BASEMENT WALL WITH SEISMIC EARTH PRESSURES AND NOVEL EXPANDED POLYSTYRENE FOAM BUFFER LAYER

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ABSTRACT

This paper summarizes the geotechnical aspects considered in the analysis and design of a reinforced concrete basement wall with up to four underground levels. The structure is founded on "firm" (very dense) glacial soils, however the walls have to retain soft/loose and potentially liquefiable soils which could exert large seismic earth pressures on the structure. Ground improvement and a novel Expanded Polystyrene buffer layer were incorporated in the design in order to reduce lateral seismic loads on the underground structure. The limit equilibrium (pseudo-static), finite element (pseudo-dynamic analysis using the program SOILSTRESS), and dynamic analysis (using the program FLAC) methods used to estimate the basement wall seismic earth pressures are described. The analyses indicated an approximately 50% reduction in seismic earth pressure loads by use of the Expanded Polystyrene buffer layer.

INTRODUCTION

This paper discusses geotechnical considerations involved in the seismic design of a reinforced concrete basement structure which retains up to approximately 10 m of loose and soft soil. The basement forms part of a highrise building located near the waterfront in the City of Vancouver.

This case history is considered to be of interest in that it represents a particular design condition for which analysis indicates seismic design soil pressures against the basement wall which are very much greater than are normally considered in the design of structures of this type. The key features of this design condition are:

1. The structure is founded on and keyed into dense soils, which would be subject to "firm ground" earthquake motion.

2. The soil retained by the structure consists of soft, fine grained marine bottom deposits, overlain by loose and liquefiable sandy fill, resulting in a significant difference between the earthquake induced motion within the retained soil and the stiff soils on which the structure is founded.

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3. The structure, a typical parking basement for a highrise structure, is considered to be relatively stiff. Therefore it would be subject to earthquake motion similar to that of the stiff foundation layer and significantly different from the motion of the retained soil.

Description of Project and Subsoil Conditions

The project consists of a commercial and residential building development situated in an area of reclaimed land. The development includes a reinforced concrete highrise tower, a number of low-rise apartments, and commercial space over a basement parkade structure, which extends beyond the footprint of the above-grade buildings. The lowest floor of the parking basement is located between 6.7 and 10.7 m below site surface grade. On its north side the basement adjoins previously constructed parking basements of similar or shallower depth. On its east and west sides there are city streets and on the south side there is a Transit right-of-way. This paper is concerned with the south basement wall of the project. Typical sections are shown on Figure 1.

Along the south side, adjacent to the Transit right-of-way, a stepped configuration as shown in Figure 1(a) was utilized where possible for the parkade layout in order to minimize the impact of the basement excavation on existing structures in the Transit right-of-way. Where this was not possible the profile as shown on Figure 1(b) was used.

The subsoil profile includes a layer of reclamation fill to a depth of 6 to 7 m below grade, resting on original bottom mud deposits which in turn rest on dense glacially overridden silts/sand/gravel mixture (glacial drift). The glacial drift rests on Tertiary bedrock.

The thickness of the reclamation fill and bottom mud deposits increase in a southerly direction across the project site, reaching their greatest thickness at the southern boundary of the project and then decreasing again toward the south. At the southern edge of the basement, a typical soil profile with cone penetrometer plot is shown in Figure 2. It will be seen that the fill consists of relatively clean sand with some shell fragments, medium dense to dense at the surface and becoming loose with depth. The fill thickness is in the order of 6 to 7 m. The bottom mud, which attains a thickness of 2 to 4 m consists of soft to firm sandy to clayey silt with shell fragments and minor layers of sand and silty sand. The glacial drift consists of a dense silt/sand/gravel mixture with its surface at a depth of 8.5 to 10 m below site surface grade. The surface of the Tertiary bedrock occurs at a depth of about 17 m.

Prior to excavating the basement the groundwater table was approximately 4 m below grade.

Design Problem and Methodology (during preliminary design)

It was recognized that a major seismic design consideration would be the requirement for the basement structure to resist lateral soil forces from the soft and potentially liquefiable soils to be retained on the south side.

Preliminary calculations indicated that the presence of a liquefied layer adjacent to the wall would result in forces exceeding the normal basement wall design loads. A decision was made to densify the zone of soil adjacent to the wall, as shown in Fig. 1. Densification was carried out using vibro-replacement stone columns at 2.5 m centres on an equilateral triangular grid. The width of the zone that could be treated was limited to about 10 m due to the presence of existing structures within the Transit right-of-way. Post-densification testing showed excellent improvement in the sand layer (see Fig. 2). The stone columns were extended through the bottom mud silt layer to provide additional strength and stiffness. The subsequent analyses confirmed that this was beneficial in reducing the potential soil deformation.

Typically, the magnitude of seismic soil loads on building basement structures are evaluated using pseudo-static limit state design methods wherein the inertial forces due to the earthquake motion are represented as static horizontal or vertical body forces acting on the potentially sliding soil mass. The widely used Mononobe-Okabe method (Okabe, 1926) is a particular case of this method involving a planar wedge failure.

The seismic coefficient (the ratio between the body force to be used for analysis and the bulk weight of the soil mass) is a matter of judgment, and dependent on the degree to which the wall being designed is capable of permitting yielding of the soil mass by either sliding of the wall as a whole, or by rotation about the base. For the case of basement walls for buildings, which can undergo essentially no sliding and only very limited rotation, it has been suggested that a seismic coefficient equal to the peak acceleration for the earthquake being considered, expressed as a fraction of gravity would be suitable, except for structures founded on a sharp rock/soil interface (Whitman, 1991).

Limit state analysis assumes the development of resisting shear stresses at the boundary of the sliding soil mass under consideration. Soil deformation ("strain" deformation) sufficient to mobilize these shear stresses is assumed. The limiting shear stress is controlled by the soil strength (the "active" case).

If the inertial body forces render the sliding mass unstable under the restraining influence of the maximum attainable soil shear stresses and other restraining forces, then sliding would initiate and additional plastic soil deformation occurs. Under earthquake conditions, the inertial body forces change with time, and the state of instability is transitory.

"Strain" deformation is usually evaluated empirically based on model tests and field observations. Typical deformation allowances for walls designed for the "active" case (Ebeling & Morrison, 1992) range from 0.1% of the wall height for retained soil consisting of dense sand under drained conditions to 0.4% of the wall height for loose sand. For undrained conditions and cohesive soils, greater strain deformation would be expected.

"Sliding" deformation can be evaluated using the sliding block approach proposed by Newmark (Newmark, 1965). This theory has been combined with the Mononobe-Okabe approach for vertical walls by Richards & Elms (1979). These theories take no account of strain deformation (rigid-plastic theory).

