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REINFORCED EARTH RETAINING STRUCTURES FOR EARTHQUAKE LOADING CONDITIONS

By

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ABSTRACT

Reinforced earth retaining structures are of recent origin. As such, very little information is available about the their performance during earthquakes. Nevertheless, composite reinforced earth can with stand compressive as well as tensile stresses. If reinforcements are designed to fail in pullout resistance, these reinforcements exhibit ductile behaviour. These properties make the use of reinforced earth ideal for earthquake loading conditions. Tensile failure of reinforcements may lead to catastrophic failure of retaining structures which should be avoided. The available information indicates that dynamic frictional resistance of reinforcements is less then static resistance. Besides, the reinforcement should extend beyond rupture surface predicted by Mononobe-Okabe to provide effective resistance required for stability of such retaining structure. This presentation proposes a method of analysis of such structures under earthquake loading conditions by using Joshi-Prajapati (1982) Method of analysis with modification proposed by Dass (1988).

INTRODUCTION

Earth retaining structures are common in civil engineering. With passing time the height of such structures has increased substantially making them important and expensive. Dynamic stresses in the top quarter of conventional retaining structures are often very severe to cause failure. The behaviour of such walls exhibits significant degree of plastic deformations. Analysis of such structures subjected to earthquake induced forces possess many problems and their construction is often time consuming.

The concept of reinforced earth was first proposed by Henry Vidal (1965) which is similar to that of reinforced concrete. The soil is reasonably good in taking compression but not so in tension. On the other hand, reinforcements are good in taking tension and not so in compression. The composite reinforced earth is good in taking compression as well as tension and is much better than plain earth. The friction between soil and reinforcement provides the required bond between two materials. Reinforced earth shows elastic behaviour over a larger range of stresses. Many reinforcing materials as ductile. These features make reinforced earth ideal for earthquake conditions. This is supported by their performance during past earthquakes. This presentation reviews available information and proposes a pseudostatic method of analysis of such structures. Research investigations on these lines are currently in progress in the Department of Earthquake Engineering. One of the objectives of this presentation is to invite comments on proposed method.

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BASIC CONCEPT AND MECHANISM OF REINFORCED EARTH

To understand the mechanism of reinforced earth it is essential to understand the behaviour of plain earth and reinforcing elements separately as well as in the form of composite material. Figure 1a shows an element of plain earth subjected to vertical stress, σ_1. If the confining stress, σ_3, is equal to zero the soil fails, because, Mohr stress circle crosses Mohr envelope as shown in Fig. 1b. The minimum value of σ_3 required to just reach the failure state is shown in Fig. 1c and d. Under these stresses shape of the element changes from ABCD to IJKL. If the soil is reinforced as shown in Fig. 1e, the tendency of the soil to spread laterally tends to stretch reinforcements due to friction between the two. Since modulus of reinforcements is relatively larger, lateral deformation of elements is very low as shown in Fig. 1e. Tensile stresses in reinforcements thus get transformed to soil element in the form of compressive stresses resulting in to apparent confining pressure ($\sigma_3 + \Delta\sigma_3$). The lateral stress ($\sigma_3 + \Delta\sigma_3$) is the confining pressure required to restrict the lateral strain in the plain earth to that corresponding to reinforced earth. The major principal stress ($\sigma_1 + \Delta\sigma_1$) is required to reach failure for a contining pressure of ($\sigma_3 + \Delta\sigma_3$) for the plain earth, whereas, the actual vertical stress σ_1 is considerably less than ($\sigma_1 + \Delta\sigma_1$). This shows that the soil can take up plenty of additional vertical stress. The lateral deformation of the composite material is much smaller than that of the plain earth, whereas, the apparent lateral confining pressure of the reinforced earth is much larger than that of the plain earth under the same vertical stress σ_1.

COMPONENTS OF REINFORCED EARTH RETAINING STRUCTURES

Backfill and reinforcing elements are main components. Reinforcements may be in the form of strips, planks, grids, nets, sheets, geotextiles, geomembrance and ties. Materials of reinforcements have good tensile strength. Their modulus of elasticity is many times that of the soil. Hence, for a given strain in the composite reinforced earth, the stress in the reinforcing material is many times that in soil. Reinforcing materials include metals, prestressed concrete, fibre reinforced plastics, woven/nonwoven synthetic fabrics, worn out automobils tyres, jute, polymers, bamboo, polypropylene, polyethylene, P.V.C. etc. If fabrics are used as reinforcing elements, they are folded back in to the backfill near the front end of the wall to contain soil and to provide required confining pressure (Fig. 2a). If strips or planks are used as reinforcing elements, special skin elements in the form of metal plates (Fig. 2b) or concrete panels (Fig. 2c) are necessary to which reinforcements are connected using fastners. Figure 2d shows schematic arrangements of a wall with skin elements. Junctions between adjacent elements make the wall sufficiently flexible to allow strains within the composite material due to lateral pressures. Use of ferrous reinforcements has problems of corrosion. Therefore, inert like polypropylene are more desirable for long term use.

DUCTILITY OF REINFORCED EARTH

The ability of the material to undergo large deformations beyond the deformation at ultimate strength without appreciably losing the strength is ductility. Ductile behaviour of materials during earthquake loading conditions is helpful in avoiding catastrophic failures. Plain earth possesses residual strength even at large strains and hence may be considered to exhibit ductile behaviour.

Soil reinforcements may fail either in tension or due to inadequate frictional bond at soil-reinforcement interfaces. If reinforcing material is not ductile, it may fall in tension which may lead to similar failure of neighbouring reinforcing elements due to transfer of additional stresses. This may ultimately lead to catastrophic failure of such retaining structures. This is highly undersirable and can be avoided by using ductile reinforcements like steel, polypropylene and a variety of geotextiles. Figure 3 shows stress-strain relationships for some of these materials. It is interesting to note that geotextile have exceptionally large failure strains of the order of 50 to 100 percent and exhibit fair degree of ductility.

