

REINFORCED EARTH WALLS: A CENTRIFUGAL MODEL STUDY

by

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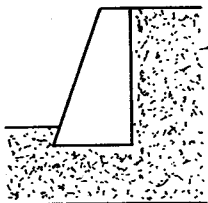
Objectives

A research programme has been under way since 1975 at the University of Manchester Institute of Science and Technology (UMIST) with the objective of clarifying and proving a reliable method for the design of reinforced earth retaining walls. The first task of any designer must be to use his judgment in order to select that one from the many available types of solution which appears most favourable to his client in the circumstances. After exercising his option, the designer's second task is to so proportion his design that it would meet his client's wishes on safety, utility, economy, life-expectancy, individuality or appearance.

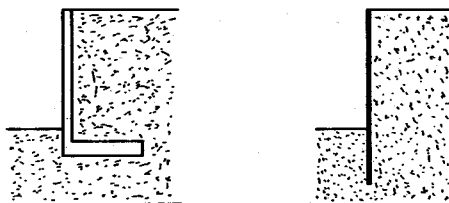
Suppose that a client demands some separation of ground levels. His designer will presumably first check to ensure that a simple soil slope will not meet the client's wishes, and will then use his judgment to select a type of retaining wall construction. The possible range of options is symbolised in figure 1 and includes (a) mass walls, (b) cantilever walls, (c) anchored cantilever walls, (d) reinforced earth walls and (e) cantilever walls with relieving platforms. Types (a), (b) and (c) are well understood in terms of the mechanics of structural elements in tension, compression or bending.

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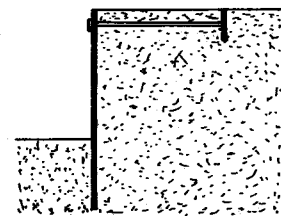
(a.) Mass Wall.



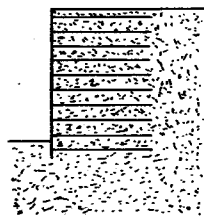
(b.) Cantilever Walls.



(c.) Anchored Cantilever.



(d.) Reinforced Earth.



(e.) Cantilever with Relieving Platforms.

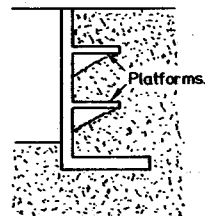


Fig 1. Classes of Retaining Structures.

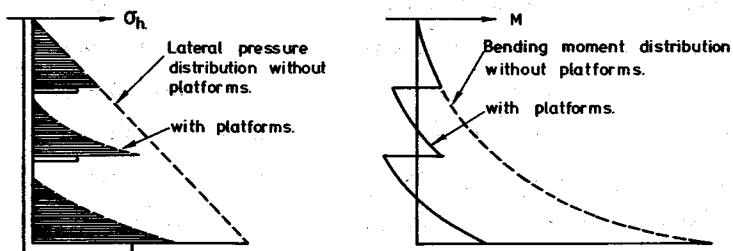


Fig 2. Tsagareli's Relieving Platforms.

The remaining systems of reinforced earth and relieving platforms are less conventional solutions which are remarkable for the extreme diversity of their authors' styles of description. Vidal (1969) describes the action of implanting frequent horizontal layers of tensile reinforcement in terms of the generation of a new composite material which possesses 'cohesion'. Vidal speaks of the whole reinforced mass as 'the wall' and casts doubt on the relative importance of the role of the front skin which is in contact with the soil. Tsagareli (1969) on the other hand, describes the benefits of the rigid connection of one or more relieving platforms in purely structural terms. He points out firstly that the outward moments of the lateral earth pressure acting on the wall above a shelf can be equilibrated at the joint by the inward moment due to the weight of earth lying on the shelf. He also remarks that if the earth was loosely filled, perhaps leaving a triangular gap underneath the shelf, the weight of earth above the shelf would not rest on the soil beneath and would therefore not cause lateral pressure on the portion of the wall beneath the shelf. Tsagareli's platforms can be seen to reduce the pressures acting on the face of the wall and, of greater importance, to be capable of altering the bending moment diagram for the face of the wall to almost any desired shape, as shown in figure 2.

Our first objective in researching reinforced earth walls was to attempt to clarify the function of Vidal's components in the same sort of structural terms as those used by the designers of each of the alternative systems reviewed in figure 1. This seemed to be a necessary pre-requisite if a designer was to be able to make a rational option. Our second objective was to review the design procedures of Schlosser and Vidal (1969) and Lee et al (1973) based upon their many laboratory

and few field scale experiments. By conducting a number of centrifugal model tests of small reinforced earth walls which approached some sort of collapse, we hoped to be able to comment on the rational choice of the proportions of such walls in practice.

Centrifugal Modelling

The centrifugal model technique is ideally suited to the clarification of the mechanics of new soil constructions such as reinforced earth. The centrifugal modeller simply attempts to construct a $1/N$ scale model of some prototype field structure which interests him, using the same materials in a similar geometry. He then observes the model as it is accelerated up to N times earth's gravity. Pokrovski and Fyodorov (1968 A & B) describe the development of the art and science of centrifugal modelling in the USSR since its introduction by Pokrovski in the 1930's. Avgherinos and Schofield (1969) set out the requirements for similarity between model and prototype which have since been used by research workers in the U.K. The central injunction is to employ identical materials and geometries, within boundaries which are themselves similar or so remote as to be negligible. The use of scale factors N on

$$\begin{array}{ll} \text{Length} & L_m = L_p / N \\ \text{Acceleration} & g_m = g_p \cdot N \end{array}$$

with suffices m and p for model and prototype respectively, guarantees that the vertical stress $L_p \rho_p g_p$ in the prototype is equal to its analogue radial stress $L_m \rho_m g_m$ in the model constructed in identical materials ($\rho_m = \rho_p$, etc.). From the equality of analogue vertical stress and the similarity of boundaries and landforms should flow the equality of analogue horizontal stress and indeed every other stress

component. If the changes of stress are correctly modelled, so should the strain be, and therefore the rates of dilation and the mobilised angles of shearing resistance. The tests to be reported concerned only the statical equilibrium of dry sand in contact with metals, so that the problems of dynamic similitude recently discussed by Rowe (1977) do not arise. The models were also proportioned and oriented as shown in figure 3 so as to reduce the errors in stress due to the non-uniformity of the acceleration field to below 10% in the region of interest.

