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SYNOPSIS: This paper describes a study performed to evaluate the seismic behaviour of a 10 m high bridge end sand fill placed upon soft organic foundation soils and supported on piles. Under static conditions the fill load is essentially carried by the piles by "arching action", and little deformation was observed to occur in the field. The results of both model tests and finite element analysis are in agreement with this finding. Concern arose as to the likely response of this structure under earthquake loading and a model sand embankment supported on 400 model piles was built and tested on the shake table. The model and testing procedures are described in some detail in the paper.

The results of the shaking table study indicate that during shaking the load is transferred from the piles onto the foundation resulting in large deformations of the fill. Analysis of the model tests based on this assumption gave deformations that were in good agreement with observed settlements. A similar analysis of the prototype indicates that seismic loading sufficient to cause such transfer would result in a settlement of the fill of about 0.4m, and that deformation would cease once the shaking stopped.

INTRODUCTION

The purpose of the shaking table test series was to evaluate the seismic behaviour of proposed bridge end fills placed upon a soft foundation soil and supported by piles. The fills comprise of compacted sand 10 m high overlying about 15 m of soft organic soil. Because such soil could not support the weight of fill, piles which are driven through the soft soil and which project into the fill are used to transfer the load to a bearing layer beneath the organic soil. Experience indicates that under static conditions the fill load is carried by the piles by an arching mechanism within the sand so that little or no deformation occurs. However, under earthquake loading the concern is that such arching would break down causing the fill load to be transferred from the piles to the soft foundation soil resulting in large deformations. The prime purpose of the shaking table test was to investigate this concern.

The shaking table test was performed on a 1 to 65 scale model. The soft organic soil was modelled by a gelatin based solution while the embankment was modelled by a compact sand. The piles were modelled by lengths of 6.4 mm diameter aluminum tubing. The model was built within a perspex box which was bolted to the shaking table and subjected to a series of uniform cycles of acceleration simulating seismic excitation. The model was instrumented and videotaped allowing its response to be observed. This paper describes the model test, the test results and their implication for analysis and design of the prototype fills.

DESCRIPTION OF TEST APPARATUS AND MODEL

The test was performed on a 1.2 m by a 2.7 m shaking table of welded aluminum and weighing 4.5 kN. It was supported by two v-slotted needle bearings and two flat bearings capable of moving in a horizontal direction. An MTS Earthquake Simulator console provided a controlled acceleration motion to the table. A reference accelerometer was located on the table.

The model was constructed in a plexiglass container bolted securely to the shaking table. The dimensions of the box were 1.84 m x 0.91 m x 0.6 m as indicated on Figures 1a and 1b. The soft organic foundation soil was modelled by a gelatin solution. The gelatin had a shear strength of 2.2 kPa. Such a shear strength was considered appropriate since it provided a

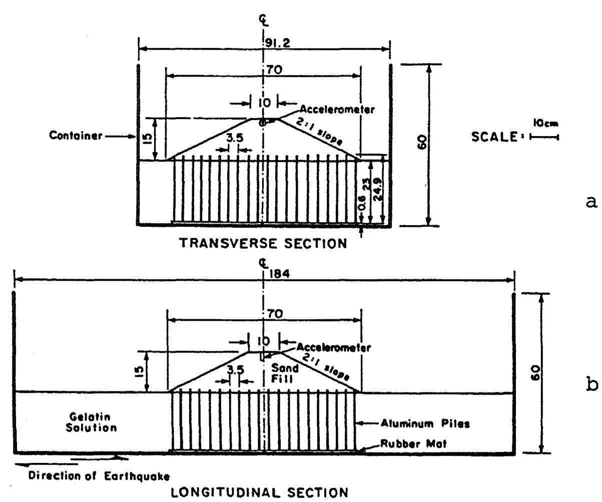

FIG. 1: Details of Model Set-Up.

reasonable factor of safety against a bearing capacity failure upon placement of the sand fill. The Young's modulus of the gelatin solution was approximately 10 kPa. Details of the type of gelatin and its preparation are outlined in Appendix A.

