PREDICTIONS OF SEISMIC DISPLACEMENTS BY FINITE ELEMENT ANALYSIS OF A SENSITIVE CLAY SITE NEAR VANCOUVER, BRITISH COLUMBIA

M. Zergoun [1] and J.A. O'Brien [2]

ABSTRACT

Earthquake induced displacements are predicted for a typical site in an area near Vancouver (British Columbia) consisting of granular Fill, Peat, organic Silt, and soft sensitive Clay overlying dense glacial Till with artesian water pressure. The site has an essentially flat topography but includes a creek channel with weak banks and has a history of high compressibility and static instability under preload fill. The dense glacial till base is inclined towards the creek which adds further cause for instability of the soft sensitive clay. Deformation analysis is carried out using the pseudo-dynamic finite element procedure proposed by Byrne et al in extension to Newmark's method. In the procedure used, both earthquake inertia forces and soil softening effects were considered. The peak horizontal ground acceleration and ground velocity corresponding to the high intensity design earthquake anticipated for the area were used in the analysis. The relevance of the proposed procedure for predicting seismic deformations of sensitive clay sites is discussed.

INTRODUCTION

The prediction of displacements due to seismic loading of soils whose strength and deformation properties change markedly with cyclic loading is a difficult problem. The difficulty mainly arises from the complexity of the stress-strain relations of the soil particularly when large strains occur. The deformation analysis applied here is a pseudo-dynamic finite element procedure proposed by Byrne (1990). This procedure allows both the seismic inertia forces as well as the softening effect of the soil to be considered. The method is essentially an extension of Newmark's procedure from a rigid-plastic single-degree-of-freedom system to a flexible multi-degree-of-freedom model. The procedure was initially proposed for the deformation analysis of earth dams where soil liquefaction would occur under earthquake loading (Byrne et al, 1992, 1994). In this paper the procedure is applied to predict seismic free-field displacements of a soil-structure system where softening similar to liquefaction is anticipated in a sensitive clay zone. This paper presents the method used, its applicability and limitations, together with a general description of the area considered and the results of the analysis.

[1,2] **MACLEOD GEOTECHNICAL LTD.** Suite 150 - 1451 Marine Drive, West Vancouver, B.C., V7T 1B8

SITE DESCRIPTION AND SUBSOIL CONDITIONS

Site description

The site considered is located within the Willingdon Area near the TransCanada highway east of Vancouver, British Columbia (see Figure 1). The site is within the former marsh area formed by the basin of the existing creek flowing through the area. This area has a history of geotechnical difficulties. During the construction of the TransCanada highway an extensive failure occurred which caused blockage of the creek. Previous site filling and preload treatment was followed by a large depression of the ground surface in some areas and a block and graben type slip failure in other areas. A major slide failure occurred later causing another blockage of the creek for a distance of about 250 m. Thereafter, the slide was reactivated which caused further blockage of the creek for a distance of about 150 m. The slides occurred by slumping of large blocks of fill with a translation of soil towards the creek. During construction of the approach ramps for a bridge crossing the creek, significant soil movements occurred which resulted in damage to the pile supported bridge wing walls and timber retaining walls and heaving of the base of the creek. The soil movements extended laterally about 75 m from the bridge location with ground displacements of the order of 1 to 2 m (Bradshaw and Fitzell, 1993).

Subsoil Conditions

Surficial geology maps of the Geological Survey of Canada indicate that the stratigraphy of the Willingdon Area consists of bog, swamp, and lake deposits including up to about 8 m thick Peat deposits. The Peat layers overlay glacio-marine or marine deposits which, in turn, overlay glacial drift. It is believed that the marine deposits have been leached after deposition to create a metastable soil structure (Rosenqvist, 1953; Terzaghi, 1955). The generalized subsoil profile within the area was estimated based on several boreholes including field and laboratory tests, and is summarized on the cross-section shown in Figure 2. We expect that the subsoil profile would include a surficial FILL zone (TILL-like soil and possibly lightweight weathered Hogfuel) overlying in order of increasing depth:

- brown, fibrous to amorphous PEAT
  thickness: 3 to 4 m
  water content: 350 to 630 %
  total unit weight: 7.8 kN/m^3

- brown to grey, soft to very soft, sensitive, organic SILT
  thickness: 1.5 to 1.8 m
  water content: 200 to 400 %
  total unit weight: 12.6 to 15.7 kN/m^3
  Atterberg limits: LL=78%; PL=39%; PI=39%; LI=4.1-9.2

- bluish grey, soft to very soft, sensitive, silty CLAY/CLAY becoming more sensitive with depth and somewhat sandier at depth
  thickness: 10 to 12 m
  water content: 75 to 200 %
  total unit weight: 14.9 to 15.7 kN/m^3
  Atterberg limits: LL=50-75%;PL=20-40%;PI=30-35%;LI=1.8-4.6

- grey, sandy SILT/silty SAND, fine gravel, trace of clay (TILL-LIKE) extending to unknown depth, dense to very dense

Water levels were measured in testholes at about EL. 12.2 m to 13.2 m (Geodetic Datum). We expect some variation of these levels with seasonal changes and during periods of sustained wet or dry weather. A condition of upward groundwater gradient is believed to exist in the glacial Till prior to fill placement due to measured water pressures higher than hydrostatic. Water pressures in excess of hydrostatic were also measured in the glacio-marine deposits due to the slow dissipation of excess pore water pressures generated by fill placement.

