A_{crit(rock)} = (0.2-0.0166\ M)f \qquad (3)$$

The probabilities of triggering liquefaction for the various $A_{crit(rock)}$ assumptions were computed using the PROLIQ program, and are shown in Table 4 together with the values shown for the Method 1 study discussed earlier.

Table 4. Probability of Liquefaction, P per annum

Method	Probability Per Annum	Return Period
1	0.011	91
2(f = 0.5)	0.0215	46
2(f = 1.0)	0.0076	131
2(f = 2.0)	0.0022	454

The probabilities for Method 2 are based on Earth Physics Branch seismogenic zones and attenuation relation.

The probability of liquefaction determined by Method 2 can also be expressed in terms of the liquefaction resistance of the soil and is shown in Fig. 9. The probability calculated from Method 1, when plotted on this diagram, indicates a resistance ratio of about 0.08, which is in reasonable agreement with a very loose sand having an $(N_1)_{60}$ = 4.

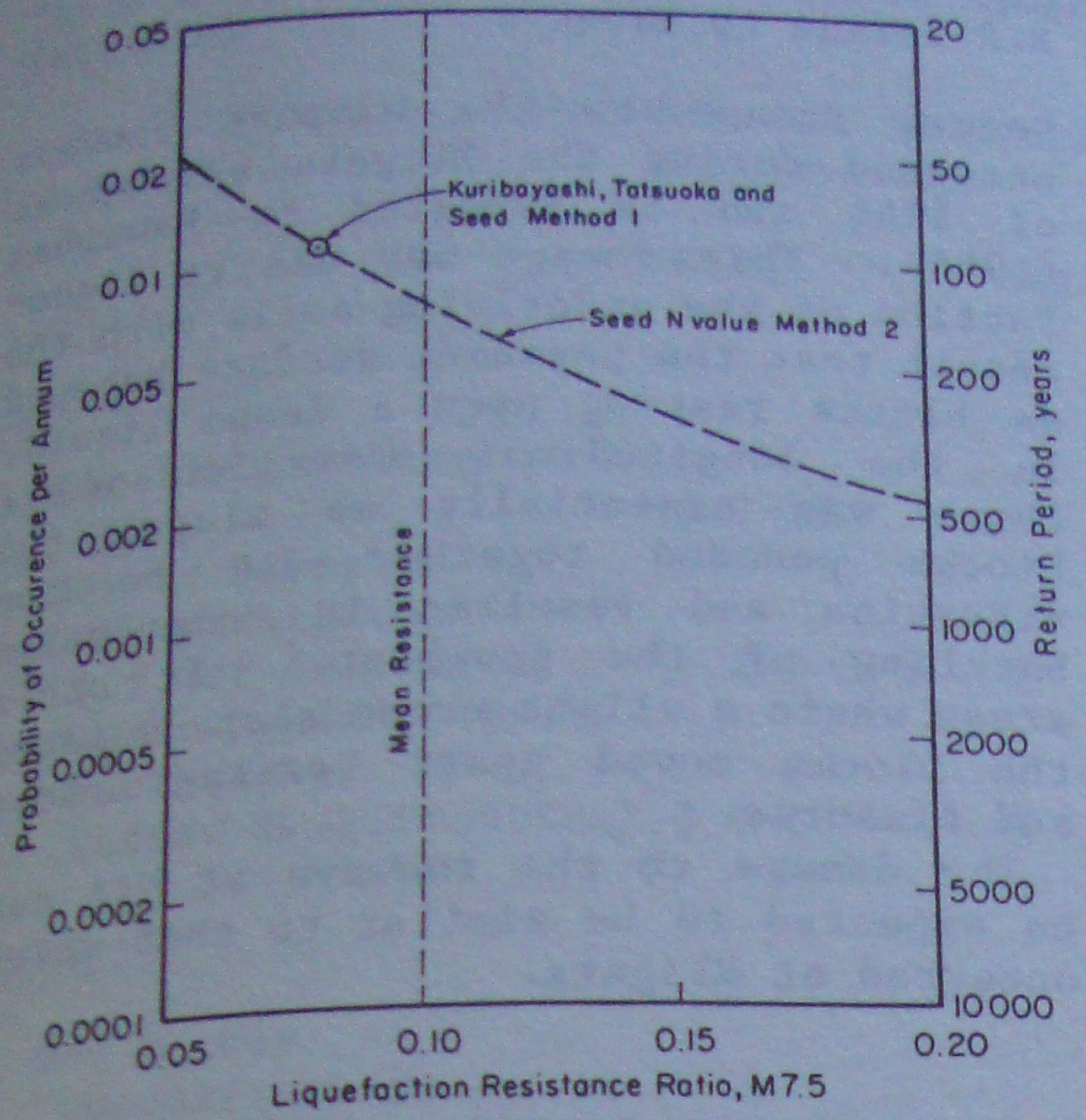

Fig. 9. Probability of Liquefaction vs. Liquefation Resistance Ratio.