If reinforcements are strong enough to avoid tension failure, the failure will be due to inadequate frictional resistance along soil-reinforcement interfaces. Mobilization of such unit frictional resistance gradually increases with the pullout displacement per unit length of the reinforcement and reaches a maximum. After reaching the maximum value, the unit frictional resistance does not increase with further increase in pollout displacement (Fig. 4). Therefore, the pollout resistance of reinforcements shows a ductile behaviour. It is also clear that it is possible to obtain a ductile behaviour for reinforcements eventhough material of reinforcement may not show much ductility in tension. Frictional resistance under dynamic conditions is much smaller than that under static conditions but remains ductile.

PERFORMANCE OF REINFORCED EARTH RETAINING STRUCTURES DURING EARTHQUAKES

Eventhough there are many reports on failure of conventional earth retaining structures, there is only one report by Fukuoka and Imamura (1984) that the authors have come across which describes the performance of a reinforced earth retaining structure subjected to earthquake ground motion. Nevertheless, it gives some idea about how such structures perform during earthquakes. This structure, situated in Tokyo, Japan, retains 5 m of fill with cohesion, c_u, equal to 14.6 kN/m^2 and angle of shearing resistance, ϕ , equal to 15 degrees. The reinforcements are in the form of steel rods of 20 mm in diameter and anchored to vertical concrete plates of 40 cm x 40 cm x 10 cm situated at all fill end of the rod. In the instrumented segment of the structure, each rod is provided with wire strain gauges to measure static tention in rods and a reinforcement gauge to measure dynamic increament in the rod tension as a function of time during earthquakes. Besides, two accelerographs were used to measure the ground acceleration during earthquake : one at the foot of the wall and the other at the surface of the fill. Skin elements are in the form of concrete plates 1m x 1m x 15cm secured to concrete columns of 20cm x 20cm x 550cm. Figure 5 shows schematic arrangement of this instrumented retaining structure. On February 27, 1983 this structure experienced an earthquake with distance of 17 km and focal depth of 70 km. Figure 6 shows records of ground accelerations measured at the top and at foot of the wall as well as rod tensions. Earth pressures were not measured directly but were computed by using dynamic data obtained. No failure surface was observed. Fig. 7 shows peak values of measured rod tensions together with Coulomb earth pressures for static case and Mononobe-Okabe pressures for dynamic case. Peak ground acceleration recorded was 0.146 g at the fill surface.

They have reported that computed peak earth pressures occur at instances of time when peak inertia forces occur which is reasonable. The difference between variations of Coulomb static earth pressures and Mononobe-Okabe dynamic earth pressures is not appreciable in view of small values of shear parameters of the fill. Besides, the static top tie tension is comparable to dynamic top tie tension. This appears to be so, because, the top tie (A_4) is totally situated with in dynamic active rupture wedge, if ϕ equals 15 degrees, α_h equals 0.146 and wall back is smooth. Largest outward movement occurs at the level of second tie from the top (A_3) as indicated by recorded tie tensions. Such outward movement would reduce considerably at lower ties which is reflected by relatively smaller tie tensions recorded (A_2 & A_1). Rupture surface was not developed during the earthquake. This indicates that structure was capable of with standing a higher level of earthquake accelerations than what was recorded. Obviously, smaller pollout resistance may have been mobilized during earthquakes indicating conditions closer to dynamic at rest conditions. This indicates that mobilized ϕ_m is considerably smaller than ultimate shear parameter ϕ equal to 15 degrees. This means that angle of rupture surface, ρ, with horizontal for such mobilized ϕ could be much smaller than 38.28^o as per Mononobe-Okabe theory assuming no wall friction. Under these conditions also, top tie does not experience appreciable tension during earthquakes. Therefore, it is reasonable to allow development of active rupture wedge within the reinforced earth structure for optimum functioning of reinforcing elements during earthquakes.

LABORATORY STUDIES ON REINFORCED EARTH WALLS

Richardson and Lee (1975) have reported results of shake table investigations for reinforced earth walls. The table was driven horizontally by MTS hydraulic ram of 5 tonne capacity. Test box, 30 in (760 mm) wide was formed out of 19mm thick plywood. Mylar magnetic tapes were used at reinforcement tie secured to facing elements in the form of curved aluminium strips of 1 in (25 mm) size. Cohesionless sand with relative density, D_r, equal to 63%, unit weight, equal to 93.5 pcf ($1{,}500\ kg/m^3$) and plain strain, ϕ, equal to 44 degrees. The 12 in deep fill extended to a length of 30 in (760 mm) behind facing elements. This box was rigidly mounted on shake table. Instrumentation included accelerometers mounted on shake table top as well as on top of the fill, LVDT's to measure wall deformations, special gauges to measure tie forces and Sanborn strip chart recorder for recording data. Vibration frequency was 11.6 Hz for most tests. Base acceleration ranged from 0.05 g to 0.5 g.

When aluminium plates were used as reinforcement under dynamic conditions, second and third ties from bottom end failed under tension leading to catastrophic failure. Such tension failure during earthquakes is undesirable, particularly if tie material is not ductile.

They measured friction between tape and backfill under static and dynamic conditions. Figure 8 shows tape friction (sliding stresses) as a function of normal stress at the tape level for conditions before, during and after shaking. It is clear that the dynamic frictional resistance is appreciably smaller. Figure 9 shows a typical record of dynamic tie forces. As the shaking starts there is a rapid increase in the tie force with increasing lateral movement of the tie. After reaching the peak, the tie force reduces gradually to reach a residual value indicating that the dynamic friction also shows desired ductile behaviour which is useful in avoiding catastrophic failures.