Limit-state methods (pseudo-static methods)

The design case presented the following number of problems for analysis:

(i) Complex geometry, due to the stepped wall configuration.
(ii) Non-uniform soil conditions, with variations both vertically (layering) and horizontally due to drainage drawdown, the zone of ground improvement and spatial variation.

Limit-state analysis was carried out using the SARMA slope stability computer program (Hoek 1985). This program takes into account of body forces and external restraining forces. A typical analysis section is shown in Figure 3. A reduced shear strength was used for the clayey SILT to account for strain softening due to cyclic loading. The "critical acceleration ratio" (seismic coefficient to initiate sliding) was determined for different external restraining forces. Sliding deformation was then assessed by the Newmark method for each restraining force, using the calculated critical acceleration and the peak design acceleration, taken as 0.25g. In this way a curve of sliding deformation versus external restraining force was developed. In order to obtain the total deformation, strain deformation was established from the shear strain required to mobilize the reduced shear strength of the clayey SILT. These strains were estimated from the results of post-cyclic monotonic triaxial tests on silts of similar characteristics. By adding the strain deformation and sliding deformation a design curve of force versus displacement was obtained (see Figure 3).

Finite-element Method

Because of the complexity of the design problem and uncertainty in the applicability of the pseudo-static method previously discussed, a separate finite element analysis was conducted to provide a check on the design curve obtained from the pseudo-static analysis. Dr. Byrne of the University of British Columbia developed a method for estimating the deformation of liquefied soils by extended the Newmark-rigid-plastic-sliding-block analysis concept to consider both strength loss and modulus reduction (Byrne, 1990). With this approach the kinetic energy from the soil mass and that induced by soil stiffness reduction is equated to the strain energy from deformation of the soil mass. This model was then incorporated in the static finite element program SOILSTRESS,(Byrne & Janzen, 1981; Byrne et al., 1992; Byrne et al., 1994). Within SOILSTRESS a seismic coefficient is used to incrementally strain the soil block until an energy balance is achieved. The inertial energy from the earthquake is input by giving the soil mass a velocity. For the SOILSTRESS analyses conducted for the site discussed in this paper an initial velocity of 0.25 m/sec (for one pulse) was used. This was increased from 0.21 m/s peak velocity from the NBCC seismic model to 0.25 m/s in order to account for possible amplification within the soft soils at the site.

Figure 4 shows a typical finite element mesh and assumed soil properties used in the analysis. For estimating the load on the basement wall in the SOILSTRESS analysis, the end nodes of the mesh were initially fixed horizontally to model a rigid basement wall. The initial velocity was directed toward the basement wall and seismic coefficients incrementally increased until an energy balance was achieved. The output stresses in the end elements were then summed over their height in order to estimate the loads on a rigid basement wall. Following this the analysis was repeated, but instead of fixing the end nodes, they were allowed to move horizontally.. Constant external horizontal restraining forces were applied at the end nodes to model the resistance from a yielding wall. The output then provided the end node displacements associated with the input forces. The input forces were then reduced in a subsequent analysis to obtain the load/displacement curve shown in Figure 5.

Expanded Polystyrene Seismic Buffer

The results of the analyses clearly demonstrated that the wall would either have to undergo considerable deformation in order to reduce the forces to levels approaching those normally used for design, or alternatively would have to withstand very high forces to maintain deformation within tolerable levels.

Since neither option was practical, a third solution was adopted which involved the incorporation into the backfill adjacent to the wall of a compressible buffer material (Expanded Polystyrene foam, [EPS] - see Figure 1).

The buffer material should:

(i) be strong and stiff enough to withstand the static condition soil forces with low deformation and creep, but at the same time exhibit a pronounced yield plateau at stress levels corresponding to the seismic soil forces;

(ii) be inert, and not subject to deterioration due to chemical attack, aging, etc.,

(iii) be of modest cost compared to the potential structural costs.

EPS has an established track record in geotechnical applications and was considered to satisfy requirements (ii) and (iii). The conflicting demands of requirement (i) were met by utilizing the correct density and thickness of EPS (since strength is dependent on density) and by limiting the level of the static stresses.

This last condition was achieved (a) by taking advantage of reduced static stresses due to the stepped configuration of the parkade and (b) where the stepped configuration was not practical due to space restrictions, by using permanent soil anchors in the shoring design.

Published stress-strain data for EPS based on the ASTM D-1621 test procedure involves a loading strain rate of about 10%/minute. For the design application, more rapid loading was of interest, and testing was carried out to confirm the behaviour. Creep tests, followed by rapid loading compression tests were performed to simulate anticipated design conditions. It was found that for the required purpose, a very low density EPS was required. Typical rapid-loading compression test result are shown in Figure 6.

The required buffer thickness was determined based on the design seismic restraining capacity of the wall (determined in conjunction with the structural engineer) the soil face deformation associated with that restraining capacity, and the force-deformation curve of the EPS.

For this design case, the EPS thickness used ranged from 450 to 610 mm, installed in the form of Polyethylene wrapped billets, 1200 mm wide and up to 2500 mm in length.

Dynamic Analyses

A preliminary dynamic analysis was conducted to check the previous design. The explicit finite-difference program FLAC (Cundall and Board, 1988) was used for the analysis. Figure 7 shows the grid and soil properties assumed. The basement wall and dense glacial soils were modelled as being rigid and rigidly connected. The earthquake motion was simulated with both a velocity sine wave with a frequency of 1.57 Hz and with an earthquake time history. Both were scaled to a velocity of 0.21m/s and both were input to the rigid base and basement wall. A Mohr Coulomb constituency model was used for the sand fill and a strain-softening "modified Mohr Coulomb" model was used for the underlying silt layer. The EPS foam fill was modelled using the double yield constituency model with volumetric stress/strain approximately following the curve obtained from the laboratory tests (Fig. 6). Analyses were made for the cases listed in the table below:

Table 1 FLAC Analysis Cases

CASE	DYNAMIC INPUT	FOAM BUFFER	REMARKS
A1	SINE FUNCTION	NO	RESULTS ON FIGURE 8
A2	TIME HISTORY	NO	RESULTS ON FIGURE 9
B1	SINE FUNCTION	YES	RESULTS ON FIGURES 8 AND 10
B2	SINE FUNCTION	YES	PROGRESSIVE SOFTENING OF SILT LAYER. RESULTS ON FIGURE 8
B3	SINE FUNCTION	YES	COMPRESSIBILITY OF FOAM REDUCED BY 1/2. RESULTS ON FIGURES 8 AND 10
B4	TIME HISTORY	YES	RESULTS ON FIGURES 9 AND 10

The lateral displacements of the foam layer, average lateral strain of the foam layer, lateral pressures applied to the wall, and total load applied to the wall were monitored during the analyses.