The difference between sequential construction and wholistic construction is as much a problem to the centrifugal modeller as it is to the analyst or computer modeller. It is inevitable that a model constructed wholly at 1 g and then wholly accelerated up to N_g will generate greater soil strains than if each layer of soil had been placed at N_g in sequence on a partly deformed sub-structure. Strain path dependency is unlikely to have played a large part in the present series of models which were principally intended to offer data of collapse.

The Models

The interpretation of our data is exceedingly simple. Each model in the present phase was constructed to the dimensions given in table 1 and explained in figure 4 and was then observed as it was accelerated up to some acceleration ratio N_{\max} which was noted in each case. It is permissible to multiply every physical dimension of the model by a factor $N \leq N_{\max}$ in order to arrive at a theoretically stable prototype at field scale. In most cases it is not necessary to imagine that the soil grains themselves are modelling larger grains in the field: it is usually true that the actual size of grains is irrelevant if they are

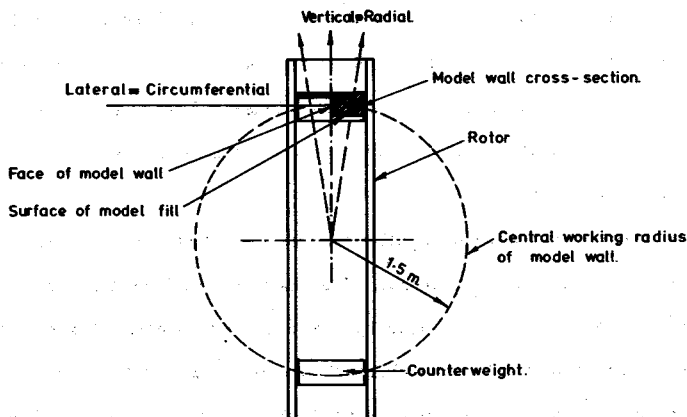


Fig 3. Plan View of Centrifuge and Model Cross-Section.

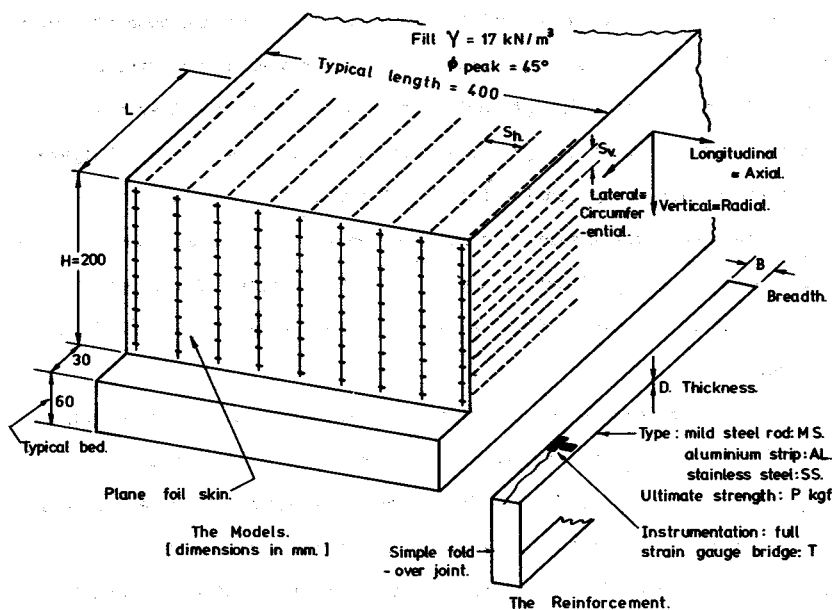
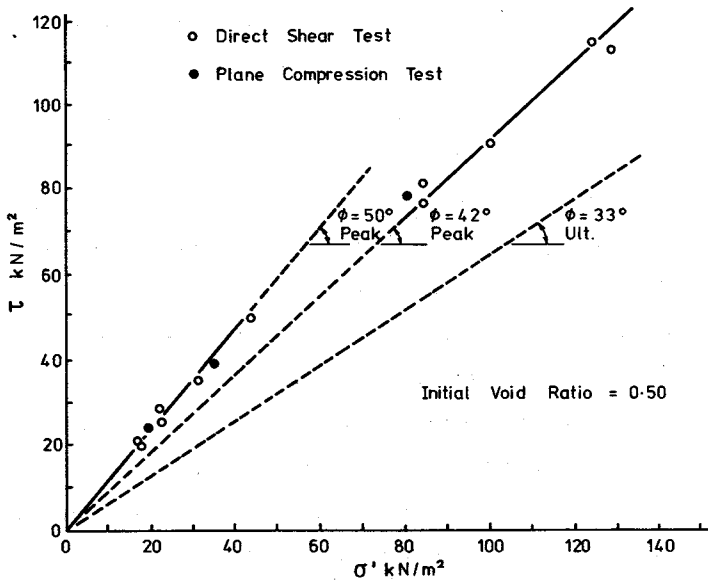
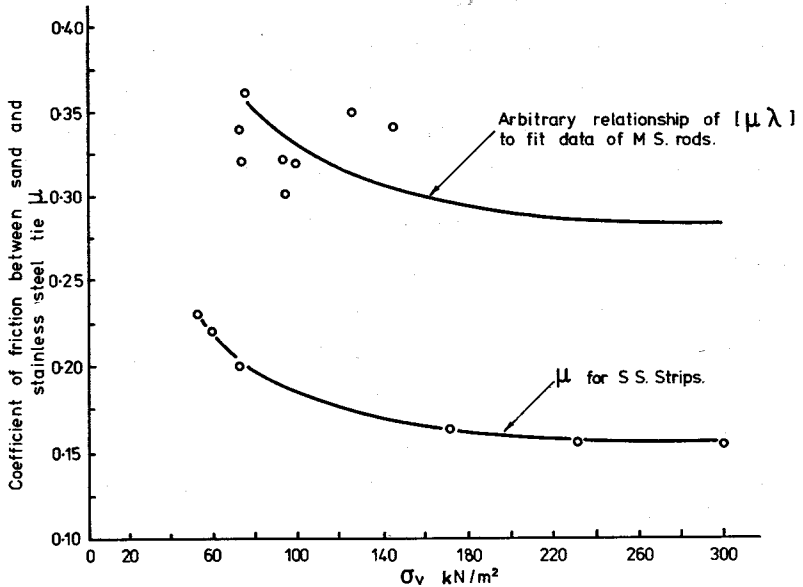


Fig 4. Configuration.

characteristically much smaller than the structural elements. Models which collapsed at small values of N are axiomatically less strongly reinforced than those which collapsed at higher values: a supposedly universal limit-state theory should be shown to be capable of predicting any of these failures.

