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IDEALIZING REINFORCED EARTH EMBANKMENTS IN DYNAMIC ANALYSIS

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SUMMARY

Loose sandy soils can be strengthened by reinforcing. Micro-reinforced earth has ductility, enhanced elastic stress limit, suitable for analysis by linear methods of analysis, can be idealized to be homogeneous and is ideal for use in earthquake engineering. Free & forced vibration tests were carried out on large 1.5 m x 0.75 m x 0.9 m high reinforced embankments on shake table excited by D.C, motor-oscillator drive. Plane strain conditions were created in lab. From tests, strain dependent shear modulus were obtained. Response computed fron dynamic 2D FEM analysis of test embankments by idealising them with 1-layer and 2-layer system agrees well with response based on test data. This suppots idealisation of microreinforcd earth to be homogeneous.

INTRODUCTION

Dynamic studies on reinforced earth (RE) are recent with very few reported studies. It hould be idealised suitably in dynamic studies. Micro-reinforced earth may be idealised to be homogeneous. To confirm it, large RE embankments were tested using shake table to obtain strain dependent shear modulus. These embankments were analysed by using 2D FEM analysis for same excitation. Dynamic response computed agreed favourably with that based on test data. This confirms validity of idealising micro-reinforced earth to be homogeneous.

TEST EMBANKMENTS, SAND AND GEOTEXTILES USED

Test embankments, M1, M2 & M3 (1.5 m long, 0.75 m wide & 0.9 m high) are shear beams (radius of gyration of 0.432 m & slenderness ratio of 4.16 in direction of vibration) & were fixed to shake table. Transverse sides (parallel to plane of vibration) has 6 geotextile strips joined together & secured to 6 rigid facing elements on longitudinal face. One reinforcement (75 mm wide) in each of the 6 layers is secured suitably to facing element at its mid-height. To simulate free field conditions, plane strain conditions were created at transverse faces & hinges provided at base of lowest facing elements. M1 has all 6 geotextiles running continuous from one set of facing elements to other. M2 has such continuous geotextiles in top four layers. Those in bottom two layers are discontinuous at mid-length. M3 has continuous geotextile in top layer & remaining are discontinuous at middle.

Angular Solani sand (D_{10} 0.15 mm, D_{30} 0.19 mm, D_{60} 0.23 mm, maximum relative density, D_r, 1.75, minimum D_r 1.39 t/m3 & specific gravity 2.59) classified as SP (fine sand) as per Indian Standard Code of practice (IS:2720-1972) has angles of shearing resistance of 33.42^0, 35.44^0 & 44.44^0 at relative densities of 54, 62.2 & 70% respectively. Reinforcements & transverse embankment cover were polypropylene woven geotextile with following properties supplied by manufacturers of cloth: specific gravity 0.91; weight per square meter 276 grams; breaking strength 245.7 kg warp way & 182 kg weft way using 50 mm x 200 mm specimens; elongation at break point 46.9% warp way & 27.8% weft way; grab strength (76.2 mm x 25.4 mm specimen, ASTM-D-1682) 214.8 kg warp way &152.8 kg weft way; tear strength (single grip, ASTM-D-2261) 21.2 kg warp way & 18 kg weft way; mean pore size of 25 microns and maximum pore size of 69 microns.

TEST SET UP

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Sand rain apparatus, suspended at a specified distance above, rained sand simultaneously over whole test bed (**Fig. 1**). Maximum relative density, D_r, achievable was 76.64%. Embankment was formed in 75 mm thick layers. In a layer, 14 containers measured densities to assess uniformity. Mean standard deviation in density was 1.08 %, which is better than 1.47% reported by Pasalacqua [Pasalacqua, 1991] & 2.39% worked out for data reported by Fairless [Fairless ,1989]. Largest % deviation was 0.76%. Average % deviation 0.47% (better than 1% reported by Pasalacqua). Horizontal shake table (2 m long, 1 m wide) moving on rails, is excited sinusoidally by a mechanical oscillator-5 H.P. D.C. motor assembly is secured to table from below (**Fig. 2**). Motor runs from zero to desired speed. Setting eccentricity of oscillator masses varies dynamic force. Six plywood formwork boxes placed one over other in stages were used to construct embankment. Facing elements were part of formwork during construction. Reinforcements were tied to facing elements at mid-height using nuts and bolts.

