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Seismic response, liquefaction, and resulting earthquake induced displacements in the Fraser Delta

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ABSTRACT

Seismic response analyses carried out in the Fraser Delta indicate higher amplifications of accelerations than have previously been considered. The analyses are based on recent information on soil properties and layer thicknesses and the results are in accord with amplifications recorded in Mexico City, 1985, and San Francisco, 1989. The computed cyclic stresses for the NBCC design earthquake would trigger widespread liquefaction in the delta and result in large vertical and horizontal displacements that would likely cause severe damage to lifeline facilities.

INTRODUCTION

The coastal area of mainland Southern British Columbia lies within a highly active seismic region. Seven earthquakes in the magnitude range M5-M7 have occurred in the recorded past 100 years. Geological evidence suggests that very large subduction earthquakes of the order M9 have occurred in the past. The recurrence period of these earthquakes is thought to be about every 700 years on average, with the most recent such event having occurred about 300 years ago.

The Fraser Delta lies within this region and is particularly prone to damage in the event of a major earthquake. This is because it is underlain by deep deposits of relatively loose or soft soils. The presence of such soils can: amplify the intensity of shaking; lengthen the predominant period of the motion, and cause strength loss or liquefaction of saturated sandy soils.

Experience at Mexico City during the 1985 earthquake showed that a major cause of damage was the very high amplification of acceleration that occurred as the motion propagated upward through the soft clay lakebed deposits. A similar amplification occurred in the San Francisco bay muds and caused much of the damage in San Francisco and Oakland during the 1989 Loma Prieta earthquake (Idriss, 1990). In addition, liquefaction of loose sand fill placed on top of the Bay mud greatly added to the damage where it was present.

In much of the Fraser Delta, natural deposits of loose to medium dense sands overlie deep silt and clayey deposits, so that the combined effects of both amplification and liquefaction are a possibility in the event of a major earthquake, and should be considered in design. The major design considerations from the foundation point of view are the depth of liquefaction and the resulting displacements. These aspects were discussed in

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detail by Byrne and Anderson (1987). However, the larger than expected amplifications in San Francisco, and recent more detailed examinations of liquefaction induced displacements by Hamada et al. (1987), and Youd & Bartlett (1988), warrant an additional study of this region.

In addition, more data are now available on sediment thickness as well as moduli and damping properties. Because the depth of liquefaction is very important in estimating earthquake induced displacements, effective as well as total stresss analyses will be carried out. Previous analyses carried out by Byrne (1978) indicated that significantly smaller depths of liquefaction were predicted from effective stress in comparison to total stress analyses.

SOIL CONDITIONS AND PROPERTIES

The Fraser Delta has a plan area of about 350 Km^2 as show in Fig. 1. The area has a mix of commercial industrial and residential development with a population of about 1/4 million people. The geology of the Delta is discussed in some detail by Wallis (1979). Basically the area is underlain by: (1) a surficial deposit comprised of a thin discontinuous veneer of clays, silts and peats up to 8 m in thickness, underlain by; (2) a sand and silty sand stratum generally 20 to 45 m in thickness, underlain by; (3) a silt-clay stratum in the North and more granular material in the South with a thickness generally in the range 100 to 300 m (Britton, 1990), underlain by; (4) a glacial till stratum with a thickness in the range 90 to 600 m (Britton, 1990), underlain by; (5) bedrock.

The water table is generally within a metre of the surface. A typical section in central Richmond is shown in Fig. 2.

Dynamic and liquefaction analyses require the following soils information: (1) G_{max} vs. Depth; (2) modulus reduction with level of shear strain; (3) damping as a function of shear strain; and (4) liquefaction resistance parameters

The maximum shear modulus versus depth relationship used in the analysis is shown in Fig. 2 and is based on Byrne and Anderson (1987) together with more recent in situ shear wave measurements reported by Finn et al. (1988) and Hunter (1990). Modulus reduction and damping values used were based on laboratory tests on similar materials as reported by Seed and Idriss (1970), Seed et al. (1986), Sun et al. (1988), and Idriss (1990). Recent test data on silt and clay material from the Fraser Delta reported by Zavoral (1990) were also examined and found to lie within the range of values considered. Liquefaction resistance of the sands is based on in situ penetration resistance values, both standard penetration and cone values (SPT and CPT). Average and lower bound values of the normalized standard penetration of the sands, $(N_1)_{60}$, are shown in Fig. 3 and are based on Byrne and Anderson (1987). The pore pressure rise parameters for the effective stress analysis are based on $(N_1)_{60}$ values as described by Byrne (1991).

ANALYSIS PROCEDURE

Total stress dynamic analyses were carried out using the computer code SHAKE. Effective stress dynamic analyses were carried out using the computer code 1D-LIQ (Byrne and Yan, 1990) based on the procedure outlined by Finn, Byrne and Martin (1976). The soil section analyzed and the soil properties used were as outlined in the previous section. Two different time histories of base acceleration were used. These were the CALTEC and GRIFFITH PARK records generated during the 1971 San Fernando earthquake. They were scaled to a peak base acceleration of 0.2 g, which corresponds approximately with the NBCC 1990 code value of 0.21 g for a probability of 10% in 50 years in Vancouver. They are both rock records, and from previous studies, have been found to be generally more severe than other rock records scaled to the same peak acceleration value.

The predicted amplifications of acceleration from base to surface are shown in Fig. 4. Also shown on this figure are results of recent analysis carried out by B. C. Hydro in connection with seismic assessment of transmission towers in the eastern Fraser Delta. The layer thicknesses varied widely in the Hydro analyses and the depths to firm ground were generally less than considered in Fig. 2. Also, significant peat layers were present in some of the Hydro profiles. The high surface accelerations computed by HYDRO were associated with a hard layer at a depth of 35 m. The low surface accelerations were associated with a weak surface layer.