When ties were strong enough to avoid tension failure it was observed that with increasing duration of shaking the wall initially rotated about the base as a plane surface. Soon after, facing elements near top end moved out appreciably. Simultaneously, the bottom most facing element rotated and ultimately rested flat on the bottom. At this stage the facing element immediately above this element started behaving similarly. However, no catastrophic failure occured. This is reasonable if tension induced in the facing element does not cause tensile failure. The relatively large outward movement of upper facing elements indicates that inertia forces of the soil wedge is much larger in that region. The observed failure plane within the reinforced earth structure during such tests was in agreement with that by the Mononobe-Okabe theory which is an important finding.

Figure 10 shows measured tie forces at various elevations during tests. Open dot curves indicate tie forces observed during construction. The figure also shows tie forces assuming static at rest and active earth pressures. The bottom tie force is appreciably smaller than that in the tie immediately above, because, it receives one half the force acting on the bottom most facing element. Measured dynamic tie forces at different acceleration levels are also shown. The forces associated with horizontal acceleration of about 0.05 g were comparable to those obtained by using K_o condition. They recommend earth pressures shown in Fig. 11 which consists of static pressures and the dynamic earth pressure increments. They also recommend the use of a seismic design coefficient E as a function of coefficient (a/g) as shown in the same figure where a is the horizontal table acceleration and g is the acceleration due to gravity. They recommend following expression for E :

$$E = 1.4\,(a/g) \tag{1}$$

Eventhough recommended earth pressures at the bottom end as well as at the top end of the wall are larger than what may be expected, they are conservative and safe.

Figure 12 shows frequency response relationship as well as shear strain amplitude, ε , as a function of frequency for the reinforced earth structure during shaking. It appears that the structure shows elastic behaviour to a reasonable extent. This is not so for gravity walls which show predominantly plastic deformations under dynamic conditions. They recommended the following expressions for damping ratio, λ , and shear modulus, G :

$$\lambda = 1/(2MF) \tag{2}$$

$$G = 16h^2\ \rho_m/T^2 \tag{3}$$

$$G = 1{,}000\ K_2\ \sigma_m{}^{1/2}\ \mathrm{PCf} \tag{4}$$

where, σ_m is the mass density of the soil; h is the thickness; T is the fundamental period of the layer; MF is the magnification factor (defined as the ratio of surface to base acceleration at resonance); K is the shear modulus factor and σ_m is the mean normal stress.

Using the data thus obtained, they analysed the reinforced earth structure using a finite element method. Reinforcing elements were represented by special tie elements. A typical earthquake accelerogram as well as response spectra developed for the same accelerogram were used for the analyses. They recommend the analysis in first and second modes only. Design acceleration, A_{des}, is given by :

$$A_{des} = 1.25\ Sa_1 + 0.5\ Sa_2 \tag{5}$$

where Sa_1 and Sa_2 are values of spectral acceleration for periods of first and second modes respectively. They recommended emperical expressions for obtaining the period of vibration in first and second mode of vibrations. They reported good agreement between results obtained by finite element method of analysis and those by the analysis using first and second modes only.

OBSERVATIONS FROM THE STATE OF ART ON REINFORCED EARTH RETAINING WALLS

There are very few reports on behaviour of reinforced earth walls during past earthquake and laboratory tests under dynamic conditions. Nevertheless, some interesting and useful observations are made from available information. Rupture surface given by the Mononobe-Okabe theory may develop within the reinforced earth, structure under dynamic conditions if vibrations are sufficiently strong. This suggests that rupture wedge predicted by Mononobe-Okabe theory is reasonable for analysis of such structures. However, if the structure is over reinforced, it is likely to experience relatively smaller stresses and perform in elastic domain. Under such conditions the strain levels and corresponding values of mobilized ϕ are also small. The soil may be considered to be always in state of failu e depending upon the choice of strain. For very small values of failure strains the rupture surface developed and the associated displacements may not be percepitable. Therefore, absence of visible failure surfaces during dynamic loading does not preclude their presence. For small strain levels and the mobilized ϕ , the rupture wedge is very large and the rupture surface is flatter. This increases the earth force and hence leads to corresponding increase in the strain level. When strains tend to grow they result into larger mobilization of ϕ which in turn tends to increase the angle between the rupture surface and the horizontal, decrease the size of the rupture wedge and hence reduce the earth force. When equilibrium is reached the earth force is just balanced by resisting forces mobilized.

When reinforcements are provided, a new constraint is imposed on the system, namely, the strains with in the soil mass should be compatible with the redistribution of the stresses due to presence of reinforcements. Therefore, when soil with in the reinforced earth structure undergoes certain degree of deformation, it results in to mobilization of certain value of ϕ and the corresponding size of rupture wedge and earth pressures due to this wedge. The earth pressures are taken up by reinforcing elements in the form of tension which has to be resisted by friction between the soil and the reinforcement along the segment of the reinforcement extending into the backfill beyond the rupture surface. If the reinforcing element lies totally within the dynamic rupture wedge, it does not contribute any resisting force required for equilibrium.

It may be desirable to design reinforced earth wall for design seismic condition such that disturbing forces on reinforcing elements are just in balance with resisting friction forces. Under these conditions, such a structure is in critical plastic equilibrium. If dynamic earth pressures are large in magnitude, the wall suffers greater plastic deformations but no catastrophic failure is likely to occur. If the intensity of seismic vibrations is smaller than the design value, the associated strains are also smaller and the structure tends to behave increasingly in the elastic domain. Therefore, the analysis in elastic domain is required for reinforced earth structures under working stress conditions and under conditions of maximum credible earthquakes, the analysis should be in plastic domain. In following articles, the outline of a method of analysis in plastic domain is indicated.