The surficial fill layer covering the site varies in thickness and has a consistency from soft to firm and a low to moderate compressibility. The thickness of the fill layer generally increases from 1 to 2 m with distance away from the creek.

The Peat layer varies with depth from fibrous to amorphous. It is highly compressible under loads in excess of existing overburden. Typically, primary Peat compression occurs in less than 2 to 3 months and essentially involves squeezing out of water from the soil mass. Secondary Peat compression takes considerably longer than primary compression and develops over several decades usually at a constant rate of settlement with logarithm of time. It involves plastic creep deformations that are dependent on the Peat initial thickness and water content but generally independent of load magnitude. The compression of Peat usually causes a marked increase in its strength under static loading (Brawner and Lea, 1960; MacFarlane, 1969).

The response of the Peat to earthquake loading was characterized in a series of laboratory cyclic triaxial tests carried out under 14 and 50 kPa effective confining pressures on specimens with about 500 % natural water content. For cyclic strain amplitudes ranging from 0.1 to 2.0 % the degradation in deviatoric stress amplitude for 3 cycles was generally small. Since the applied cyclic stress ratios ranged from 0.05 to 1.20, it is considered that the performance of the Peat layer under earthquake loading would be good.

The response of the organic Silt layer to cyclic loading was not tested in the laboratory. Its behaviour under earthquake loading may be compared to similar organic Silts from other sites in the greater Vancouver area which were subjected to cyclic triaxial and simple shear tests. Previous test results on a similar organic Silt from a nearby site indicated that for cyclic stress ratios ranging from 0.19 to 0.24, cyclic strain development varied from 0.60 to 7.0 % for a consolidation stress equal to 50 kPa. The undrained strength ratio S_u/σ'_v decreased from a pre-cyclic value equal to 0.28 to a post-cyclic value ranging from 0.22 to 0.24. For this site, the organic Silt is very soft, sensitive, and has a considerably higher liquidity index than the Port Mann bridge organic Silt. Moreover, the thickness of the Organic Silt layer is small compared to the underlying sensitive Clay layer. Therefore, the properties of the organic Silt are considered identical to the properties of the underlying soft sensitive silty Clay zone.

Field Vane tests indicated that the Clay layer has an undrained strength ranging from 10 to 16 kPa. Such low values of undrained strength corresponded with Clay natural water contents equal or higher than its liquid limit. The Atterberg limits indicate a plasticity index ranging from 30 to 35 % and a liquidity index ranging from 1.8 to 4.6. The relationship betwen sensitivity and liquidity index suggested by Bjerrum (1954) indicates that the Clay sensitivity at this site could range from 50 to 500 which would correspond to a very quick to extra quick clay. Direct measurement of the Clay sensitivity at the site was attempted using the field Vane apparatus. The field test results indicated sensitivity values of about 20 or higher as the in-situ remoulded strength was below the resolution of the apparatus available for the site investigation. It is believed that the Clay undisturbed undrained strength could be higher than measured as the initial insertion of the Vane can cause significant disturbance in soft clays (Roy and Leblanc, 1988). Shear wave velocity measurement indicated values of about 50 to 60 m/s for a depth ranging from 2 to 11 m below existing grade (Elevation ranging from 13.0 m to 3.0 m, Geodetic Datum). Measurement of shear wave velocity was also difficult because of the low undrained strength and high sensitivity of the Clay.

Also because of the very soft nature of the Clay, attempts at sampling undisturbed specimens were generally ruled out. However, the Clay response to earthquake loading may be estimated by comparison with similar sensitive clays which experienced slide failures under static conditions or seismic events and were subjected to laboratory monotonic and cyclic loading tests. An extensive literature is available on the behaviour of sensitive clays from various sites and with a wide range of physical properties (MacKay, 1979, 1982; Locat et al, 1984; Maerz, 1985). The observation of flow slides of sensitive clays under both seismic and static conditions indicates that the clay behaves as a liquid and spreads considerably even on very gentle slopes (Liebling and Kerr, 1965; Mitchell and Markell, 1974). The results of static and cyclic loading tests available in the literature on similar sensitive clays (Bjerrum, 1961; Bjerrum and Kenney, 1968; Lee and Focht, 1976; Lee, 1979; Lefebvre and Leboeuf, 1987) indicate the existence of a low threshold strain level (generally less than 1 %) beyond which the clay looses its structure and starts to develop large strains rapidly. These observations on the response of sensitive clays to seismic loading are similar to the observations reported on the behaviour of loose sand under both monotonic and cyclic loading (Bjerrum et al, 1961; Castro, 1969). Therefore, the methods that were developed to predict seismic deformations of liquefiable loose sands can be applied to estimate soil displacements in sensitive clay sites such as the site considered in this paper.

For the subsoil conditions described above, site development would generally require preload treatment to prepare the area to the specified design grade. A staged fill placement with careful instrumentation to monitor ground response with time is recommended. Most structures on the site would require a pile supported foundation. A minimum setback distance from the adjacent creek would be required and the effect of lateral loading of piles due to soil displacements under seismic conditions should be analyzed.