The results of both analyses indicate that base accelerations of 0.2 g could generally be amplified to 0.3 g in the Fraser Delta. This is consistent with observations in San Francisco and Mexico City, and in agreement with the median recommendation proposed by Idriss (1990) also shown in Fig. 4.

The thickness of the till could range between 90 and 600 m. Additional analyses were carried out and indicated that the predicted surface accelerations were not sensitive to the till thickness. This is to be expected as the till under these high confining stresses would act much like a rock.

The thickness of the silt-clay layer could range between 100 and 300 m. Analyses carried out with a silt-clay layer of 270 m rather than 120 m indicated essentially no change in the predicted peak surface acceleration.

Analyses were also carried out to represent the condition of a sandy rather than a silt-clay layer in the depth range 30 to 150 m. This condition occurs in the southern portion of the Delta and resulted in essentially no amplification.

The predicted peak surface acceleration is approximately 0.30 g and represents an amplification factor of 1.5 for a base rock motion of 0.2 g. In a previous study Byrne and Anderson (1987) computed an amplification of 1.05. The amplification is essentially independent of the thicknesses of the till and silt-clay layers for the thicknesses likely to be encountered. Lower amplifications are predicted in the southern portion of the Delta where sands rather than silt-clays are present at depth.

The accelerations cause cyclic stresses in the ground, and it is these that may cause the soil to liquefy. Because the computed accelerations are higher than previously calculated by Byrne and Anderson (1987), the cyclic stresses will also be higher. The computed equivalent cyclic stress ratios (CSR) as a function of depth for the top 30 m are shown in Fig. 5. CSR is defined as 0.65 times the ratio of the computed peak shear stress to the effective overburden pressure. The values shown are for the soil conditions in Fig. 2 and vary significantly depending on the earthquake record and the choice of modulus reduction and damping values used. The higher CSR values correspond with surface accelerations that are in agreement with amplifications recorded in San Francisco. For this reason CSR's indicated by the dashed line shown on Fig. 5 are considered appropriate for design.

Liquefaction is triggered when the cyclic stresses from the design earthquake exceed the cyclic resistance of the soil (CSR > CRR). The CRR values were obtained from the $(N_1)_{60}$ values of Fig. 3 and the Seed et al. (1984) liquefaction chart, and are shown in Fig. 6. A correction value of 1.08 was included to modify the CRR for an M7 event rather than an M7.5 event. An M7 event is thought to be appropriate for a probability of 10% in 50 years. The design CSR from Fig. 5 is also shown on Fig. 6 and the results indicate that for the lower bound $(N_1)_{60}$ condition, liquefaction to the full 30 m depth is predicted to occur. For the mean $(N_1)_{60}$, liquefaction to a depth of 16 m is predicted.

Effective stress dynamic analysis were also carried out using the computer code 1D-LIQ. The pore pressuremeter parameters for the model were derived from $(N_1)_{60}$ values such that under constant amplitude cyclic load conditions, liquefaction would occur in 15 cycles in agreement with the Seed et al. (1984) liquefaction chart . The detailed procedure for doing this is outlined by Byrne (1991). The results are shown in Fig. 7a in terms of the excess pore pressure rise as a function of depth. It may be seen that in the depth range 3 to 23 m the pore pressure rises to equal the initial effective stress σ'_{vo} indicating liquefaction in this region. Below 23 m, while excess pore pressures are high, liquefaction is not predicted to occur.

The total stress 1D-LIQ analysis for the same lower bound $(N_1)_{60}$ condition is shown in Fig. 7b for comparison. It may be seen that the CSR exceeds the CRR for the complete depth of sand, indicating liquefaction to the full 30 m depth of the layer. Thus the total stress analysis is seen to give a conservative estimate of the depth of liquefaction, perhaps too conservative.

The reason for the reduced depth of liquefaction in the case of the effective stress analysis is that as pore pressure rise and liquefaction occurs in the layer most susceptible to liquefaction, this layer loses its ability to transfer shear load, and subsequent dynamic stresses are significantly reduced in all layers.

LIQUEFACTION INDUCED DISPLACEMENTS

Triggering of liquefaction may result in large horizontal and vertical displacements, and these can be estimated from the effective stress analysis carried out here. The computed maximum of vertical displacement at the surface was 0.22 m for the lower bound $(N_1)_{60}$ conditions. Liquefaction induced vertical displacements can also be estimated from empirical equations based on field experience during past earthquakes. Tokimatsu and Seed (1987) have presented a chart for predicting liquefaction induced strains based on in situ $(N_1)_{60}$ values and field experience. The predictions based on their chart for the various conditions are shown in Table I.

Table I

Liquefaction Induced Displacements

Type of Analysis	In Situ $(N_1)_{60}$ State	Thickness of Liquefied Layer, m	Vertical Displ., m	Horizontal Displacement, m	
				Byrne	Hamada
Effective stress	Lower bound	20	0.2	2.5	
Total stress	Lower bound	20	0.5*	2.7**	3.3
Total stress	Lower bound	27	0.7*	3.4**	3.9
Total stress	Mean	13	0.2*	0.6**	2.7

*Based on Tokimatsu and Seed, **Based on Byrne (1991a).

It may be seen that for the same $(N_1)_{60}$ conditions, and the same depth of liquefied layer, the vertical displacement based on total stress analysis and field experience is more than twice that computed from the effective stress analysis.

The computed horizontal displacements of the crust from the effective stress analyses are shown in Fig. 8. It may be seen that prior to triggering of liquefaction the displacements are small, but upon triggering, large displacements occur in one direction of the order of 2.5 m.