NUMERICAL APPROACH FOR ANALYSIS OF REINFORCED EARTH WALLS

There are two conditions under which reinforced earth walls are required to be analysed. Under the OPERATING BASIS EARTHQUAKE (OBE), the structure predominently behaves in elastic domain. Under the MAXIMUM CREDIBLE EARTHQUAKE (MCE), the structure is expected to show plastic behaviour. The proposed, method of analysis is for the reinforced earth structures subjected to MCE only.

As plastic deformations are expected to occur, it is reasonable to consider occurence of a rupture wedge which is supported by experimental evidence (Richardson and Lee, 1975). It is also reasonable to assume full mobilization of frictional resistance along soil-reinforcement interfaces, if frictional resistance is in critical balance with tension in reinforcing elements. Figure 13 shows reinforced earth wall together with rupture surface predicted by Mononobe-Okabe theory. In view of the absence of rigid retaining wall, it is reasonable to consider wall friction to be absent. To determine the tensile force in earth reinforcement due to dynamic earth pressures, it is necessary to obtain distribution of dynamic pressure behind the facing element. For this purpose, the numerical method proposed by Joshi and Prajapati (1982) together with modifications suggested by Das (1988) would be useful.

The Joshi-Prajapati method of analysis assumes Coulomb's rupture wedge for static case and Mononobe-Okabe rupture wedge for dynamic case. As shown in Figure 14, the rupture wedge may be discretized into a number of smaller wedges with vertical interfaces. The earthforce on each vertical interface is horizontal. From force equilibrium conditions for each discrete wedge, it is possible to determine magnitude of horizontal interface earth forces and soil reactions along the rupture surface. From this, it is possible to obtain pressure distribution due to soil reaction along the entire rupture surface, BC. The rupture wedge may be further discretized into a number of discrete wedges with horizontal interfaces as shown in the same figure. For a discrete wedge, such as ACFF', the force due to weight of the wedge is known in magnitude and direction. The soil reaction along CF is known in magnitude and direction as per Joshi-Prajapati method of analysis. The earth force on the face AF' is horizontal. The angle of mobilized value of shear, ϕ_m, between the vertical and the soil reaction, R_y, acting on the horizontal interface FF' may be obtained from the wall movement required to realize the active state as well as the triaxial test data for the backfill material (Das, 1988). Therefore by using force equilibrium conditions, it is possible to determine the magnitude of earth force acting on the face of the wall AF'. This procedure may be repeated on similar lines to determine the earth force acting on each segment of the face of the wall AB. From this information, it is possible to obtain the earth pressure distribution along AB. For earthquake loading conditions, the procedure remains the same except that inertia forces in vertical and horizontal directions are also considered in addition to other forces. Further details are cited in original references.

For a reinforced earth wall, the earth force transfered to the top most reinforcement is equal to earth force acting on segment AM of the wall face AB which is known from the above analysis. This is the force in this reinforcement which has to be resisted by frictional resistance along the segment of that reinforcement projecting in to backfill beyond the rupture surface BC. This resistance can be obtained from the ultimate resistance due to friction per unit area of the surface of reinforcement obtained experimentally. As such, length of the segment M_1M_2 of the reinforcing element may be obtained by dividing the magnitude of the earth force acting on segment AM by frictional resiatance of the reinforcement per unit length. The length N_1N_2 of the next reinforcement may also be obtained on similar lines. Thus, it is possible to obtain the length of each reinforcement to withstand earth forces under earthquake loading conditions.

PLASTIC DEFORMATION OF THE REINFORCED EARTH WALLS

Under MCE conditions, plastic deformations are expected. Reinforcement situated at the bottom is not likely to slide horizontally due to large frictional resistance. As such, forward motion of wall during earthquakes would be largest near the top end and will reduce gradually towards the lower end as indicated by experimental results.

When the distribution of earth pressure along the back of facing element is known, it is possible to obtain the earth force transferred to reinforcement in the form of tension. Besides, from experimental investigations it is possible to obtain the largest value of friction along the soil-reinforcement interface. Knowing this information it is possible to compute the yield acceleration required to initiate sliding movement of the reinforcement, whenever earthquake accelerations are larger than such yield accelerations. Integration of the design accelerograme out side the level of yield acceleration gives the damage potential. Further integration of this damage potential diagram gives sliding displacement of the reinforcing element as a function of time. Such computations may be carried out for all reinforcing elements to obtain the deformed shape of the reinforced earth wall at any instant of time during the earthquake.

OVERALL STABILITY OF THE REINFORCED EARTH WALL

So far the discussion has been with respect to forces and displacements associated with reinforcement by using the formation of a rupture wedge behind facing element of the wall. This is useful in designing reinforcing elements of earth wall suitably. However, it is also necessary to examine the stability of such a wall against dynamic earth pressures acting behind it. In view of the large weight of the reinforced earth wall, it may be expected that the wall may not slide along the base. Similarly, it may also be expected that due to large base width, the wall may not fail due to inadequate bearing capacity. Under large dynamic earth forces acting on wall, it may undergo shear deformations particularly because unlike the nearly rigid masonary gravity walls the reinforced earth wall is deformable. Such shearing deformations of wall caused by earth pressures behind the wall may be computed by using numerical approach. Such an analysis of reinforced earth wall as whole would be desirable to ensure that the performance of structure is satisfactory during earthquakes.

ADVANTAGES & LIMITATIONS OF THE PROPOSED ANALYSIS

In view of the plastic behaviour of structure under MCE conditions, it is an advantage to use a method of analysis in plastic domain. This method is simple, direct and explicit. The design data on material properties required for the analysis may be obtained from experimental investigations. The numerical analysis proposed by Joshi and Prajapati (1982) together with modifications to it proposed by Das (1988) is useful for such an analysis. The disadvatage stems mainly from the lack enough of reported experimental results. Eventhough some laboratory investigations and results of field performance of one such structure during earthquakes are available, it is desirable to have a wider data base and benefit of many more such investigations.

