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The development of multi-anchor earth retention systems

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Introduction

The technology of earth retention has undergone a revolution world-wide over the last 15 years. Tsagareli (1969) set out a number of Soviet advances beyond the conventional reinforced concrete cantilever retaining wall. These included the introduction of buried, precast relieving platforms offering increments of reverse bending moment where they were fixed to the wall stem with the objective of limiting the overall maximum bending moment and thereby reducing the stiffness and cost of the whole construction. Vidal (1969) initiated an alternative technology which he called Reinforced Earth, featuring flexible strands of horizontal tensile "reinforcement" attached at regular intervals to a flexible or articulated facing system. This latter method has proved seminal in the creation of a new generation of retention systems relying on the tensile strength of buried strips or sheets, many of them employing new materials such as glass-fibre reinforced plastics or oriented polymers.

The development of centrifugal modelling, at least in the West, has occupied a similar span of time, and it is natural that a number of earth retention research projects have been carried out on centrifuges. This paper discusses two such projects, the first relating to tests on 'reinforced earth' conducted on the 1.5 m radius centrifuge at U.M.I.S.T. and the second concerning a short pilot study on the T.R.R.L concept of 'anchored earth' carried out on the 5 m radius Cambridge Geotechnical Centrifuge. The principal objective of the paper is to introduce evidence which demonstrates that failure mechanisms of novel forms of construction

(i) are often difficult to <u>predict</u> analytically
(ii) are relatively easy to create and observe in centrifuge models
(iii) are often easy to match <u>retrospectively</u> against an elementary theoretical model.

The paper closes by proposing a role for centrifugal model testing in design evaluation, whether by the designer himself, his certification authority, a rival patent holder, or a research worker.

Reinforced sand

The reinforced earth principle was initially tested using the UMIST centrifuge on a variety of models constructed using paper facing, sand fill, and nylon fishing line of various strengths as the reinforcement. The qualitative understanding gained in this period 1972-75 was subsequently of use as the basis for a more thorough study of the limiting equilibrium of a large number of carefully engineered models in the period 1975-1980. The outcome of the work on sand reinforced with metallic ties was reported by Bolton and Pang (1982), and will be summarized briefly below.

Firstly, it was shown that model walls with a uniform distribution of reinforcement similar to that shown in figure 1 could conservatively be analysed at failure as a flexible bulkhead carrying active earth pressures and supported at regular intervals by friction strip anchors. Figure 2 shows the outlines of a method of plastic stress analysis, first proposed by Schlosser and Long (1974), which was used to create simple expressions for the safety factor F_F against friction failure and F_T against tensile failure. The active thrust on the facing area served by a single anchor was taken to be enhanced by the overturning effect of the wall's backfill outside the reinforced zone:

$$T = S_v\, S_h\, K_a\, \gamma Z\ (1 + K_a\, Z^2/L^2) \qquad (1)$$

This was compared with the tensile strength of the anchor, P, and with an assumed friction resistance

$$Q = 2BL\, \mu\, \gamma Z \qquad (2)$$

$$\text{Then } F_T = \frac{P}{T} = \frac{P}{S_v\, S_h\, K_a\, \gamma Z\ (1 + K_a\, Z^2/L^2)} \qquad (3)$$

$$\text{and } F_F = \frac{Q}{T} = \frac{2BL\, \mu}{S_v\, S_h\, K_a\ (1 + K_a\, Z^2/L^2)} \qquad (4)$$

Figures 3 and 4 indicate the results of back analysing various model collapse scenarios using these expressions. The direct shear apparatus was employed for both μ and ϕ', and the mean value of the peak strength of samples tested at an appropriate density and confining pressure was used in the analysis. The figures relate to the lowest calculated safety factor of any single anchor. The bulk of the Phase I tests of walls with flexible facing are fully reported in Choudhury (1977).

The accuracy of the friction criterion was quite good, but there was an apparently erratic underprediction of tensile strength by a factor of upto two in some cases. Furthermore, every model test collapse was dramatic, whether due to loss of the dilatant component of ϕ'_{max} in the sand as the centrifuge acceleration increased, or whether due to tension failure in the anchors. This raised the question of whether the failures could in some way be described as brittle. These conflicting concerns were investigated in Phase II of the testing programme using the fully instrumented models depicted in figure 5, and including a large number of strain gauge bridges recording tension in the anchors at various positions. They are fully described in Pang (1979).

Three sources of strength not accounted for in the simple analysis were identified. Firstly friction against the back surface of the facing could, if the facing were stiff enough to carry the accumulated thrust, reduce the neighbouring vertical stresses in the soil and thereby the

active pressure on the wall. It was shown that flexible foil facing usually permitted the full trapezoidal enhancement of bearing stress assumed in figure 2, whereas with self-supported, stiff, panel facing the neighbouring vertical bearing stress was roughly equal to the simple overburden pressure.

Secondly, although the earth pressures on the facing in the upper three quarters of the wall caused tensions to increase with depth in line with (1), the anchor tensions in the lowest quarter failed to increase with depth. This was not wholly attributable to the reduction in vertical soil stress due to the skin friction referred to above. A further 40% reduction in tension at the base was attributed to the effects of friction on the stiff horizontal surface upon which the models were constructed.