In order to establish the required setback distance for structures adjacent to the creek and to analyze the response of foundation piles to lateral loading, a prediction of free-field soil movements under seismic conditions is required. The methodology used to estimate free-field soil displacements for this site is described in the following section.

METHODOLOGY

Analysis Procedure

Complex effective stress dynamic analysis procedures have been proposed to predict the displacements due to seismic loading of soils. These methods require elaborate soil models and parameters based on advanced laboratory testing methods and calibration techniques that are not normally available in practice. Generally, these methods are used for research purposes.

A simple deformation analysis procedure was proposed by Newmark (1965). In this method soil deformation response to seismic loads is considered by analogy with a sliding block modelled as a single-degree-of-freedom rigid plastic system. Displacements can be computed by numerical integration for any prescribed time history of acceleration. The maximum displacements at the end of a shaking period can then be estimated using simple formulae where the earthquake load is approximated by a number of pulses.

The application of Newmark's simple procedure to soil-structure systems where liquefaction can occur raises two concerns: (a) the soil is generally not rigid plastic and does not take into account the softening effect of the liquified soils; and, (b) the single-degree-of-freedom model does not allow the pattern of displacements to be computed. Byrne (1990) discusses these concerns and proposes a method that allows for a general stress-strain relation as well as extending Newmark's approach to a multi-degree-of-freedom system (see Figure 3). Basically a pseudo-dynamic finite element procedure is used, in which earthquake induced displacements satisfying energy considerations are computed by use of a seismic coefficient. The appropriate seismic coefficient is the one which satisfies the work-energy equation and is found by trial-and-error. The extended Newmark procedure has been incorporated into the finite element computer code SOILSTRESS (Byrne and Janzen, 1981, 1989). In carrying out analyses where liquefaction is triggered, only one pulse is considered. This is justified based on the observation that, in general, liquefaction or strain softening starts at a small strain level compared to the strains that occur once the peak strength is mobilized. Once liquefaction or strain softening occurs, it is considered that no further major pulses would occur.

The application of the extended Newmark procedure to the prediction of displacements due to seismic loads is limited to soil elements with a stress-strain response that conforms to the characteristic behaviour of liquefiable soils such as loose sands or quick sensitive clays. Typically these materials are strain softening when loaded under undrained conditions beyond a given small strain level (generally less than 1 %). The application of the extended Newmark model also implies the existence of a hypothetical residual strength value S_r that the liquified soil elements would reach after considerable strains (generally larger than 20 to 50%). Although conceptually plausible, the existence of a residual strength for liquified sands has been questioned. In contrast, for clays the concept of a residual strength has been established and well documented in the literature (Ladanyi et al, 1968; Kenney, 1967, 1977; Lupini et al, 1981).

Cases Analyzed

The water level was considered at the design flood level of the creek and the peak ground acceleration was set at 0.25g based on the seismic hazard analysis carried out by the Pacific Geoscience Centre of the Geological Survey of Canada for this site with a corresponding velocity equal to 0.25 m/s. For this site, the velocity was applied uniformly to all the layers of the subsoil profile along the cross-section in Figure 4.

The only variable in the analyses was the level of post-cyclic undrained strength ratio S_u/σ'_v. The post-cyclic undrained strength ratio for the sensitive Clay and the organic Silt layers was varied from 0.20 to 0.05 with intermediate values equal to 0.15 and 0.10. This range of values was considered because of the difficulty of measurement of the peak undrained strength within the organic Silt and the sensitive Clay zones. This approach will allow to relate the predicted seismic displacements to the degree of strength deterioration caused by earthquake loading.

It was assumed that the granular Fill cover and Peat layer would not be affected by the earthquake shaking since the fill is compacted and mostly above the water table and laboratory data indicated a good performance of the Peat under cyclic loading conditions. The zones of different materials along the East-West section of the subsoil profile and the water level considered are shown on the initial finite element mesh in Figure 4.

Soil Parameters Used in the Analyses

The soil parameters used in the analyses were based on the hyperbolic model, and were obtained from available laboratory data on similar materials following the method described by Duncan et al. (1980). The soil is treated in the analysis as equivalent elastic using secant and bulk moduli that vary with stress level as follows:

$$G = k_g P_a(\sigma'_m/P_a)^n(1-\tau R_f/\tau_f)$$

and

$$B = k_b P_a(\sigma'_m/P_a)^m$$

in which k_g and k_b are shear and bulk modulus numbers, n and m are modulus exponents, τ_f is the failure strength, and R_f is the ratio of the strength at failure to the ultimate strength from the best fit hyperbola, σ'_m is the mean normal stress, P_a is atmospheric pressure, and τ is the mobilized shear stress. The soil parameters used in the analyses are listed in Table 1. The soil parameters also include the initial tangent modulus E_i, and the total unit weight γ_t for each material.