CONCLUSIONS

The soil is usually strong in compression and weak in tension whereas reinforcing elements are strong in tension and weak in compression. As such, reinforced earth as a composite material is strong in both compression and tension. Failure of reinforcements in tension during dynamic loading leads to catastrophic failure of retaining structures which is undesirable. The force deformation relationship of reinforcements pollout tests show ductile behaviour under static and dynamic conditions. Ductility is a desirable property of materials under earthquake loading conditions. Hence, reinforcements should be designed on the basis of ultimate pollout resistance to preclude catastrophic failure during earthquakes.

Experimental investigations have indicated that rupture wedge predicted by Mononobe-Okabe theory occurs within the reinforced earth structure. This is a reasonable basis to develop methods of analysis of structures under dynamic loading. To offer resistance to plastic movement of this rupture wedge, reinforcements should extend beyond the rupture surface into the backfill behind. Pollout resistance of reinforcements under dynamic conditions is much less than that under static conditions.

The proposed method of analysis assumes Mononobe-Okabe rupture wedge under maximum credible earthquake conditions. The distribution of soil reaction along the rupture surface and earth pressures along the facing element may be obtained by Joshi-Prajapati method of analysis with modifications proposed by Das. From this information, it possible to obtain pollout force in each reinforcement and hence the length of the reinforcement beyond rupture surface knowing the dynamic pollout resistance of each reinforcement. The pollout plastic displacement of each reinforcement may be obtained by using the concept of yield acceleration. It is also desirable to check the overall stability of such retaining structures by treating them as gravity walls subjected to earth pressures from the backfill behind the structure under static as well as dynamic conditions.

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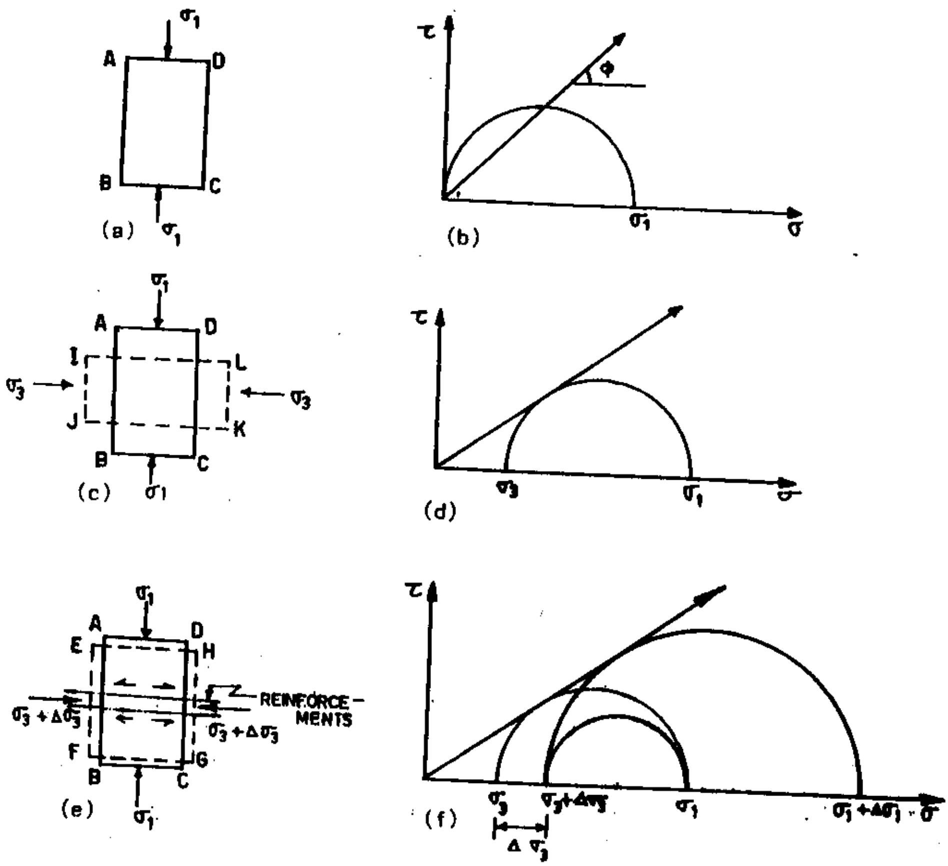