Thirdly, there was clear evidence of redistribution of stresses following the attainment of full tensile strength in anchors at some level. It was this variable potential for redistribution which caused the erratic nature of tension-induced collapses. It was shown that the data from identical models suffered negligible scatter until some anchor first reached its tensile strength. If plastic redistribution were possible, it was shown that the self weight of a model could then be increased a further 46% before collapse. However, apparently identical models could suffer the premature rupture of some anchors if they were brittle (extension to rupture in the work hardened aluminium strips being only of the order of 0.3%). This did not necessarily lead to immediate collapse, but it did lead to comparative weakening. The weakest of the group of identical models collapsed just as the most heavily loaded anchors attained their tensile strength. It was further observed that plastic redistribution of tensions failed to take effect when the anchors were also on the point of pulling through due to insufficient friction.

The following conclusions were drawn.

(i) Vidal's concept of reinforced earth as a new composite material was of no help in organising the data of collapse. On the contrary, his technique could best be analysed using the concept of an anchored bulkhead.

(ii) A simple method of stress analysis was shown to be conservative in relation to all observed collapse scenarios, when appropriate parameters derived from conventional tests were used.

(iii) The extra degree of security sometimes observed in tests was attributed to frictional arching and plastic redistribution of limiting tensions. Neither of these effects could easily be allowed for in design.

(iv) The nature of tension collapses of reinforced earth retaining walls were not as they had been supposed. Peak tensions had been observed in a zone parallel to, and only a short distance behind, the facing. Critical anchors at ¼ height usually reached their tensile strength first. If the anchors were brittle, some could break without this being obvious externally. The final rupture sequence frequently propagated dynamically either along the facing joints (even though these were full-strength) or along the hypotenuse of an active failure wedge, breaking anchors at positions which were remote from failure an instant before. Back analyses based on the assumption of an instantaneous plastic mobilization of strength were therefore as erroneous and misleading as their analytical counterparts based on the method of assumed rupture mechanisms.

Reinforced clay

Pang (1979) describes a single centrifugal model test on reinforced clayey soil using the package already depicted in figure 2. The clayey soil was a 'brick-earth' from Berkshire, a material laid down in periglacial times, and consisting of 15% sand, 50% silt and 35% clay size particles. It can be found at a natural moisture content of about 18%, compared with its plastic limit of 20% and its liquid limit 37.5%, whilst possessing an effective angle of shearing resistance of 39° (data from Pomfret (1976)). It appears that this material may fairly be taken to represent a wide category of superficial, low-plasticity, glacial clays which can now be found in a firm to stiff condition, and which may be considered the most suitable clays for exploratory use as compacted backfill. In order to remove gravel, the soil was dried, milled and sieved (No. 14): this amended its engineering properties to a small extent which was determined.

It was found that the optimum water content for standard compaction was 16% at which its shear strength (taken as half the unconfined compression strength) was 160 kN/m^2. It was decided, however, to attempt to replicate behaviour at, or slightly wetter than, the natural water content. The shear strength of the soil after standard compaction at 18% water content was roughly 110 kN/m^2, with an air content of 2%. The relatively high degree of saturation of the clay compacted in its natural condition led to the decision not to compact the soil in layers in the model wall but rather to preconsolidate the reconstituted soil to the appropriate condition, extrude it, and then slice complete sheets of the clay which could be interleaved with reinforcements.

One dimensional compression under 300 kN/m^2 brought a 545 x 300 x 300 mm clay block to roughly 20% moisture content and a shear strength of about 125 kN/m^2, being almost saturated. Having been sliced, incorporated in a reinforced wall, and tested quasi-undrained, the average condition of the clay was then found to be

water content	m	= 19.5%
bulk weight	γ	= 20.4 kN/m^3
dry weight	γ_d	= 16.8 kN/m^3 (94% of optimum standard compaction value)
air content	A	= 2%
undrained strength	c_u	= 108 kN/m^2
secant modulus	E_u	= 13000 kN/m^2 (at ε_1 = 1%).

It is not thought that strength variations during testing would have exceeded ± 10%. It was therefore felt that the soil used in the model was not dissimilar to the condition which would have been achieved with the brickearth in the field.

Oedometer tests indicated a coefficient of consolidation C_v = 1.32 mm^2/min, so that the influence zone of consolidation during the 75 minute test may be estimated to be $(12\ C_v t)^{\frac{1}{2}} \simeq 35$ mm. This is consistent with the observation that the average bulk properties of the soil changed little in the course of testing, allowing the description 'quasi-undrained' with regard to the negligible capacity for drainage at the model boundaries. The centrifuge test carried the model to 116 g in 75 minutes: an average rate of construction in the field of 3 m per month can be deduced on the assumption that equivalent drainage times are to be scaled by n^2 at n gravities. This rate of construction is not unreasonable from a practical standpoint.