The test embankment was modelled by a uniformly graded fine to medium grained sand. The sand had a specific gravity of 2.7 and maximum and minimum void ratios of 0.9 and 0.5 respectively. When in place, the sand was estimated to have an average density of 16.2 kN/m^3, a void ratio of 0.60 and a relative density of 75%.

A schematic representation of the fill cross-section and the location of the accelerometer are also shown on Figure 1. For the given size of the box, it was decided that a scaling factor of 1/65 would be the most appropriate since it places the fill at a reasonabe distance from the end boundaries without reducing the model to an impractically small size.

The piled foundation proposed for the embankment was simulated by 400 piles held at the base by a fairly rigid rubber mat 0.6 cm thick. The piles were set out at 3.5 cm spacing over a mesh 70 cm x 70 cm. The piles comprised of 6.4 mm diameter, hollow aluminum tubes having a wall thickness of 0.9 mm, a bulk density of 16 kN/m^3 and a length of 25 cm.

Sequence of Construction

The rubber mat was positioned in the centre of the box and glued firmly to the base. The piles were placed into prebored holes in the mat which resulted in their being effectively held in a pin ended manner at the base. They were kept vertical during placement of the gel by means of a top plywood template through which locating nails were placed. This template was later removed when the gel had set. The gel was poured to 23.0 cm above the level of the rubber mat resulting in approximately 1.5 cm of the pile heads extending above the gel surface. The gelatin solution was then left for 5 days to allow it to fully set and attain its peak strength.

After this waiting period, the sand embankment was constructed on top of the gel by placing sand inside a pyramid shape former. The form was placed centrally over the piled area and sand was poured in four equal lifts, ensuring that each lift was uniformly placed with no visible gaps between the mold and the slopes. This necessitated compacting the sand as it was being pressed by hand into the corners of the forming mold. The form was then removed leaving a truncated pyramid shaped sand embankment with cross-sections as shown on Figure 1.

The surface of the sand was sprayed with a thin coat of black paint for the following reasons:

(1) by binding the sand particles, the possibility of loose sand grains rolling down the slope was reduced;

(2) zones of large movements could be detected from formation of cracks on the painted surface;

(3) the thin crust provided a good surface for measuring the fill profile using a dial gauge;

(4) the black surface produced a better photographic image of the fill.

Instrumentation

The principal instrumentation consisted of accelerometers mounted on the table and on the crest of the sand fill. A miniature piezo-electric accelerometer was used for the latter. The output of the accelerometers was filtered before being amplified and recorded on a oscilloscope. The filtering of high frequency noise levels was essential, particularly at low levels of excitation which would have otherwise been dominated by those higher frequencies. A cut-off frequency of 6 Hz and 20 DB noise cut out was used throughout the experiment.

The deformation of the test fill was measured by means of a dial gauge attached to the end of a graduated bar moving against a fixed vernier scale. This bar was then mounted on wheels over two parallel bars which in turn were fixed on wheels over two outer bars bolted firmly to the top of the container. Such an assembly enabled the dial gauge to be moved longitudinally and transversely over the fill. The profile was determined by lowering the dial gauge until it came into contact with the surface. Measurements were taken on a grid spacing of about 10 cm.

The surface movements were also monitored by video and movie cameras.

TESTING PROGRAM

The model was tested by subjecting it to a cumulative series of constant amplitude sinusoidal base acceleration motions. Each series comprised of about 15 cycles and the amplitude of acceleration was increased in each subsequent series. A total of 8 series were performed on the one model with base acceleration amplitudes of 2.5, 7.5, 10, 12.5, 15, 20 and 30 percent g.