Byrne (1991a) presented a simple extension of the Newmark (1965) procedure for predicting liquefaction induced lateral displacements taking into account the post liquefaction stress-strain and strength behaviour and its dependency on $(N_1)_{60}$ value. He showed his procedure to be in good agreement with field and laboratory observations. The estimated displacements from this procedure are shown in Table I (Col. 5) and good agreement is obtained with the effective stress analysis (2.5 vs 2.7 m).

Lateral liquefaction induced displacements can also be estimated from empirical equations based on field experience. One such equation that is commonly used was proposed by Hamada et al. (1987) as follows:

$$D = 0.75\,(H)^{1/2}\,(\theta)^{1/3} \tag{1}$$

in which D = the liquefaction induced displacement, m; H = the thickness of the liquefied layer, m; and θ = the slope of the ground surface in %.

The predicted liquefaction induced displacements based on this equation (θ = 1%) are also shown on Table I (Col. 6). They are significantly greater than the prediction from the effective stress analysis However, most of the Hamada data were associated with very loose sands, and this is discussed in more detail by Byrne (1991a).

It should be noted that all of the empirical methods of predicting liquefaction induced displacements are strongly dependent on the thickness of the liquefied layer. The total stress analysis procedure which predicts a greater thickness of liquefied layer will also predict greater liquefaction induced displacements as may be seen from Table I. Thus the commonly used total stress procedure may be unduly conservative when estimating both the extent of liquefaction and the magnitude of liquefaction induced displacements.

Extensive zones of liquefaction are predicted to occur in the Fraser Delta in the event of a major earthquake and are likely to result in severe damage. Experience at Niigata, Japan 1964, and San Francisco 1989 indicate that damage to buried services such as water, gas, sewer, electricity and telephone would be very severe due to the large differential movements of the surface crust. Damage to bridge and overpass structures, and the George Massey Tunnel could also be severe. The dyking system will likely suffer severe cracking, and flooding is a possibility. Light wood structures supported on the crust are likely to suffer light to moderate damage. However, older taller buildings supported on piles could suffer very severe damage due to loss of pile support.

SUMMARY

Dynamic analysis carried out in the Fraser Delta deposits indicate that in the event of a major earthquake, bedrock motions of 0.2 g may be amplified to 0.3 g at the surface. This is in accord with amplifications recorded in San Francisco in 1989 and Mexico City in 1985. The surface accelerations are not greatly influenced by the thickness of the till layer nor the thickness of the silt-clay layer. However, significantly lower surface accelerations are predicted if the silt-clay layer is not present as appears to be the case in the southern Delta.

These amplified accelerations would trigger liquefaction over most areas of the Delta for earthquakes with M7 or greater. The density and liquefaction resistance varies considerably in the various locations throughout the Delta and for the Lower bound condition liquefaction to the full depth of the sand layer could occur. This could result in vertical displacements of about 0.7 m and horizontal displacements of up to 3 or 4 m. Such movements could result in very severe damage to structures and lifeline facilities. Effective stress analyses indicate significantly smaller depths of liquefaction and lower displacements. Site specific evaluations are needed for major structures.