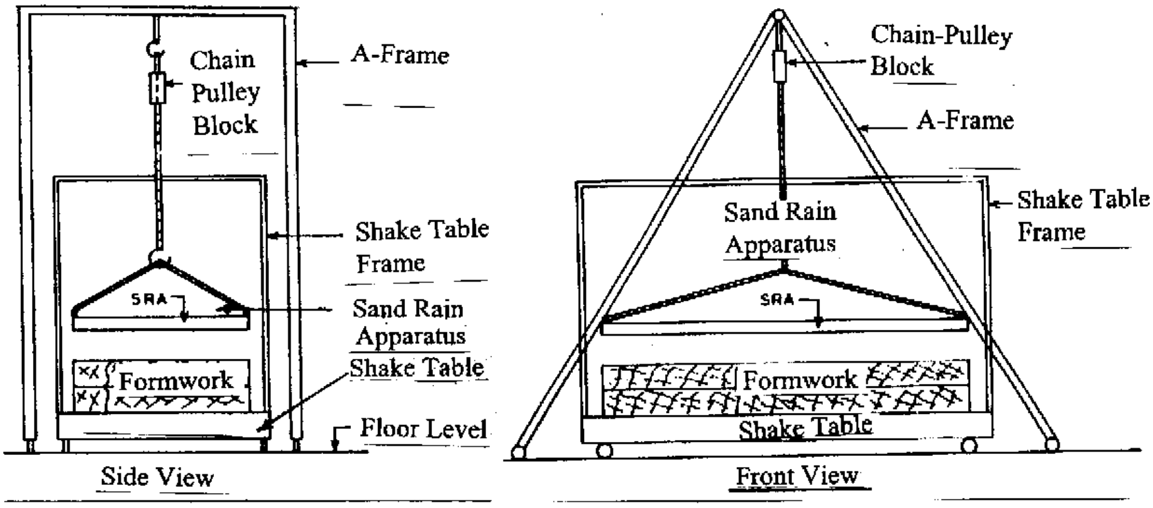

Fig. 1 Schematic Diagram of Sand Rain Apparatus.

