Reinforced Earth*

by

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Historical Review and Early Stages of Development

Earth, being the cheapest and readily available construction material, has been popular with the civil engineers, even though it suffers from being poor in mechanical properties. It has been the constant endeavour of research workers to put forth innovative ideas to improve its mechanical properties to suit the requirements of engineering structures.

Amongst the recent developments is a new construction material obtained by the combination of earth and reinforcement and termed as "Reinforced Earth". Reinforced earth is formed by the association of frictional soil and tension-resistant elements in the form of sheets, strips, nets or mats of metal, synthetic fabric or fibre-reinforced plastics and arranged in the soil mass in such a way as to reduce or suppress the tensile strain which might develop under gravity and boundary forces. It is wel known that most granular soils are strong in compression and shear but weak in tension. The performance of such soils can be substantially improved by introducing reinforcing elements in the direction of tensile strains in the same way as in reinforced concrete.

Soil reinforcement has been in vogue in crude form since ancient times. Some of the existing historical monuments bear testimony to the use of earth reinforcement technique (Jones, 1978). No rational study of soil reinforcement had, however, been made till a French engineer, Henri Vidal, published his investigation on soil reinforcement in 1966 and started the use of the term "Reinforced Earth." Few new materials or techniques have aroused so much interest and awareness amongst civil engineers in recent times as soil reinforcement has done. The apparently simple mechanism of reinforced earth and the economy in cost and time has made it an instant success with research workers and field engineers alike for temporary as well as permanent structures.

Reinforced earth possesses many novel characteristics, which render it eminently suitable for construction of engineering structures. It employs pre-fabricated elements which can be easily handled, stored and assembled. Ordinary frictional soil constitutes most of its bulk and the soil needed for the construction can be placed in position by modern hauling and compaction equipment. The flexible nature of reinforced earth mass enables it to withstand large differential settlements without distress. Reinforced earth thus permits construction of engineering structures over poor and difficult sub-soil conditions.

By far the greatest use of soil reinforcement technique has been made

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in the construction of retaining structures. Hundreds of retaining walls have been constructed all over the world. Majority of these walls support horizontal or sloping earth fills. Quite a few reinforced earth bridge abutments and quay walls have also been built. Many special structures (coal storage slots, rock crushers etc.) have also been constructed using this technique (Vidal, 1978). In situations where the deformation and displacement of foundation soil are such that only flexible structure can be constructed, reinforced earth is most suitable.

Much of the early work was carried out in France and included research on the material properties of reinforced earth and its application in retaining walls and abutments. These studies indicated that reinforced earth can be considered as a cohesive material with anisotropic cohesion introduced due to reinforcement being a function of strength and density of reinforcement (Schlosser and Vidal, 1969). However, the concept of anisotropic cohesion did not find direct application in design of retaining structures. The early design of wall was based on an anchorage concept (Schlosser and Vidal, 1969) with each reinforcing strip or tie locally balancing the Rankine active thrust on the area of skin supported by it. Subsequent work in France and elsewhere showed local equilibrium analysis to be conservative, especially for failure due to slippage between soil and reinforcement (Lee et. al. 1973).

New models for strength properties of reinforced soil and new methods of analysis of reinforced earth walls were proposed in the latter half of the last decade. Overall equilibrium methods which considered a biplanar or curved failure surface were advanced which were significant improvements over the earlier methods. Field data from instrumented structures further served as useful feedback for the design considerations.

Principle of Reinforced Earth

The basic mechanism of reinforced earth can be explained in several ways. A simple method of explaining the concept is by Rankine state of stress theory. If a two-dimensional element of cohesionless soil is subjected to uniaxial stress, it will not be able to remain in equilibrium as the Mohr circle of stress will cut the strength envelope of the soil (Fig. 1*a*). If the element is subjected to equal bi-axial stresses, it will undergo uniform compression (Fig. 1b). If one of the stresses (say σ_1) is increased while maintaining the other constant, a compression of the element in the direction of σ_1 and an expansion in the direction of other stress σ_3 will result. When the lateral strains reach critical proportions, failure of element results similar to the failure of a sample in triaxial compression test and σ_1 at the stage is related to σ_3 as $\sigma_3 = k_a \sigma_1$, where k_a is the coefficient of active earth pressure. At this instant the Mohr circle of stresses is tangential to the strength envelope. To hold the element without failure, the lateral stresses must be increased. If reinforcement is provided in the direction σ_3, interaction between the soil and reinforcement will generate frictional forces along the interface. Tensile stresses will be produced in the reinforcement and a corresponding compression in the soil element, as long as there is no slippage between the soil and reinforcement. It will be analogous to the existence of a pair of plates which prevent lateral expansion of the soil element (Fig. 1c). The additional lateral pressure will move the Mohr circle to the right and away from the failure envelope and the soil element will remain in equilibrium.

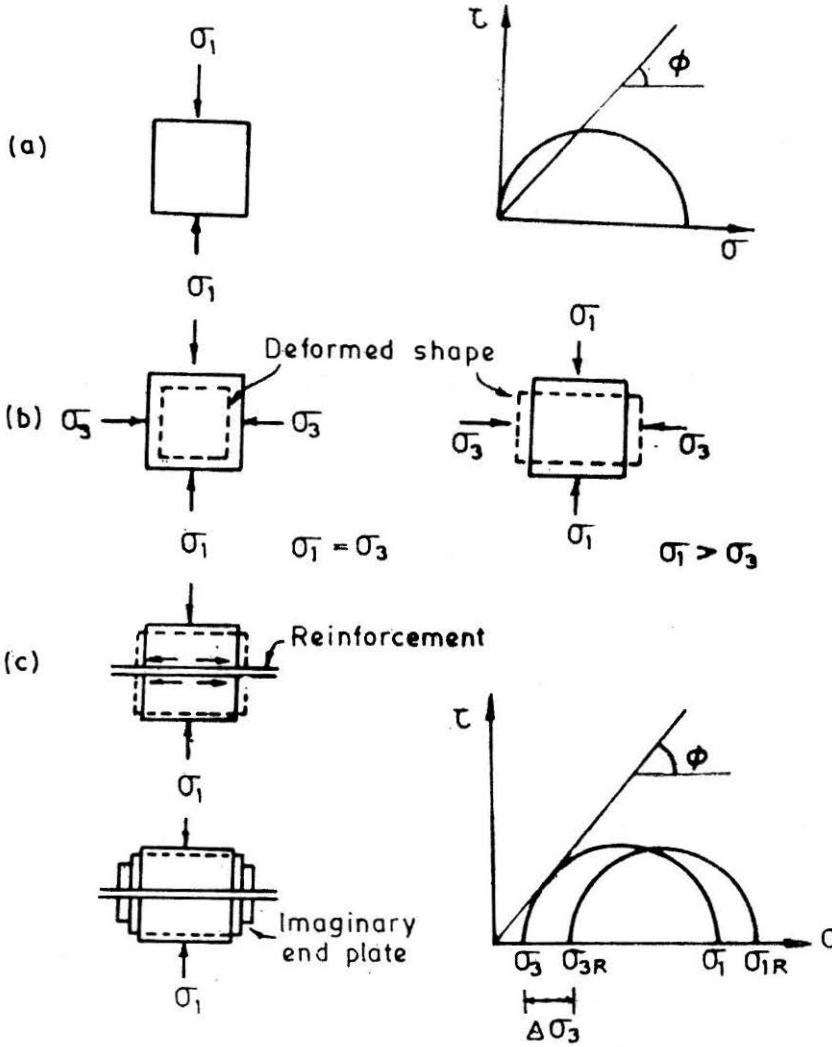

FIGURE 1 Basic Mechanism of Reinforced Earth

Thus soil-reinforcement friction is fundamental to the concept of reinforced earth.