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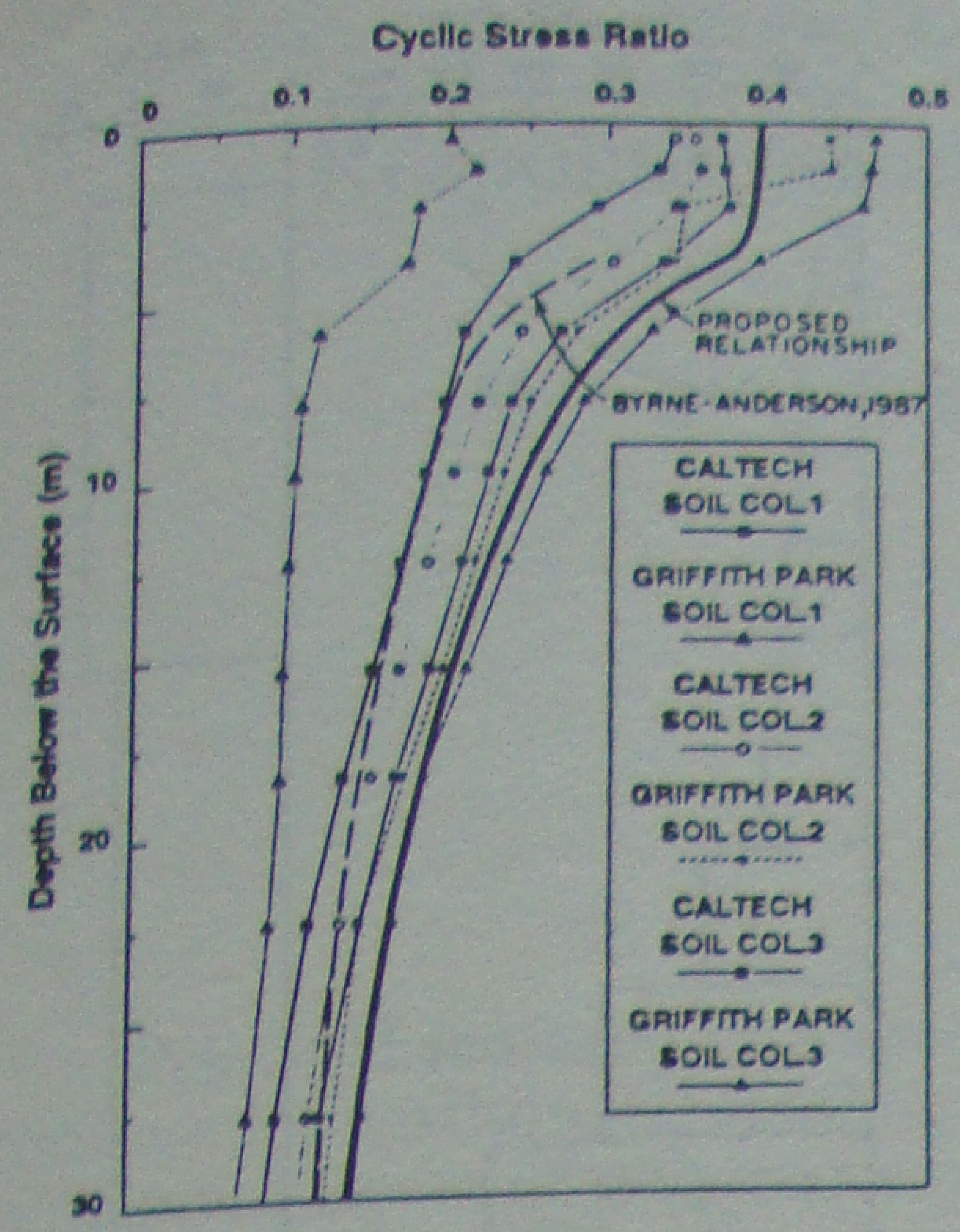