The following observations were made from the dynamic FLAC analyses:

- the EPS buffer results in an approximately 50% reduction in lateral loads on the rigid basement wall;
- lateral earth pressures at the upper metre of the wall alternates between zero and the passive capacity of the soil both with and without the EPS buffer layer and then increases with depth;
- compression of the EPS buffer layer increased approximately linearly with depth;
- progressive softening of the silt layer from a shear modulus of 10,000 kPa to 160 kPa over 3 cycles did not significantly alter the loading on the basement wall compared to the case where the modulus did not soften.
- Increasing the compressibility of the EPS buffer layer beyond a given threshold results in increased lateral compression but no reduction of lateral loading on the wall (Figures 5 & 10).
- the lateral loads from the dynamic (FLAC) analyses are similar to and possibly slightly lower than those interpreted from the finite element pseudo-dynamic SOILSTRESS analyses.

CONCLUSION

The case history described in this paper is an example of a design problem which is occurring more frequently in the Vancouver area, as previously reclaimed foreshore areas are developed in a manner requiring the construction of deep basements below tidal levels. Where the structure is founded on "firm" ground, and the surrounding old reclamation fill is soft and or liquefiable, very large wall loading may occur under earthquake conditions, well in excess of traditional design loadings for these types of structures.

The use of an Expanded Polystyrene buffer material in this case represented an attempt to address this particular design problem in a way which would not lead to excessive extra structural cost.

All methods of analysis used on this project to estimate wall loadings indicated a beneficial effect from yielding at the soil face in reducing the wall loading. Clearly judgment is required in the interpretation and application of the results.

In this case a buffer consisting of Expanded Polystyrene was used in order to permit yielding at the face of the basement wall. When using this material consideration should be given to the ratio of static to seismic loading so as to avoid excessive creep under static conditions, but still provide sufficient compressibility under seismic loading. Development of a more satisfactory buffer material with low creep and a pronounced yield plateau would be a useful initiative. Decreasing the compressibility of the buffer near the surface may also be beneficial.

A preliminary dynamic analysis of the soil-structure system using the program FLAC provided insight into the behaviour of the wall/buffer/soil system and indicated a significant load reduction on the wall by providing a yielding buffer layer.

Acknowledgements
We wish to thank Marlin Gee and Glen Merkley for drafting the figures and Carol Oddy for assistance in typing the manuscript. We also wish to thank Dr. Peter Byrne for encouragement toward using the FLAC program and Dr. Loren Lorig of ITASCA for assisting with setting up the mesh.

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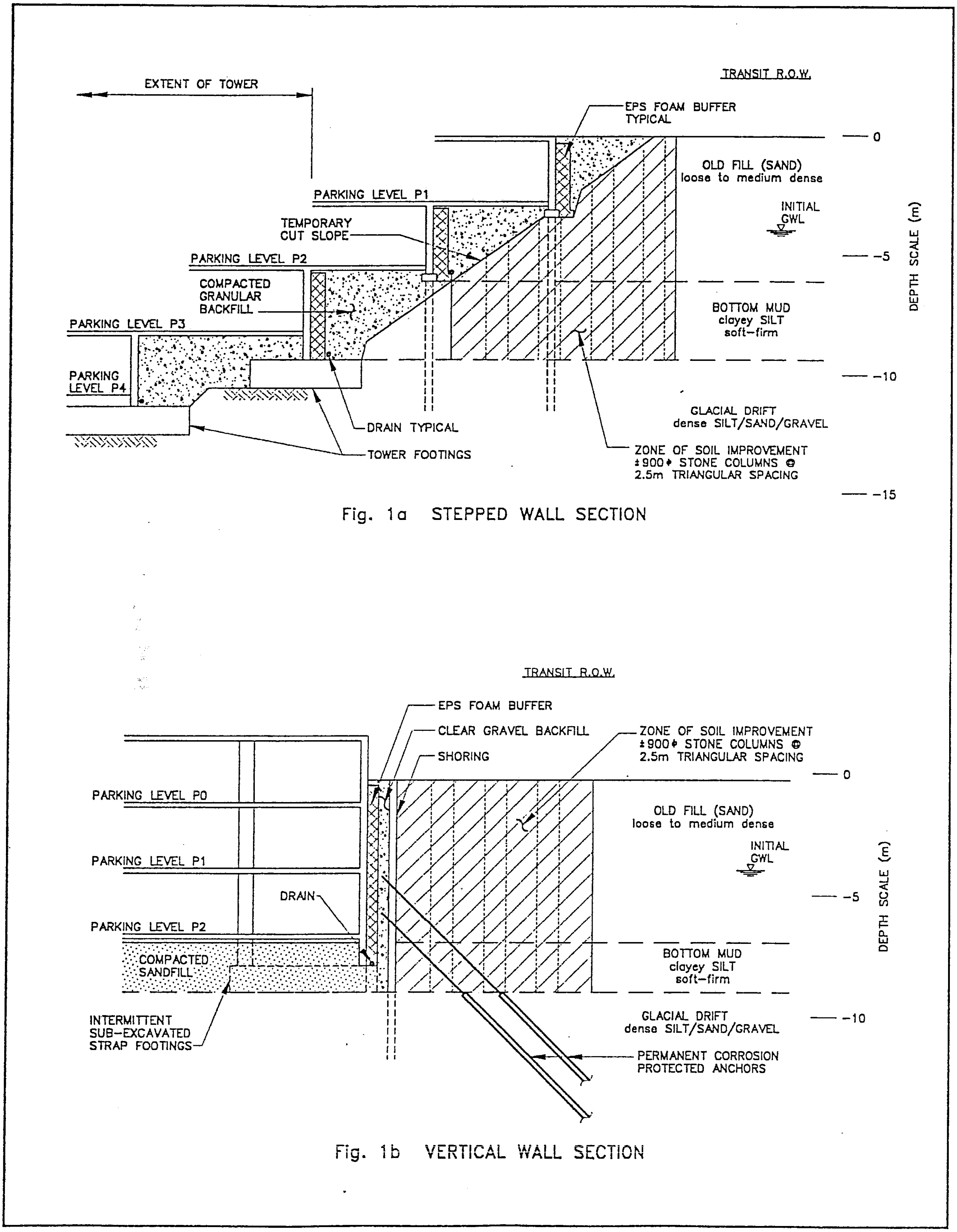


Figure 1 Sections at south basement wall (a) stepped wall section (b) vertical wall section