A frequency of 3 Hz was used for all of the above test series. This frequency was selected based upon a preliminary low level amplitude shaking test. A base acceleration of 1% g was used for this low level preliminary test and the crest acceleration was monitored allowing the dynamic amplification factor, which is the ratio of the crest to base acceleration, to be computed. The results are shown in Table 1 and indicate the dynamic amplification factor increases from 1.4 at 1.0 Hz to 3 at 4.5 Hz. There were no dramatic changes in amplification with frequency and consequently a convenient testing frequency of 3 Hz was selected.

Vertical displacements at the surface of the embankment were taken over a grid spacing of approximately 10 cm, covering the entire geometry. Measurements were taken prior to shaking and after the termination of the test. Some measurements were also taken after the application of 10% g base acceleration. A complete list of the readings (Table B1) and the locations where measurements were taken (Figure B1) are presented in Appendix B.

Table 1.

Response of the sand embankment to a base acceleration of 1% g.

Test Number	Applied Base Displacement (mm)	Frequency (Hz)	Induced Acceleration in the Fill (% g)	Amplification Factor
1	3.61	1.	1.4	1.4
2	0.13	1.6	1.4	1.4
3	0.08	2.0	1.5	1.5
4	0.05	2.6	1.6	1.6
5	0.03	3.0	1.7	1.7
6	0.02	3.4	1.8	1.8
7		4.0	2.0	2.0
8		4.5	3.0	3.0

Table 2.

Values of the base excitation and the induced acceleration in the Fill.

Frequency (Hz)	Number of Cycles	Applied Base Displacement cm	Applied Peak Base Acceleration	Peak Induced Acceleration in the Sand Fill	Amplification Factor = Fill Acceleration Base Acceleration
3	15	0.05	0.02 g	0.03 g	1.5
3	15	0.13	0.05 g	0.08 g	1.6
3	15	0.19	0.075 g	0.12 g	1.6
3	15	0.25	0.10 g	0.21 g	2.1
3	15	0.32	0.125 g	0.22 g	1.7
3	15	0.37	0.15 g	0.24 g	1.6
3	15	0.52	0.20 g	0.26 g	1.3
3	15	0.79	0.30 g	0.33 g	1.1

During the test series, it was observed that the movements were predominantly deep seated resulting from movements in the gel. However, some downslope movement of sand grains beneath the surface skin of paint also occurred. Movements began to manifest themselves at a base acceleration of 5% g and became significant at 10% g. At that point, the crest of the embankment had deformed approximately 2 cm, and the piles in the central region around the perimeter of the mesh had deflected outward by approximately 1 cm. The sand level at the toe of the fill increased. This could have resulted from either heave of the gel or from downslope movement of sand or a combination of the two effects. Increasing the amplitude of the base motion resulted in further movements in both vertical and lateral directions. Base excitation of 30% g led to large movements with crest deformation of over 5.5 cm and lateral spreading in the toe areas of approximately 10 cm in the direction of the applied motion. Lateral movements and spreading of the sand also occurred in the direction normal to the base motion and was approximately half the above magnitude.

From the post-failure examination of the embankment and upon its removal it was observed that significant deformations had occurred beneath the base of the fill. In addition, approximately 0.5 to 1 cm intrusion or penetration of the sand into the gel's surface had occurred. Such penetration is not likely to occur in the prototype foundation material. However, in relation to the total amount of deformation (5.5 cm) this contributes not more than 20% of the total movement and so has only a minor effect on overall movements beneath the embankment. The deformation of the surface of the gel was not monitored during shaking. However, it appears that the major portion of the 5.5 cm of crest settlement was due to deformation of the gel rather than penetration of sand into the gel.