Vidal (1978) describes reinforced earth as a cohesive material. The cohesion is assumed to be induced due to introduction of the reinforcement in an otherwise cohesionless soil. The anisotropic cohesion is produced in the direction of reinforcement and the concept is based on the behaviour of triaxial samples of reinforced earth. It has, however, not been possible to define this cohesion in a way as to enable its use in the design of reinforced earth structures.

A different concept of the influence of reinforcement on the behaviour of reinforced soil mass has been advanced by Bassett and Last (1978). It is suggested that introduction of reinforcement modifies the dilatancy characteristics of soil with possible rotation of principal strain directions. The concept is based on the fact that if the dilation of the soil is restricted, the shear strength mobilised will be higher. The presence of reinforcement in soil imposes a condition of restricted dilatancy. It also predetermines the principal incremental strain directions and rotates them relative to the unreinforced case. The principal stress directions are forced to follow suit and a redistribution of stresses results.

Strength Characteristics of Reinforced Earth

The study of reinforced earth as an equivalent homogeneous material has been undertaken by research workers in an attempt to understand its behaviour and to determine its strength characteristics. Long et. al. (1972) and Yang (1972) have been the pioneers in carrying out strength studies of reinforced earth. They have reported the results of triaxial compression tests on cylindrical samples of sand containing thin horizontal sheets of tensile reinforcing material. Yang used woven fibre glass netting and Long et. al. used aluminium foils, which were also used by Schlosser and Long (1974) in their studies. These studies concluded that :

1. strength increases with increase in confining pressure,
2. strength increases with increase in amount of reinforcement, and
3. failure of samples is due to rupture of reinforcement.

Typical sets of data from each investigation are presented in Figs. 2 and 3. Both investigators concentrated their subsequent analysis of the data on the portion of strength envelope above critical confining pressure where tensile failure developed in reinforcement, ignoring the condition where failure was governed by soil-tie sliding. Both investigators analysed the data in terms of single composite material and noted that within the tie breaking range, the strength envelope for reinforced specimen was parallel to the strength envelope for unreinforced sand. At failure maximum strength of sand and the soil were both assumed to be mobilised simultaneously. Beyond these basic assumptions, the two research workers took different paths, in proposing a working hypothesis.

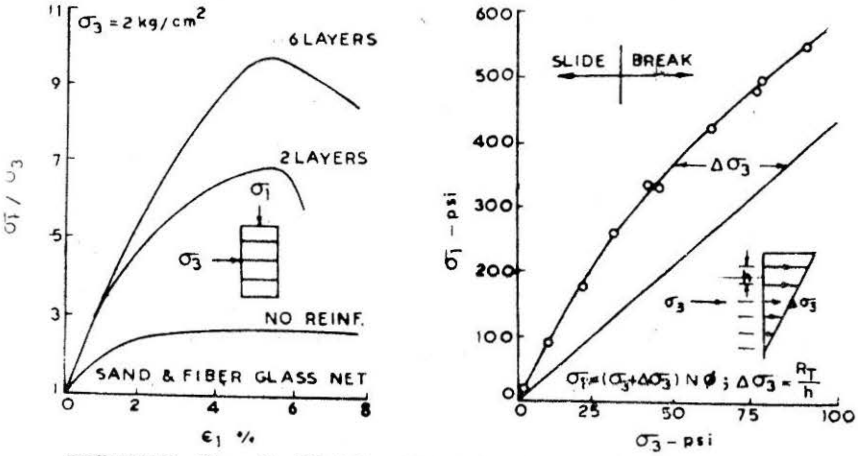

FIGURE 2 Strength of Reinforced Sand Specimens (After Yang, 1972)
(1 kg/cm² = 14.2 psi)

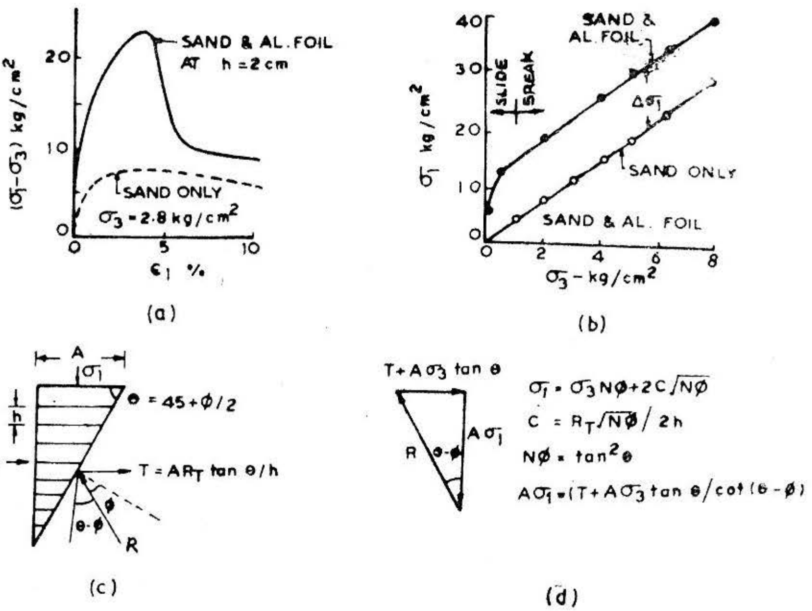

FIGURE 3 Strength of Reinforced Sand Specimens (After Schlosser and Long, 1974)

Yang suggested that the tensile stresses built up in horizontal reinforcing layers were transferred to the soil through sliding friction and caused an increase in confining pressure $\triangle\sigma_3$. It follows that :

$$\sigma_{1f} = (\sigma_3 + \triangle\sigma_3)\, N_\phi \qquad \ldots(1)$$

where,

σ_{1f} = major principal stress at failure

σ_3 = applied confining pressure on the test specimen,

$N_\phi = \tan^2\left(45 + \frac{\phi}{2}\right)$, where ϕ is the angle of internal friction of unreinforced sand.

Schlosser et. al. (1974) and Long et. al. (1972) interpreted the strength envelope for reinforced sand as that of a cohesive frictional Mohr-Coulomb material (Fig. 4) with the strength defined by

$$\sigma_{1f} = \sigma_3 N_\phi + 2c\sqrt{N_\phi} \qquad \ldots(2)$$

The additional strength $\triangle\sigma_1$ in excess of frictional strength of unreinforced specimens was interpreted as being the effect of a cohesion developed in new composite material. An analytical expression was derived for this cohesion term for the case of a unit thick plane section,

$$c = \frac{R_T\sqrt{N_\phi}}{2h} \qquad \ldots(3)$$

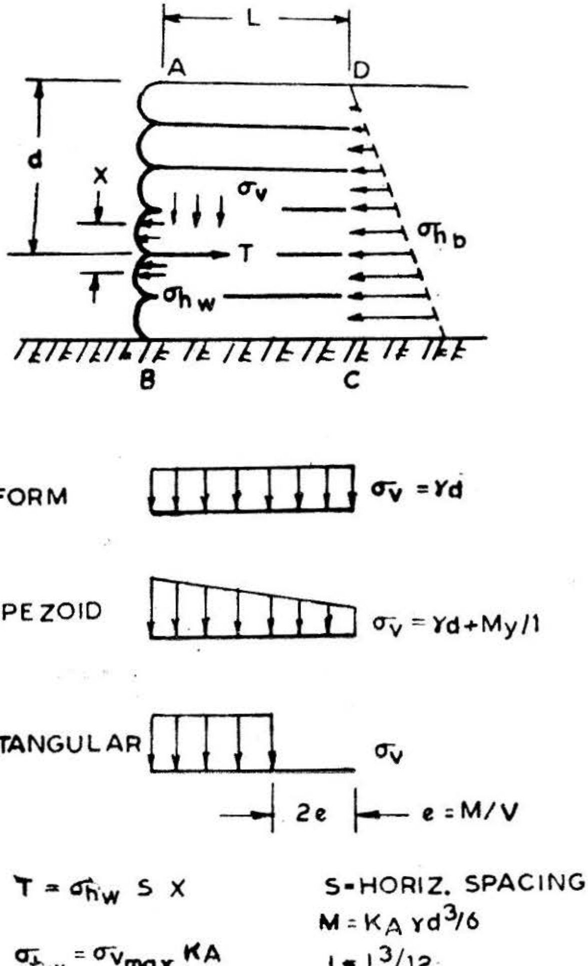

FIGURE 4 Assumption in Some Working Hypotheses on Reinforced Earth Walls

where,

R_T = tensile resistance of the reinforcing unit thick section and

h = vertical spacing between adjacent horizontal layers of reinforcement.