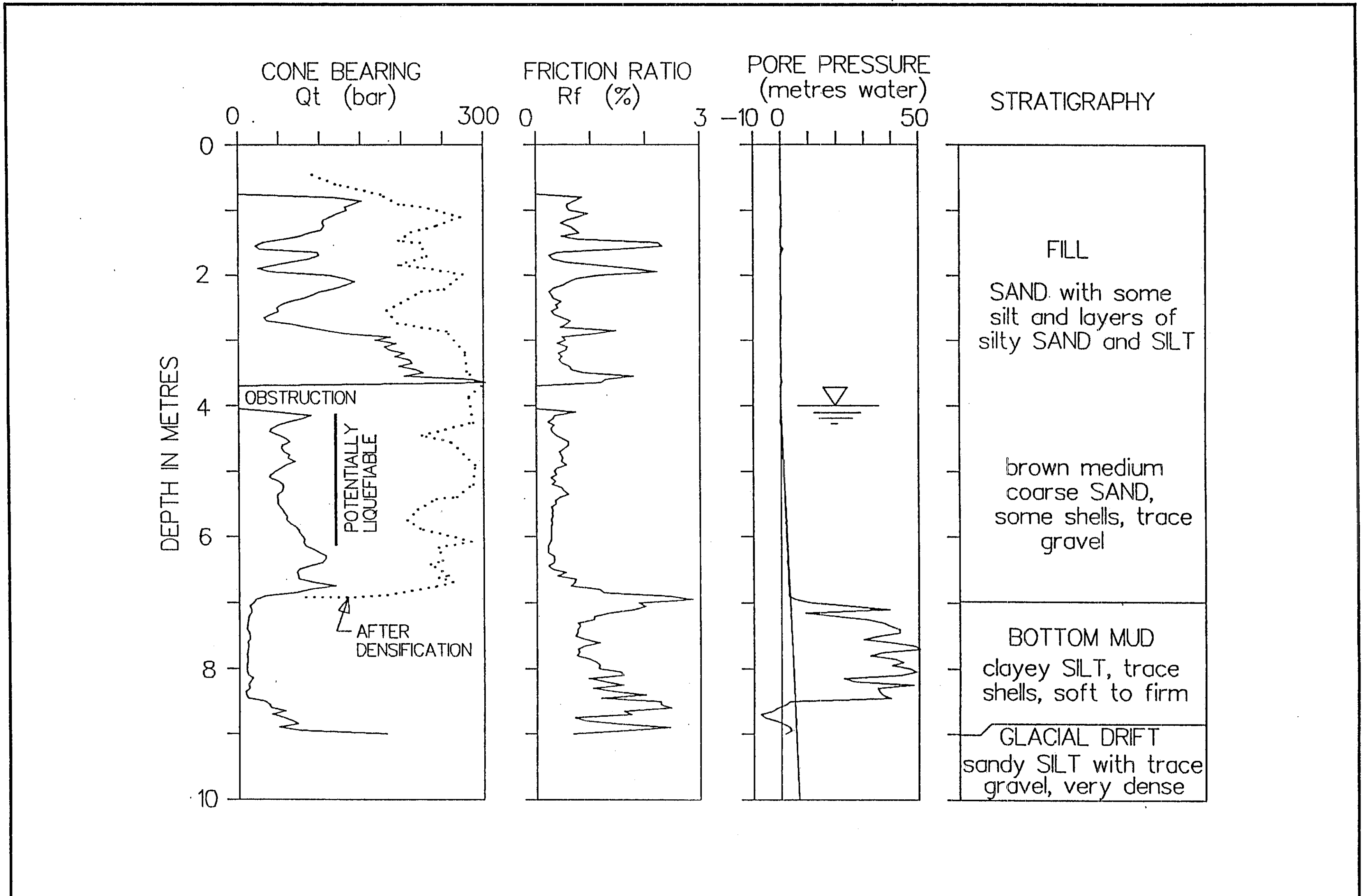


Figure 2 **Typical Soil Profile with Cone Plot**

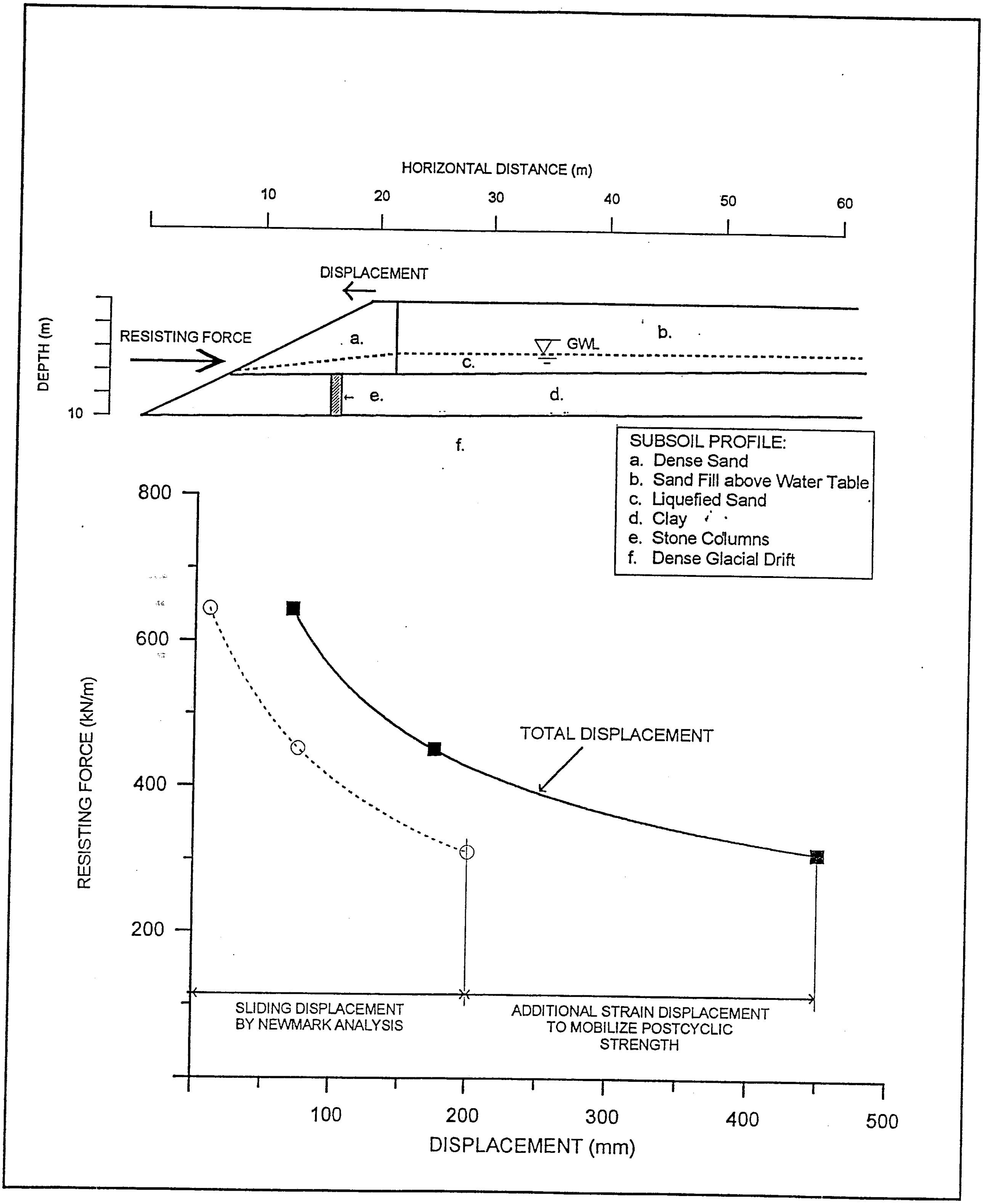


Figure 3 **Pseudo-Static Analysis Section and Force/Displacement Curve for Stepped Wall Section**