7. DISCUSSION OF LIQUEFACTION PROBABILITY

The results indicate a range in probability per annum of liquefaction from about 0.02 to .002 depending upon the liquefaction resistance of the soil. These probabilities correspond with A_{50} to A_{500} year events. Since liquefaction has not been observed to occur at the site, perhaps the assumed low values of liquefaction resistance can be discounted from an examination of performance during past earthquakes.

Examination of the earthquake history in the past 120 years indicates that two of the major earthquake events shown in Table 2 would plot just below the line in Fig. 2 indicating the possibility of liquefaction at the airport for these events, for materials having $(N_1)_{60} < 4$. Since liquefaction was not recorded, the resistance of the soil must be at least equal to a material having $(N_1)_{60} > 4$ or a liquefaction resistance ratio > 0.08. Thus a lower bound resistance ratio of 0.08 rather than 0.05 should be used for Method 2 and leads to a lower bound probability of liquefaction 0.011.

The extent of liquefaction will depend primarily on the resistance of the soil and the shaking level. If all of the soils had the same resistance ratio as the average value of 0.10, then liquefaction would be predicted to occur everywhere for the 130 year earthquake (A_{130}). However, because the soil strength varies somewhat along the length of the dyke, the length of dyke to liquefy will vary with the earthquake level. The estimated length of liquefied dyke versus the probability per annum or earthquake return period is shown in Figure 10. It may be seen that liquefaction is predicted to commence for the A_{90} event, that 50% of the dykes would liquefy for the A_{130} event and that all of the dykes would liquefy for the A_{1000} year event. This estimate is based on the assumption that the length of liquefied dyke is proportional to the number of penetration values indicating liquefaction. Since the penetration values were obtained from 22 boreholes distributed along the length of the dyke this assumption is not unreasonable.