The aluminium reinforcing strips were 160 mm long, 4 mm wide and 0.1 mm thick. Their tensile strength had been determined to be 50 N, and 25 of the strips had been instrumented with a fully active strain gauge bridge at some location, and had been calibrated in tension by the application of weights. All strips were painted with a water resistant coating (Mcoat D) in order to ensure constancy of surface conditions and in order to protect the gauges. Shear box tests were conducted in which an upper block of the clay from the press (in the same condition as that used in the model) was forced to slide over the painted strips. The rates of sliding used were 0.2 and 1.2 mm/minute, the faster of which certainly correspond to effectively undrained shear on the interface. For the normal stresses in the range 0 to 120 kN/m^2 the mobilised shear strength at each speed was very well modelled by an interfacial friction angle ϕ' of 29°, leading to shear stresses in the range 0 to 66 kN/m^2. It is not envisaged that in an undrained test the interface shear stress could exceed the strength of the parent clay, which was 120 kN/m^2. It was anticipated, but was not proved, that in the range of normal stress above 120 kN/m^2 the mobilised shear stress would depart from a 29° friction relation and become asymptotic to some particular value not exceeding 120 kN/m^2 and not less than 66 kN/m^2.

The initial friction condition based on total stress σ, notwithstanding that extremely strong pore water suctions must have been in existence within the sample, and notwithstanding the habitual use of effective stresses σ' rather than total stresses σ in the friction equation, is taken to relate to the retreat of the water menisci inside the soil surface at its interface with the reinforcing material, so that the interfacial pore pressure was zero, corresponding to atmospheric air. By this argument, the strength of the interface would remain frictional against total normal stress until the internal pore pressures became positive, allowing water to invade the interfacial air pockets. The undrained shear strength of the interface under larger normal stresses would then remain constant, as the effective normal stress remained constant. By this argument the undrained shear strength of the interface can be expressed as

$$\tau_i = (\sigma_i - u_i)\tan\phi'_i \tag{5}$$

where the normal interfacial stresses (σ_i, u_i) might be related to the values (σ, u) in the hinterland by the following.

In the absence of 'arching'

$$\sigma_i \simeq \sigma \tag{6}$$

$$\left.\begin{array}{ll} u_i = u & \text{for } u > 0 \quad (a) \\ u_i = 0 & \text{for } u < 0 \quad (b) \end{array}\right\} (7)$$

and

The hinterland pore pressure could only be derived after an assessment of the whole scenario in terms of stress history and present boundary conditions. If the average effective stresses p' in the hinterland were taken not to have altered since compactive remoulding as shear strength c_u, figure 6 demonstrates that

$$p' = \frac{c_u}{\sin \phi'} .$$

The average vertical total stress must of course be

$$\sigma = \gamma Z$$

so that the average total stress p in the hinterland will lie between γZ and $(\gamma Z - c_u)$ depending on the existing degree of mobilization of shear strength in the hinterland. Since larger pore pressures are more critical, a conservative estimate for u should be

$$u = \gamma Z - \frac{c_u}{\sin\phi'} \quad (8)$$

Taking (5), (6), (7) and (8) together we get

$$\left.\begin{array}{lll} \tau_i = \gamma Z \tan \phi_i' & \text{for } \frac{c_u}{\gamma Z} > \sin \phi' & (a) \\ \tau_i = c_u \frac{\tan \phi_i'}{\sin \phi'} & \text{for } \frac{c_u}{\gamma Z} < \sin \phi' & (b) \end{array}\right\} (9)$$

Substituting parameters relevant to the model test we get

$$\left.\begin{array}{lll} \tau_i = 0.55\ \gamma Z & \text{for } \gamma Z < 172 \text{ kN/m}^2 & (a) \\ \tau_i = 0.88\ c_u = 95 \text{ kN/m}^2 & \text{for } \gamma Z > 172 \text{ kN/m}^2 & (b) \end{array}\right\} (10)$$

The details of the model wall are reported fully by Pang. The clay mass was initially 202 mm high, resting on a polythene sheet over a wooden base. The length was 483 mm between PTFE-coated stainless steel and glass end faces, creating plane strain conditions. The facing consisted of articulated aluminium panels 1 mm thick with slits to receive the reinforcing strips, which were made as integral pairs bent into a squared - U shape which could be interlaced into a pair of slits so that the strips were spaced 20 mm vertically and 80 mm horizontally. The reinforcing strips went back 160 mm from the face; unreinforced clay then continued for a further 140 mm up to a PTFE coated stainless steel sheet lying against the steel endwall.

The deformation of the model as it was centrifuged at successively higher accelerations over a period of 75 minutes was obtained by 5 horizontal and 10 vertical displacement transducers and is presented in figure 7. A crack began to open in the clay surface above the rear ends of the strips at 40 g. It was wide enough to be visible in the closed-circuit TV picture at 80 g. At 105 g a second crack formed about 110 mm behind the facing, after which the rate of deformation became large before complete collapse at 116 g.

The data of tension measurements, shown in figures 8 and 9 are even more revealing. The pattern of variation of tension along the 160 mm long reinforcing strips was deduced from gauges placed at different locations on different strips at the same level. The general pattern was of a reasonably constant tension between 40 and 90 mm from the wall face, with evidence of a reduction towards zero at the face. Of course, the tension must also reduce to zero at the free end; this must have occurred over a length not exceeding 70 mm, but there was insufficient data to offer more detail. Nevertheless a force of 50N was absorbed over a contact area of no more than 70 x 4 x 2 = 560 mm^2 in each of the layers below mid-height, so that the interfacial cohesion has been observed to be not less than 89 kN/m^2 corresponding to 83% of the average undrained shear strength. The vertical stress at collapse was 470 kN/m^2 at the base of the wall which would be more than sufficient, according to (10), to generate positive pore pressure and to lead to 10(b) as a fairly close predictor of the observed cohesion.