Comparing Eqns (1), (2) and (3), a direct relation between Yang's $\triangle\sigma_3$ and Schlosser's c is

$$c = \frac{\triangle\sigma_3\sqrt{N_\phi}}{2} \qquad ...(4)$$

Rearranging the coefficients from Eqns (3) and (4) leads to

$$\triangle\sigma_3 = \frac{R_T}{h} \qquad ...(5)$$

Thus either the '$\triangle\sigma_3$' or the 'c' approach could be used equally well for analysing the behaviour of reinforced earth for maximum strength conditions where failure occurs by breaking of the reinforcement.

These hypotheses are both limited to condition where failure occurs by breaking rather than sliding or pull out failure. Sliding failure is the more difficult of the two mechanisms to understand because of uncertainities in the basic soil-reinforcing sliding or pull-out properties. Furthermore, these approaches are also limited to failure conditions, with no provision for deformation prior to developing peak strength of the reinforcement. For these reasons, the '$\triangle\sigma_3$' or 'c' hypotheses have both been used directly in field design.

The increase in confining pressure $\triangle\sigma_3$ due to reinforcement was found to be a function of σ_3 and strength and concentration of reinforcement. It was found to increase linearly with σ_3 initially and then become constant. The value of σ_3 at breaking point being called the equivalent critical confining pressure. Failure due to slippage between soil and reinforcement was detected for σ_3 values less than critical value of σ_3 and failure due to rupture of reinforcement for values greater than that. The value of $\triangle\sigma_3$ was constant for the latter case. The increase in strength of reinforced sample was attributed to increase in confining pressure due to presence of reinforcement.

Hausmann's and Brom's investigations confirm the hypothesis that slippage failure leads to increased friction angle. Yang's experimental results as presented by Hausmann and Vagneron (1977) also support the hypothesis. The French test results (Long et. al. (1972)) suggest that a pseudo-cohesion is introduced when failure is caused by rupture of reinforcement.

Principal Types of Reinforcement

A wide range of alternatives exist in making a choice of reinforcing materials. However, these materials are prone to corrosion, and therefore suitable preservative treatment must be given before using the desired reinforcing materials.

Amongst the commonly used materials are Bamboo, Polypropylene, and Polyethylene. A comparative evaluation of some materials (Datye 1981) is listed in Table 1. Bamboo as a reinforcing material is subjected to the hazard of insects, fungus or virus attack. Considerable difficulty is also experienced in splicing the bamboos. However, if adequate preservative treatment is accorded, it can largely enhance the life of the bamboo. ASCU, a water soluble preservative or similar chemicals can prove to be quite effective.

Prof. H.Y. Fang of the Lehigh University has reported that bamboo strips embedded in concrete have a very long life. It has also been reported that bamboo members placed in annular holes filled with lime mortar in a composite construction of sun-dried bricks and bamboo have served for over 100 years. This bamboo was also free from insect attack.

Bamboo is commonly available in lengths of 3 m or so. However the development of designs of splices suitable for field application have also been reported and tests have confirmed joint efficiencies better than 70

TABLE 1

Comparative Evaluation of Reinforcing Materials

(a) Characteristics

Sl. No.	Chara-cteristics	Bamboo		Poly-propylene		Poly-ethylene		P.V C. (rigid)
		Poor variety	Good variety	High density	Low density	High density	Low Density	
1	Specific Gravity	—	0.7	0.9	—	0.97	—	1.35 to 1.50
2	U.T.S. kg/cm²	1000	2000	2100	1000	260	150	470 to 710
3	Elonga-tion at break	—	—	20	—	500	600	—

(b) Costs

Parameter / Material	Bamboo	Poly-propylene	Mild Steel bars	High tensile steel
UTS, kg/cm²	1500	6000	5200	—
Yield, kg/cm²	1200	—	3000*	12,000
Allowable stress, kg/cm²	600	3000	1400	9,000
Cost/litre, Rs.	3	60	28	60
**Equivalent Sectional Area, cm²	2.5	0.5	1	0.16
Cost of 1000 cm length with equivalent sectional area	7.5	30	30	9.6
Cost ratio	0.25	1	1	0.32

* Average value for commercial grades

** Area of member having a strength equal to a mild steel section with an area of 1 cm².

per cent. There is potential for further improvement in the splicing technique.

Polypropylene Strips have become available in India for the packaging industry. It is known to be a durable material and reported to have long service life. Some of its characteristics (Datye, 1981) are :

Size	: Range 9 to 12 mm (width) 0.5 to 0.6 mm (thickness)
Ultimate tensile strength	: 2100 kg/cm²
Cost/litre	: Rs. 22.00
Elongation at break	: 20 per cent (approx.)

The following considerations should be borne in mind while using bamboo as a reinforcing material :

(*i*) All bamboo used for engineering purpose should be seasoned attaining an age of approximately 3 to 6 years.

(*ii*) The best time for harvest of a bamboo is from late summer to mid-autumn, because at that time the natural moisture content of bamboo is low, and therefore the swelling-shrinkage potential is also low.

(*iii*) Sulphur-sand treatment of bamboo gives it a higher strength and a low water absorption potential.

(*iv*) The bamboo earth mat is composed of bamboo strips installed either horizontally or vertically in the soil mass. The length and spacing of the bamboo strips will depend on the surcharge weight above the location where the mat is to be placed. For reinforcement of existing or natural slopes, the vertical type of mat can be used; however, for the new embankment or for the mat foundation, the horizontal type is more suitable.

Methods of Analysis Based on Classical Earth Pressure Theories

Static Analysis of Reinforced Earth Retaining Walls : Figure 4 illustrates the working hypothesis for the static design of a reinforced earth wall. The zone of reinforced fill ABCD behind the wall is assumed for analysis purposes to behave as a cohesive composite body. The length L of the reinforcing ties is selected by experience for a first trial analysis ($L \geqslant 0.8\ H$). The horizontal stress σ_{hb} acting on the vertical plane DC in the backfill at the ends of the ties is assumed to be defined by the minimum active earth pressure equation

$$\sigma_{hb} = K_A\ \lambda\ d \qquad \ldots(6)$$

where d is the depth at which σ_{hb} is evaluated and K_A is defined by

$$K_A = \frac{1}{N_\phi} = \tan^2\left(45° - \frac{\phi}{2}\right) \qquad \ldots(7)$$

The internal stability of the reinforced mass ABCD is next evaluated by calculating the driving tie force T and comparing it with the maximum tie resistance in breaking and in pullout. The driving tie force is defined as the product of the horizontal earth pressure on the wall σ_{hw} and the tributary wall area supported by the ties spaced X vertically and S horizontally,

$$T = \sigma_{hw}\ SX \qquad \ldots(8)$$

The ties are flat strips with the wide side lying horizontally, with known breaking strength. The sliding pullout resistance is a direct function of the vertical normal stress σ_v on the tie and the angle of soil-tie sliding friction.