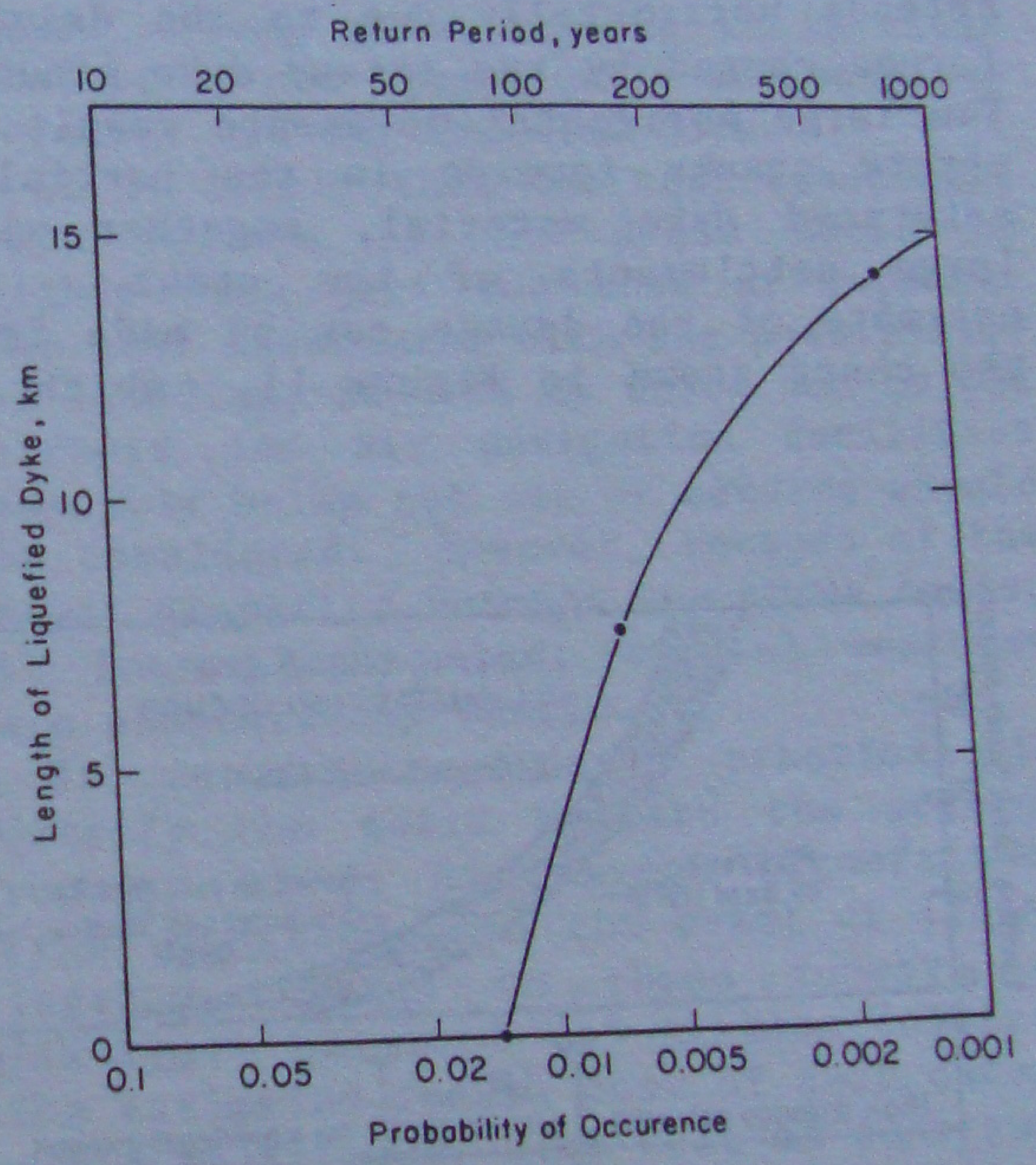

Fig. 10. Length of Liquefied Dyke vs. Probability of Occurrence.

8. EARTHQUAKE DAMAGE DUE TO LIQUEFACTION

If the soils beneath the dykes and runways liquefy, severe damage to these structures can be expected to occur. Such damage is essentially caused by movements induced by loss in stiffness and/or strength in the liquefied zones. A quantitative evaluation of such movements is very difficult, and at best can be considered a rough estimate. However, the pattern of movements derived from such calculations is helpful. There is much field evidence of liquefaction induced damage in past earthquakes and such evidence will be the principal source of damage estimation.

8.1 Dykes

Very severe damage to dykes occurred in Japan during a number of past earthquakes and this is described by Yokomura (1966), Kawasuma (1964), Suzuki, (1971) and Moriya and Kawagughi (1970). The damage involved very severe longitudinal cracks along the crest. Crest settlements of 1/2 to 2m were observed. The soils underlying these dykes were loose alluvial or deltaic deposits such as are present at the airport. The mechanism involved is basically one of liquefaction of the underlying soils causing a loss in strength, with the result that the slope spreads horizontally due to the driving forces caused by the raised dyke ground. The large horizontal movements result in severe cracks forming in the partially saturated dyke material, together with large settlements of the crest. An estimate of the damage can be made from the chart shown in Figure 11, which is