In order to assess the mobilized shear stress under small confining pressures, consider the observation of a 100 mm 'tail' from a tension of 7.5N to zero at a depth of 30 mm and an acceleration of 30 g. The normal stress of 18 kN/m^2 is much less than that required to create positive pore pressures, while the shear stress of 9.4 kN/m^2 is consistent with a friction angle of 28°, justifying 10(a) within 5%. At an intermediate stress level of 172 kN/m, the shear force mobilised at 77 g at a depth of 0.11 m was 49.5N over an area not less than 560 mm^2, implying a mobilised shear stress of 88 kN/m^2 which is within 8% of that implied by (10). Although more tests should be performed before (9) were put to general use, it is evident that the proposed concepts of interfacial cohesion may have some merit.

Regarding the collapse of the reinforced clay wall, it is possible to relate the vertical and horizontal soil stresses in the zone of constant tension by the relation

$$\sigma_h = \gamma Z - 2c_u \quad (11)$$

on the assumption that maximal shear stresses are mobilised and that the major compressive stress is vertical. These horizontal stresses, tending to translate and overturn the facing and the first 40 mm of soil, can be resisted by tensile forces in the reinforcement and by horizontal shear stresses in the soil up to the facing. If these supportive shear stresses are neglected, and the ability of the reinforcement to redistribute forces at various depths is also ignored, a pessimistic estimate for maximum σ_h becomes simply

$$\sigma_h = \frac{T}{S_v S_h} \quad (12)$$

where T is the tensile strength of the reinforcement, and S_v, S_h are the vertical and horizontal spacings. Equating (11) and (12) gives

$$(\gamma Z)_{max} = 2c_u + \frac{T}{S_v S_h} \qquad (13)$$

Inserting appropriate values

$$(\gamma Z)_{max} = 2 \times 108 + \frac{50 \times 1000 \text{ kN/m}^2}{20 \times 80}$$

$$\text{or } (\gamma Z)_{max} = 216 + 31 = 247 \text{ kN/m}^2 \; . \qquad (14)$$

This offers a pessimistic estimate of collapse acceleration

$$N_{max} = 247/(0.190 \times 20.4) = 64 \text{ g}$$

taking the lowest layer of reinforcements at 190 mm depth to be critical.

It will be seen in figure 9 that the first strips to attain their tensile strength were at 150 mm depth and did so at about 66 g. It can also be seen that substantial plastic redistribution of forces took place as the acceleration was increased to 81 g when gauged strips at this level began to break. Progressively larger deformations, cracks, and more ruptures continued up to 105 g with the advent of the second surface crack. Final collapse of the now shattered model occurred at 116 g. It should be noted that the capacity of the simple stress analysis (13) to predict the onset of progressive collapse, although excellent in this case, should ideally be supported by further observations. Pang showed that the ultimate scaled collapse height corresponded quite well to the theoretical collapse of an unreinforced vertical face, following Taylor's chart.

The most important aspect of (13) is the relative magnitudes of the soil strength and reinforcement strength in preventing collapse, the latter contributing only one eighth of the total in equation (14). Compared with the large undrained strength of any clay which might credibly be used as backfill, the reinforcement is negligible. This paradox can be explored further by translating the collapse at 116 g of 0.2 m high model to its field equivalent of 23 m height. It is extremely unlikely that clay fill would be employed to this height; 7 m would be a useful target height for road-related walls and bridge abutments, for example. At such a height any clay strong enough to be trafficked ($c_u > 70$ kN/m^2), would be free-standing in vertical faces ($H > \frac{2c_u}{\gamma}$ i.e. 7 m). Furthermore, (8) would generally offer negative pore pressures, so that according to (9) the interfacial shear strength should be considered to be frictional with zero pore pressures. The lesson to be learned here is that the undrained collapse problem is unlikely to be met in practice. If such a conclusion were to appear complacent, it should be noted that a sequence of models identical to that reported here, but using dense sand backfill ($\phi' \simeq 48^\circ$) collapsed earlier than the clay model at accelerations no higher than 92 g.

This does not mean, of course, that reinforced clay can be recommended as a routine procedure. Apart from the problems of controlling the consistency of the compacted clay during construction, the effects of the long term dissipation of pore suctions must be considered. The post-compaction suctions predicted in (8) would usually exceed 100 kN/m^2. As these relax, due for example to the establishment of steady seepage through a water-retaining construction, or to the downward percolation of rainwater, the tendency for the clay matrix to swell would be counteracted by tensions in the reinforcement. If the pull-out resistance of the reinforcement were to exceed its tensile strength, the result could be the progressive rupture of the reinforcement. Further research is clearly required before the long-term integrity of reinforced clay can be determined.

Anchored earth

A series of centrifuge model tests were conducted during the development of the TRRL concept of anchored earth. Figure 10 shows a Z-anchor which was one of a series of earth anchors proposed by Murray and Irwin (1981), and which was incorporated in simple multi-anchor models as indicated in figure 11. The objective of the tests was to provoke tensile failures and pull-through failures so that the new method could be compared with the reinforced earth concept. However, the intrusion of a different mode of failure - anchor bending - proved to be more significant.