The choice of models naturally limits the behaviour which is observed. The global equilibrium of a heavily reinforced block has been considered by Smith and Bransby (1976). The present tests however, have been focussed on to the problem of the slippage or fracture of the reinforcing strips in a fairly wide wall which was built on a medium dense bed of sand, thereby minimising the likelihood of monolithic collapse. The strips were buried flat in the dry sand and connected with a simple folded joint through slits in a foil skin which was typically 0.05mm thick. Their strength and roughness was so chosen that the typical model could be brought to a state of collapse either by slippage of strong strips or by tensile failure of rough, weak strips. Table 1 shows that model characteristics are distributed about a norm which corresponds at 50 gravities to a prototype approaching failure with a 10m height and 7.5m long strips at intervals of 1m vertically and 2.5m horizontally.

By using models twice as long as the wall was high, and by employing smooth end surfaces, the deformation of the wall was effectively constrained to be plane. In order to validate any limit state analyses it was necessary, therefore, to assess the strength of the well graded coarse to fine sand fill in plane strain. The dry sand was placed in the model in layers by free-fall from a suspended hopper and a density of 17.0 to 17.5 kN/m³ was achieved, corresponding to a relative density of about 0.70 to 0.75 and a void ratio in the vicinity of 0.55. Tests were

Fig 5. τ/σ' on plane of $[\tau/\sigma']$ peak.Fig 6. Pull-out Tests on Stainless Steel Strips
60mm \times 6mm \times 0.10 mm, and 1.5 mm dia.
Mild Steel Rods buried in sand at an
initial Void Ratio $e_0 = 0.50$

conducted using both 60mm square shear box samples and 80mm square plane strain compression samples, on the sand at this density. By following Arthur et al (1977) in the assumption that the horizontal plane through a direct shear box is a plane of maximum obliquity of stress, and by analysing the Mohr circle of stress for the compression test, it has been possible in figure 5 to amalgamate the data in order to display the peak stress ratio. At a small normal stress of 20 kN/m^2 the peak angle of shearing resistance is seen to be at least 50° , but this drops to 42° as the normal stress rises to 120 kN/m^2 . The ultimate angle of shearing resistance corresponding to a critical state appeared, both from the trend in shear box results after the peak and from loose slope angles, to be in the region of 33° almost irrespective of stress. Ponce and Bell (1971) are typical of recent authors who report an angle of effective shearing resistance increasing with the reduction of stress. Cornforth (1973) establishes that peak angles will exceed ultimate angles by up to 17° in a typical dense sand in plane strain. Our own results may be considered typical of granular fill, therefore.

Three types of reinforcement have been used in the UMIST research programme: MS, mild steel welding rod 1.5mm diameter and copper-coated to prevent corrosion; AL, aluminium foil of thickness 0.05mm; and SS, stainless steel strip of thickness 0.1mm. The first and third of these cannot conceivably break in the centrifugal models but only slip, the second was so proportioned that it could not break free by lack of friction but would rupture according to various tensile tests at an average nominal tensile stress of 70 N/mm^2 having yielded at 50 N/mm^2 . The friction between the stronger reinforcements and the sand was subjected to close scrutiny. The chosen method was to extract 60mm

lengths of the 6mm wide model SS strips from their position, buried horizontally in a shear box containing the sand at the appropriate density. This, at model scale, corresponds to a carefully instrumented pull-out test which might be attempted on a full-scale structure. Figure 6 represents the results for stainless steel strips expressed as a coefficient of friction μ over a range of working stresses. As with the sand alone, the stress was found to be an important determinant of the angle of friction developed. The 1.5mm diameter copper-coated MS rods were much more difficult to use in the pull-out apparatus due to their relative rigidity: figure 6 shows the arbitrarily assumed relationship through the scattered data. The round bars also afforded the problem of an unknown stress distribution, whereas the pull-out resistance of a flat strip $B \times L$ was taken as $2BL \cdot \mu \cdot \sigma_v$, it was necessary to characterise that of the round bars by $\pi BL \mu (\lambda \cdot \sigma_v)$ where $\lambda \cdot \sigma_v$ represents the average stress normal to the bar. Figure 6 actually plots $(\mu \lambda)$ measured at various vertical stresses σ_v : it was assumed that both μ and λ would be shared by the pull out test and the plane strain model test, so that it would not be necessary to decouple them.

Electric resistance strain gauges were used in a few later tests to find the tensile stress at various positions on the model ties. Whole bridge circuits employing 2mm gauges on the 6mm wide stainless steel strips have been shown to generate reliable results when calibrated in a Hounsfield tensometer. A high proportion of breakdowns has had to be accepted due to the extremely hostile environment in which the gauges were placed. This led to our repeating a critical experiment many times in order to accumulate sufficient information: models 19,20,27,33 and 36

were identical except as regards the disposition of functioning strain gauge bridges. Their design was chosen specifically to avoid failure so as to minimise the damage to the instrumentation: comparison with test 22 shows that failure due to insufficiency of friction was quite close. Choudhury (1977) shows a very close agreement between signals emanating from the same location in models which were meant to be identical. An attempt has also been made to assess the distribution of vertical pressure under the model reinforced-earth walls. Two total stress transducers with 10mm diameter diaphragms were calibrated both by air pressure and by disposing them under a uniform layer of the sand and centrifuging them: they were then installed at various positions beneath the model walls. All the instrumentation was restricted to the central third of the wall, well away from its ends.