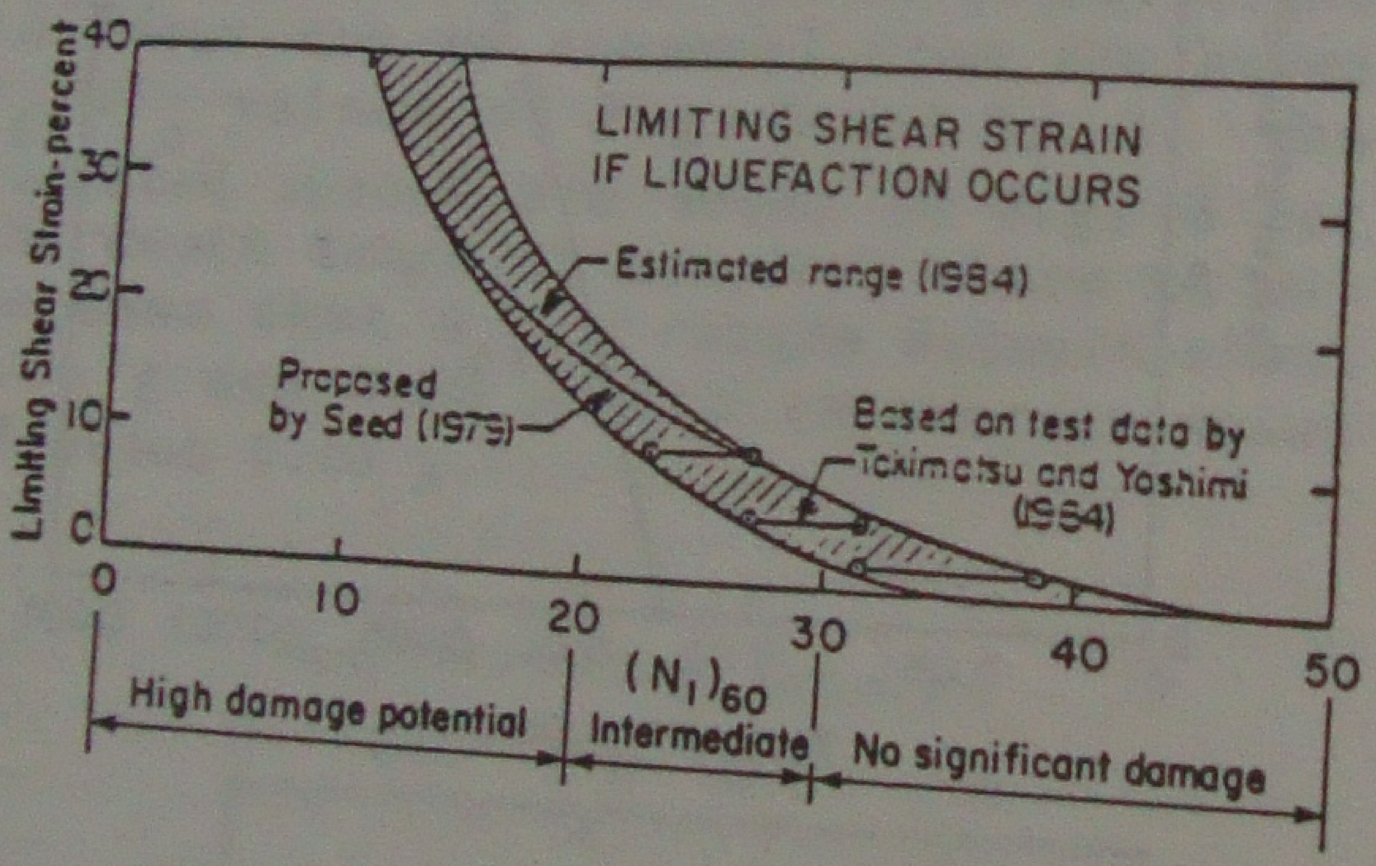

Fig. 11. Limiting Shear Strains vs. $(N_1)_{60}$ for Clean Sands.

based upon field experience. For $(N_1)_{60}$ values less than 10 such as are present at the airport, this chart indicates that the soils have high damage potential.

In summary, liquefaction will result in severe cracking of the dykes. Settlements of the crest can be expected to range between 0.5 and 2m.

8.2 Damage to Runways

Severe damage to the airport runways occurred during the Niigata earthquake of 1964 and is reported by Kawasuma (1964). This damage was due to liquefaction of the underlying soils with the result that the pavement surface behaved as blocks resting upon a dense liquid. In the longitudinal direction where there was essentially no slope, the blocks pounded together due to the vibration and resulted in compression buckling of the pavement. In other areas where a slight cross slope existed the blocks moved apart leaving cracks and fissures.

The damage to the runways at VIA can be expected to be similar to that which occurred at Niigata.

9. OPTIONS FOR COPING WITH LIQUEFACTION PRONE SOILS

There are basically two options for coping with liquefiable soils:

1) Stabilize the soils so as to essentially prevent the occurrence of liquefaction.

2) Accept the risk of liquefaction and repair the damage after it occurs.

The loose material in and beneath the dykes can be stabilized by densification so as to prevent liquefaction. The loose material beneath the existing runways and aprons cannot be densified without destroying the paved surfaces. Thus if a liquefaction proof runway is desired it would essentially require construction of a new runway.

The practical options are therefore:

1) Densify the loose deposits in and under the dyke.