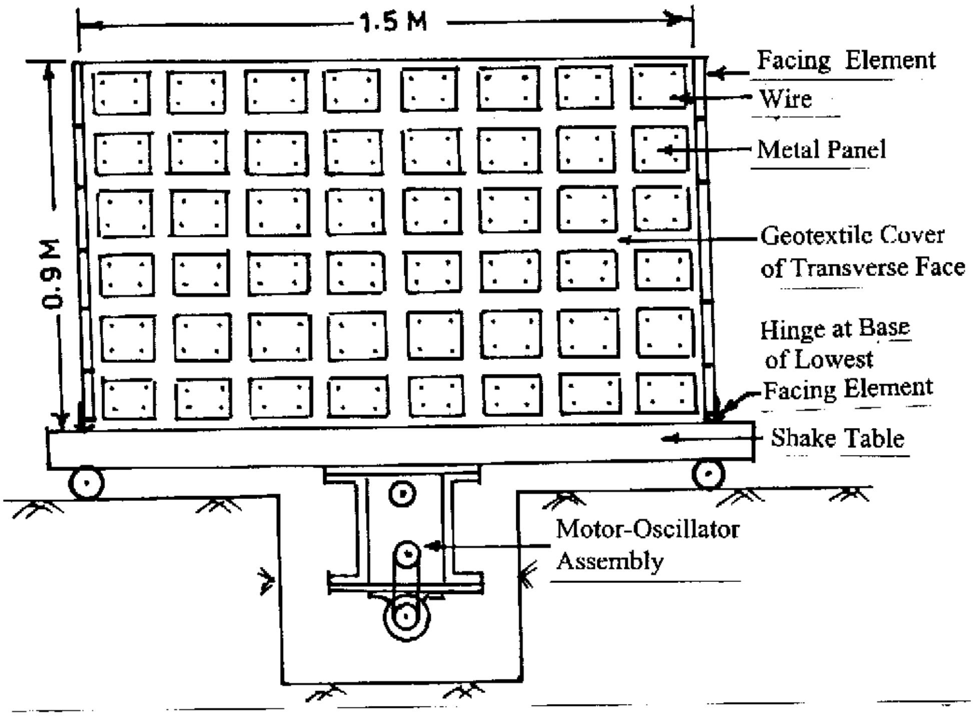

Fig. 2 Schematic Diagram of Shake Table with Transverse View of Test Embankment.

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o create plane strain condition on transverse planes, 8 metal panels (180 mm x160 mm x1 mm thick) secured to outer face of geotextile transverse coveres of each of the 6 layers of embankment. Each panel was connected to corresponding one on opposite face by 1 mm diameter steel wire (**Fig. 2**). Gaps between panels allow free

movement of embankment in vibration plane but disallow relative transverse movement between transverse faces. Lowest facing element with T- iron shoe resting in V-groove of angle welded to table allows rotation but no translation. Geotextile covering inner side of hinge all along stops sand spilling into it. Geotextile cover, metal panels, steel wires & hinges simulate plane strain conditions of field which are difficult to simulate. Reluctance type pickups secured to facing elements at 425 & 875 mm above table & to the table measure accelerations. Universal amplifiers and pen recorders with speed upto 125 mm/sec recorded output of pickups.

PREPARATION OF TEST EMBANKMENTS AND TEST PROCEDURE

Embankments were constructed in layers of 75 mm. At first a unit of formwork along with transverse side geotextile covers was placed in position & secured to table. Eight metal panels were connected to corresponding panel on opposite face by wires. A 75 mm deep sand layer was rained. Reinforcement was placed in position & tied to facing elements with nuts, bolts & angle iron. A 75 mm deep sand layer was again rained. Next unit of formwork was placed & tied to previous one & process repeated till embankment was ready. Embankment top was covered with geotextile tied down to stop spilling out of soil during tests. All planks except facing elements were then removed. Test embankment was then ready for testing (**Fig. 2**).

Free vibration tests were performed by sledge hammer impact on table. Response of embankment and the table were recorded for three trials. **Figure 3** shows a typical record. Forced vibration tests were carried out for frequency-response study. Initially, oscillatory mass eccentricity (OME) values of oscillator was set to 6 and run at 5 Hz. Frequency was raised in small steps. At each frequency, steady state accelerations were recorded till excitation frequency was higher than fundamental frequency of embankment. Then, OME was increased in steps of 6 up to 36. Higher OME setting increases eccentricity of eccentric masses and hence dynamic force.

PROCESSING TEST DATA

Acceleration-time record data from free vibration gives fundamental frequency and three prominent peaks of accelerations in one direction & two in between peaks in other direction which could be measured accurately. For embankment vibrating with mean acceleration amplitude, a_{free}, at fundamental frequency, F_n, the circular frequency, ω_n, is given by $(2\pi F_n)$ and mean displacement amplitude, d_{free}, by [afree/(4π2 F_n^2)] and average shear strain, γ_r, by (d_{free}/H), H being embankment height. From fundamental vibration of shear beam, circular frequency, ω_n, & shear modulus, G_m , at mid height of embankment at resonance are:

$$\omega_n = \sqrt{\frac{K_s G_m}{\rho}}\,\frac{(2r-1)\pi}{2H} \qquad \text{and} \qquad G_m = \frac{\omega^2 \rho 4H^2}{K_s (2r-1)^2 \pi^2} \qquad (1)$$

where ρ is mass density & r is unity in fundamental mode. Constant shape factor, K_s is 5/6 [Krishna et. al., 1994]. Values of γ_r and G_r were thus obtained for test embankments. **Table 1** shows accelerations & natural frequencies from free vibration tests & computed displacements, average shear strain, γ_r, & shear modulus, G_r .