Tension Data

The instrumented tests can be used to generate some notions concerning the distribution of stresses in a reinforced earth wall which is on the verge of collapse. Of greatest concern to the designer is the distribution of tension in the reinforcement, and figure 7 depicts the form of the data accumulated in models 19, 20, 27, 33 and 36 which is concentrated in the front zone of the reinforcement in order to locate and quantify the maximum tensions. The line of maximum tension was independent of acceleration and is roughly vertical and 40mm inside the 200mm high face. The tensions must obviously drop to zero at the loose ends, but they are also seen to drop quite steeply towards the joint at the face, especially near the base of the wall. Each strand of reinforcement serves a face area $S_v S_h$ and it is therefore clear that the average lateral earth pressure acting on the face (σ_h) can be computed

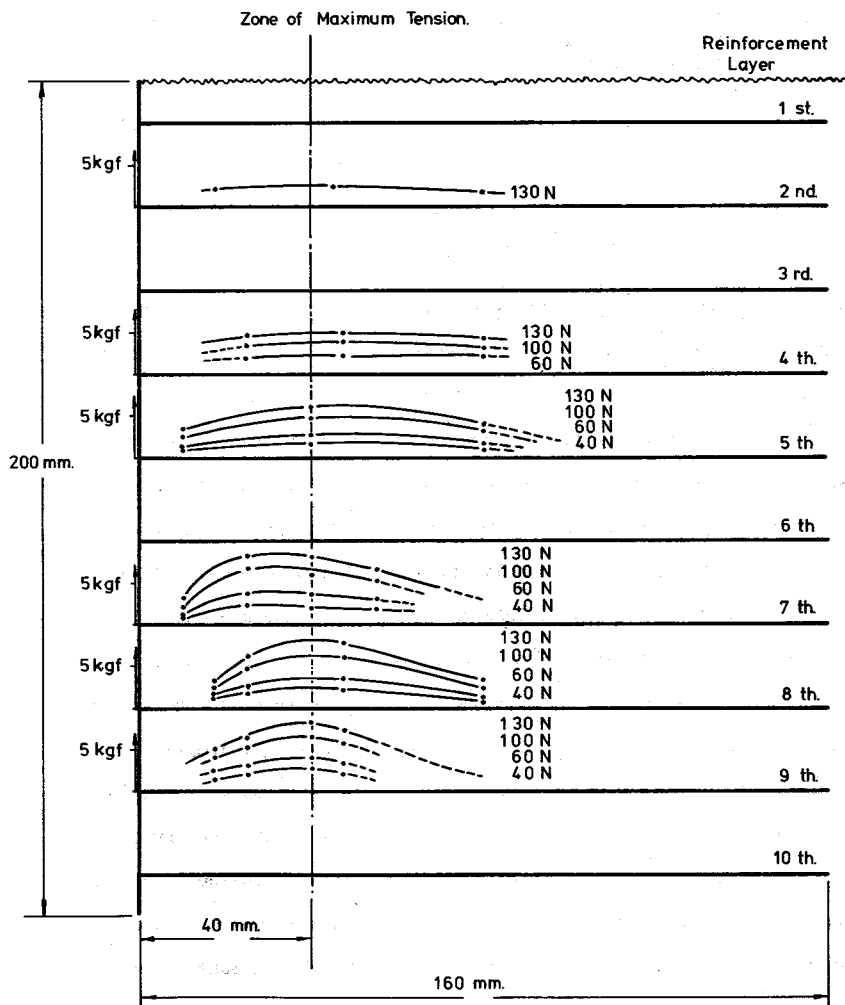


Fig 7. Tension Distribution in Models 19, 20, 27, 33, and 36 at N Gravities up to $N = 130$.

in the locality of a reinforcing strip whose tension at the face is

T_{face} , simply by

$$\sigma_{h \text{ face}} = \frac{T_{\text{face}}}{S_v S_h} \quad (1)$$

Bolton et al (1977) demonstrated that

$$\sigma_{h \text{ max}} = \frac{T_{\text{max}}}{S_v S_h} \quad (2)$$

is also a reasonable approximate relationship between the lateral earth pressure and reinforcement tension at any depth in the zone of maximum tension if this is only a short distance behind the face.

If equation 2 provides an acceptable way of estimating the lateral earth pressures in a zone of maximum tension just behind the face of the wall, it only requires a corresponding estimate of vertical earth pressure in the same zone in order to turn the data of figure 7 into a map of earth pressure coefficients. Figure 8 displays data of vertical pressure under the base of the models. As the acceleration was increased there was little tendency for any change in shape of the distribution, which was almost uniform and equal to the simple overburden pressure γH where the unit weight of the sand $\gamma = N/\rho g$. There was nevertheless, an increase of up to 25% in the base pressure in the region previously identified as that of maximum tension. Closer to the face, the pressure appeared to drop to at least 10% below the average overburden pressure. Choudhury (1977) showed that an increase of 30% in the base pressure was not unusual in the maximum-tension zone, and that equal reductions in pressure frequently occurred at the buried end of the base of the reinforced zone. It is necessary to attempt to generalise this observation, and this is usually accomplished by reference to the thrust of the backfill exerted on the buried side of the

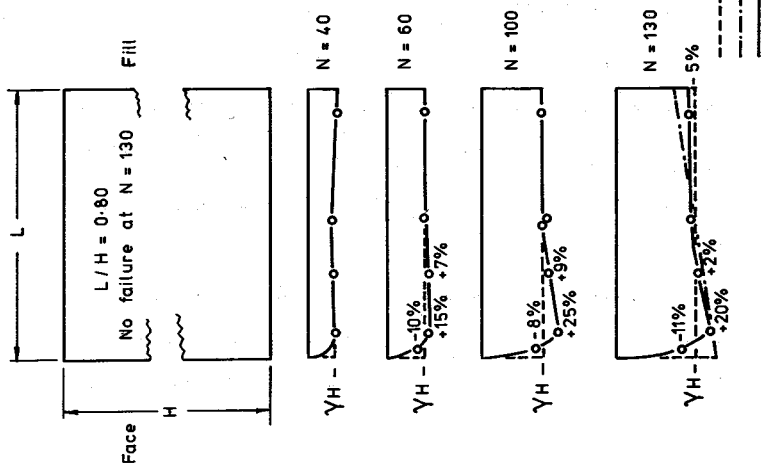


Fig 8. Distribution of Vertical Stresses at the Base of Models 27, 33, 36.