FIG. 1. BASIC CONCEPT AND MECHANISM OF REINFORCED EARTH

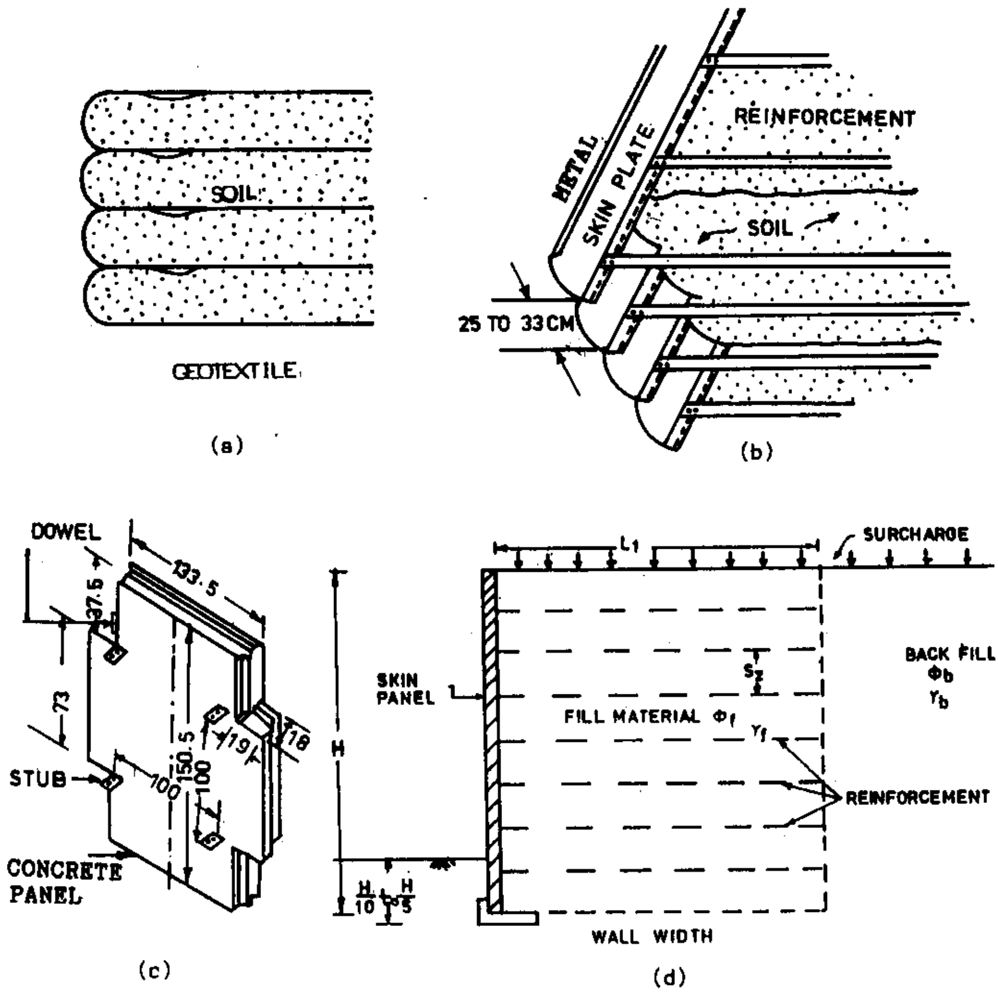
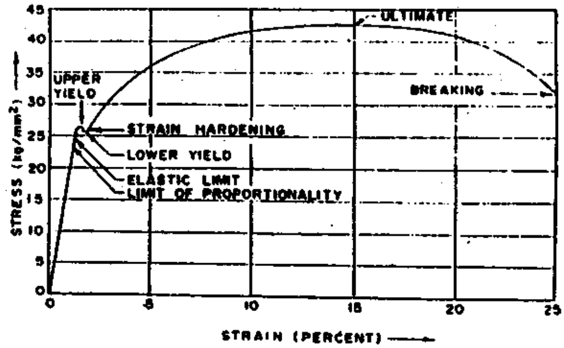
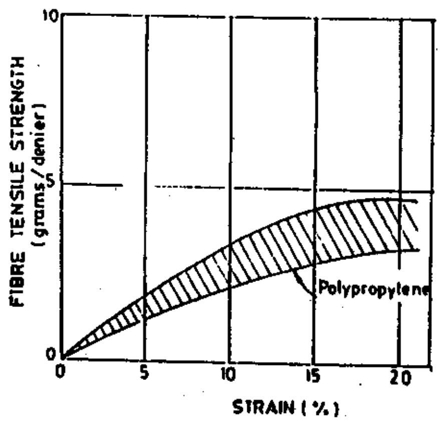