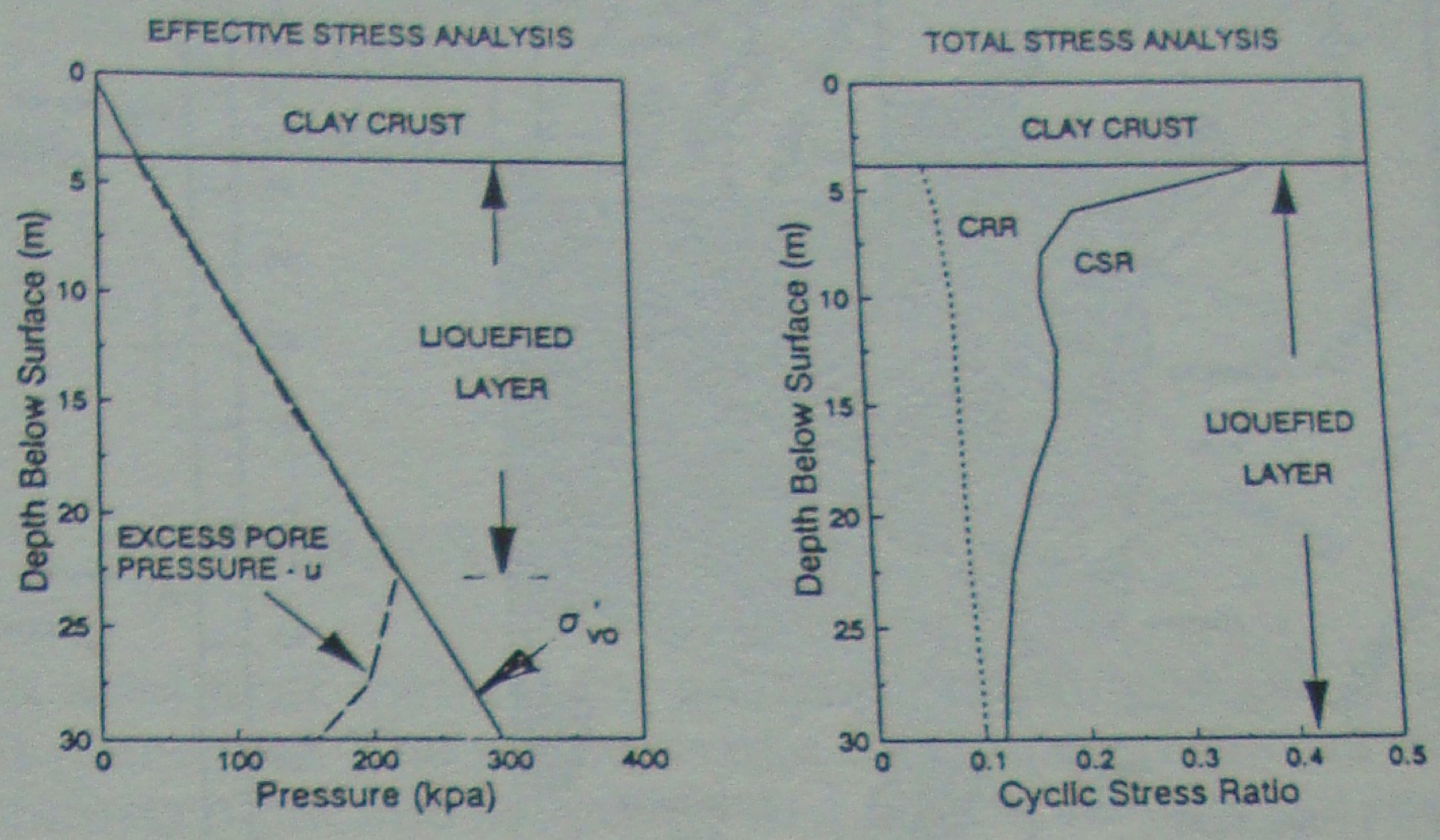
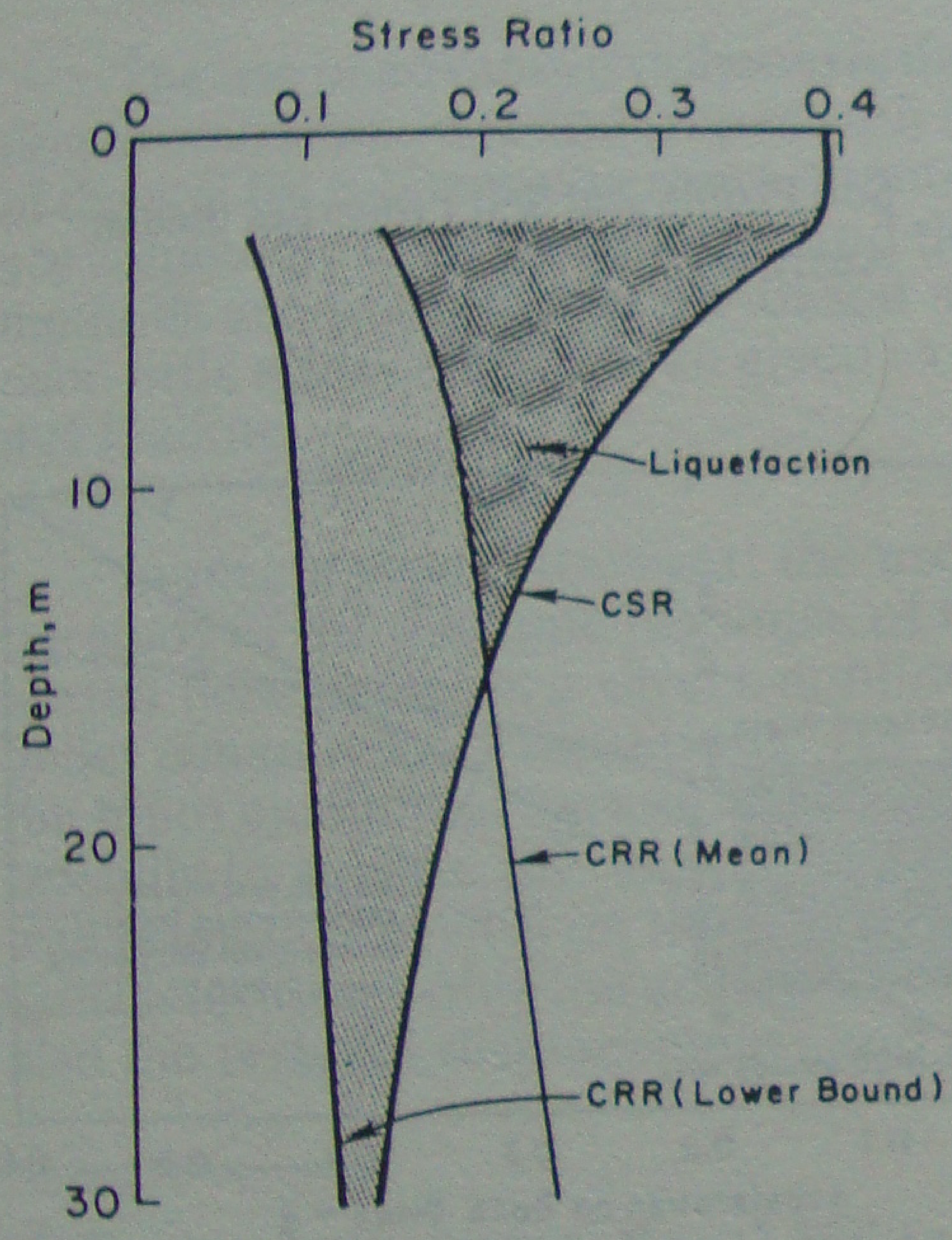