RESULTS

The predicted deformations for the three first cases considered with a post-cyclic undrained strength ratio equal to 0.20, 0.15, and 0.10 are shown in Figure 5. This figure shows the deformed finite element mesh immediately east of the creek magnified by a factor of 10. The creek bank is predicted to undergo increasing larger horizontal and vertical movements towards the centre of the creek with deterioration of the post-cyclic undrained strength. The pattern of soil movements correspond to the field observations described above for this site during previous slide failures that occurred under static fill loads. It is interesting to note that major lateral soil movements occur in the organic Silt transition layer between the overlying Peat and imported Fill and the underlying soft clay zone. This result is consistent with the fact that the organic Silt has a liquidity index ranging from 4 to 9. Generally larger soil deformations are associated with higher liquidity index in sensitive soils as shown by Mitchell and Markell (1974). Larger lateral soil displacements occur in the lower Clay layers when a marked deterioration in post-cyclic undrained strength is considered (cases 3 and 4). The smaller lateral soil displacements in the Clay zone may be related to the 'bowl-shaped' Clay deposit between the organic Silt layers and the glacial Till hard bottom as shown in Figure 4. The shape of the Clay deposit in the section considered tends to limit the magnitude of lateral soil displacements within the Clay zone.

A summary of predicted soil displacements is shown in Table 2. The predicted lateral soil movements at the creek top of bank increase from 1.5 to 2.5 m for a decrease in post-cyclic undrained strength ratio from 0.20 to 0.05. The corresponding downward soil movements at the creek top of bank range from 1 to 2 m. At a distance of 25 m from the creek top of bank, the predicted lateral soil displacements at the surface range from 200 to 500 mm and the corresponding downward ground subsidence varies from 10 to 20 mm. The lower displacement values predicted by the application of the simple Newmark's analysis using the same soil strength properties as indicated on Table 2 are mainly due to the fact that the sliding block model is considered for a rigid-plastic material. The extended Newmark analysis considers both elastic and plastic deformations that occur within the block and takes into account the softening of the soil stress-strain response due to seismic loads.

The above results indicate that a setback of minimum 25 m from the creek top of bank would be sufficient to limit the free-field soil displacements to a tolerable level for pile-supported structures. Moreover, the distribution of horizontal soil displacements with depth may be used for the analysis and design of pile foundations subjected to seismic lateral loads in addition to static axial loads.

Figure 6 shows the relationship between post-cyclic undrained strength ratio and predicted lateral seismic displacements at the creek east top of bank and at the ground surface 25 m east of the creek. This relationship may be used to estimate the minimum post-cyclic undrained strength required to limit lateral soil displacements within an acceptable range. The required minimum undrained strength may then be used to derive performance criteria to be met by soil improvement techniques such as lime stabilization. However, this relationship is specific to the site considered using the analyses described above.

SUMMARY

Soil deformations due to seismic loads are predicted based on an extended Newmark model that takes into account soil softening effects and earthquake inertia forces. The model which was initially developed to estimate displacements for loose liquefiable Sands is applied to a site that contains Peat, soft organic Silt, and soft sensitive silty Clay overlying dense glacial Till. The site considered is near a Creek and has a history of high compressibility and slide failures under moderate fill loads. The analyses were carried out for various levels of post-cyclic undrained strength deterioration of the organic Silt and sensitive Clay layers. The results indicate the required setback distance from the creek to limit free-field displacements within a tolerable range. The distribution of lateral soil displacements with depth can be used for the analysis and design of piles subjected to horizontal earthquake loads. The relationship between post-cyclic undrained strength and lateral soil displacements can be used to derive ground improvement performance criteria to limit soil movements under seismic loads.

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TABLE 1 **Soil Parameters Used in the Analysis**

Material & Parameters	Fill	Hogfuel	Peat	Organic Silt	Sensitive Clay
E_i (kPa)	120000	10800	1628	8300	25000
K_g	400	325	325 (275)	325 (255)	325 (130)
n	0.50	1.00	1.00	1.00	1.00
K_b	4000	2500	2000	2500	2500
m	0.25	1.00	1.00	1.00	1.00
ϕ (deg.)	35	33	28	26 (0)	26 (0)
$\Delta\phi$ (deg.)	0.2	0.0	0.0	0.0	0.0
Su/σ'_v Case 1 Case 2 Case 3 Case 4	-	-	-	(0.20) (0.15) (0.10) (0.05)	(0.20) (0.15) (0.10) (0.05)
R_f	0.90	0.90	0.70	0.70	0.70
γ_t (kN/m^3)	18.8	10.3	10.3	16.7	18.0

Notes:
a) Numbers between brackets indicate post-earthquake soil properties;
b) Pre- and Post-earthquake properties for Fill, and Hogfuel are assumed identical since these soils were assumed to be not affected by seismic loading.

TABLE 2 **Summary of Predicted Seismic Soil Displacements (m)**

Case Number	Seismic Coefficient k_h	Using Newmark's Method (Hor.)	Using Byrne's Energy Method & Soilstress: At Creek Bank (Hor.)	At Creek Bank (Ver.)	25 m from the Creek (Hor.)	25 m from the Creek (Ver.)
-	0.0400 [a]	0.42	-	-	-	-
1	0.0650 [b]	0.14	1.49	1.05	0.17	0.010
2	0.0495 [b]	0.26	1.82	1.42	0.23	0.015
3	0.0425 [b]	0.37	2.43	1.73	0.39	0.017
4	0.0255 [b]	1.10	2.86	2.05	0.45	0.019

Notes: [a] Seismic coefficient or yield acceleration from slope stability analysis.
[b] Seismic coefficient for internal/external energy balance less than 5 %.