Assuming that the reinforced block ABCD acts as a cohesive composite body, the vertical normal stress σ_v along a tie may be defined by one of three different assumptions illustrated in Fig. 4 following the well known equations for the stress at the base of an eccentrically loaded footing; uniform, trapezoidal or rectangular. By any of the assumptions, the maximum vertical stress σ_{max} is located at the wall. The horizontal wall stress is then calculated as

$$\sigma_{hw} = K_A (\sigma_{Vmax} + q) \qquad \ldots(9)$$

where q is any vertical surcharge loading. After comparing the driving and resisting forces on ties at various elevations, if the first design assumptions are not adequate, new tie arrangements are selected and the process repeated. In calculating the tie pullout resistance, the entire tie length L is assumed to be effective in mobilizing soil-tie frictional pullout resistance.

A similar, but simpler approach assumes that the reinforced body ABCD is too flexible and similar to the rest of the backfill to warrant treatment as a separate structure, giving

$$\sigma_{hw} = K_A (\lambda d + q) \qquad \ldots(10)$$

This approach was originally suggested by Schlosser and Vidal (1969) and was used by Lee et. al. (1973). It is identical to the uniform pressure assumption in Fig. 4. In addition, Lee et. al. (1973) assumed that only the length of the tie extending behind a potential active Rankine failure plane should be used in calculating the tie pullout resistance, but the test generally showed this assumption to be conservative.

The results of some typical laboratory scale reinforced earth wall tests to failure performed at UCLA are shown in Fig. 5. For a tie breaking condition the failure height predicted by the simple Rankine approach

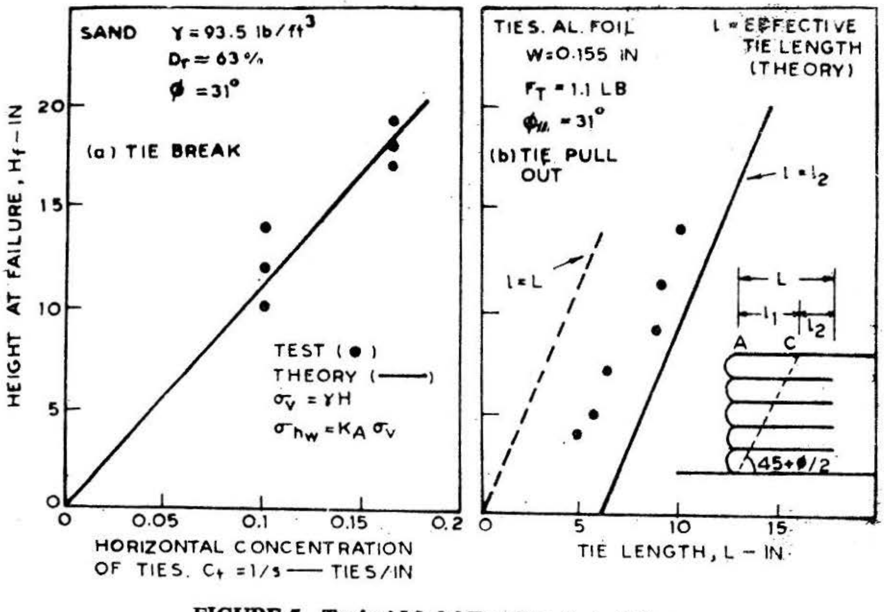

FIGURE 5 Typical Model Test Results to Failure
(1 cm = 0.394 inch)

gave adequate agreement with the experimental results. The experimental failure height by tie pullout indicated that walls could be built higher than predicted by the simple Rankine Theory, if the effective tie length counted only on the portion l_2 (Fig 5b) of the tie behind the Rankine failure plane. Apparently, the soil within the sliding wedge ABC makes some contribution to the pullout resistance. On the other hand, comparison of the measured data with the theoretical dashed line, assuming that the total length L is effective in resisting tie pullout, is unconservative.

Although one may question whether a failure wedge of soil actually exists, the test results similar to those shown in Fig. 6a are commonly observed in models when walls are loaded to failure. Whether failure is by tie breaking or by tie pullout the observed outer boundary of the failure zone always approximates the theoretical Rankine failure plane, strongly suggesting the existence of Rankine failure wedge. However, more research is needed to define the tie pullout resistance and especially the contribution of the soil within the zone defined by this failure wedge.

Comparatively little information has been published to aid in the design of reinforced earth retaining walls to resist dynamic loading. A typical set of tie force data obtained from laboratory scale tests at UCLA (Fig. 6b) shows that tie forces increase with increasing base accelerations.

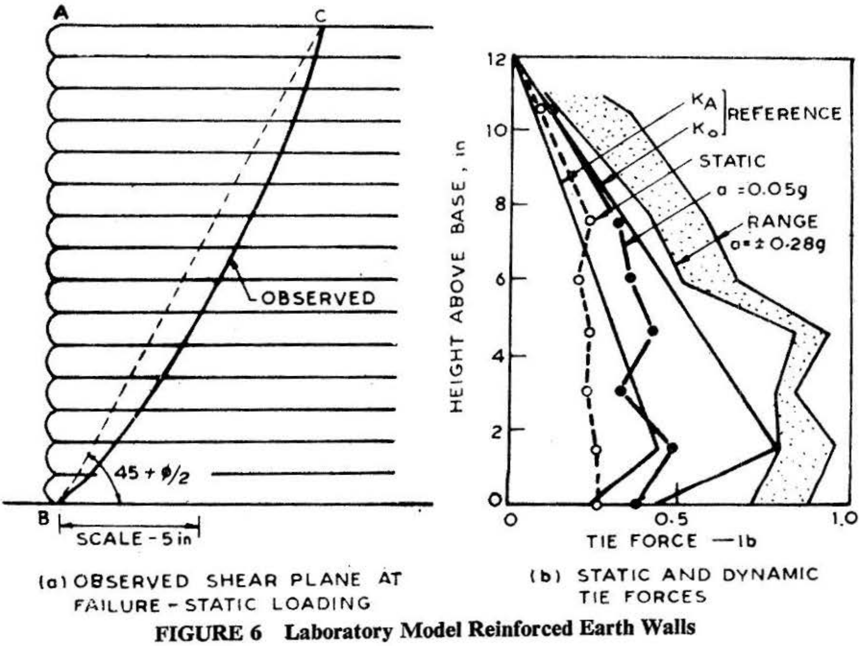

FIGURE 6 Laboratory Model Reinforced Earth Walls

Analysis As a Composite Material

Under the effect of the compressive stress σ_{n_1}, a cube of particles will deform plastically by contracting in direction of the compression and expanding in the direction of reinforcement. The reinforcement becomes elongated and exerts a tension between the two plates. This force is equal to the compression σ_{n_1} which is exerted on the earth in the direction of reinforcement. Sliding of particles stops when this compression reaches the value $K_a\ \sigma_{n_1}$, K_a being the active lateral earth pressure coefficient, and the initial cube of earth after deformation reaches a stable state of equilibrium. Hence, the traction force in the reinforcement can be calculated.

The apparent elasticity and hence elastic deformation depends only on the cross-section of reinforcement and material of elasticity of material of which it is made, as in case of deformation of steel-rubber composite beam (Fig. 7).