Fig. 5 Computed Cyclic Stress Ratios for the Fraser Delta



a b

Fig. 7 Liquefaction Assessment



CSR = Cyclic Stress Ratio caused by Design Earthquake

CRR = Cyclic Resistance Ratio

CSR > CRR => LIQUEFACTION

Fig. 6 Liquefaction Assessment for Richmond, 0.2 g and M7

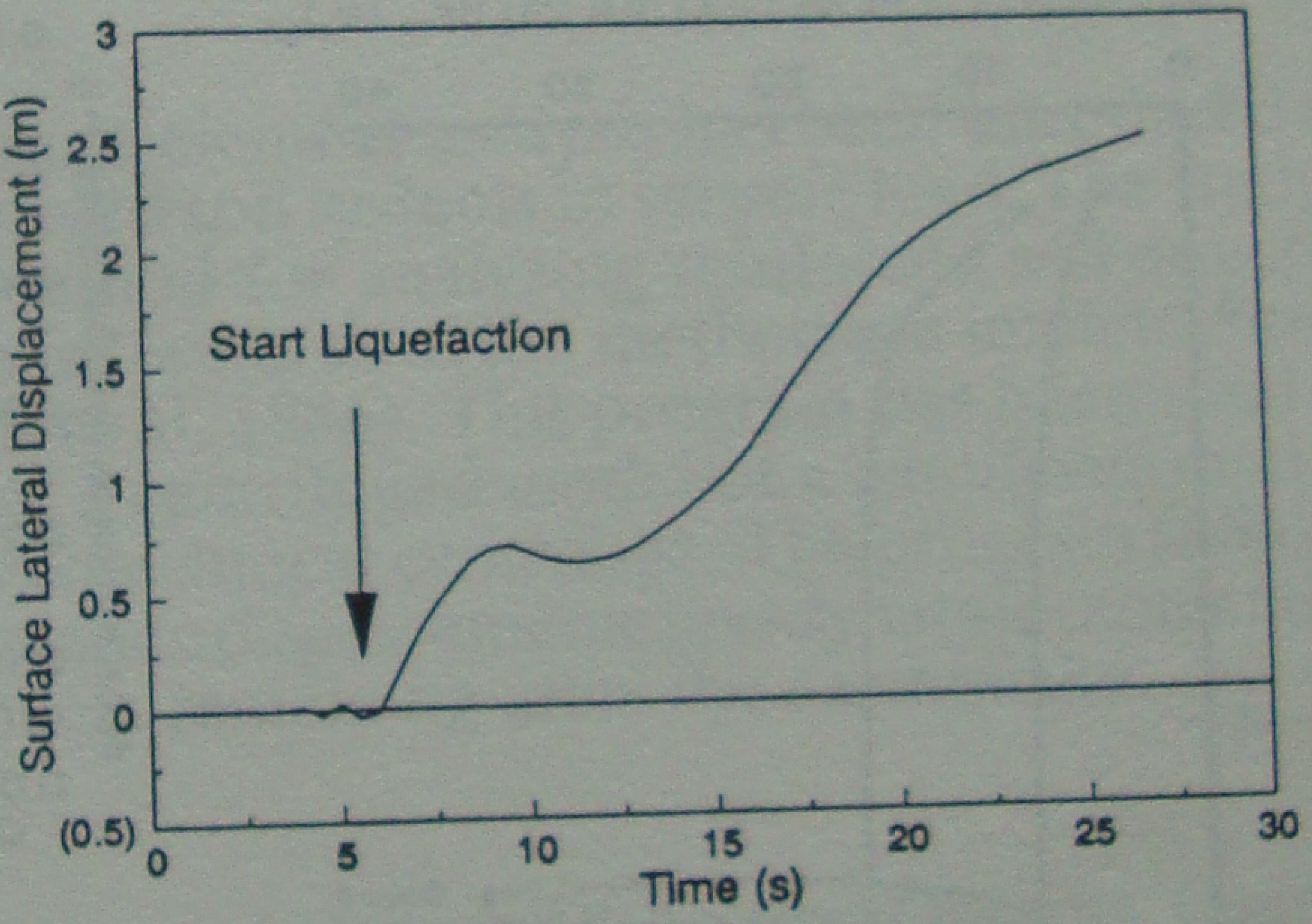


Fig. 8 Predicted Surface Displacements for Lower Bound $(N_1)_{60}$ Values, 0.2 g and M7

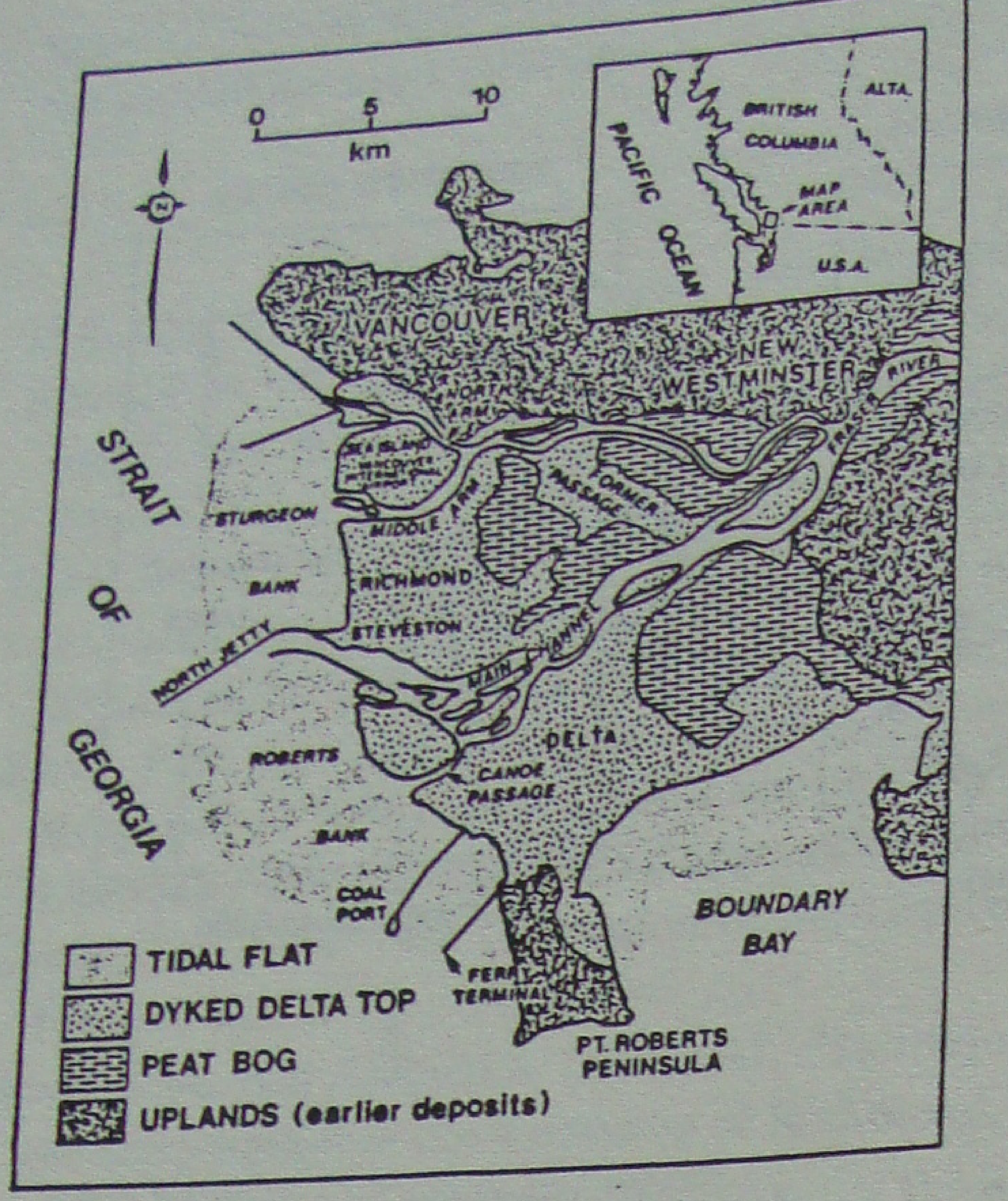


Fig. 1 Fraser Delta (adapted from Luternauer and Finn, 1983)