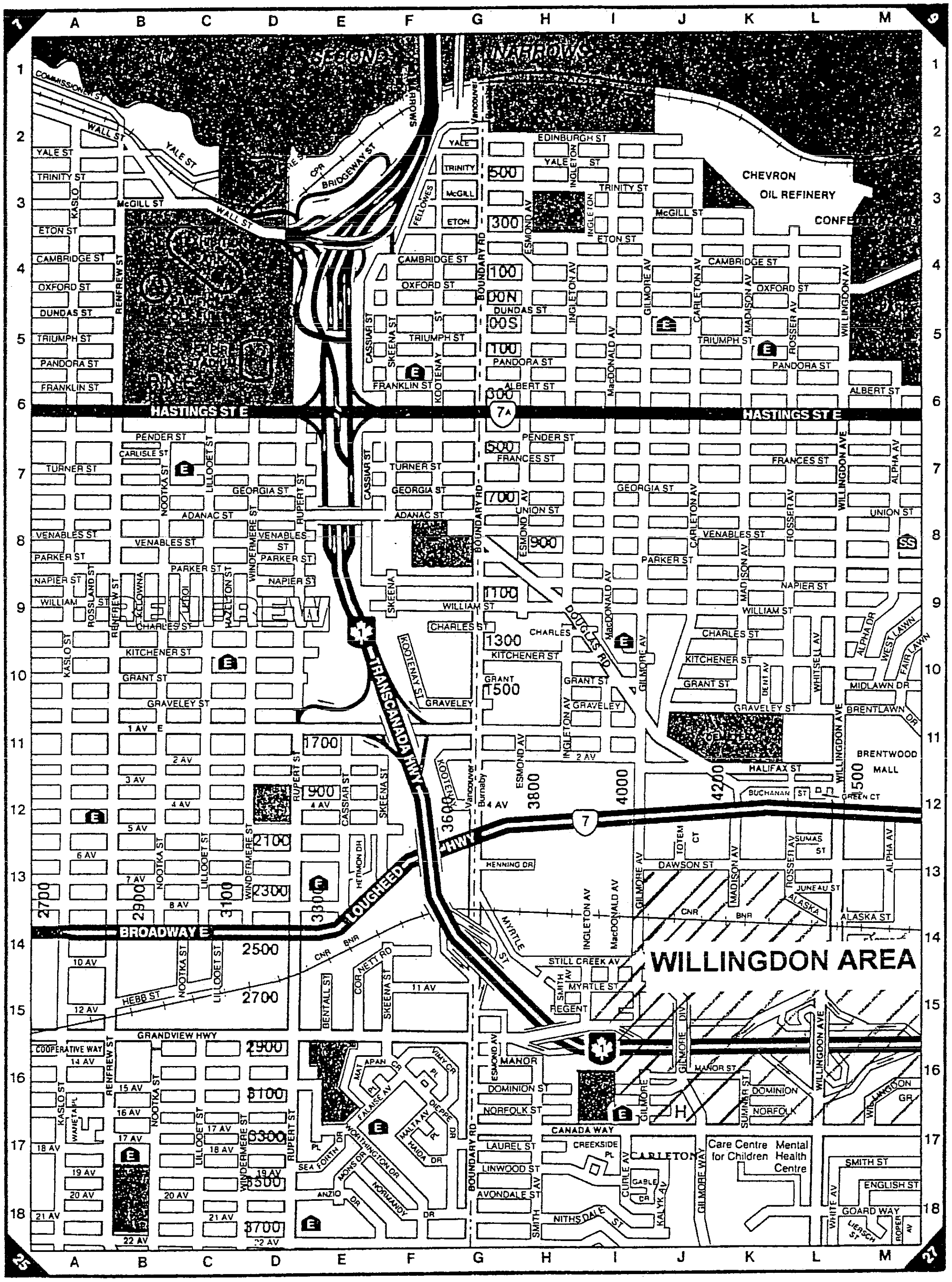


Figure 1 Location Plan - Willingdon Area, Burnaby, British Columbia.