This is exemplified by the behaviour of an articulated model wall constructed in six sections and backfilled with nominally dry, medium fine (52 - 100 sieve) Leighton Buzzard sand placed at a relative density of 80%. The wall was anchored at 50 mm intervals vertically (S_v) and horizontally (S_h) by copper wires of diameter D = 0.5 mm, preformed into a Z-anchor with a straight length L = 240 mm and a Z-detail of gross width B = 15 mm, each placed with minimum disturbance on the horizontal bench created as the fill was raised in 50 mm layers. The wires passed throgh holes drilled in the centres of the 3 mm thick facing panels, were bent back so as to bridge the hole, and were retained in that position by a drop of solder.

Tests using a Hounsfield tensometer indicated that the wire yielded at 153 N/mm^2 and rupture at 231 N/mm^2 after 25% plastic extension, offering a tensile strength of 45 Newtons at a minimum measured diameter of 0.495 mm. Shear box tests on the sand indicated that at 80% relative density the peak angle of shearing was 49° at a normal confining pressure of 40 kN/m^2 falling roughly 2½$^\circ$ for every doubling of stress thereafter.

The deflections of the wall are displayed in figure 12. The model collapsed at 22 g with the lowest facing strip pivoting about its base and the remainder translating forward uniformly by about 40 mm up to the arresting structure. It is important to appreciate that the horizontal and vertical scales of figure 12 are distorted: the rotation of the lowest facing strip could only have been of the order of $^1/_{30}$ at the onset of collapse.

On examination of the model the conditions sketched in figure 13 were discovered. There was evidence of an 'active' zone of collapse behind the facing consisting initially, perhaps, of a triangular wedge projecting roughly 100 mm behind the facing at the crest. The 'Z' details of the anchors were all straightened out but to various extents: the final projected width B' was 11 mm in the top layer, 2 mm in the next, and no more than 1 mm in the rest with the zig-zag being almost completely returned to a straight line. There was no distress of the solder joints, nor was there any evidence of plastic reductions in the wire diameter, so that it must be presumed that the average tensile stress in the anchor had at no point greatly exceeded the yield stress. The failure may best be described as a bending failure of the 'Z' - detail leading eventually to a pull-through collapse.

If simple active earth pressures are invoked without friction on the bulkhead, the maximum tension in the lowest layer of anchors at 275 mm depth would have been 37 Newtons at collapse compared with their strength of 45 Newtons. An equivalent strip anchor wall, following the earlier discussion, could have been two or three times stronger, since frictional arching and plastic stress redistribution would have enhanced the performance. There is no reason to doubt that the Z-anchor wall could have withstood an equivalent load had the bending failure not intervened.

In retrospect, the incidence of bending in the anchor head was not surprising. It must be appreciated that the bending strength of round rod is very small in comparison with its tensile strength. Let the anchor material be considered isotropic, with a yield strength σ_Y. The force which we wish to develop is the yield strength of the straight leg,

$$Y = \frac{\sigma_Y . \pi D^2}{4} \qquad (15)$$

Let us calculate the lever arm e at which a force equal to Y will just cause plastic bending of a rod of the same material subject to no tension or shear force. The plastic moment can be shown to be related to the yield strength by

$$M_Y = \frac{\sigma_Y D^3}{6} \qquad (16)$$

so that $$e = \frac{M_Y}{Y} = 0.21\ D \qquad (17)$$

or of the order of one fifth of a diameter. No system of uniformly distributed normal stresses on the legs of the Z - detail, whose overall dimension is typically thirty wire diameters, can be found which will generate anchor forces of the order of Y without contravening eccentricity conditions equivalent to (17). Since such forces have been carried, it must follow that the strength of the Z - detail is chiefly associated with localised stress distributions inside the corners of the Z which are able to minimise the bending moments in the wire.

The advantage of conducting centrifuge tests is that new ideas can swiftly be optimised. For example, it was appreciated that a semicircular anchor head forming a U - anchor could generate distributed supporting stresses within the semicircle without distorting the anchor at all as shown in figure 14. This notion was quickly checked. The same 0.5 mm wire was used with a configuration identical in every way to the previous test except that U - anchors were used. This model was precisely twice as strong as its predecessor, collapsing at 44 g after only 2 mm forward movement. The efficiency of the anchor system was clearly high, since an autopsy showed that most anchors had ruptured in tension a short distance behind the facing.

Conclusions

Design methods based on metaphysical rules governing the proportioning of materials are, correctly, going out of fashion. Limit state design offers an objective methodology in which the performance criteria are clearly stated in terms of the various possible limit modes, and the critical design situations which might induce them. This paper has offered evidence in support of the view that limit modes are not obvious until they have been observed, and that observations of centrifuge models are useful in support of the imagination and frequently generate a simple understanding of behaviour which can be related directly to the design problem. Computer programmes, by contrast, create elaborate descriptive analyses which often leave the essential facets submerged in a morass of irrelevant detail. Field trials usually skirt away from collapse scenarios and fail to investigate critical design situations. Their great cost also militates against a proper consideration of the range of possible behaviour which could be elicited in different circumstances.