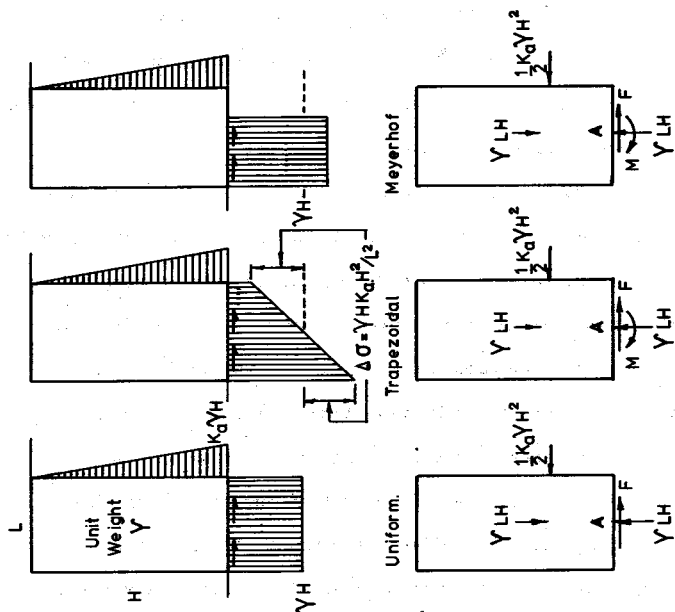


Fig 9. Stress Distributions around a Reinforced Block.

reinforced mass. It is very difficult to draw the 'free-body' diagram for the control volume of a reinforced earth mass, due to its redundancy. As a guide, the prime requirement in structural design is that components should be in equilibrium with the applied loads. A second requirement is that no component should deform excessively: this may be guaranteed in rigid-plastic material if the yield criterion has nowhere been approached. Figure 9 demonstrates that a simple trapezoidal distribution of pressure at the base can be sufficient to satisfy the broad requirements of a 'plastic' design in which equilibrium and the non-violation of the Mohr-Coulomb criterion are paramount. An alternative uniform base pressure defies equilibrium of the control volume by rotation about the mid point A of the base. An alternative Meyerhof-type distribution defies the Mohr-Coulomb criterion at the buried heel of the reinforced mass with zero vertical effective stress. The assumption of a horizontal thrust from the backfill is a pessimistic gesture typical of Rankine's analysis of walls: the analyst damages the backface of the mass by drawing a frictionless wound through it, before analysing it. If the simple trapezoidal distribution be adopted, the vertical stress at the front of the wall is increased to,

$$\sigma_{v \max} = \gamma H (1 + K_a H^2/L^2)$$

in order to counterbalance the overturning moment. A vertical stress

$$\sigma_{v \max} = \gamma Z (1 + K_a Z^2/L^2) \quad (3)$$

may therefore be a reasonable and pessimistic estimate in the zone of peak tension.

The data of figure 7 can be used to assess the usefulness and validity of equations 2 and 3. Figure 10 displays the estimated values of σ_h and σ_v in the zone of peak tension at each level and at each

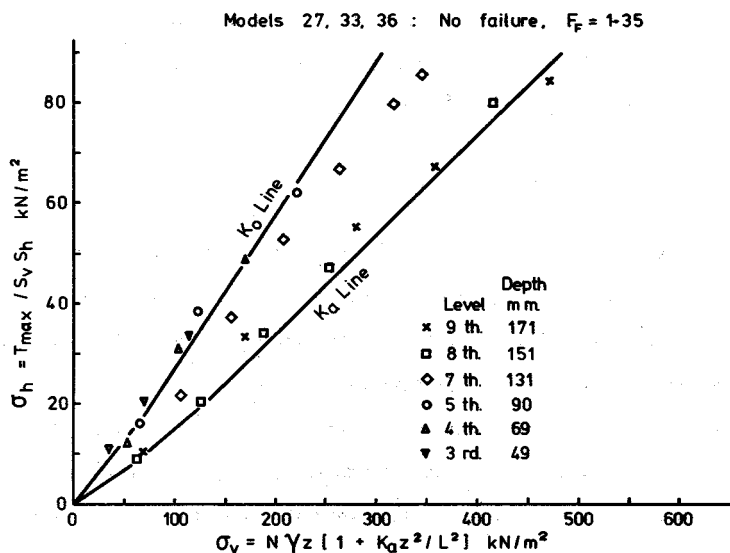


Fig 10. Estimated Earth Pressures in the Zone of Maximum Tension.

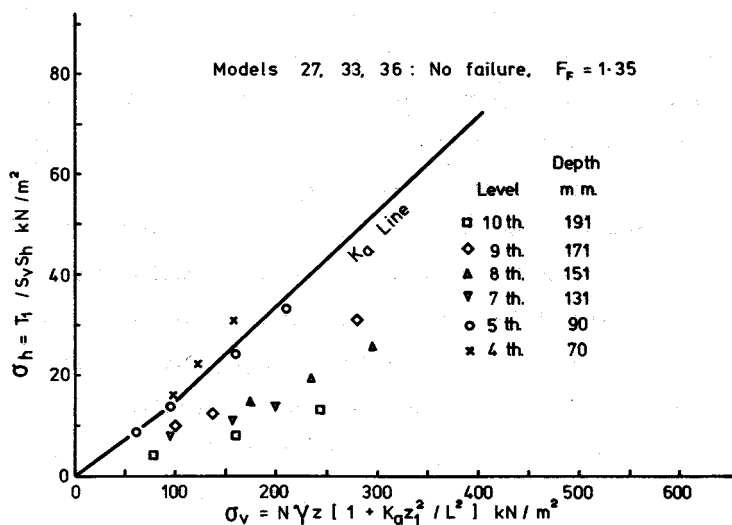


Fig 11. Estimated Earth Pressures near the Face of the Wall. [T_1 measured 10-15mm from face.]

acceleration increment up to the maximum of 130, recalling that there was no collapse. Superimposed on the diagram are the conventional earth pressure coefficients

$$K_a = (1 - \sin \phi') / (1 + \sin \phi')$$

for soil at failure with its effective Mohr circle of stress tangential to ϕ' lines, and Jaky's $K_0 = (1 - \sin \phi')$ for soil in one-dimensional plastic compression. The K -lines are seen to be curved due to the dependence of ϕ' on σ' seen in figure 4, taking care to account for the stress on the active planes. The data is very well organised, with the lower quarter of the construction in a clearly active condition in the zone of peak tension, and the upper half in an apparent conformity with the K_0 parameter. Very similar results were obtained for model 28 which eventually came to a friction failure, except that the majority of data fell close to the K_a line. Occasional tension readings in other models confirmed the picture of figure 10.