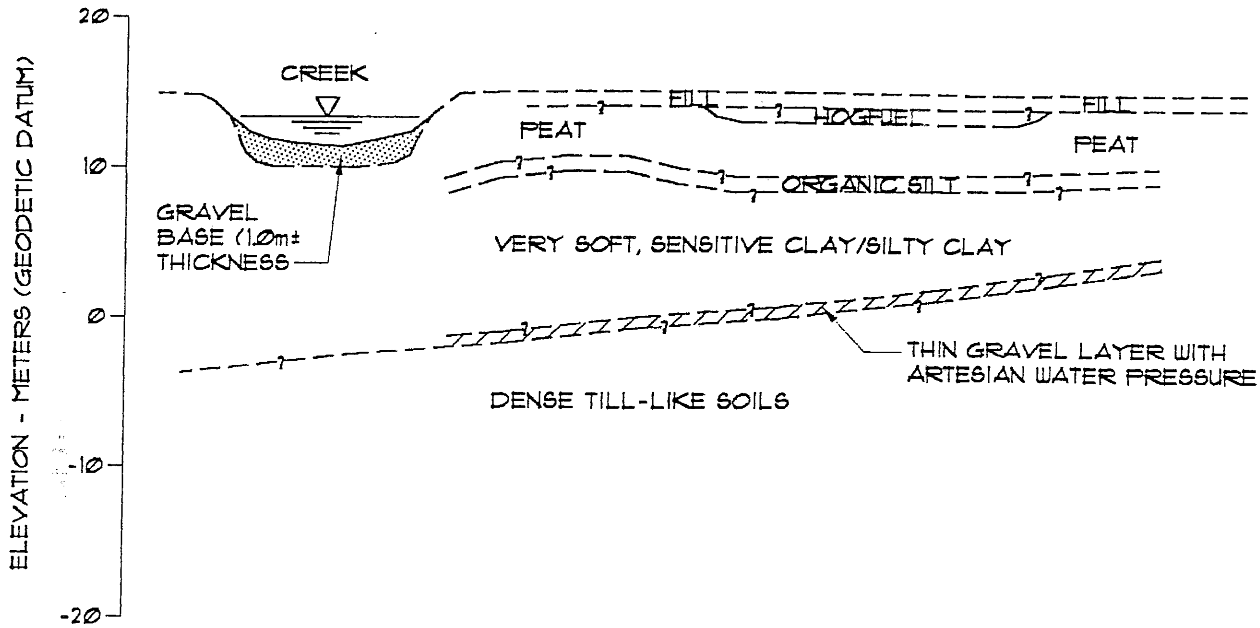


Figure 2 Generalized Subsoil Profile - East-West Section Through the Creek