The sequence of movements of a section of the model prior to dynamic testing, after 10% base excitation and after 30% base excitation are depicted in Figure 2. These movements are based upon surface movement measurements, video and movie recordings, and visual observations during and after testing together with volume compatibility constraints. Details of the calculations involved are shown in Appendix C. Prior to dynamic testing, the piles at the toe of the embankment had moved outward about 1 cm due to construction of the fill. Such outward movement would induce downward movement of the sand-gel interface as shown. Crest deformation of 2 cm resulted after application of the 10% g base excitation motion. Additional outward movement of the toe piles together with movement of the gel surface occurred at the same time as shown in the Figure. The profile after 30% g is also shown in Figure 2 and indicates a crest movement in excess of 5 cm and lateral spreading of the fill of some 10 cm. The lateral movement of the pile heads at the toe of the slope was about 4.5 cm or about half the spreading movement of the

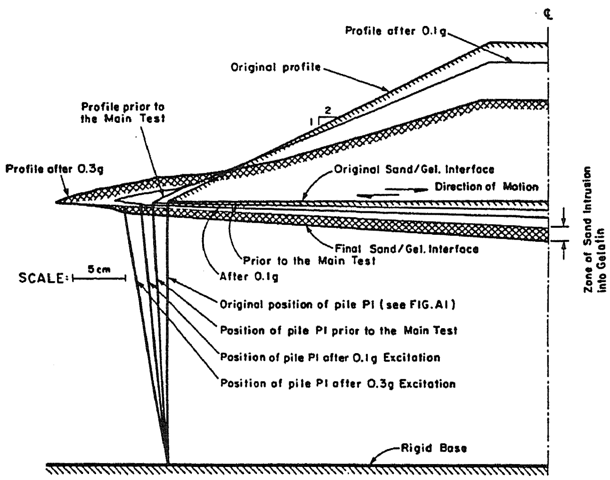

FIG. 2. Deformed Profiles at Various Stages of Shaking (Section 5-5, Fig. B1).

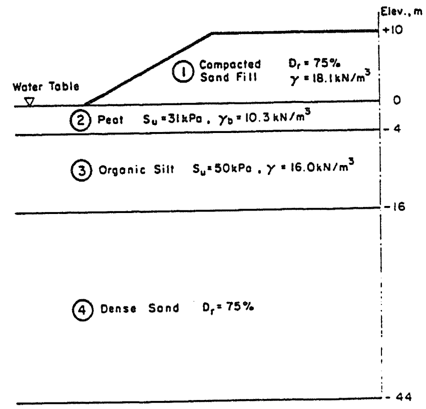

FIG. 3. Prototype Soil Profile.

sand. The additional movement of the sand was caused by sand particles moving down the surface of the slope and rolling outward in the toe area.

APPLICATION TO PROTOTYPE FILLS

The base acceleration level of 10% g causes a crest acceleration of 21% g and corresponds approximately with the design earthquake. The model tests suggest a crest settlement of about 1.8 cm or 12% of the height of the model embankment for this level of excitation. For the prototype with an embankment height of 10 m, deformations corresponding to 12% of the height would result in a crest settlement of 1.2 m.

Prior to the shaking most of the weight of the embankment is carried by the piles as a result of an arching mechanism in the sand. The observed vertical movements recorded during the shaking table tests suggest that the arching is gradually lost with increasing levels of shaking. Thus during shaking the vertical load is gradually transferred from the piles to the soil foundation and results in large deformations.

Deformations of the foundation soils after the shaking ceases will be small because such movements will cause the load to be transferred back to the piles and thus deformations should essentially cease after the shaking ceases. The model test results are in agreement with this hypothesis.

Seismic deformations of the fill are therefore mainly due to deformations of the foundations caused by the transfer of vertical load from the piles to the foundation during the period of shaking. The maximum seismic deformations can be estimated for the prototype from a static finite element analysis in which all of the weight of the sand embankment is assumed to be transferred to the foundation soil during the shaking. Under these conditions the organic soil would behave in an undrained manner. Based on the soil profile and properties shown in Figure 3, a crest settlement of 0.4 m together with a horizontal longitudinal displacement of 0.4 m was predicted for the prototype fills. Details of the analysis are included in Appendix D.