Forced vibration test data at each OME in the form of steady state accelerations at various frequencies was obtained from which frequency-response curves were obtained. Displacement, d_{fq}, at excitation frequency, F_{rq}, & associated acceleration, a_{fq}, is given by [$a_{fq}/(4\pi^2\ F_{eq}^2)$]. Frequency-response curve for a given embankment & OME gives resonant frequency, F_n, together with peak response. Shear modulus, G_r, at first mode resonance is got from equation (1). Average shear strain, γ_r, is (d_{fq}/H) at resonance. At any time, accelerations are measured at 3 points on embankment face. Accelerations at other points above base were interpolated by a quadratic: y=(a + b x + c x^2), y being acceleration at a distance x from base. Constants a, b & c were evaluated knowing accelerations at 3 points. Further details are discussed elsewhere [Mehdi, 1998]. With this procedure, resonant accelerations & frequencies were obtained at various OME values to obtain frequency-response curves for M1, M2 & M3 (**Figure 3** for OME of 24& for top pickup position). Figures for other cases are not reported for want of space. **Figure 4** shows that curves of strain dependent shear modulus for M1, M2 &

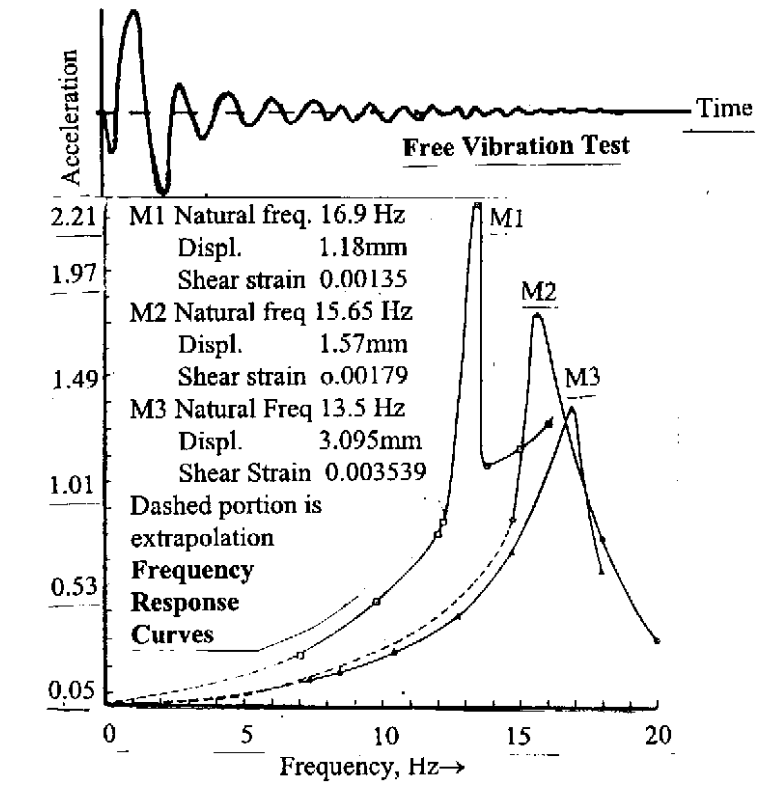

Fig. 3 Free Vibration Test Record and Frequency Response Curves for RE Embenkments (OME=24)