This does not imply that centrifuge tests can be used easily as a direct model of a field-scale problem. Consider, for example, the progressive failure of brittle earth anchors. The future pattern of corrosion in a buried strip of metal is unknown. However, it is clear that if some notional sacrifical layer is to be used in design, that layer may be removed only in some localised region - near the joint for example. It is also clear that the joint of an anchor may be initially weaker than the rest even before corrosion. The effect of severely weakened sections may be that the anchor reaches its tensile strength here before it has even yielded elsewhere. The result is brittleness: the overall extension of the anchor is 10% or 15% of the length of the weakened portion, which may be a fraction of one percent of the anchor length.

The response to this problem was to conduct centrifuge tests on work-hardened anchors which might imitate this brittleness at model scale. The outcome was a new understanding of this piece of applied mechanics, rather than a measurement of some model behaviour which could directly be scaled up for design purposes. The proposal is, therefore, that a prime function of the centrifuge model test is to clarify design problems in relation to the principles of mechanics and the accepted behaviour of soils.

It needs to be explained to patent holders and their bankers, patent officers, certificating authorities, designers and their insurance companies, that new technologies can, and should be proved prior to their use. In ground engineering the most economical method of proving new ideas is to test a carefully engineered sequence of centrifugal models subjected to a variety of critical situations.

Acknowledgements

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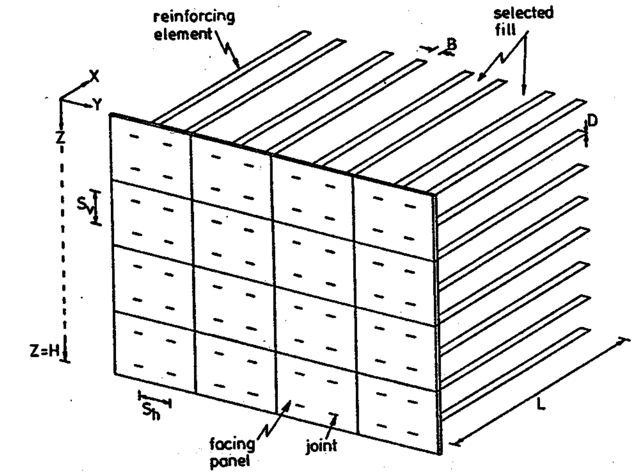


Figure 1 Reinforced Earth retaining wall

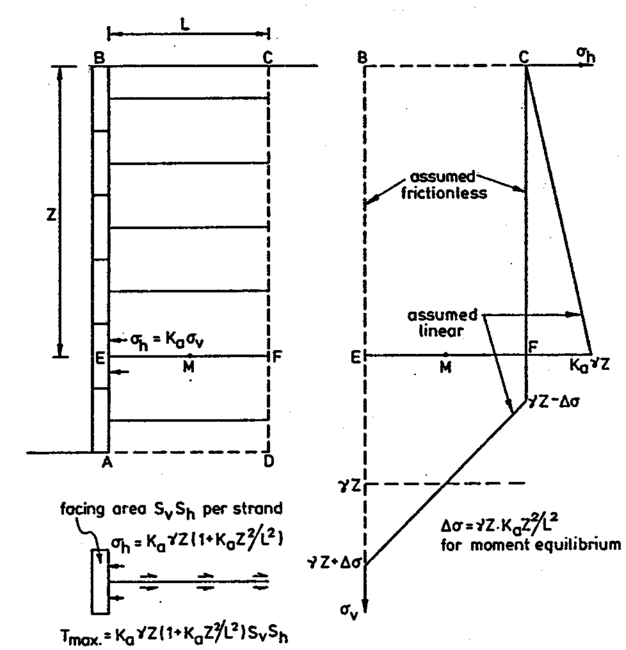
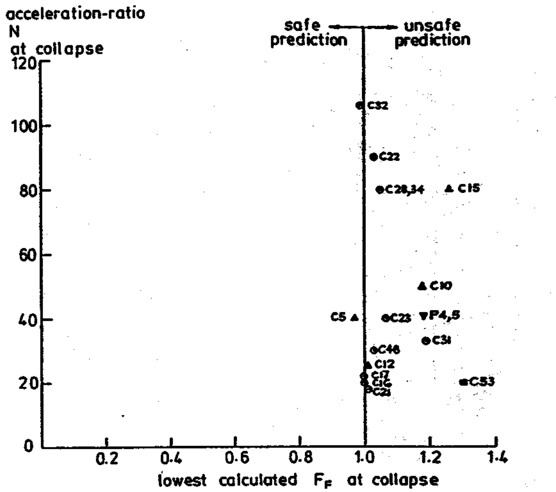
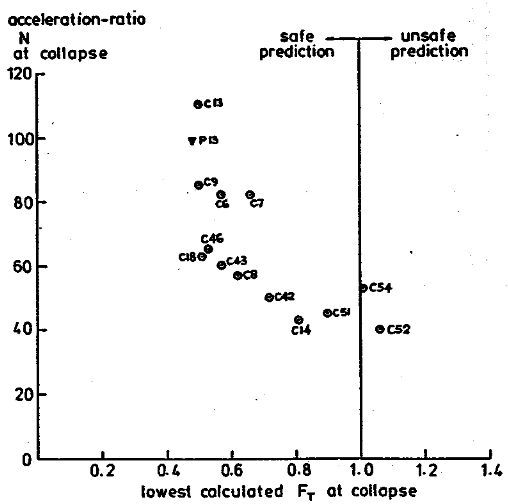