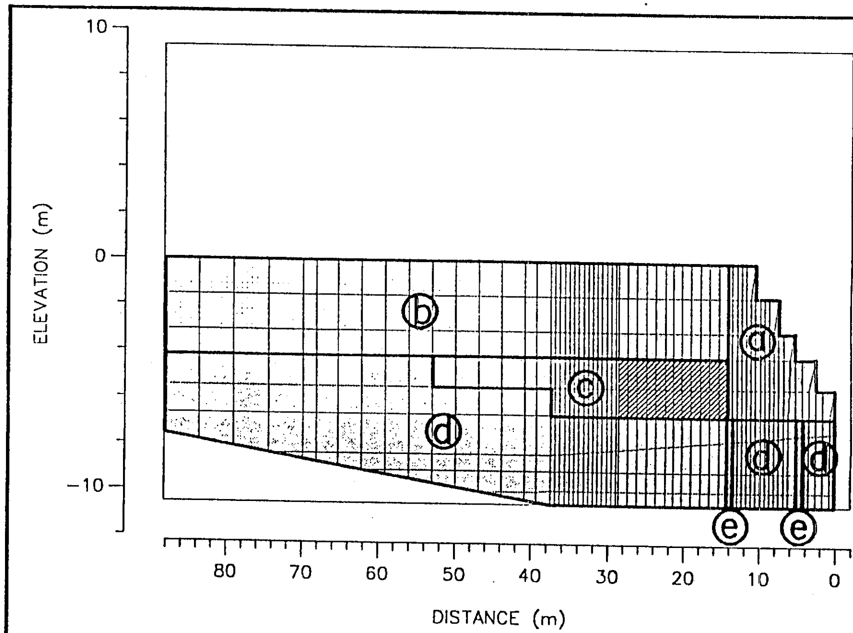


Fig.4a Underformed Mesh

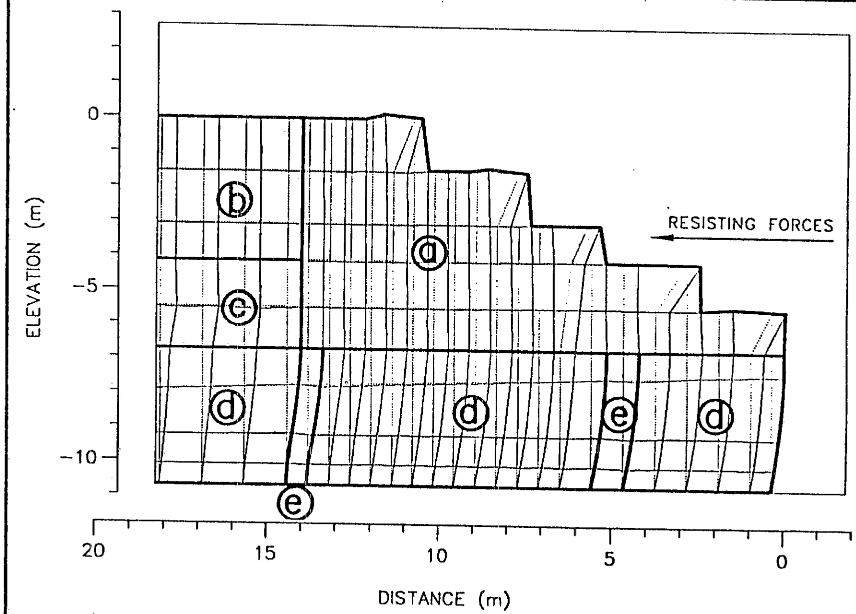


Fig.4b Deformed Mesh (2 x Exaggeration) for external force = 835kN/m

a Dense Sand
b Sand Fill above Water Table
c Liquefied Sand
d Clayey Silt
e Stone Columns

	a,b		c		d		e	
	BEQ	AEQ	BEQ	AEQ	BEQ	AEQ	BEQ	AEQ
Ø	34	34	34	0	0	0	38	38
c KP_a	0	0	0	7.5	30	25	0	0
K_G	200	200	200	0.15	100	1.5	400	400
n	0.5	0.5	0.5	0.0	0.4	0.5	0.5	0.5
K_B	2000	2000	2000	2000	2000	2000	2000	2000
m	0.25	0.25	0.25	0.25	0.25	0.25	.25	.25
R_f	0.7	0.7	0.7	0.001	0.7	0.001	0.7	0.7
γ_T kN/m^3	19	19	19	19	18	18	19	19

$$G_{max} = K_G P_a \left(\frac{\sigma_m}{P_a}\right)^n$$

$$M_B = K_B P_a \left(\frac{\sigma_m}{P_a}\right)^m$$

P_a = Atmospheric Pressure

BEQ = Before EQ

AEQ = After EQ

Figure 4 SOILSTRESS Finite Element Mesh (Stepped Wall Section)