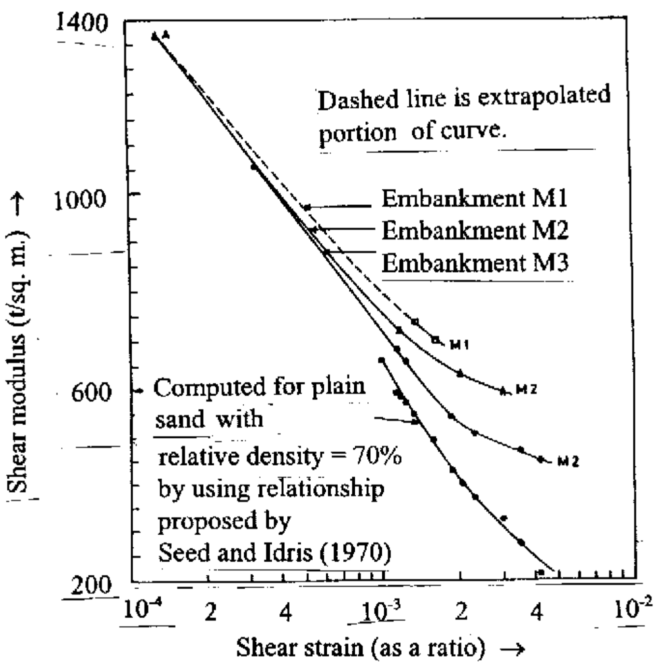

Fig. 4 Strain Dependent Shear Modulii for Reinforced Earth Embankment and for Plain Sand.

M3 tend to merge at A with very low strain of the order of 10^{-4} for which reinforcing action is negligible. The figure also shows shear modulus, G_s, for plain sand at D_r of 70% [Seed and Idris, 1970]. So, it is logical to extend curve for M1 up to A as shown. Such strain dependen G_r is very useful data for dynamic analysis by Finite Element Method. Shear modulus of plane sand at same relative density (70%) at low strain of 10^{-3} is about 12.2% smaller than that for M1. The discrepancy reduces for M2 and further reduces to 3.87% only for M3 which is logical and expected, because, M3 is weakest with most reinforcements discontinuous at mid-length. At higher strains, G_r of RE embankments is much larger than Gs of plain sand, because, at higher strains reinforcing action is more prominent leading to increased G_r. This high lights advantage of RE over plain sand.

Table 1

S. No.	Particulars	M2	M3
1	Natural frequency, F_n, in Hz	23.250	20.833
2	Horizontal acceleration coefficient, α_{ht}, at top pick up level	0.2522	0.4928
3	Horizontal acceleration in m/sec at top pick up level	2.47417	4.83508
4	Displ., $d_{free} = a_{free}/(4\pi^2\ F_n^2)$ in mm at fundamental F_n.	0.11593	0.28218
5	Average shear strain, $\gamma_r = d_{free}$/H at fundamental frequency.	1.325 x 10-4	3.225 x 10-4
6	Shear modulus, G_r, in t/m^2 at fundamental frequency.	1316.278	1056.8304

FINITE ELEMENT METHOD OF DYNAMIC 2D ANALYSIS OF EMBANKMENT IN TIME DOMAIN

It was performed in elastic domain by idealising embankment with 120 isoparametric four noded quadrilateral elements with 12 along height & 10 along length (**Fig. 5**). Computer programme for the same uses lumped mass matrix, [M], stiffness matrix, [K] & damping matrix, [C], given by [α [M] + β [K]], where α & β are constants obtained by damping in first two modes. Embankments were analysed with 1 & 2-layer idealisations. One layer idealisation considers embankment homogeneous. More realistic two-Layer idealisation has 2 homogeneous layers of equal depth since shear modulus varies with depth & is proportional to root of octahedral stress [Seed & Idris, 1970]. For M1 & M3 density, relative density, Poisson'sratio & excitation duration were1.625 t/m^3, 70%, 0.254 sec. respectively. For M1, shear modulus, shear strain, Young's modulus, excitation frequency & peak acceleration coefficient were 735.773 t/m^2, 0.001.35, 1839.432 t/m^2, 16.9 Hz and 0.152 g respectively. Corresponding values for M3 were 546 t/m^2, 0.001625, 1365 t/m^2, 13.75 Hz & 0.152 g respectively. If Gm, G1 and G2 are shear modulii & σoctm, σoct2 and σoct2 are corresponding octahedral stress at mid-depths of 1-layer idealization, top and bottom layers of 2-layer idealisation respectively, G1 and G2 are given by:

$$G_1 = G_m\left(\sigma_{oct1}/\sigma_{octm}\right)^{0.5} \quad \text{and} \quad G_2 = G_m\left(\sigma_{oct2}/\sigma_{octm}\right)^{0.5} \tag{2}$$

Knowing G1 and G2, properties are obtained for 2 layers in 2-layer idealisation extendible to 3 or more layers of desired depth. For M1, shear modulus & Young's modulus were 20.241 t/m^2 and 1300.604 t/m^2 respectively for top layer & 901.108 t/m^2 & 2252.771 t/m^2 respectively for bottom layer. For M3, they were 386.08 t/m^2 & 965.2 t/m^2 for top layer & for bottom layer, they were 668.71 t/m^2 & 1671.77 t/m^2 respectively.