2) Construct a new runway on previously densified ground.

3) Do nothing now and repair the dykes and runways as necessary if they are damaged during an earthquake.

Each of these options is considered in the sections which follow and approximate cost estimates are provided in order to place them in economic perspective.

10. SUMMARY OF COSTS

A summary of the costs of densification and repair of the dykes and runways is shown in Table 5. Densification of the

Table 5. Cost of Optional Programs (Rough Estimate)

Item	Cost in Millions of Dollars Range	Avg.
Dykes		
Densify Existing Dykes	\$ 9-18	14
Repair Dykes after Earthquake (if not densified)	\$ 2.5-7.5	5
Repair Dykes after Earthquake (after densification)	\$ 0-1	0
Runways		
New Runway (Densification cost only)	\$ 5-10	7.5
New Runway (Complete cost)	\$ 55-60	60
Repair current runways	\$ 5-10	7.5
Loss of revenue if airport unuseable for two months	Not included	

soils underlying the dykes would cost \$9-18 million depending on the densification method used. If the Vibro probe method is adequate, the cost will be about \$9 million. If stone columns are necessary, the cost may be \$18 million. Even if such densification is undertaken, some earthquake damage is likely to occur and is estimated to cost about \$1 million to repair.

If no densification is undertaken and liquefaction occurs, the cost of repairing the dykes is estimated to be about \$5 million.

Densification of the foundation soils beneath a new runway are estimated to be between \$5 and \$10 million depending on the method of densification, Dynamic Consolidation or Vibro Probe. The total cost of constructing a new runway pavement including densification is likely to be in the range of \$55-\$60 million.

The cost of repairing the current runways in the event of liquefaction is estimated to be \$5 to \$10 million. The time to complete such repairs could be in excess of two months.

The average costs will be used for economic evaluation purposes.

11. ECONOMIC EVALUATION OF ALTERNATIVES

If no remedial work is done on the dykes it is estimated that the probability of having an earthquake large enough to cause severe liquefaction is about .0076 per annum. The estimated costs of repairing the dykes in the event of such liquefaction is \$5 million. The estimated cost per annum of the earthquake damage is the product of the total cost times the probability per annum of occurrence, namely \$38,000.

If remedial work is carried out, and if it is assumed that it is sufficient to prevent liquefaction for any level of earthquake, then the cost per year is the cost of the remedial work amortized over the life of the dykes, presumably forever. For remedial work costing \$14 million, and for a real interest rate of say 3% (discounting inflation), this would represent a cost of \$420,000 per year. Given these numbers it is clear that it is not economically advantageous to carry out any remedial work.

It is likely that the cost associated with failure of the dykes would be higher than just the cost of repairs. The cost associated with the possibility of flooding, as well as those due to the airport and air navigation facilities possibly being put out of service should be considered. However, because of the great disparity between the above costs, it is unlikely that remedial measures are economically viable.

It is not considered practical to densify the soils beneath the present runways so as to prevent liquefaction from occurring. In the event of liquefaction, damage to these runways is likely to be about 7.5 million dollars. The estimated annual cost of such damage would be \$57,000. The cost of constructing a new runway is about \$60 million. The annual cost at a 3% interest rate would be \$1.8 million. Thus on a cost basis alone it is not economically favourable to construct a new runway. However, the time to complete the repairs to the runways in the event of liquefaction is likely to exceed two months, and this may not be acceptable.

12. CONCLUSIONS

Significant liquefaction of the soils underlying both the dykes and the runways has a probability of occurrence of about .0076 per annum which corresponds with a 130 year return period event. It is likely that while damage to the dykes would be severe and would involve costs in the range of $5 million, severe flooding and loss of life is unlikely to occur provided remedial measures are instigated immediately. Such damage could be essentially prevented from occurring by densification of the soils within and under the dykes. The cost of such densification would be in the range $9 to $18 million and the analyses indicate that such an outlay is not economically warranted.

Liquefaction beneath the runways would cause severe damage to the overlying pavements. The cost of repairing such damage would be $5 to $10 million and the time taken for such repairs may exceed two months. It is not considered economically feasible to densify the soil beneath the existing runways because this would involve the destruction and replacement of the existing pavement with a cost approaching that of constructing a new runway.

If aircraft activity and other factors justify the construction of a new runway, then it would be appropriate to densify the subsoils to make the runway resistant to liquefaction.

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