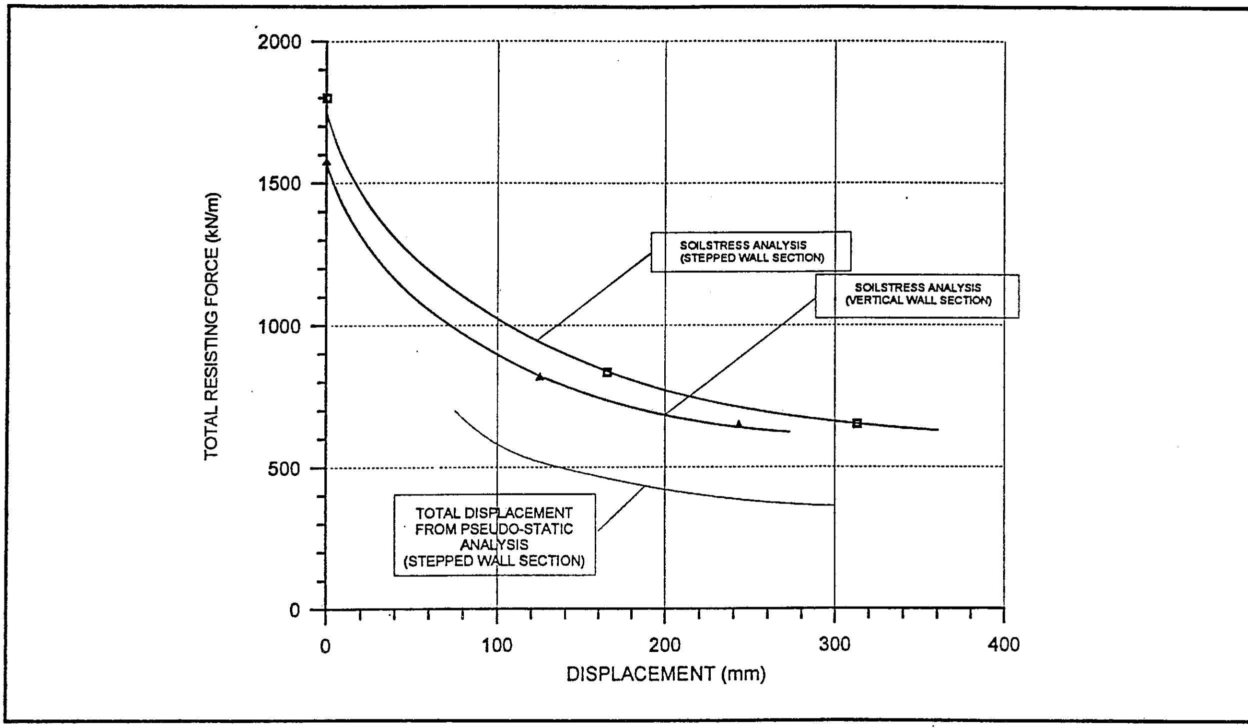


Figure 5 Force/Displacement Curves from SOILSTRESS analysis

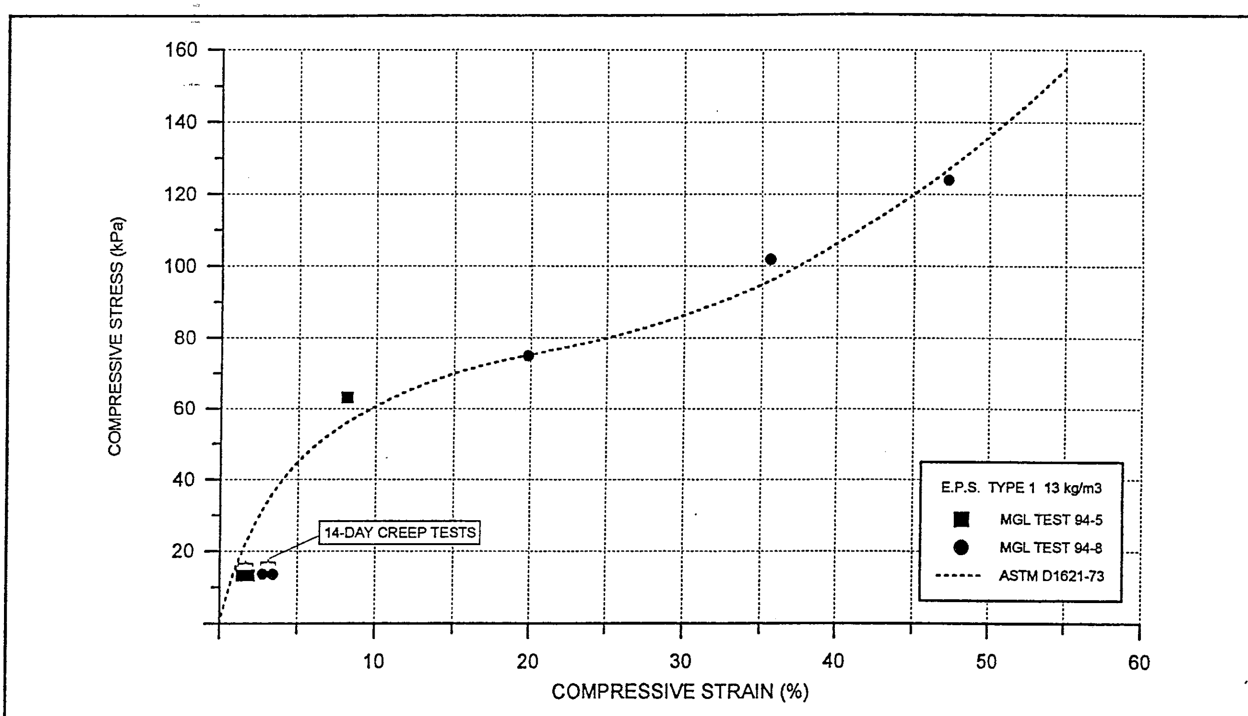
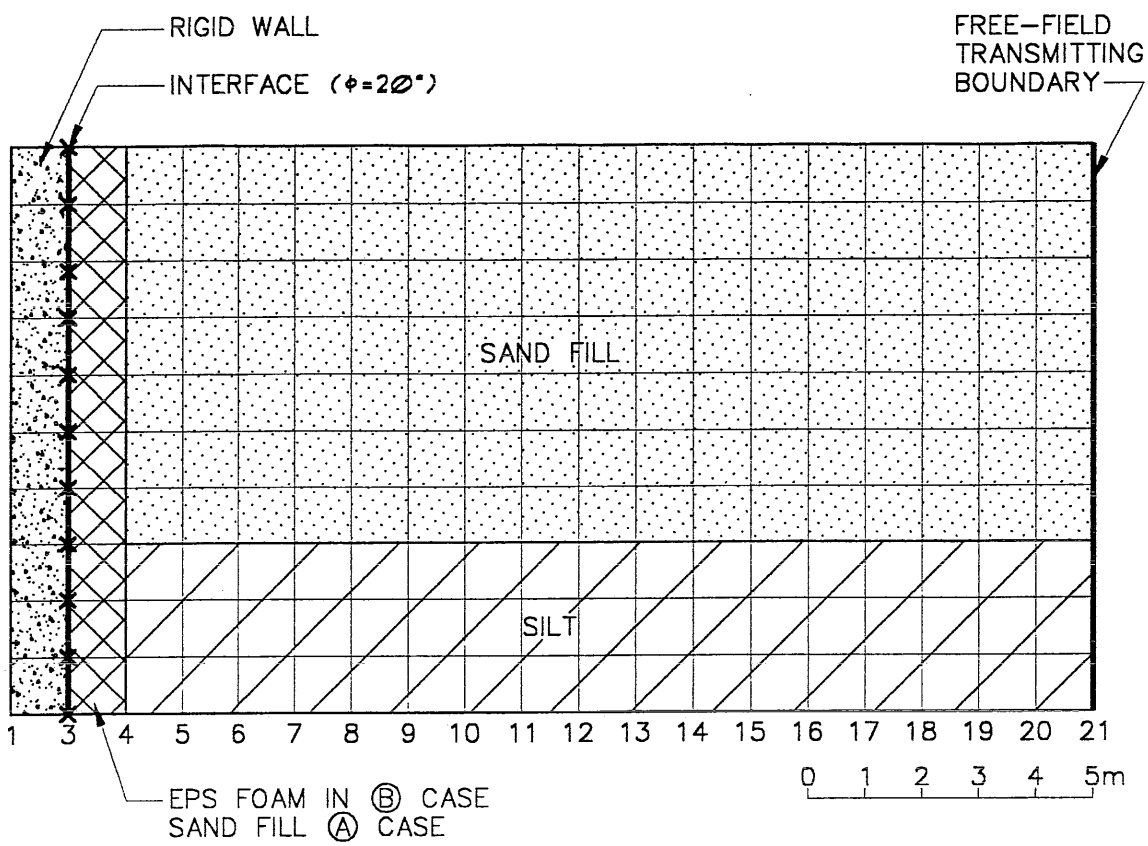