If a_{cM1} and a_{cM3} are computed maximum accelerations & a_{eM1} and a_{eM3} are those based on test data for M1 and M3 respectively, ratio, r_a, is defined as (a_{cM1}/a_{eM1}). Ideal value of r_a is raideal and equals unity. Percentage discrepancy, r_{ad}, is obtained as {100(r_a - r_{aideal})/ r_{aideal} }. **Figure 6** shows variation of r_{ad} with damping ratio, ς_1, for M1 & M3 at centre of longitudinal face (mid-depth). It gives ς_1 =0.18 for M1 & ς_1 =0.1275 for M3. Presently, no method is available to evaluate right damping for dynamic analysis with

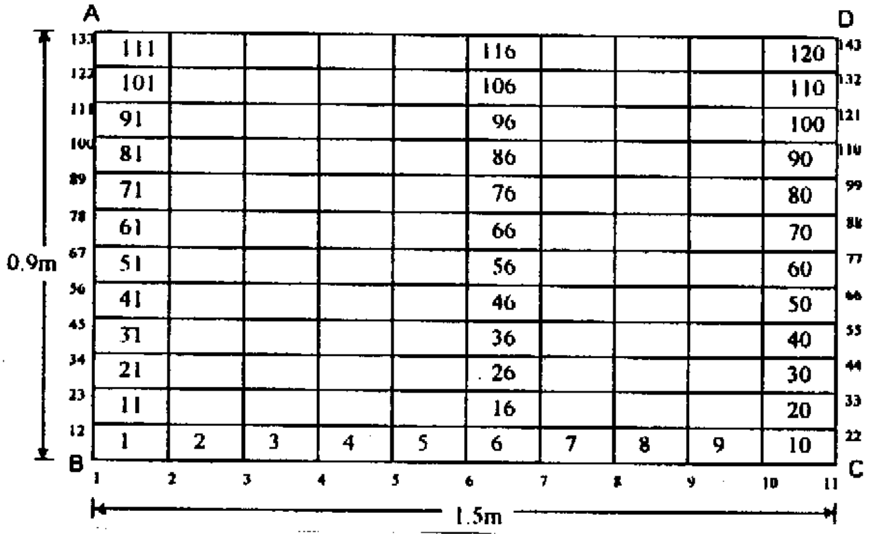

Fig. 5 Idealised Reinforced Earth Embankment for 2D Dynamic FEM Analysis.