Wellingdon Area, Burnaby, British Columbia.

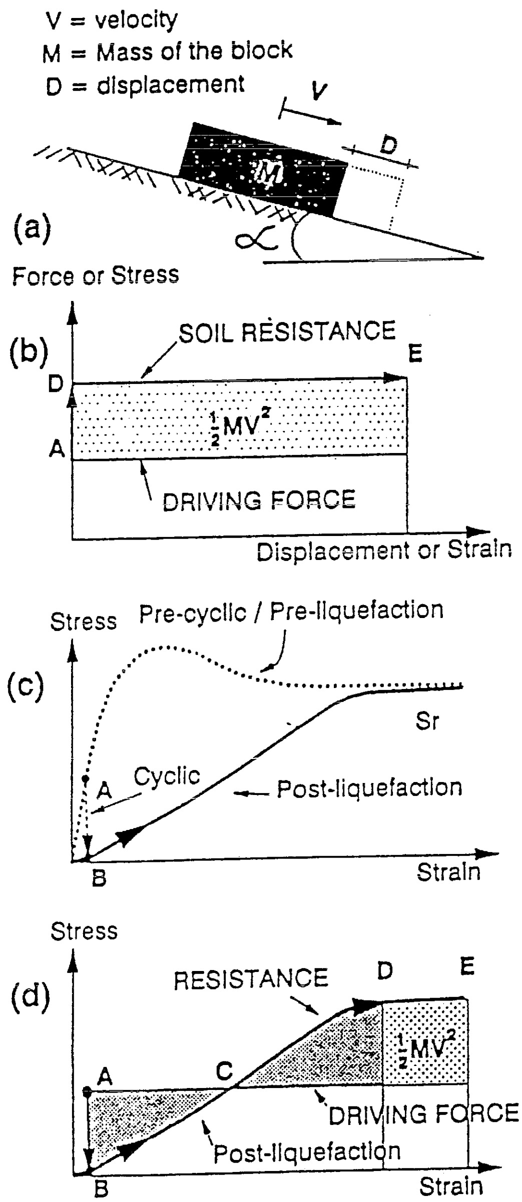
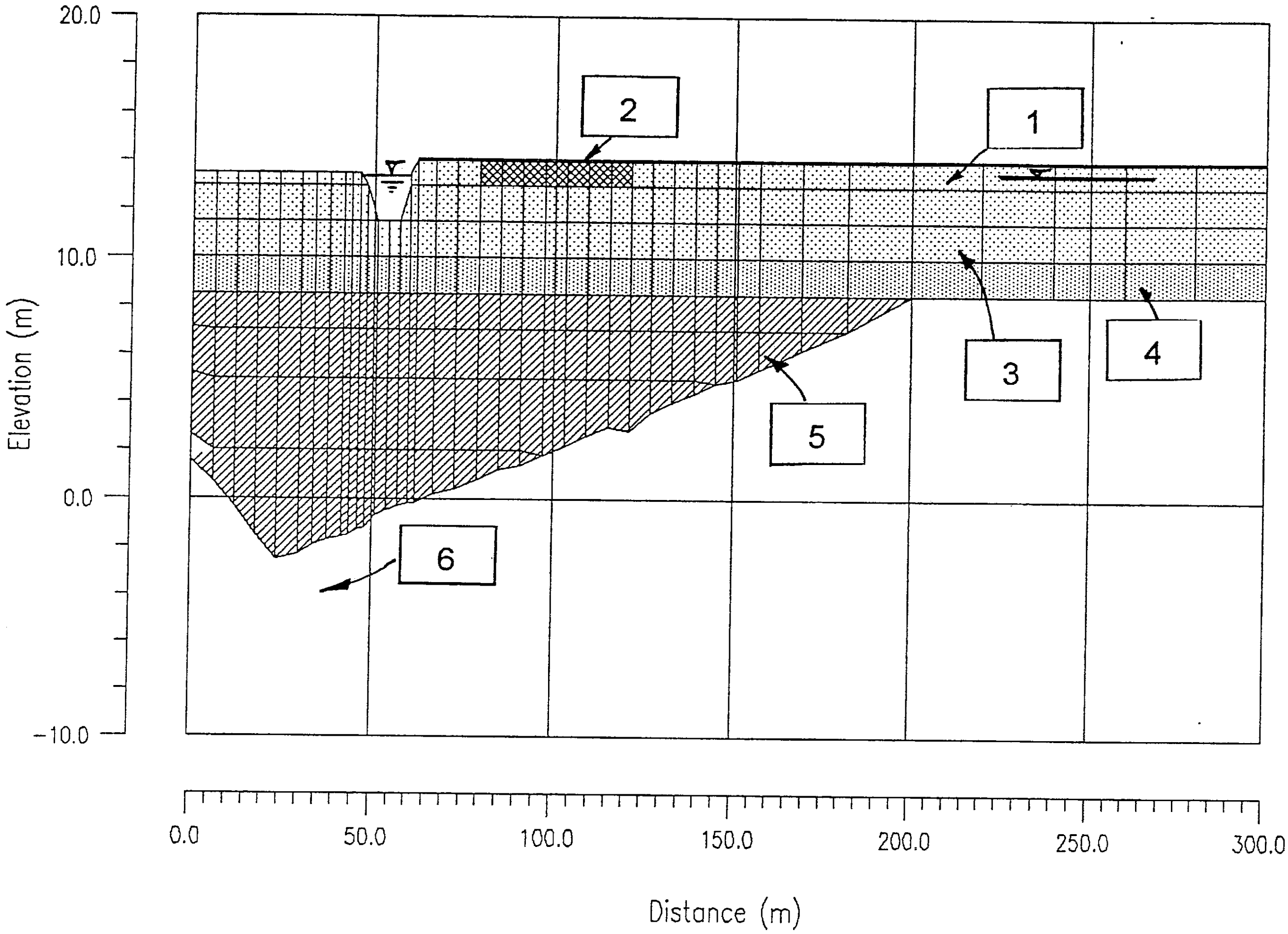