If equation 1 is used to estimate the lateral pressure close to the face of the wall, and equation 3 to estimate the vertical pressure, figure 11 results. Active earth pressures may evidently apply in the upper half of the wall, but the lateral pressures in the lower half are less than the simple prediction. Clearly the earth pressure coefficient cannot be less than K_a . The vertical stress close to the wall must simply be smaller, indeed less than one half of the prediction, for the lower zone. The data of figure 8 have forewarned of this possibility already. The inferred reduction of both vertical and horizontal stress from the peak stress zone towards the face of the wall coupled with the reduction in tension in the reinforcement, marks out this zone, referred to by Schlosser and Vidal (1969) as the active zone of the construction,

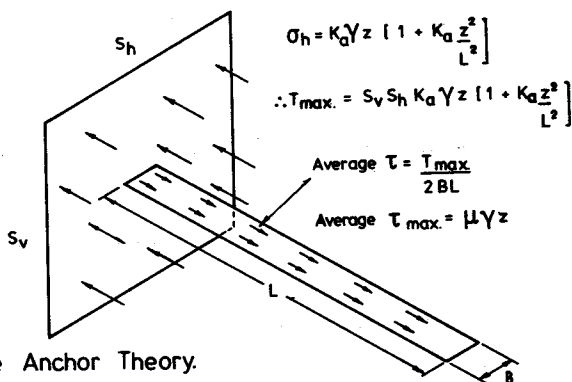


Fig 12 Simple Anchor Theory.

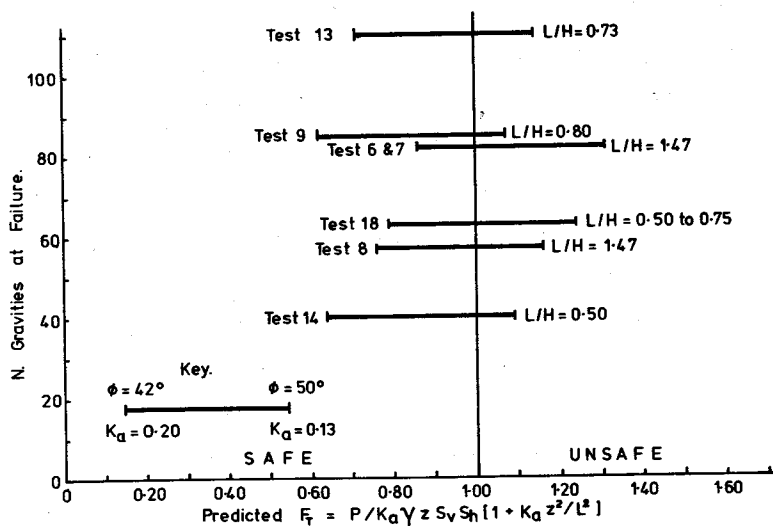


Fig 13. Tension Failures : Prediction for Lowest Broken Tie.

as one of great complexity. The stresses on the face must clearly be strain dependent.

Failure of an Anchor

The successful analysis in terms of earth pressure coefficients of the data of reinforced earth walls approaching a friction-failure suggests the possible utility of a Rankine-type analysis for the earth pressure on the face of the wall at the point of collapse. As far as the face and its joints are concerned, such an analysis would appear to be pessimistic and safe. Figure 12 depicts what must now be viewed as an anchored wall unit. The vertical stress immediately behind the facing panel is estimated pessimistically to be $\gamma Z (1 + K_a Z^2/L^2)$ where K_a conditions have been assumed in the retained fill. The lateral pressure of $K_a \gamma Z (1 + K_a Z^2/L^2)$ creates a tension at the front of the anchor of $S S_v K_a \gamma Z (1 + K_a Z^2/L^2)$ which must be dissipated in shear stress on the surface of the anchor so as to reduce to zero at its buried, free, end. It is a simple matter therefore, to declare a pessimistic factor of safety against tension failure to be:

$$F_T = \frac{P}{T_{\max}} = \frac{P}{K_a S S_v \gamma Z (1 + K_a Z^2/L^2)} \quad (4)$$

It is much less easy to deal safely with the possibility of friction failure. It is, on the other hand, easy to be optimistic and to declare that the whole upper and lower surfaces of a strip anchor be considered capable of resisting forward sliding with an average shear stress of $\mu \gamma Z$. This leads to a factor against friction failure

$$F_F = \frac{C_{SLIP}}{C} = \frac{\mu \gamma Z}{T_{\max}/2BL} = \frac{2 \mu BL}{K_a S S_v (1 + K_a Z^2/L^2)} \quad (5a)$$

or in the case of a rod of diameter B

$$F_F = \frac{\pi (\mu \lambda) BL}{K_a S_v S_h (1 + K_a Z^2/L^2)} \quad (5b)$$

These propositions effectively coincide with the working hypothesis of the Laboratoire Central des Ponts et Chaussées reported by Schlosser and Long (1974).

It is a fairly simple matter to compare the evidence of the 13 friction failures listed in table 1 with the prediction of equation 5, and the 7 tension failures with equation 4. The most significant difficulty is over the choice of the soil's angle of friction. If we have been successful with the data of figure 4 then ϕ' in the model should reach a peak value in the region 42° to 50° depending on the stress across the plane of maximum stress obliquity. At 130 gravities the vertical stress at the base of the model is roughly 450 kN/m^2 , so the stress on the plane of peak soil stress ratio will be about 125 kN/m^2 and the peak angle of friction roughly 42° from figure 4. At mid-height the peak ϕ' would be about 46° rising to 50° towards the top, again at 130 gravities. In order to reflect the inevitable uncertainty over the appropriate strength, we have chosen to use both $\phi' = 42^\circ$ corresponding to $K_a = 0.20$ and $\phi' = 50^\circ$ corresponding to $K_a = 0.13$ in the back analyses. Each predicted model failure is then represented as a horizontal bar covering the 42° to 50° interval in angle ϕ' .