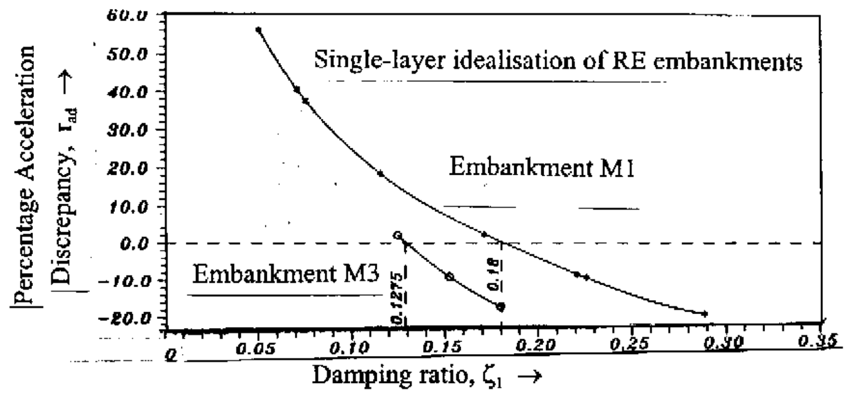

Fig. 6 Variation of Damping with Percentage Acceleration Discrepancy

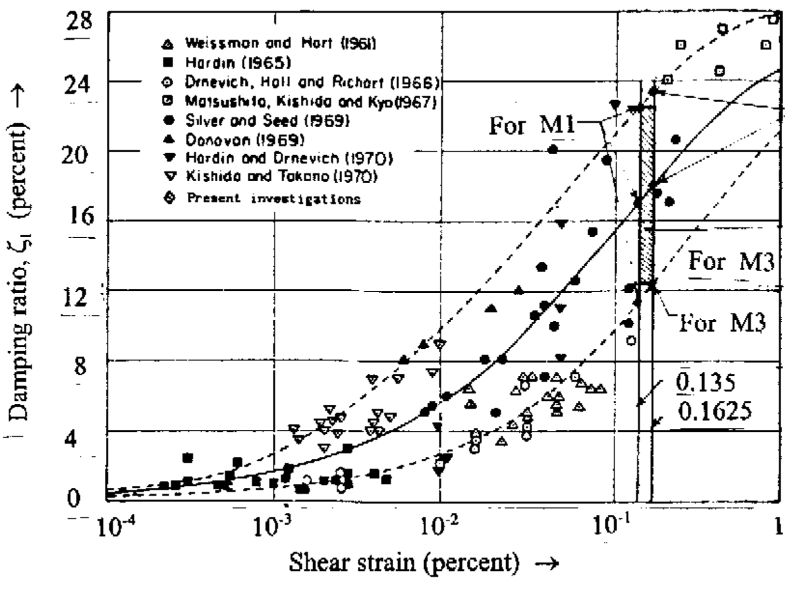

Fig. 7 Damping for Zero Acceleration Discrepancy and Damping Proposed by Seed et. al. (1984).

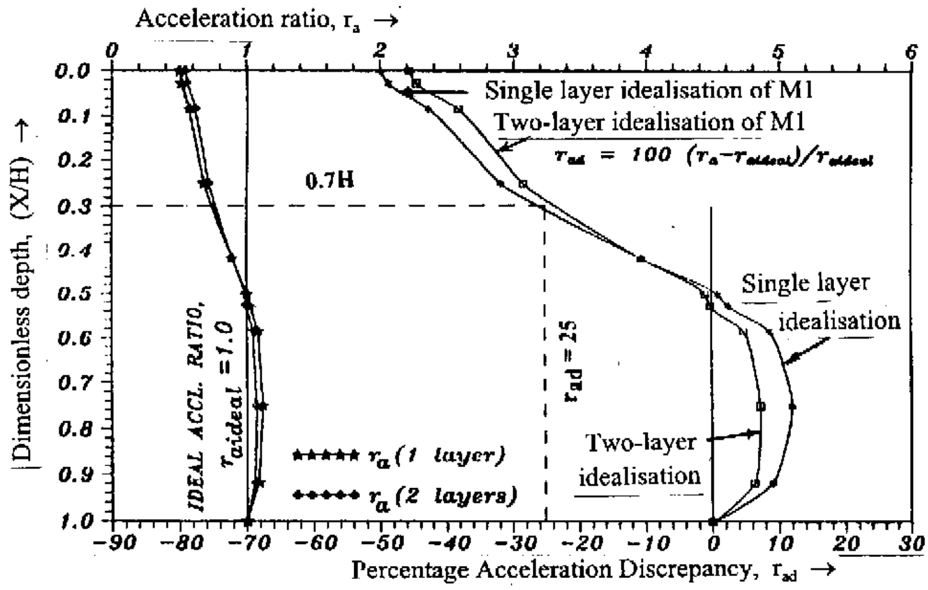

Fig. 8 Acceleration Ratio & % Acceleration Discrepancy Variation with Dimensionless Depth for M1