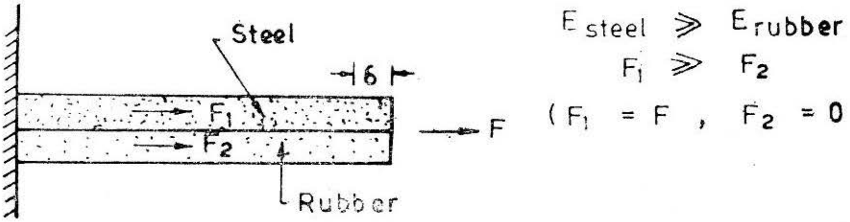

FIGURE 7 Analogy of Reinforced Earth with Steel Rubber Beam

Theoretical Models

If reinforced earth is assumed as a homogeneous but anisotropic material, the Mohr Coulomb failure criterion can be applied. Thus Hausmann (1976) postulated two theoretical models, the SIGMA model and the TAU model, for describing the strength of soil mass reinforced with horizontal reinforcement that fails by expanding in the direction of reinforcement. The essential difference between the two models relates to the assumed role of reinforcement. In the SIGMA model, the reinforcement is assumed to induce normal confining pressure σ_r in the specimen in the direction of reinforcement, whereas in the TAU model, the presence of reinforcement is assumed to introduce shear stresses τ_r. If the failure of sample occurs due to rupture of reinforcement, the induced stresses are assumed constant and related to the tensile strength of the reinforcement. When the failure is due to slippage between soil and reinforcement, these are assumed proportional to the initial vertical stress. The increase in the strength of the soil due to presence of reinforcement is reflected either by an apparent cohesion intercept c (failure due to rupture of reinforcement) or by increased friction angle ϕ_r (failure due to slippage), Fig. 8. In either case the increase in strength is a function of shear strength of soil and the tensile strength and distribution of reinforcement in the soil mass. The parameters defining increase in strength are given by Eqs. 11 to 14.

(I) *SIGMA-Model* :

$$c_r = \frac{\sigma_r}{2\sqrt{K_a}} \qquad \ldots(11)$$

and

$$\sin\phi_r = \frac{1+F_s-K_a}{1-F_s+K_a} \qquad \ldots(12)$$

where

$$F_s = \frac{\sigma_r}{\sigma_1}$$

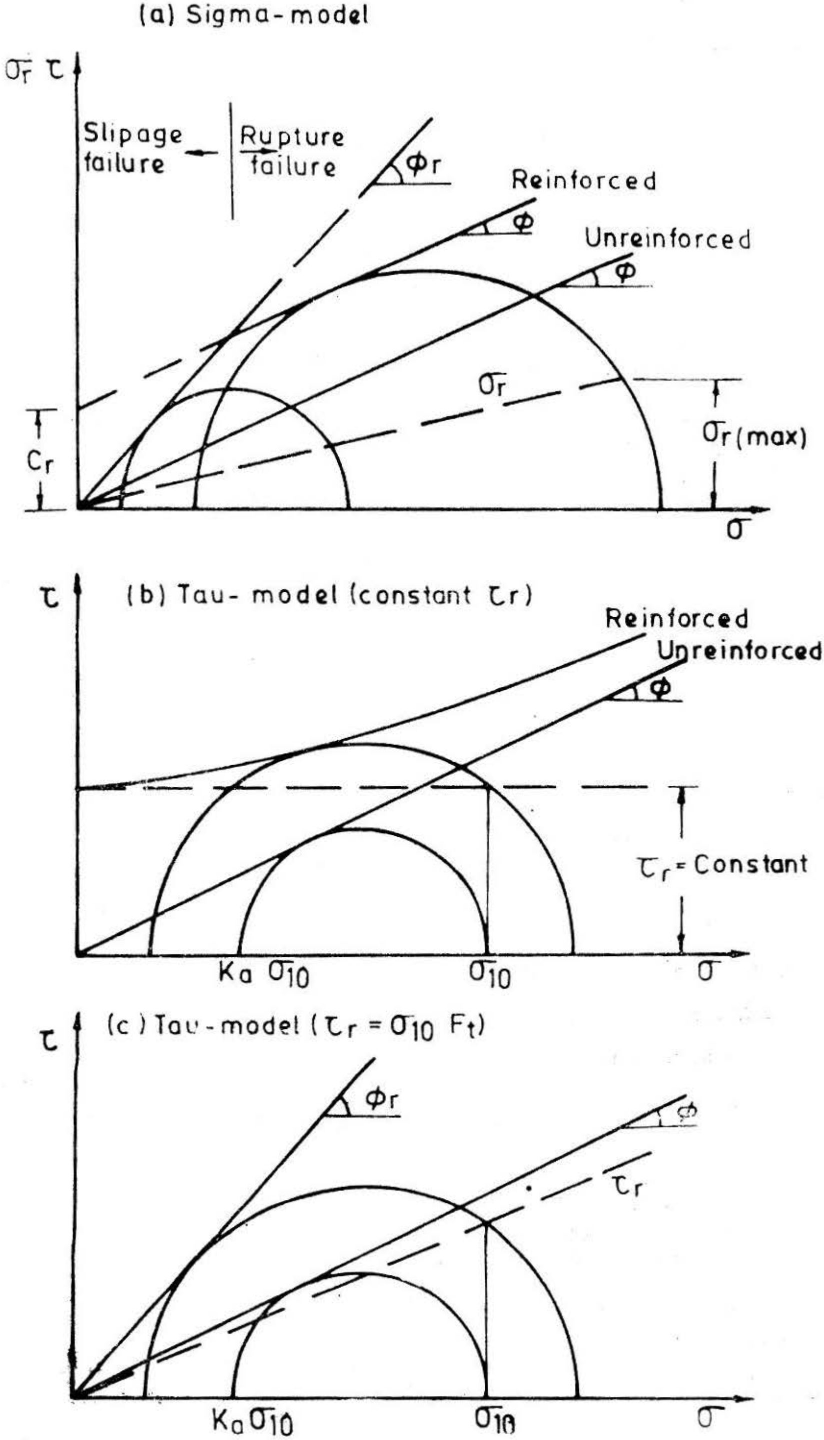

FIGURE 8 Theoretical Strength Models of Reinforced Earth (Hausmann, 1976) (1 m = 3.28 ft)

(II) *TAU-Model* :

$$\sin \phi_r = \frac{\sqrt{4 F_t^2 + (1-K_a)^2}}{1+K_a} \qquad \ldots(13)$$

where $F_t = \dfrac{\tau_r}{\sigma_1}$,

and $$\tau_r = \sqrt{\sigma_n^2 \, . \tan^2 \phi + \tau_{rmax}^2} \qquad \ldots(14)$$

where τ_r = induced shear stress

Because of imperfect transfer and non-uniform distribution of friction at the soil-reinforcement interface, reinforcing efficiency factors were incorporated in the expressions for F_s and F_t. For triaxial samples, F_s and F_t can be computed from consideration of soil-reinforcement friction angle ψ and the size and spacing of reinforcement as given in Eqs. 15 and 16 (Hausmann, 1976).

$$F_s = \frac{\tan \psi . \, r_o^2}{r_1 . \, \triangle H} e_s \qquad \ldots(15)$$

$$F_t = \frac{2 \tan \psi . \, r_o^2}{r_1^2} . \, e_t \qquad \ldots(16)$$

where e_s, e_t = reinforcement efficiency factors,

r_o = radius of reinforcing disc,

r_1 = radius of sample, and

$\triangle H$ = vertical spacing of reinforcement.

Results of Model and Prototype Tests

Tests conducted have revealed (Saran et. al. 1978, Narain et. al. 1981);

1. Triaxial compression tests on reinforced soil samples show a small decrease in the angle of internal friction but a substantial increase in the cohesion intercept if the failure of the sample is due to rupture of reinforcement.

2. Provision of reinforcement in cohesionless soil increases the ultimate bearing capacity and decreases the settlement of the footing. The optimum depth for location of first reinforcing layer below the base of footing is very close to 0.4 s.

Studies on Reinforced Earth Slab

The beneficial effects of reinforced earth slab as a foundation bed for footings have been described by Binquet and Lee (1975). They conducted model tests on reinforced earth slab overlying the following formations :

(*a*) Homogeneous deep sand.

(b) Sand above an extensive layer of very soft material simulating soft clay or peat.

(c) Sand above finite pocket of very soft material.

The sand box selected for tests was $152 \times 51 \times 33$ cm ($60 \times 20 \times 13$ in) in dimensions. A 7.5 cm (3 in), wide rigid strip footing spanned the width of the box. The footing was composed of three segments that were loaded as a unit, but only the middle section was instrumented to reduce the effect of side wall friction. The vertical loads were applied through a pneumatic bellofram piston. The reinforcing material adopted was aluminium strips $152 \times 1.5 \times 0.00127$ cm ($60 \times 0.5 \times 0.0005$ in) having a breaking strength 1.72 kg(3. lb). These strips were placed along the length of the box.