Earthquake induced movements could be reduced by placing a reinforcing cloth horizontally above the pile tops. However, it would be prudent to assume that the same loss of arching would still occur with the vertical load being gradually transferred to the cloth and thence to the pile. In the limit, all of the weight of the soil would be carried by the cloth and the deflection of the cloth could be determined by analyzing a system comprised of a cloth blanket draped over the pile tops and subjected to a downward pressure equal to the weight of the soil above the blanket. The deflection of the cloth would lead to crest deformations and such deformations could be estimated from analysis. Results from finite element analyses of the above soil-structure interaction problem indicate that the placement of a reinforced cloth considerably reduces both vertical and horizontal deformations of the embankment (Brown (1986)). This is in accord with field observations on a prototype structure in which such a reinforcing cloth was incorporated (Brown (1986)).

Horizontal movements of the piles will induce shear forces and bending moments in the piles. These loads can be computed as outlined by Byrne et al. (1983).

CONCLUSIONS

The model tests indicate that horizontal base excitation comparable to the design earthquake will induce settlements of the order of 12% of the height of the embankment. Such deformations result from a transfer of vertical load from the piles to the foundation soil as a result of a loss in arching due to shaking. Finite element analyses indicate that such a transfer of load will cause a deformation of 0.4 m for the prototype fills. A reinforcing cloth placed above the pile tops would reduce such settlements.

The horizontal earthquake induced loads can be carried by the piles provided they can withstand the earthquake induced displacements. Such displacements can be computed as outlined by Byrne et al. (1983).

ACKNOWLEDGEMENTS

Dr. R.G. Campanella's advice and comments on the model testing were most helpful. The figures were drawn by Mr. R. Brun and Mrs. K. Lamb typed the manuscript.

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APPENDIX A - PROPERTIES OF GELATIN

The gelatin used was a Bloom type 100. The Bloom unit is a measure of the force required to depress a prescribed area of the surface of a gelatin sample which is made up from a given concentration and according to a standard procedure, a set distance of 4 mm. It is simply a measure of the rigidity of the gel (1). The strength of the gel formed depends upon concentration and the intrinsic strength of the gelatin used which is a function both of structure and molecular weight. Commercially available gelatin range in Bloom between 100 to 275 and generally the higher the Bloom grade, the less gelatin will be required to produce the same strength. However, a minimum quantity of 3% gelatin is required to form a gel solution at room temperature and this largely dictates the choice of gel type if we are interested in developing solutions with low shear strength values. The gelatin solution used was made up of 6% gelatin crystals by weight. The solution was prepared at approximately 65°C which is the water temperature at which the gelatin crystals are dissolved quite readily and also result in the most stable composition. The graph showing the change of strength with time for a gel prepared in this manner is shown on Figure A1. It can be observed that the peak strength is achieved after approximately 4 days, beyond which degradation occurs by bacteria and molds. The nature of the organisms which grow in gelatin solutions depend upon a number of factors. However, it was found that the rate of growth is significantly smaller for samples prepared at 65°C in comparison with other techniques of dissolving the gelatin crystals. Extreme care was exercised to ensure that the crystals were thoroughly dissolved before removing the foam and pouring the solution into the container. The solutions were prepared in 30 litre batches and the total amount (approximately 400 litres) was poured in place within 2.0 hours. Using a miniature vane device, the shear strength of the gel at the time of the experiment was measured to be 2.2 kPa. Such a shear strength was considered to be appropriate since it provided a reasonable factor of safety against a bearing capacity failure upon the placement of the sand fill as well as leaving some margin of safety for possible weakening of the gel peak strength prior to the experiment. The Young's modulus of the material was approximately 10 kPa and was determined by measuring the depression due to a uniform load applied to the gel's surface.