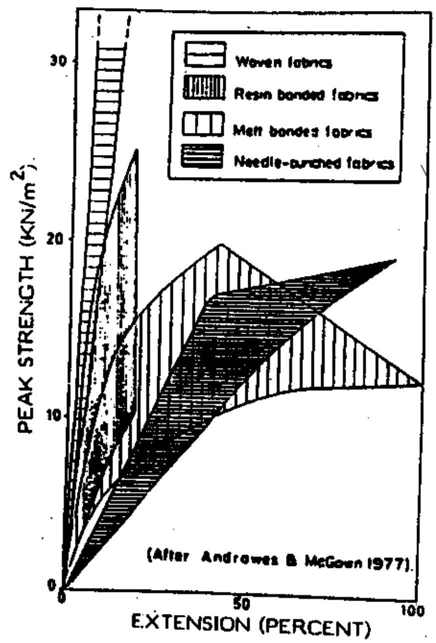
FIG. 2 SECTION AND COMPONENTS OF A REINFORCED EARTH WALL



(a) MILD STEEL



(b) POLYPROPYLENE



(c) LOAD EXTENSION RELATIONSHIP FOR VARIOUS FABRICS

FIG. 3. STRESS-STRAIN CURVES FOR SOME REINFORCING MATERIALS

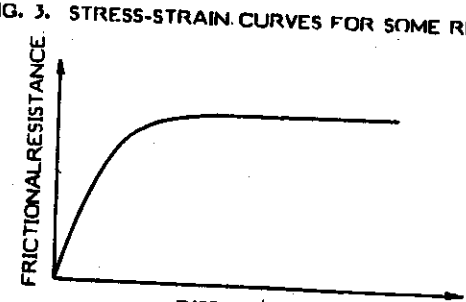


FIG. 4 VARIATION OF FRICTIONAL RESISTANCE WITH PULLOUT DISPLACEMENT

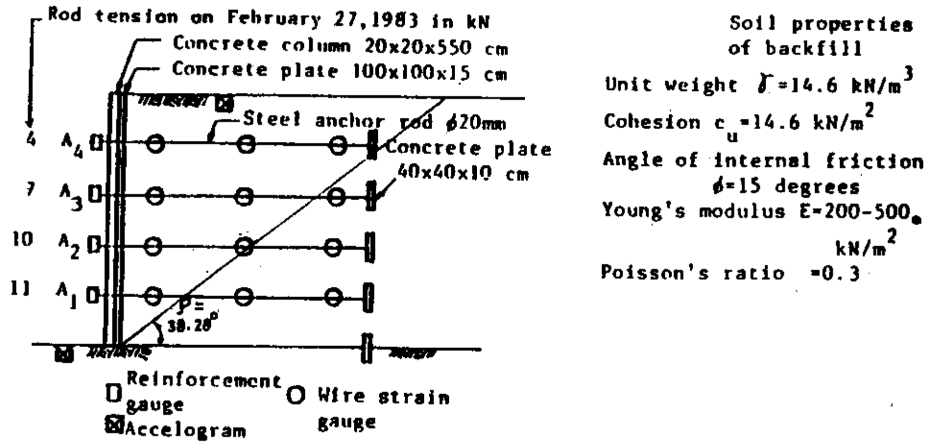


FIG. 5 CROSS-SECTION OF MULTIPLE ANCHORED RETAINING WALL

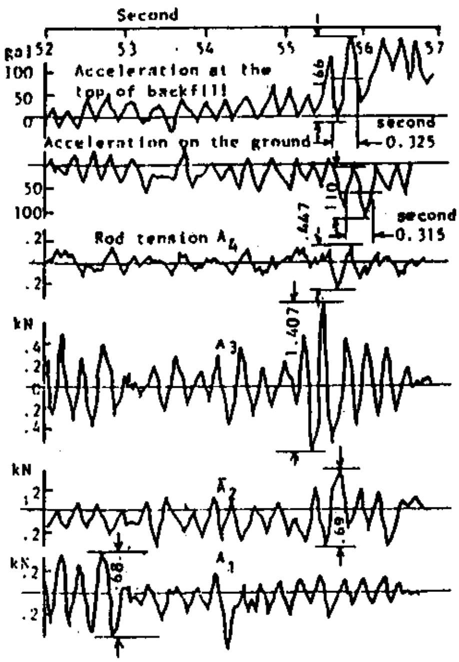


FIG. 6 ACCELERATIONS AND TENSIONS OF RODS

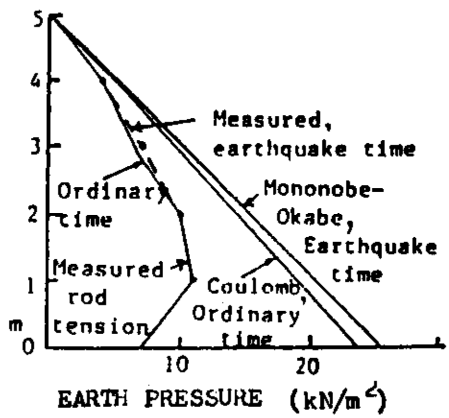


FIG. 7 EARTH PRESSURE CALCULA-TED AND MEASURED

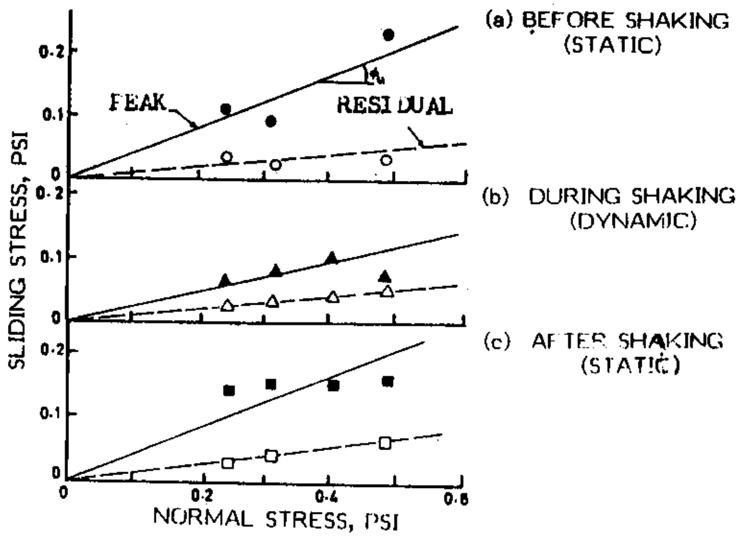