The desired density in the box was maintained by hand-held pneumatic vibrator. All the tests were performed by keeping constant density of soil (1.5 t/m^3), constant spacing between two reinforcing layer i.e. H=2.54 cm (1 in) and constant number of strips along the width of the test box (i.e. N_R=17). However, the number of layers of reinforcement N, and the distance from the ground surface to the top of uppermost layer, U, were varied.

The following qualitative trends were observed :

1. The pull-out failure generally occurred with lightly reinforced earth slab, whereas tie breaking which occurred in uppermost layers, was generally associated with heavily reinforced slab.

2. In all cases where tie did break, it was observed that the rupture was located approximately below the edges of footing.

3. Reinforcement at depth greater than 1.5 B also contributes to increase in bearing capacity.

4. For every soil condition, there is an optimum arrangement of reinforcement giving maximum increase in BCR*.

Binquet and Lee (1975) have addressed the analytical problem of bearing capacity of strip footing on granular soil containing horizontal layers of tensile reinforcement. Based on model test observations, following three possible modes of bearing capacity failure are considered :

1. Shear failure above uppermost layer of reinforcement : This failure is likely to occur if the reinforcement concentration in the uppermost layer is sufficiently large to form an effective lower rigid boundary into which shear zone will not penetrate.

2. Tie pull-out : This type of failure appears to occur if reinforcements are too short to mobilize required frictional resistance.

3. Tie Breaking : This type of failure is likely to appear with long, heavy reinforcement. Usually only one to three layers of tie were found to be broken in any particular test.

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*BCR (Bearing Capacity Ratio) = $\frac{q}{q_o}$ where q and q_o are the average bearing pressures for reinforced and unreinforced soil at desired density respectively.

Economy as Compared to Conventional Walls

The flexible nature of reinforced earth mass enables it to withstand large differential settlement without distress. Reinforced earth thus permits construction of engineering structures over poor and difficult subsoil conditions. Reinforced earth walls are consequently economical when height of structure is large, or ground conditions are unfavourable and suitable backfill materials are locally available, Savings of the order of 20 to 50 per cent have been recorded in many cases by adopting reinforced earth structures.

With reinforced earth, as with most types of construction, the key criterion for success is to do the required job at the least cost. Thus the advantages of reinforced earth must be closely associated with, and strongly dependent upon, the relative cost as compared to other possible solutions. Therefore, in order to investigate whether serious studies of reinforced earth might be considered for special applications, a comparative cost analysis was undertaken.

It was decided to compare costs of constructing reinforced concrete, metal bin, and reinforced earth retaining walls under ideal simple conditions with a definite intent to avoid complicated particular situations.

The results of this comparative cost analysis are summarized in Fig. 9, which shows the estimated cost per running foot of wall versus the wall height. These curves show that for the ideal assumed conditions,

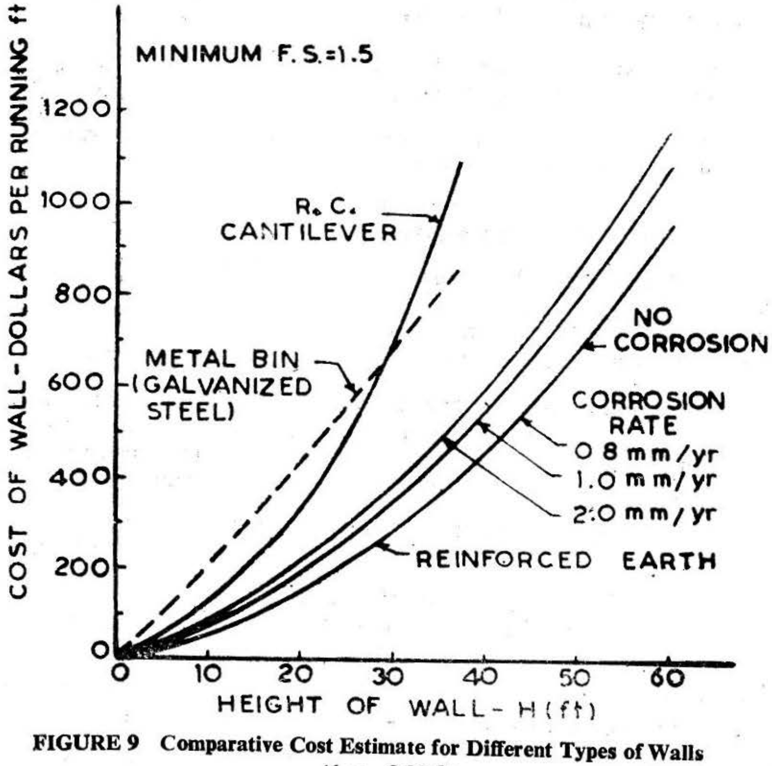


FIGURE 9 Comparative Cost Estimate for Different Types of Walls (1 m=3.28 ft)

reinforced earth walls are expected to cost about half as much as a cantilever or crib wall of the same height. Furthermore, corrosion does not appear to be a major cost item in that reasonable allowances for it does not increase the overall cost by an unduly large amount.

It is to be expected that any particular case will involve special situations leading to different costs than those shown. Nevertheless, on the basis of this comparison it is felt that the concept of reinforced earth merits further consideration as a possible solution to some earth retaining problems.

Because a reinforced earth wall uses small prefabricated components and requires no formwork, it can be readily adapted to required variations in height or shape. There is no theoretical upper limit to its height, and the probable practical height limit would appear to be greater than for conventional walls. By using precast concrete skins, a wide variety of external designs can be used for different esthetic or architectural appearance. Because it uses relatively small amounts of reinforcing, there is the possibility of using special high strength, corrosion resistant materials such as fiberglass for special installations (Lee et al. 1973).

Corrosion of Reinforcement and its Prevention

Corrosion is defined as the destructive attack of a metal by chemical or electrochemical reactions with its environment. The initiation and propogation of corrosion is governed by corrosion tendency and corrosion rate (Boyd et. al. 1978).

The chemical properties of soil of most interest in corrosion studies are those related to the solubility of its constituents in water. Acidity and alkalinity are then measured by pH. The physical properties of soil which affect corrosion are those which determine the aeration of the soil and its permeability.

Studies by the U.S. National Bureau of Standards have established that :

(*i*) For Zinc and Ferrous Metals :

(*a*) none of the commonly used ferrous metals are immune to corrosion in all soils,

(*b*) the rate of corrosion is controlled by the characteristics of the soil and varies widely in different soils, and

(*c*) the plain iron and steel metals corrode at nearly the same rate in the same soil environment.

In general, in well drained soils having high resistivity such as sandy and silty loams, tests showed that the corrosion rate of ferrous metals decreased after a few years from an initial high rate to an insignificant rate. Poorly aerated soils and well aerated soils with high concentration of chlorides and sulphates, acidity or alkalinity were shown to be corrosive to zinc (Fig 10).

The protection afforded by zinc coatings on steel (galvanised) is

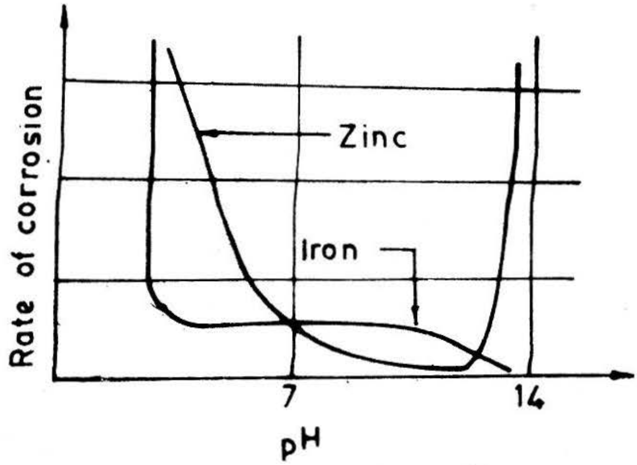


FIGURE 10 Corrosion Rate vs pH

derived from :

(*i*) The higher corrosion resistance and lower corrosion rate of zinc to steel.