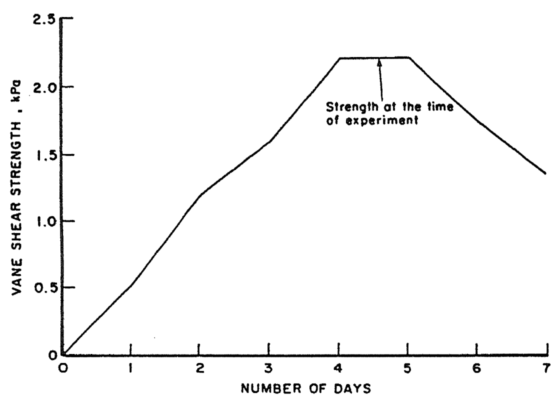

FIG. A1. Change of Shear Strength with Time for 6% Gelatin Solution Prepared at 65°C.

(1) Gelatin Manufacturers Institute of America, Inc., Standard Methods for Sampling and Testing of Gelatins, New York, 1964.

APPENDIX B - RECORDED MOVEMENTS

The recorded vertical displacements after 0.1 g and 0.3 g base excitations are shown in Table B1. The recorded horizontal displacement are shown in Table B2. The locations of the recording points are shown in Figure B1.

Table B1. Deformation of the Points Shown on Fig. B1.

Section	Cumulative Vertical Displacements (Settlement + ve)/cm	
	After 0.1 g	After 0.3 g
B1	-0.30	-0.76
B2	0.40	1.71
B3	0.61	2.13
B5	0.37	1.80
B7	0.55	1.80
B9	0.88	2.26
C1	-0.64	-1.25
C2	0.18	1.28
C3	1.31	3.60
C5	1.28	3.81
C7	1.46	3.90
C9	0.37	1.07
D1		-1.22
D2		0.98
D3		x
D4		5.73
D5		5.57
D6		5.36
D7		3.57
D8		2.23
D9		0.88
E1		-0.76
E2		0.91
E3		3.17
E4		5.64
E6		5.55
E7		3.57
E9		1.07
F1	-0.37	-0.88
F2	0.0	0.91
F3	x	x
F4	2.01	5.52
F5	x	5.61
F6	1.89	5.70
F7	1.74	3.47
F8	x	2.44
F9	x	1.158
G1		0.12
G2		1.25
G3		3.60
G5		3.05
G7		2.99
G9		1.46
H1	-0.20	0.30
H2	0.70	3.11
H3	0.12	1.16
H5	0.30	2.04
H7	x	2.38
H9	0.12	1.92

Table B2. Movement of the piles shown on Figure A1.

Cumulative Lateral Movement of the Piles from the Original Position				Lateral Movement During Each Phase	
Pile	After Embankment Construct cm	After 0.1g cm	After 0.3g cm	Up To 0.1g cm	Between 0.1-0.3g cm
P1	1.2	2.1	3.5	0.9	1.4
P2	1.7	2.6	4.1	0.9	1.5

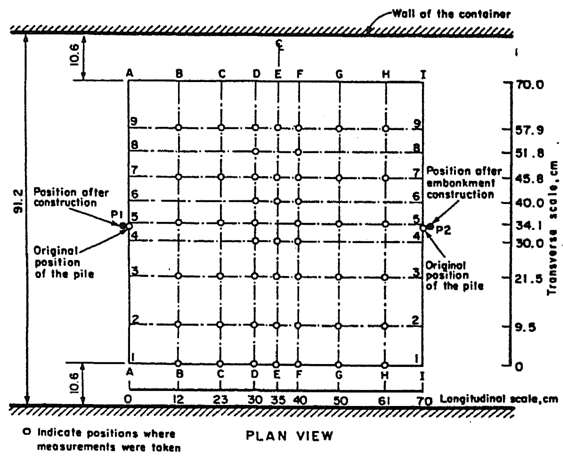

FIG. B1. Location of the Points Where Movements were Monitored.