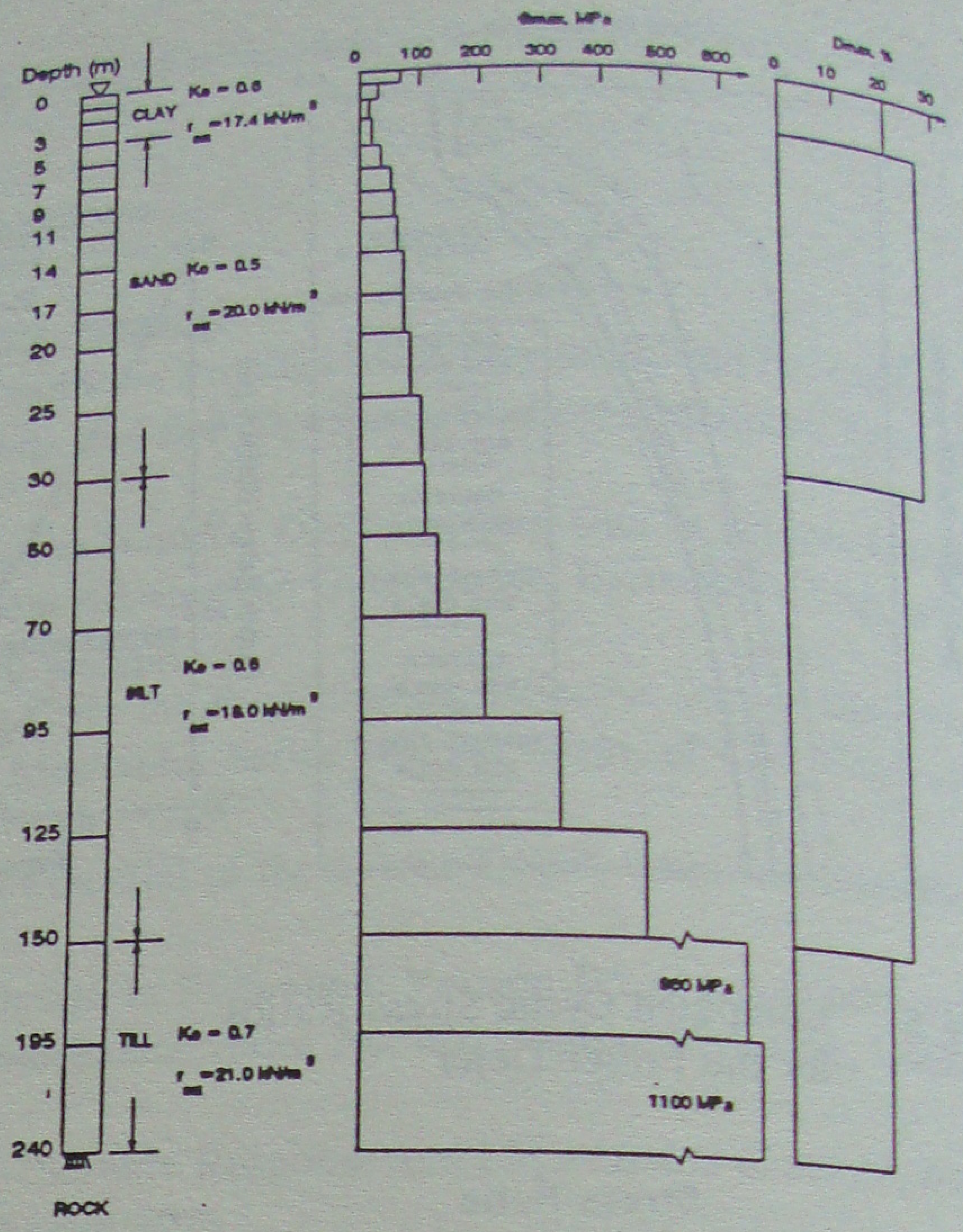


Fig. 2 Soil Profile and Soil Properties used in Analysis

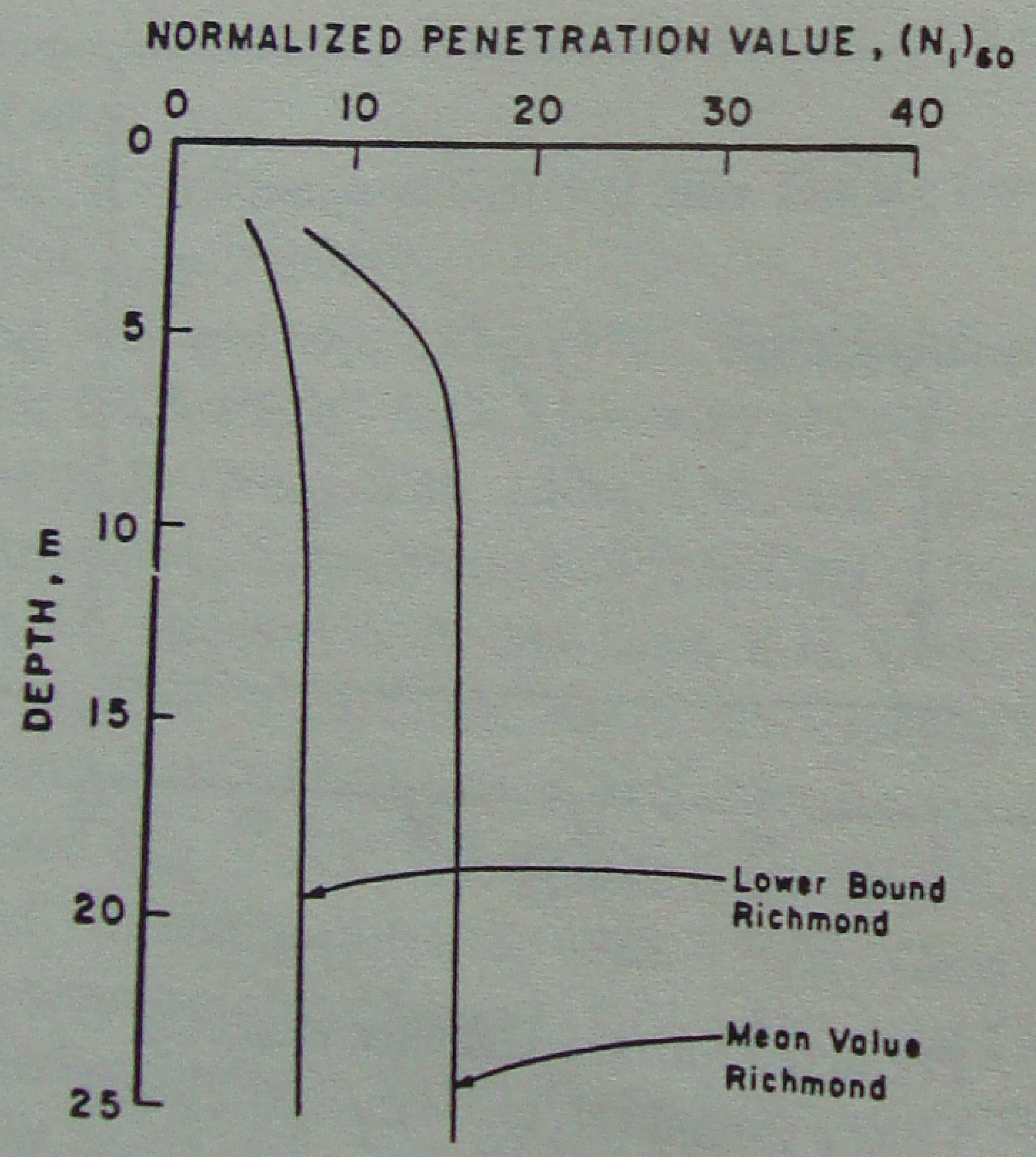