Figure 6 Results of Rapid Loading Compression Tests on EPS, with ASTM Test Curve for Comparison



	G kPa	K_B kPa	φ	C kPa
SAND FILL	20,000	200,000	34	0
SILT	10,000 TO 160 OVER 3 CYCLES	200,000	0	30 TO 2.5
EPS FOAM	900	600 PLASTIC VOLUMETRIC STRESS/STRAIN AS GIVEN IN FIGURE 6	0	50

SEISMIC MOTION INPUT AS A VELOCITY SINE CURVE OR EARTHQUAKE TIME HISTORY APPLIED TO THE BASE AND RIGID WALL.

Figure 7 Mesh and soil properties used for FLAC dynamic analyses

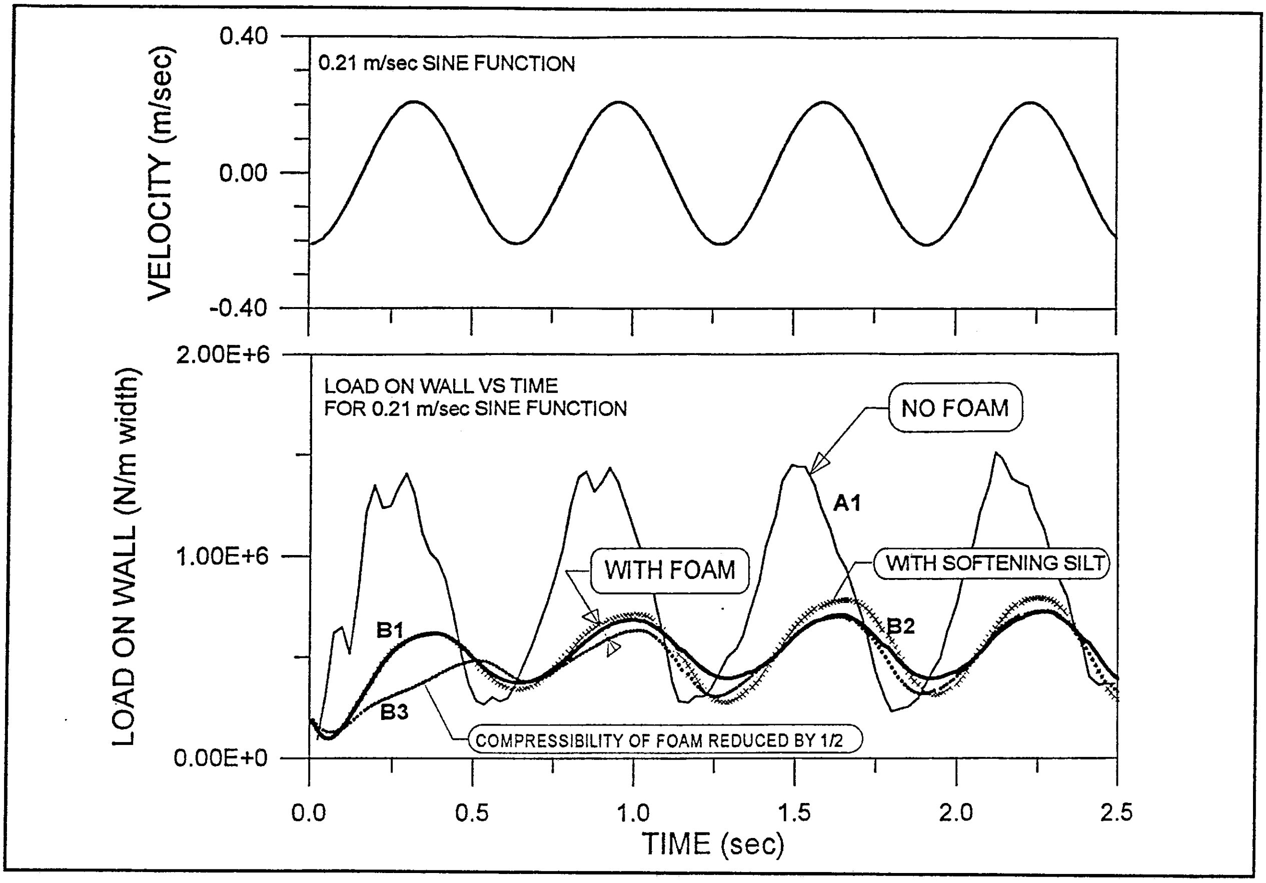


Figure 8 Input sine function and lateral load on wall versus time from FLAC analyses