Figure 13 depicts the factor of safety against tension failure calculated by equation 4 for each of the models in which an autopsy showed that some anchor had broken. The factor is calculated for the lowest anchor which actually broke, and is shown to be quite competent since the values are spread around the required value of unity irrespective of the strength of the model, which is represented by the acceleration ratio to which it could be subjected. The utility of the

trapezoidal enhancement factor $(1 + K_a Z^2/L^2)$ can be judged by the close fit between models of very different aspect ratios. Models 14 and 18 with L/H ratios of 0.5 would have had theoretical factors of safety about 1.8 times greater had it not been for the use of the factor: such a theoretical factor would have been very much in error on the unsafe side and we would have generated severe scatter in the results. Choudhury (1977) remarks that the lowest failing anchor was very seldom in the lowest level of anchors at 190mm depth but rather at 170mm or 150mm. If this had not been accounted for in figure 13 the theoretical factors of safety would have been about 0.8 times lower so that the prediction of failure would have been just a little more pessimistic.

Figure 14 depicts the factor of safety against friction failure calculated by equation 5 for each of the models in which an autopsy showed that a collapse had taken place without any structural component being broken. The analysis was made for the base of the wall at depth $Z = H$: μ is smaller, K_a greater and Z/L greater at the base of the wall, leading to the smallest safety factor for anchors. The theoretical factor is shown to be very well capable of dealing with low L/H ratios: the prediction for test 16 would have been grossly unsafe were it not for the presence of the trapezoidal factor. Likewise an enormous range of materials and spacings have been brought into consistent account.

The use of peak soil strength from figure 4 and the most optimistic anchor theory would evidently generate a safety factor of roughly 1.25 for each back analysis. This slightly unsafe condition could be eliminated in a variety of pragmatic ways: perhaps the most straightforward would be to employ a conservative angle of friction of 42° for

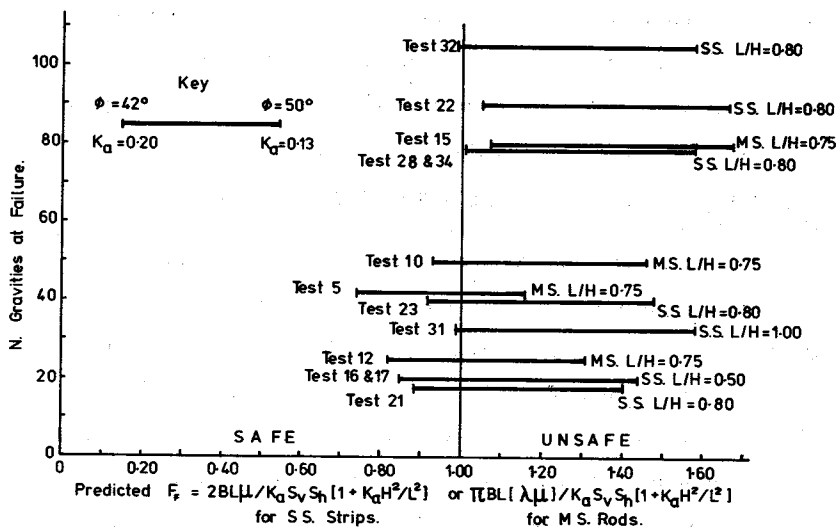


Fig 14. Friction Failures: Prediction for Lowest Tie.

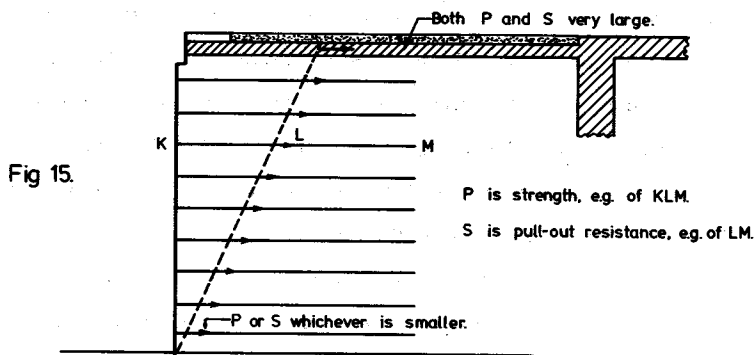


Fig 15.

sliding collapses, on the grounds that the large localised strains which were evident could well have carried the soil beyond its peak rate of dilatancy. The decision on ϕ' will dominate the accuracy of designer and research worker alike.

Advantage of Stress Analysis over Wedge Analysis in Design against Tension Failure

The back analysis of the centrifugal models has not yet revealed any rogue failure which is attributable to gross errors in the anchor theory. On the contrary it would appear that the accuracy of the conception of a reinforced earth construction at failure as an assemblage of individual anchors only just holding back small areas of facing, has been demonstrated with even more certainty than have the widely accepted conventional conceptions regarding cantilever walls. It seems that a designer would be perfectly justified in treating a reinforced earth retaining wall as a multi-anchor wall with long horizontal anchors. This may well clear his mind concerning the problem of choice of system from the options of figure 1. The advantages of the anchor concept over the cohesion concept are greatest, however, when considering the detailed design of components. The designer can safely be left to guard both against Rankine's active stresses causing global movement by acting on the rear face of the block, and against similar stresses on the front skin causing tearing of the face, rupture of the joint or anchor, or slippage of the anchor. He need not be burdened with long composite calculations which attempt to equate sliding and restraining forces of assumed Coulomb wedges which may cut across reinforcements of various spacing and strength and which will typically cut through soil with a friction angle reducing with depth.