(*ii*) The fact that zinc is generally anodic to steel thus providing *Cathodic protection* at cut edges, pits etc., and

(*iii*) The retarding effect of zinc solution on the corrosion of exposed steel by virtue of an increased local *pH*.

Use of Reinforced Earth in Foundation Embankments and Non-Conventional Uses

It is in the field of retaining structures that reinforced earth has found its widest development. Reinforced earth masses acting as heavy and flexible structures can replace conventional structures such as retaining walls, quay walls and bridge abutments. In some cases it can result in considerable economy; and in view of its flexibility it may even, in certain cases, be the only validly applicable technique.

The use of the reinforced earth technique for earth-retaining structures is often dictated by technical and economic considerations.

Reinforced earth is often chosen for the following reasons :

1. The flexibility of a reinforced earth structure allows it to withstand considerable total and differential settlements without failure of the structure.

2. The construction of a reinforced earth wall is relatively fast since it can be built in the same manner, and at the same rate, as an ordinary highway embankment.

3. A reinforced earth structure may be built in stages like an embankment when either the foundation soils or the soils to be excavated are unstable.

4. Cut slope reinforced earth retaining walls require more excavation than conventional structures. This may create conditions of marginal slope stability particularly in cases where the existing slope is steep.

5. Construction techniques for reinforced earth structures built in the dry are well developed and fully understood. However, further study is required to develop procedures for constructing reinforced earth in water as in the case of docks.

Four different applications of the reinforced earth technique may be listed for general retaining structures (Fig. 11).

I. Structures founded on good foundation soil

When the foundation soil is stable, the decision to build either a reinforced earth wall or a conventional retaining wall is made on the following criteria :

Fill slope or embankment walls

No technical considerations are involved. The criteria are economic and aesthetic, and a separate decision must be made in each case.

The savings obtainable from the use of reinforced earth walls increase with the height of the wall.

In both cases, the reinforced earth solution may present a technical advantage if the foundation soil is very stable (for example, when it consists of rock, pavement, etc.). A reinforced earth wall may in fact rest on the ground surface even though the surface may be very irregular, whereas a standard concrete wall requires a footing whose width increases as the wall becomes higher, as in the case of cantilevered reinforced concrete walls.

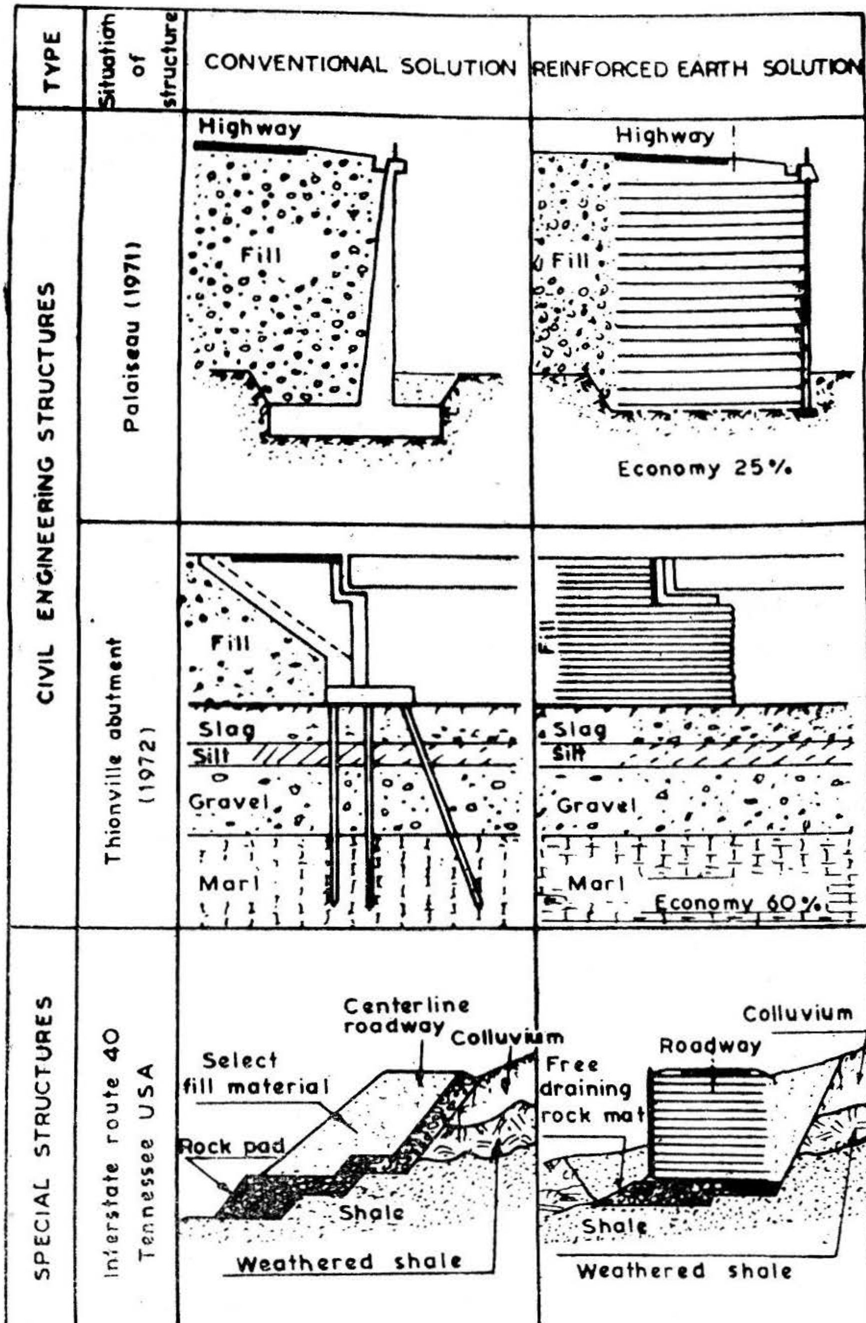
Cut slope walls

Whenever the excavation dictated by a reinforced earth solution is technically possible, the choice between reinforced earth and a standard concrete wall will be made on the basis of economic considerations. It should be kept in mind that in mountainous areas where random fill may be of poor quality, there may be difficulties in supplying adequate backfill material. On steep slopes, in cases where considerable excavation may be called for, problems of temporary stability may be encountered. This technical difficulty sometimes makes it necessary to eliminate the reinforced earth solution, and to opt instead for a reinforced concrete wall which is tied back to the slope.

II. Structures founded on poor foundation soil

When the foundation soil has a low bearing capacity, the reinforced earth solution is always very economical. Indeed, because of its flexibility and its ability to support significant settlements, it precludes the need for deep foundations which would be necessary for a conventional structure in reinforced concrete.

At the same time, the use of reinforced earth avoids certain difficult technical problems associated with deep foundations situated close to embankments : lateral pressure on piles, negative friction, etc.