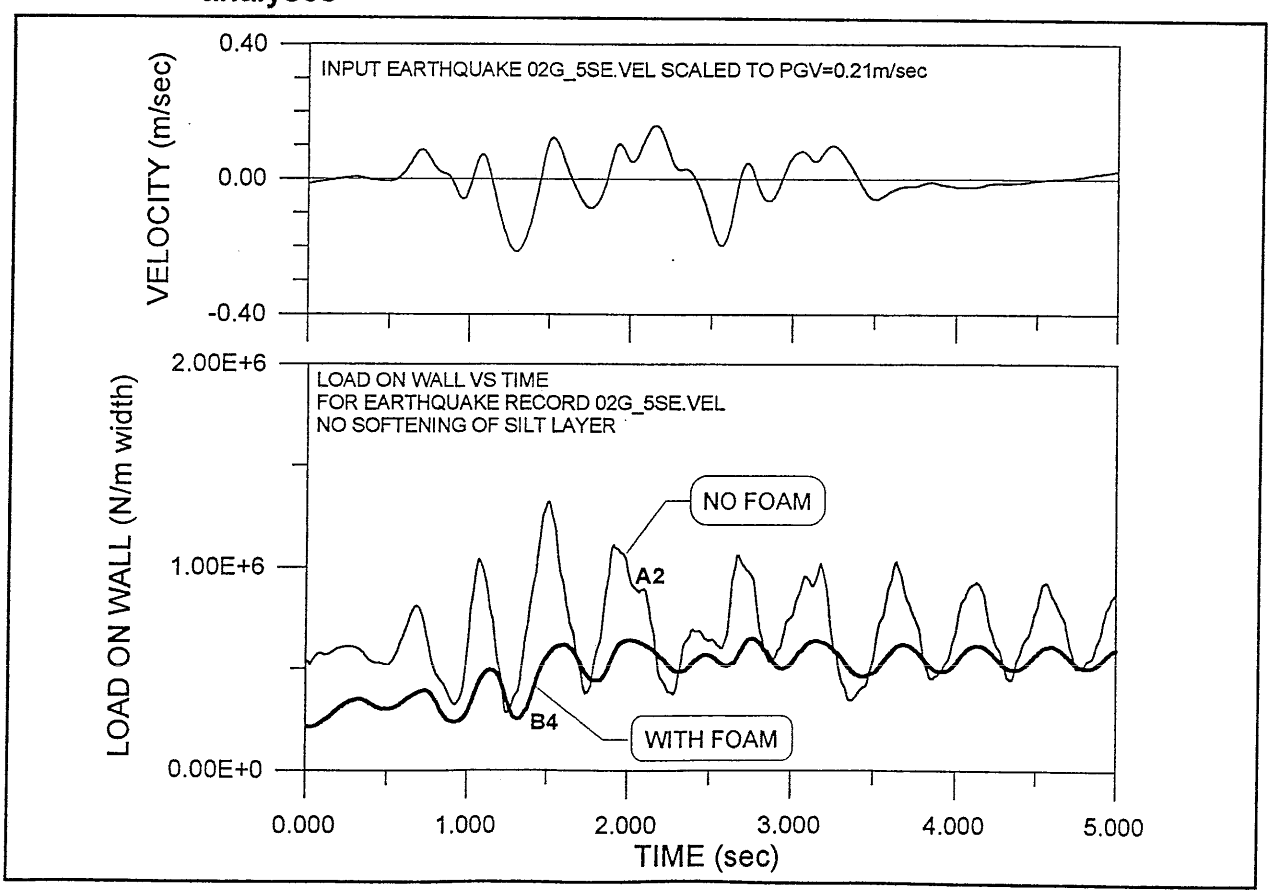


Figure 9 Input time history and lateral load on wall versus time from FLAC analyses

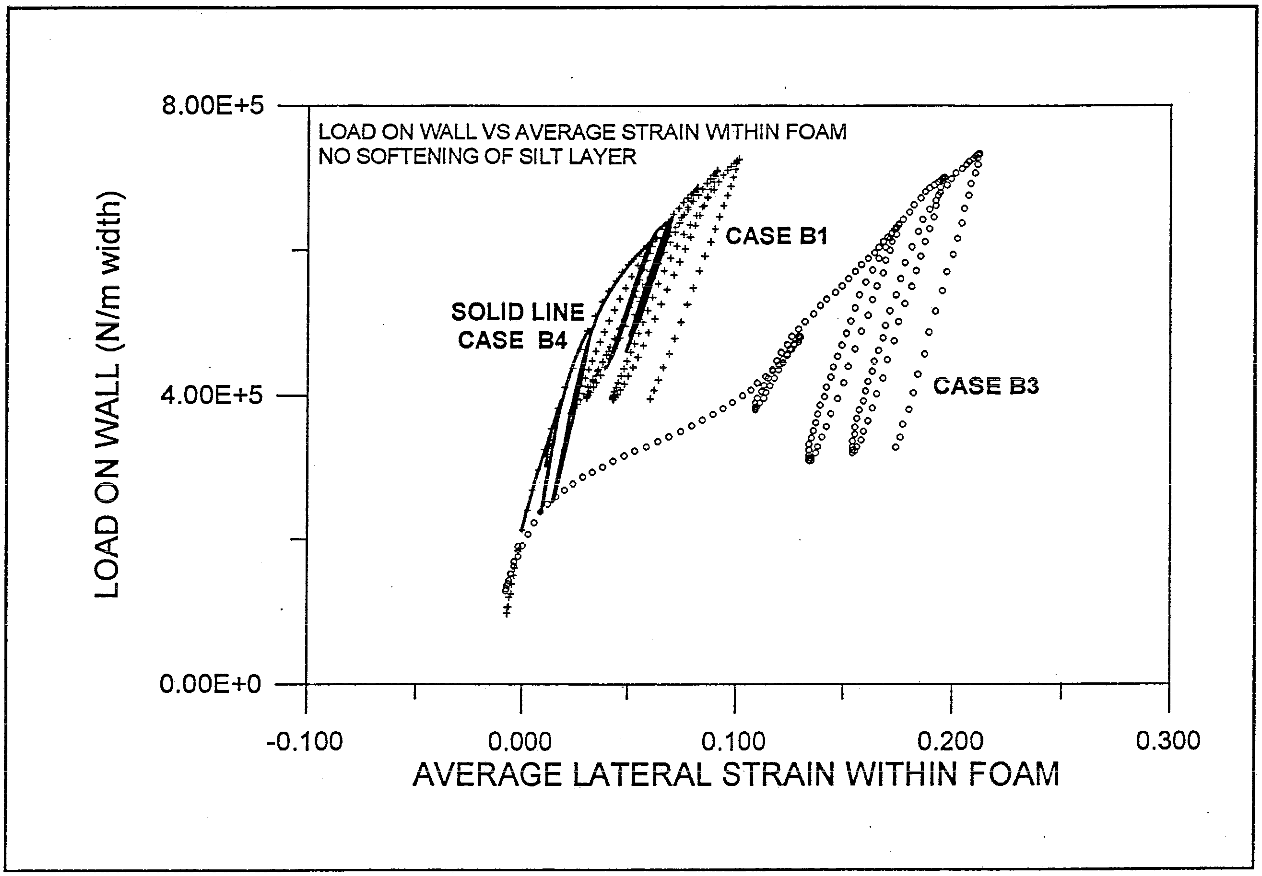


Figure 10 **Load on wall versus average lateral strain within foam buffer layer from FLAC analyses**