Figure 3 a) Model of Block Sliding on Inclined Plane Subjected to Velocity Pulse V;
b) Work-Energy Relation for Newmark's method;
c) Characteristic pre- and post-cyclic stress-strain relationship;
d) Work-Energy Relation for Byrne's extended Newmark method.
(after Byrne et al, 1994)



No.	MATERIAL TYPE
1	GRANULAR FILL
2	HOGFUEL
3	PEAT
4	ORGANIC SILT
5	SENSITIVE CLAY
6	GLACIAL TILL

Figure 4 Initial Finite Element Mesh, Subsoil, and Groundwater Conditions.

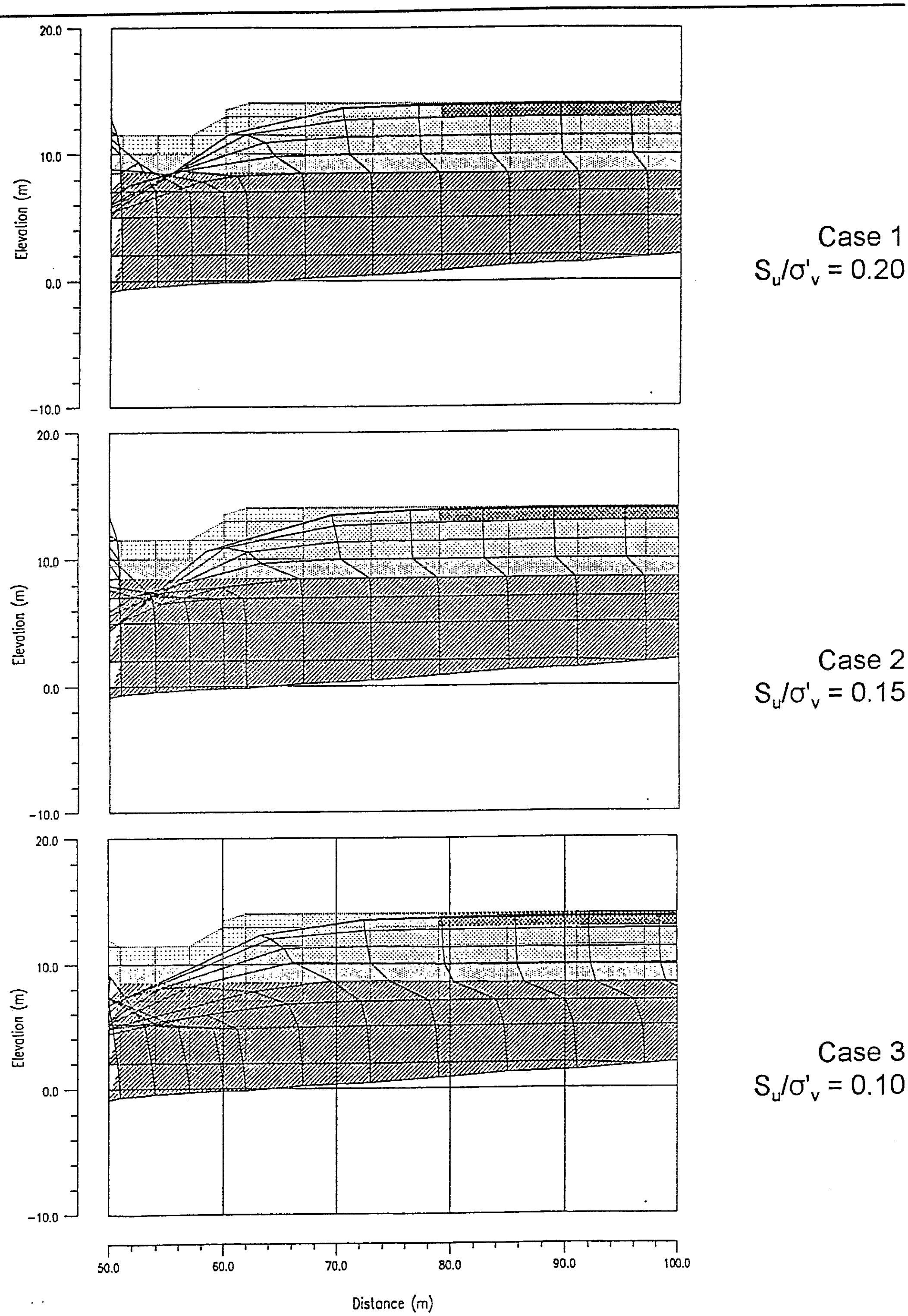


Figure 5 Displacement Pattern East of Creek - East-West Section (Dashed Lines: Initial Mesh; Solid Lines: Displaced Mesh)

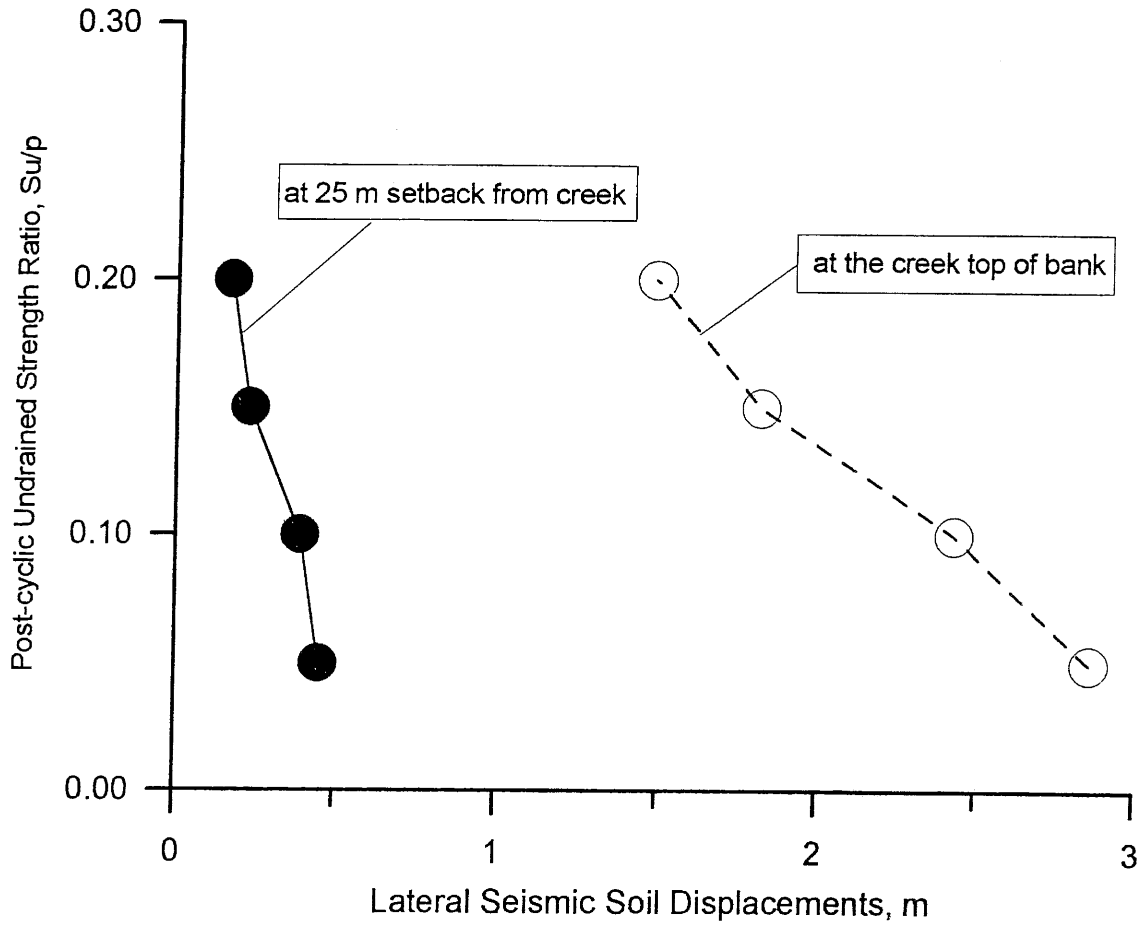


Figure 6 Predicted Seismic Lateral Soil Displacements at ground surface versus Post-cyclic Undrained Strength ratio S_u/σ'_v of organic Silt and Clay layers.