Fig. 3 Normalized Penetration Values in Richmond

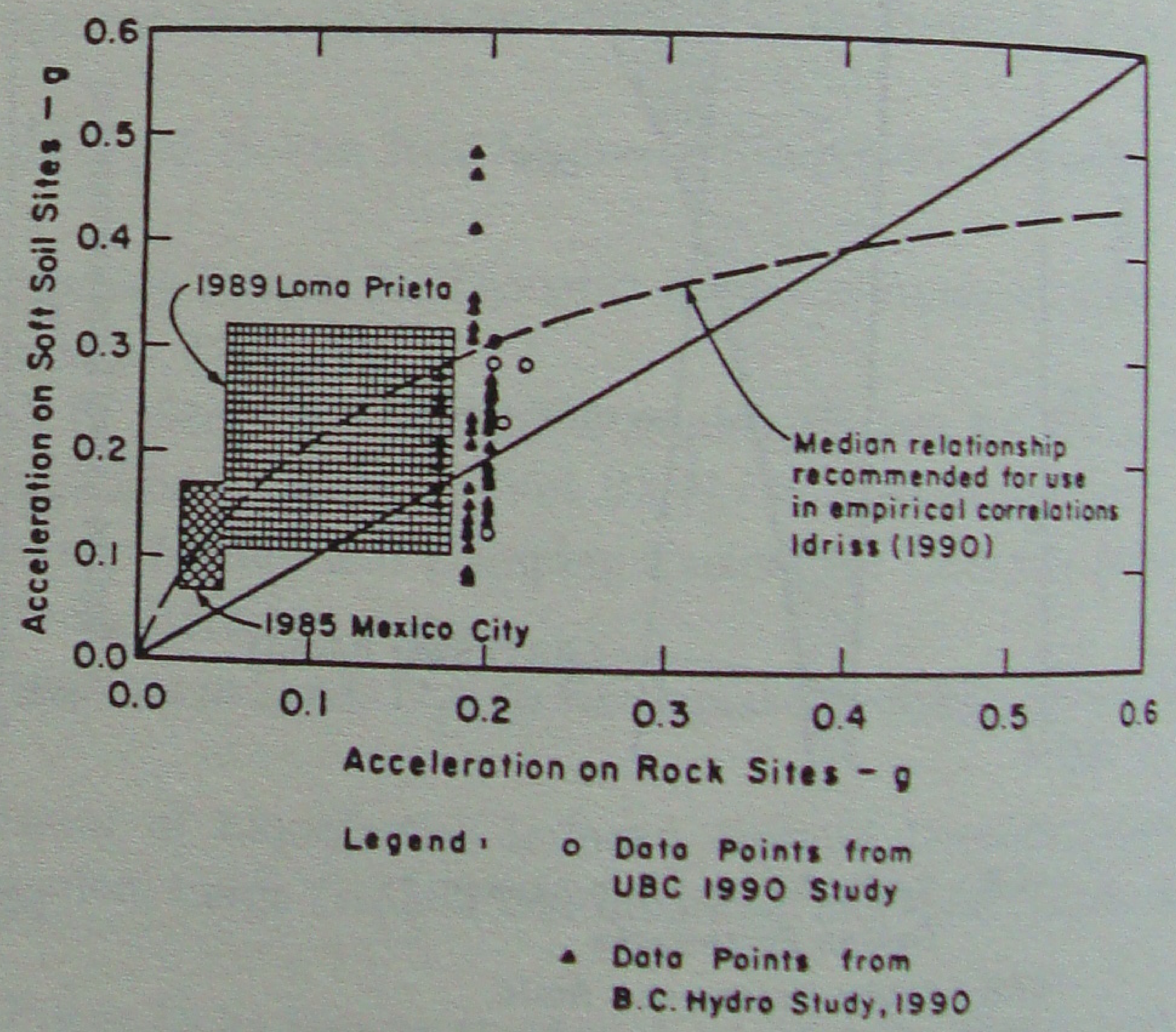


Fig. 4 Amplification of Acceleration