FIG. 8 VARIATION OF SLIDING STRESS WITH NORMAL STRESS

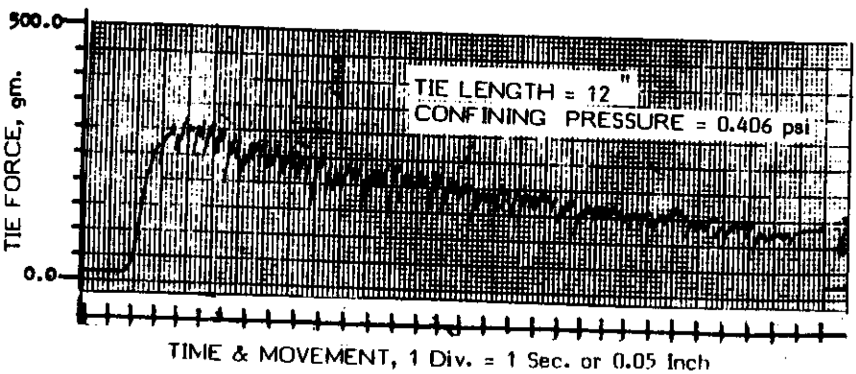


FIG. 9 RECORD OF PULLOUT TEST TO DETERMINE TIE FRICTION

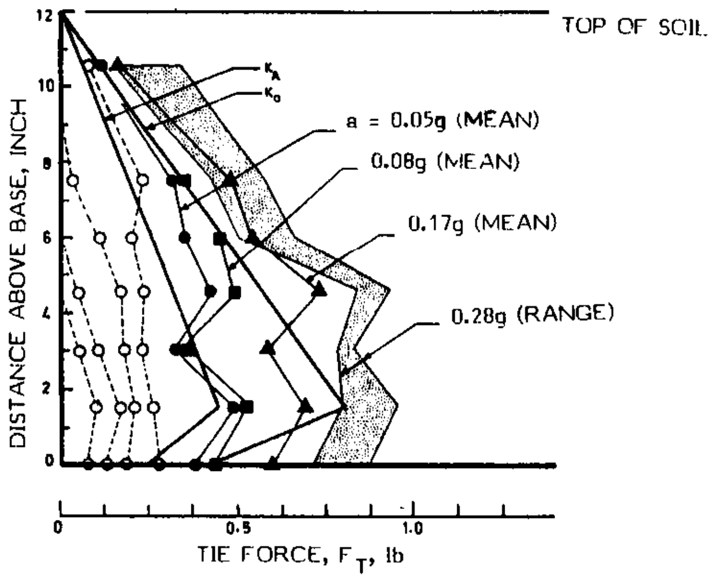


FIG. 10 MEASURED TIE FORCES FOR VARIOUS ACCELERATIONS.

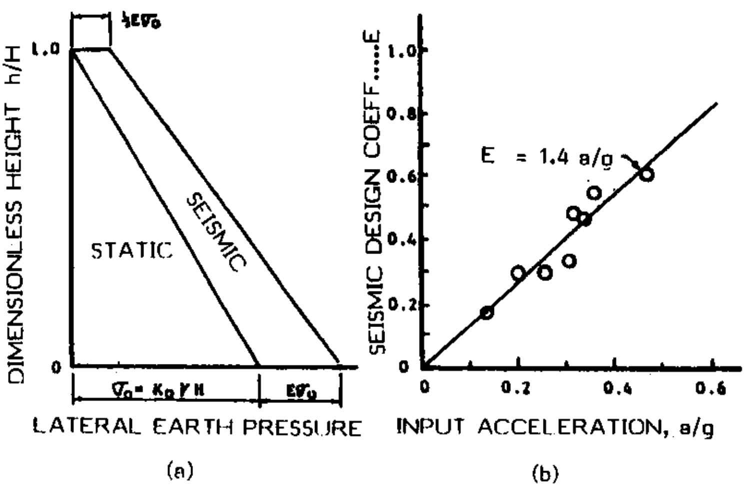


FIG. 11 ENVELOPE AT MAXIMUM SEISMIC EARTH PRESSURES

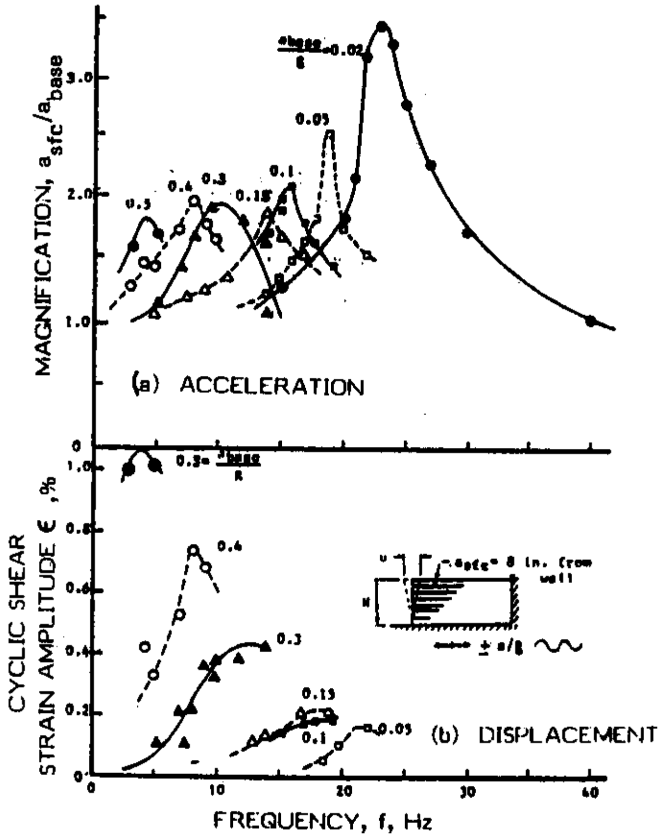


FIG. 12 RESONANCE RESPONSE OF MODEL REINFORCED EARTH WALL

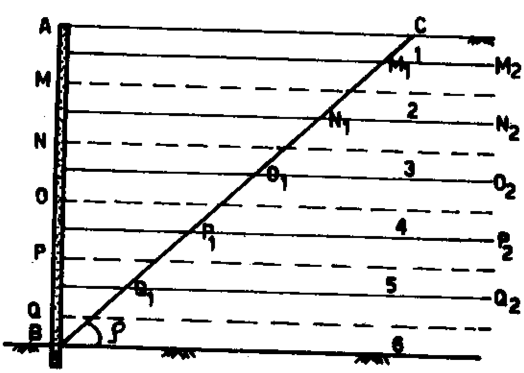


FIG. 13 REINFORCED EARTH WALL WITH MONONOBE-OKABE RUPTURE SURFACE

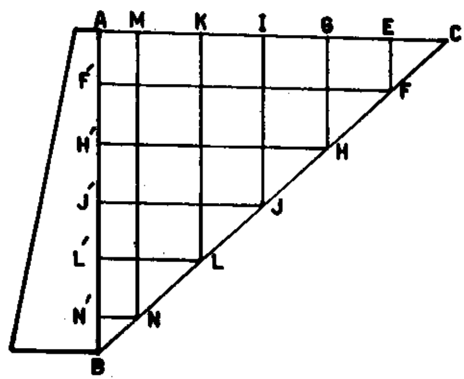


FIG. 14 DISCRETIZATION OF THE ACTIVE STATE RUPTURE WEDGE FOR JOSHI-PRAJAPATI METHOD (1982)