APPENDIX C - BASE MOVEMENTS FROM VOLUME COMPATIBILITY

Based on observation and the measurements taken, the surface profile of the fill was determined. Knowing the deflection of the piles, and assuming that the piles and gelatin move together, the lateral component of the movements was obtained. If it is assumed that there is no change in the overall volume of the sand fill, analysis based on compatibility of volumes can be performed to establish the deformation profile at the base of the embankment. Such analysis require a trial and error balancing of areas. The estimated base profile at the end of the shaking (after 0.3 g) is shown in Figure C1. The areas involved were as follows:

Zone 1) Settlement of the area below the surface of the fill + = 93cm^2

Zone 2) Area involved in the lateral movement of the piles (2.3 x 23.5/2) - = 27 cm^2

Zone 3) Area of the sluffed portion beyond the face of the slope - = 14 cm^2

Zone 4) Area settled beneath the base of the fill (45.5 x 1.6/2 - = 37 cm^2

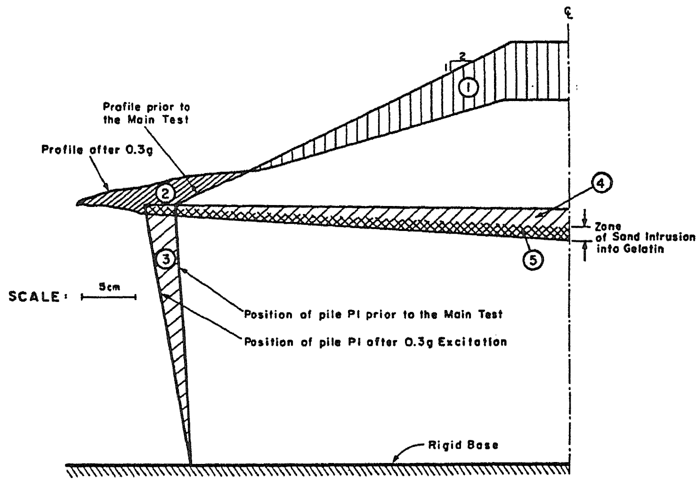

FIG. C1. Deformed Sand-Gel Interface Based on Volume Compatibility.

Zone 5) Area of sand intruded into the gel. $A_5 = 1/2(1.0 \times 45.5) = 22.8$ cm^2. Because the sand pores in this area are now filled with gel, the actual area displaced = $1/1+e\ A_5 = 1/1.5\ (22.8) - = 15$ cm

The summation of the zones 2,3,4 and 5 balance the downward movement given by zone 1.

APPENDIX D - ANALYSIS OF EARTHQUAKE INDUCED DISPLACEMENTS OF THE PROTOTYPE FILLS

The model studies indicate that the effect of seismic shaking is to break the arching effect by which the sand fill load is carried by the piles, with the result that the load is gradually transferred from the piles to the foundation soil as shaking proceeds. A worst case is to assume that all of the fill load must be taken by the foundation soil and that the piles carry no vertical load. This condition will exist during the shaking. After the shaking stops, the load will again revert to the piles as further settlement occurs either due to creep or settlement effects. Therefore the load transfer condition exists only for a short period of time and can be simulated assuming the fill must be carried by the foundation using undrained stress-strain relations for the organic soils.

A typical soil profile was shown in Figure (3) and the stress-strain parameters in Table D1. These parameters define a hyperbolic stress-strain curve that is now commonly used in practice and has been discussed by many researchers including Byrne and Janzen 1981. The parameters for the peat and organic clay were obtained from laboratory tests in which samples were first subjected to cyclic loading simulating earthquake loading and then tested in undrained shear to obtain their post cyclic stress-strain behaviour. The laboratory post-cyclic stress-strain curves are shown in Figure D1. The stress-strain parameters for sand were based on Byrne and Eldridge, 1982.