Figure 2. Simple stress analysis : the anchor analogy



Key: Mark I tests (C) ⊙ SS ; range of L/H 1.00 (C31) to 0.43 (C48)
▲ MS ; L/H = 0.75
⊡ AL ; L/H = 0.50
Mark II tests (P) ▼ SS ; L/H = 0.80

Figure 3. Friction failures of walls with flexible facing



Key: Mark I tests (C) ⊙ AL ; range of L/H = 1.5 (C6,7,8) to 0.5 (C14,18,51,52,54)
Mark II tests (P) ▼ AL ; L/H = 0.5

Figure 4. Tension failures of walls with flexible facing

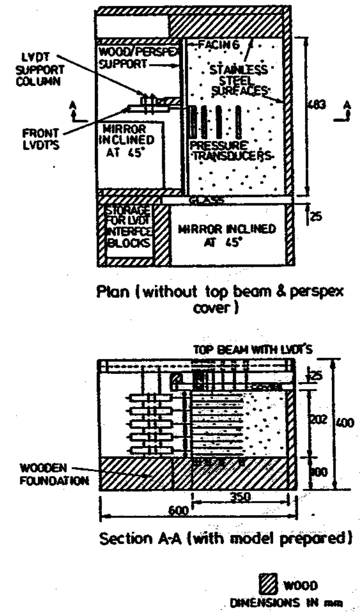


Figure 5. Mark II Model package (excluding steel container)