TYPE
Situation of structure
CONVENTIONAL SOLUTION
REINFORCED EARTH SOLUTION
CIVIL ENGINEERING STRUCTURES
Palaiseau (1971)
Highway
Fill
Highway
Fill
Economy 25%
Thionville abutment (1972)
Fill
Slag
Silt
Gravel
Marl
Slag
Silt
Gravel
Marl
Economy 60%
SPECIAL STRUCTURES
Interstate route 40 Tennessee USA
Centerline roadway
Colluvium
Select fill material
Rock pad
Shale
Weathered shale
Colluvium
Roadway
Free draining rock mat
Shale
Weathered shale

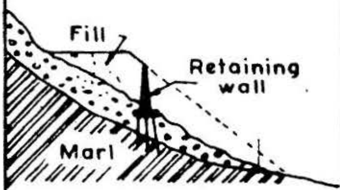
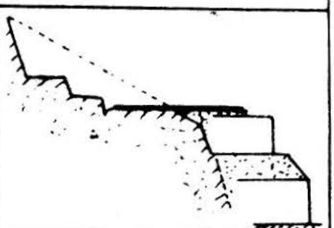
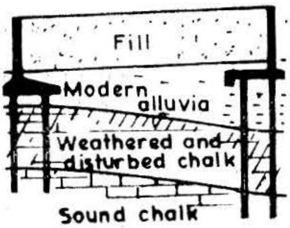
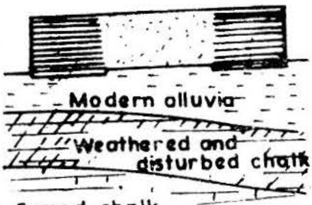
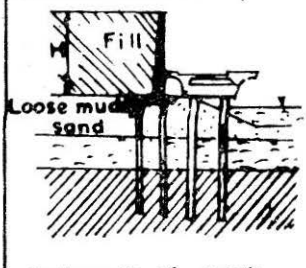
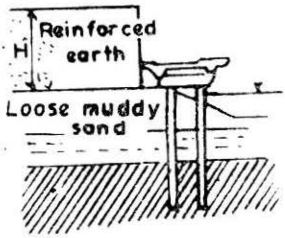
TYPE	Situation of structure	CONVENTIONAL SOLUTION	REINFORCED EARTH SOLUTION
RETAINING WALLS IN MOUNTAINOUS COUNTRY	Highway A 8 (Roquebrune - Vigna II - 1968 Menton)	 - Unstable scree - Deep foundation necessary brut risk of shearing of piles	 - Flexibility of reinforced earth in relation to movement of scree - No deep foundations Economy about 30 %
URBAN RETAINING WALLS	RN 187 - F 18 Connection with Pont de sevres (1971)	 - Subgrade of weak bearing capacity - Deep foundations necessary	 - No deep foundations - Economy 45 %
	Sete interchange (CD 2) (1971)	 - Subgrade of weak bearing capacity - Deep foundation necessary	 - No deep foundation - Economy about 50%

FIGURE 11 **Examples of Typical Reinforced Earth Retaining Structures built between 1968 and 1972**

III Special cases

In certain difficult situations, reinforced earth is the only valid technical solution. Such is the case when the retaining structures to be built are required to be flexible because of deformation and displacement of foundation soil which cannot be controlled. The most typical example is that of retaining structures to be built on a very long unstable slope, where the installation of a reinforced earth structure neither improves nor aggravates the problem of general stability.

On the other hand, it should be noted that the use of reinforced earth in slips of average size is only efficient in cases where there is an abutment at the foot of the slip combined with drainage in the same place, the abutment being sufficient to stabilize the whole slip (the reinforced earth wall, constructed with draining fill material, plays in fact the dual role of retaining wall and drainage mask). In all other cases, the conventional methods of strengthening must be used : drainage within the mass, reduction of the slope, strong abutment at the foot by anchorage or by a mass of large volume.

By using reinforced earth in combination with other types of structures (metallic or others), original solutions can be brought to complex problems. An example of this is the new redevelopment of the Port Storage Park, designed and executed by the Port Autonome de Dunkerque in France.

IV. Temporary or provisional structures

On certain sites, notably those in urban areas, it is necessary, in order to maintain traffic flow, to build temporary, resistant retaining structurets which are well designed and which can be converted into permanent structures if necessary once work is in progress. The main characteristic of a temporary retaining structure is that it can be easily dismantled, and thas the wastage of its constituent materials is kept to a minimum. In thid respect reinforced earth is a good material to use. It can be quickly ane simply dismantled, and only earthworks need to be carried out. Th. facing elements are recovered and can be used again for another structure In contrast, there is a risk of the reinforcement being severely damaged during dismantling.

In certain problems of instability, it is sometimes necessary to build a retaining wall very quickly to support a dangerous structure for a short period, in order that work can begin on more efficient methods (drainage, underpinning). Likewise in such cases reinforced earth can constitute an interesting procedure (Schlosser and Vidal 1969).

Scope of Reinforced Earth in India

Though several thousand reinforced earth structures have been built in Europe and USA since the inception of the technique in 1966, not much progress is seen in India probably due to the constraining factor of availability and cost of reinforcing materials. Consequently not much detailed model/prototype testing has been undertaken so far giving an impression that the savings achieved are not sufficient to justify import of materials and technology and an element of risk is involved if the techniques are to be used for major structures.

Reinforced earth as compared to stone masonry construction, will continue to be expensive until materials like galvanised iron strips and polymer fabrics, which are used abroad, are replaced by low cost alternative materials.

A new type of reinforcement system (Datye, 1981) consisting of loops, spirals and rings has been suggested and it is considered that this system has a very good potential for reducing the volume proportion of the reinforcement without compromising the performance. Analytical studies and model experiments are needed to investigate the structural behaviour of the new reinforcement system and to establish design guidelines.

Studies on Reinforcing of Clays

The scope for application of reinforced earth would be further enhanced if soil reinforcement can be used for cohesive soils and expansive clays. This is possible only if the system of reinforcement is such that its performance does not depend on the adhesion of soil and reinforcement. The granular soil not only has good frictional resistance but is also free draining and generally less corrosive than a cohesive soil.

Research is being carried out into the possibility of using clay as a fill material for reinforced earth. As clay is probably the most common soil encountered in many countries, encouraging results from such research would be of interest. Any potential benefits arising from the local availability of such soil could be outweighed by the penalties that might arise, such as, difficulty in handling, development of pore water pressures and the greater risk of corrosion.

Ingold has conducted work in 1982 and reported his work on reinforced clay subject to undrained triaxial loading. When fully saturated clay is reinforced with a continuous impermeable reinforcing medium and subjected to rapid undrained loading, the reinforcement causes a reduction in strength rather than an increase. Tests involving the reduction measurement of pore water pressure are consistent with the notion that such a reduction in strength is related to a reduction in minor principal effective stress caused by the outward radial migration of high pore water pressure generated at the centre of the sample. A limited number of tests on partly saturated clay with impermeable reinforcement again confirmed that strength is reduced in clay with high degree of saturation. As degree of saturation decreases the strength ratio rises, until reaching a degree of saturation of approximately 70 per cent the strength ratio achieved equals that obtained under fully drained conditions.

Ignold and Miller have also conducted studies on reinforced clay and reported their work 'Drained Axisymmetric loading of Reinforced Clay'. Studies on reinforced clay are picking up momentum despite enormous restraints and it is hoped that a breakthrough will be available in time to come.

Future Outlook

High costs being the major constraint for the development of reinforced earth techniques in India, priority should be accorded to the investigation by field trials for the use of less expensive reinforcing materials like synthetic fibres, which possess adequate surface roughness and strength.

Their use should be encouraged in reinforced earth walls of smaller heights (< 4 m) in order that the economy of such walls vis-a-vis other types of retaining walls is improved.

The material properties of reinforced earth as determined by triaxial compression tests cannot be directly utilised for design of reinforced earth structures as the configuration of reinforcement in field structures is not simulated in triaxial samples. Means should be devised for proper simulation of field reinforcement in triaxial samples.

The experimental research on reinforced earth has almost exclusively been confined to the use of frictional soil in the backfill. Methods and materials for reinforcing of cohesive soil also need to be investigated. In mining and materials handling structures requiring vertical walls such as loading ramps or conveyor feeders, reinforced earth can provide an economic arrangement. Marine structures for wharves, reclamation areas or sea walls have been built in a few countries like Australia using reinforced earth.

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