The earthquake induced displacements were computed using the finite element method and assuming that all of the fill load must be carried by the foundation soil. The computer program SOILSTRESS (Byrne and Janzen, 1981) which performs an equivalent linear elastic analysis was used. The finite element mesh is shown in Figure D2. The computed displacement pattern is shown in Figure D3. It may be seen that the crest of the embankment settles about 0.4 m and also moves out horizontally about 0.4 m. Some heave is predicted to occur in the toe area.

Table D1

Hyperbolic Parameters Used in the Analysis.

Parameter	Material Number			
	1	2	3	4
k_g	356.	5.0	23.0	356.0
n	0.5	0.0	0.0	0.5
k_b	334.0	500.	2300.	334.0
m	0.5	0.0	0.0	0.5
ϕ	37.5°	0.0	0.0	37.5°
ϕ_{cv}	33.0	0.0	0.0	33.0
$\Delta\phi$	0.0	0.0	0.0	0.0
c(kPa)	0.0	31.0	50.0	0.0
R_F	.8	.8	.8	.8

Shear Modulus, $G = k_g P_a \left(\frac{\sigma_m}{P_a}\right)^n \left[1 - \frac{\tau R_F}{c+\sigma \tan\phi}\right]$

Bulk Modulus, $B = k_b P_a \left(\frac{\sigma_m}{P_a}\right)^m$

where P_a = Atmospheric Pressure
σ_m = the mean normal stress

See Byrne and Janzen (1981) for details.

Material No.	Description
1	Compact Sand
2	Peat
3	Organic Silt
4	Compact Sand

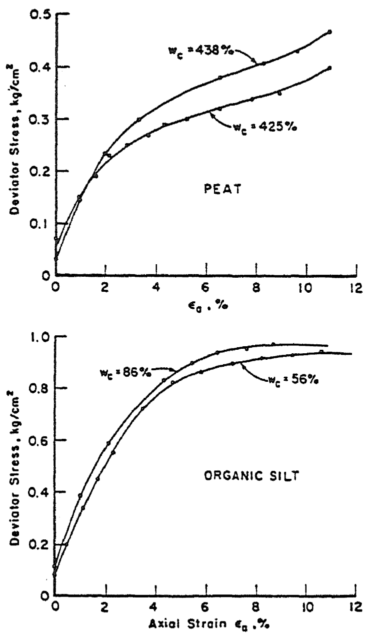

FIG. D1. Laboratory Post-Cyclic Stress-Strain Curves for Peat and Organic Silt.

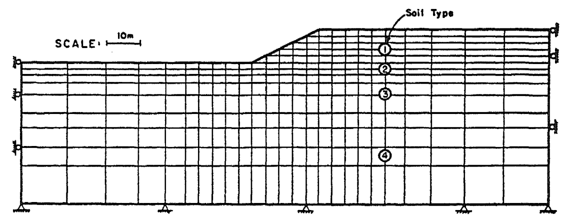

FIG. D2. Finite Element Mesh Used.

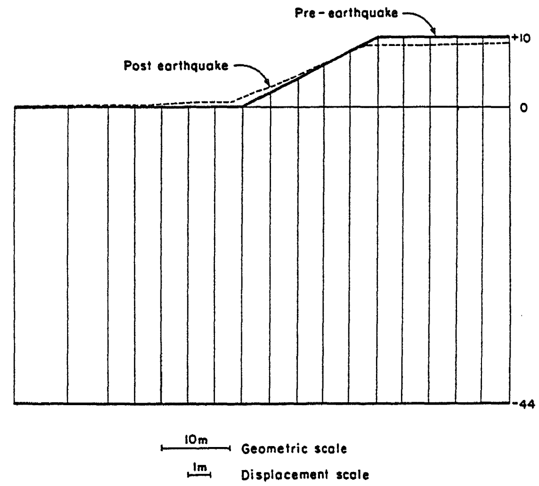

FIG. D3. Post Earthquake Deformation of Prototype.