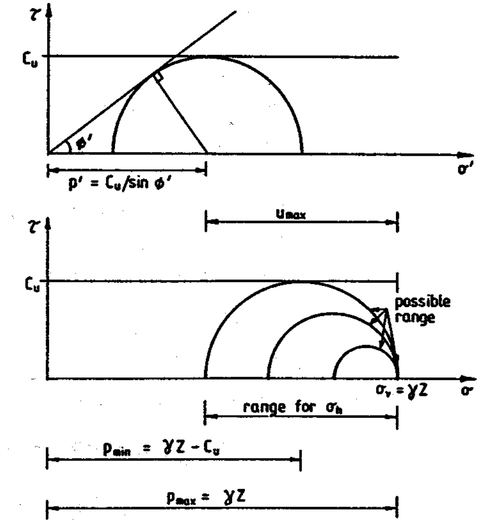


Figure 6 Stresses in compacted fill

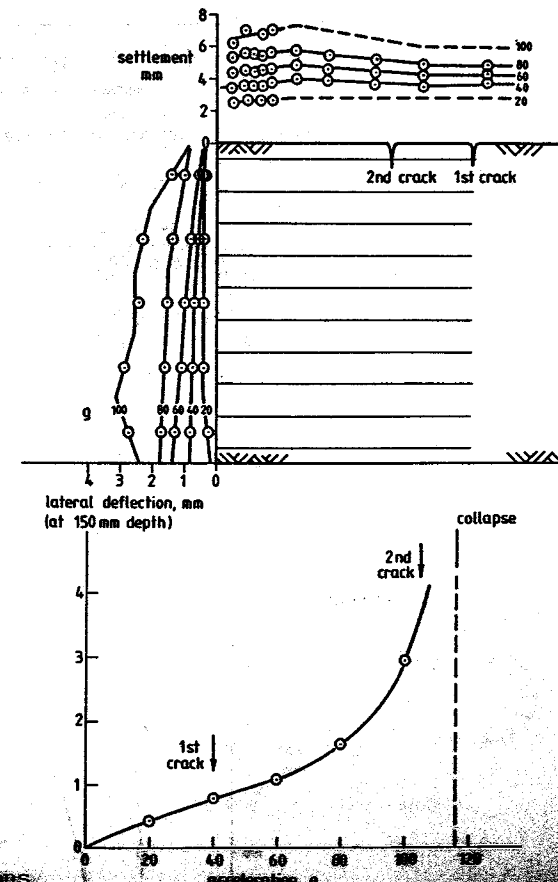


Figure 7
Clay model deformations

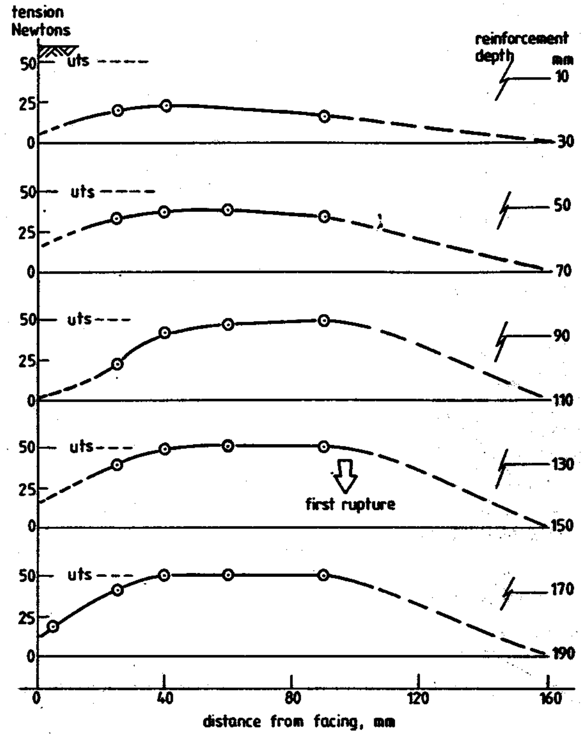


Figure 8 Tension profiles at first rupture

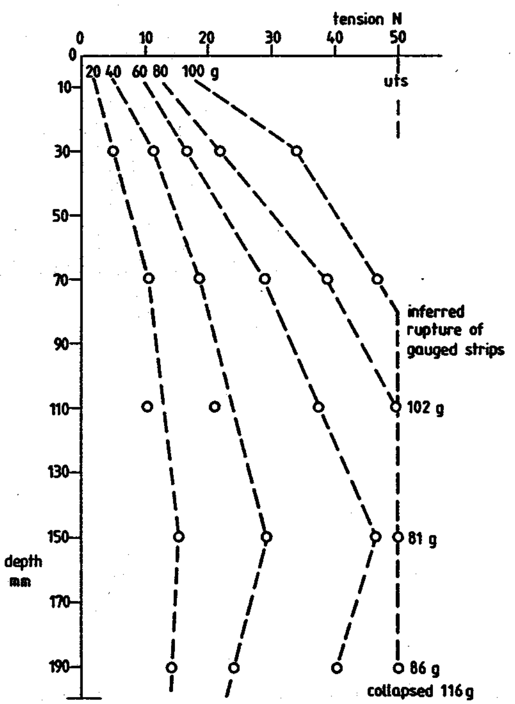


Figure 9
Peak tension
versus depth

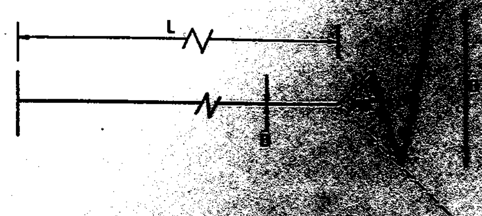


Figure 10 Z - anchor profile

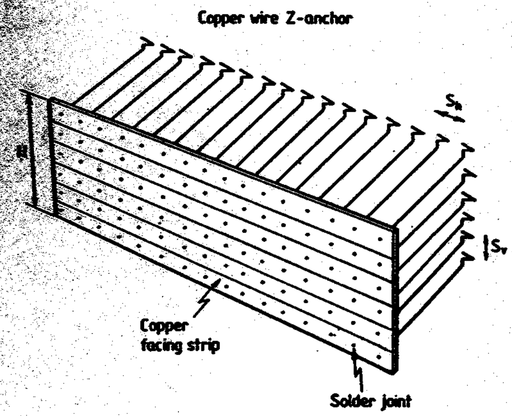


Figure 11 Assembly

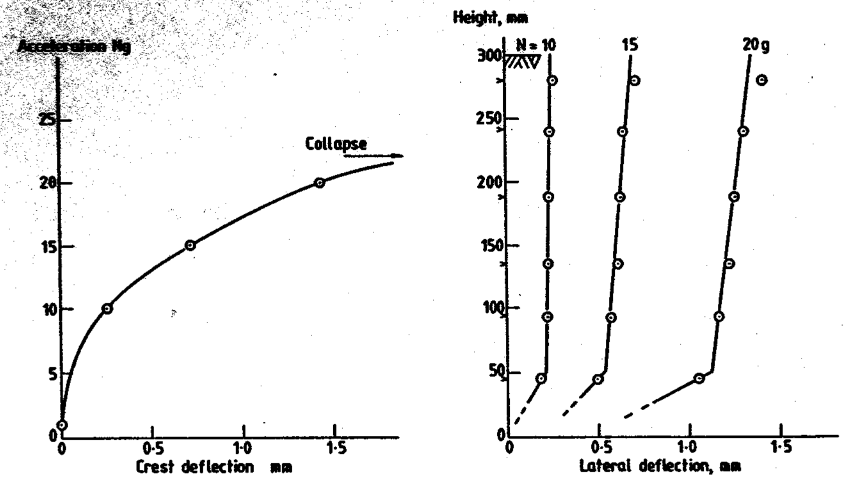


Figure 12 Deformations

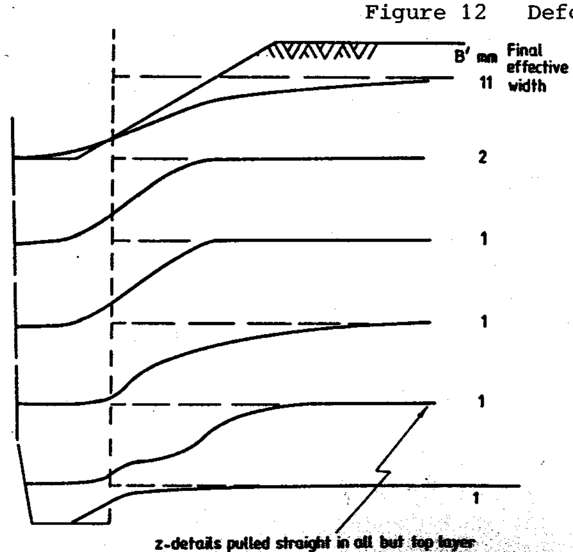


Figure 13 Disposition after test

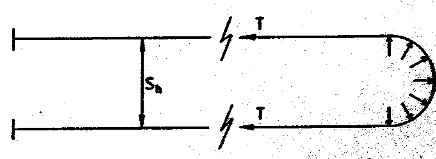


Figure 14 U - anchor