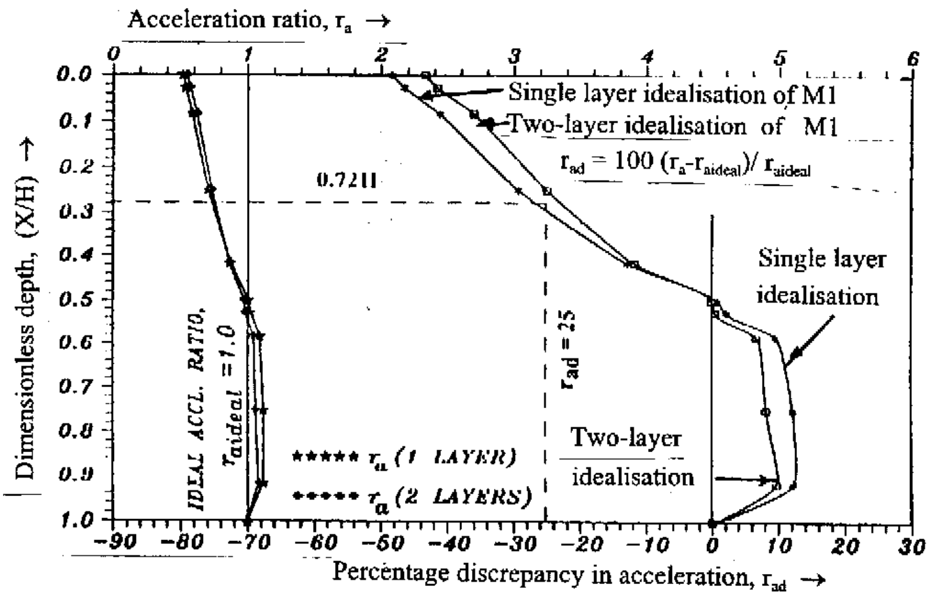

Fig. 9 Acceleration Ratio & % Acceleration Discrepancy Variation with Dimensionless Depth for M3

errorless computed response. This method of determining ς_1 for no error condition is useful when test results are available. **Figure 7** shows that ς_1 evaluated this way is within the range of damping recommended by Seed et. al. (1984). This shows that damping by proposed method is indeed quitereasonable.

Figure 8 shows variation of r_a & rad for 1 & 2-layer idealisations of M1 with dimensionless depth, (X/H), X being distance below embankment top & H being embankment depth. If 12% discrepancy is tolerable (r_a =12%), computed response agrees well with test data up to 0.61 H above base. If 25% discrepancy is tolerable (r_a =25%), this extends up to 0.7H above base. For whole depth, average ra is –15.7% which is quite small. In top 0.3 H this extends up to 0.7H above base. For whole depth, average r_a is –15.7% which is quite small. In top 0.3 H portion, response based on test data is larger than computed one. Even here, discrepancy is smaller with more realistic 2-layer idealisation, as expected. By assuming many thin layers near top, the idealisation will be more realistic and discrepancy between analytical accelerations and those based upon test data will be further reduced. **Figure 9** variation of acceleration ratio, r_a , & % discrepancy in acceleration, r_{ad} , with (X/H) for embankment M3 from which it may be noted that for 25% tolerance in discrepancy in acceleration, the good agreement of computed acceleration response with that based on test results extends upto 0.72 H above base which is a shade better than that for M1. Observations from similar plot based on displacements for M3 also indicate similar conclusions. The data on displacements was also processed on similar lines and results indicated similar conclusions. Therefore, it may be concluded that idealisation of micro-reinforced earth as homogeneous material is a reasonable.

CONCLUSIONS

Sand rain apparatus developed forms uniform sand layers with mean standard deviation of 1.08 % in relative density which is better than what is reported in state of the art. Development of devices to create plane strain conditions and to create proper hinge at base of lowest facing element in lab are significant contributions to experimental technique. Based on free vibration tests and frequency response tests on full-scale embankments, strain dependent shear modulii of 3 reinforced earth embankments were obtained over a wide range of strains which essential for dynamic analysis. Idealisation of micro-reinforced earth to be homogeneous is appropriate. By representing test embankment by appropriately large number of layers near embankment top can reduce difference between computed response and that based on test data to desired level. The proposed method to evaluate correct damping by using test data is helpful in obtaining more accurate analytical results.

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