This may not only simplify his design equations but also save his

reputation. If he were to fail to follow exactly the tortuous mathematical scheme set out by Banerjee (1975) for example, the designer could well find himself inferring that reinforcements break simultaneously at every depth, even where they may be equally strong and at equal spacing, since this seemed to be an outcome of the Coulomb slip surface. As always, the slip surface technique is optimistic. All published evidence denies the possibility that the lateral pressure can be constant with depth: our own results, which imply only a little redistribution beyond the triangular, are typical. If the designer had expected all anchors to fail in tension simultaneously in our centrifugal models, concerted in their attempt to carry the burden of the lateral soil thrust, he would have been forced to conclude that

$$P_T^* = \frac{\sum P}{\sum K_a S_a \gamma Z(1+K_a Z^2/L^2)} \approx \frac{P}{(1/2)K_a S_a \gamma H(1+K_a H^2/L^2)} \approx 2P_T \quad (6)$$

This would have doubled his estimation of the factor of safety which would then have been in gross error if figure 13 can be relied upon. The fundamental mistake of employing mechanisms such as wedges in the design against tension failure of reinforced earth walls is not rectified by following Banerjee in choosing the lowest alternative of the strength P of an anchor and the pull-out resistance S of that part which lies outside the Coulomb wedge. Figure 15 depicts the most severe test of such logic: the top anchor bed has been made into part of a neighbouring sub-structure so that both P and S may economically be made very large indeed. It ought to be clear that neither P nor S must be used in such as equation 6 since the lateral stress at the wall face will be totally unable to generate such a large force. The true collapse mechanism would clearly not involve failing the strong top anchor.

It should be recognised that the pattern of ruptures observed after a tension failure, which have been described by Lee et al (1973) as lying along a Coulomb slip plane is, as Schlosser and Vidal (1969) suggests, a purely dynamic, progressive phenomenon. Choudhury (1977) observed precisely the same sort of distribution in the centrifugal model tests. The autopsy is totally misleading, as explained above, if it leads to an imagined simultaneous rupture of anchors. It should also be recognised that the progressive failure, although catastrophic, is due to the condition of loading rather than to any 'brittle' behaviour in the materials. Rankine's 'plastic' lowerbound type of analysis has been shown to provide a very respectable group of predictions.

Conclusions

1. Centrifugal models support the proposition of Schlosser and Long (1974) that reinforced earth retaining walls can be designed to avoid collapse by treating them as multi-anchor walls. Anchors can be made safe by consideration of simple active earth pressures normal to the face of the construction, enhanced by a trapezoidal adjustment to the vertical stress caused by the thrust of the backfill. Well-spaced anchors can effectively develop their coefficient of friction over their gross area, and can seemingly be considered to be acted upon by the average overburden pressure. The determination of mobilizable angle of shearing resistance appropriate to the stress and density of the soil in the construction offers by far the greatest problem to the designer.
2. If reinforced earth walls can be considered to be anchor walls, it may well prove economic in certain circumstances to replace horizontal strip anchors by conventional vertical plates retained

by cables. In particular, the designer might benefit from the replacement of the top few metres of a proposed reinforced earth retaining wall construction either by an identical skin retained by short cables leading to vertical anchor plates, or even by a precast L-shaped wall section with a good crest detail to contain drains, barrier, posts etc. Reinforced earth walls tend to absorb an inefficient length and thickness of reinforcement in their shallow zones.

3. Lee (1976) pointed to a need for further research on the problem of pull-out or friction failure. The centrifugal models demonstrate the surprising accuracy of the very simple strip anchor calculation, even when it is extended to include rods. If the designer does not attempt to choose a geometry outside those which have been modelled in these tests ($B/S_h \gtrsim 0.35$, $B/S_v \gtrsim 0.75$, $S_h/L \gtrsim 0.5$), or those of others, it may be safe to ignore theoretical demons such as anchor interference, block pull-out of a vertical column of anchors with the intervening soil, reduced stress inclination so that μ can not be developed, or the supposed presence in the limit state of an active zone immediately behind the face. Indeed friction failure may be a less worthy candidate for research than other modes of collapse, since it may be quite economic to provide a very large safety factor against it. The weight of reinforcement is directly related to F_T , and not to F_P : in other words the friction capacity can be improved simply by spreading the same weight of reinforcement over a wider plan area.
4. Real safety is only possible if a designer is aware of every type of collapse mechanism which can exist. Field-scale tests,

even if conducted, are hardly likely to generate the necessary diversity of behaviour pattern leading to collapse. Indeed such tests are difficult to interpret unless they do lead to a collapse. Schlosser and Vidal (1969), Lee (1976), Chang et al (1977) and Finlay and Sutherland (1977) each demonstrate that lateral stresses can exceed their active values within walls which did not fail. This may simply confirm the adequacy of designs based upon active limit states, and featuring safety factors. The effect of $K > K_a$ is to move the Mohr-circle of the soil away from its failure envelope: is this not the required effect of a safety factor? If the soil is not collapsing, how can the wall? Of course, it would be possible to use such a heavy compaction machine that the soil was forced to push out the wall and permanently stretch the anchors, but this might be best accounted for by adding a generous notional surcharge to the simple Rankine calculations rather than by attempting a quasi-elastic analysis. Many similar soil deformation problems, which are strictly outside the scope of limit analysis, may be solveable in an ad hoc manner on site if the staff are vigilant.

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TABLE 1 Models : H = 200mm.

Reinforcement - Type: MS mild steel rod; SS stainless steel strip
AL aluminium; Tension measurement T

Foundation Bed: 60mm sand except models 11 and 12 on perspex.

Model length: 400mm except models 2, 3 and 13 at 760mm.

MODEL	REINFORCEMENT DETAILS							N		FAILURE
	Type	L mm	B mm	D mm	S _v mm	S _h mm	P kg f	μ min	MAX	
2	MS	150	1.5	1.5	10	20	100	0.30	130	NONE
4	MS	150	1.5	1.5	20	30	100	0.30	130	NONE
11	MS	150	1.5	1.5	20	31	100	0.30	130	NONE
15	MS	150	1.5	1.5	20	40	100	0.30	80	FRICTION
10	MS	150	1.5	1.5	20	47.5	100	0.30	50	FRICTION
5	MS	150	1.5	1.5	20	60	100	0.30	40	FRICTION
12	MS	150	1.5	1.5	20	62	100	0.30	25	FRICTION
19,20, 27,33, 36	SS:T	160	6	0.1	20.5	47.5	87	0.16	130	NONE
24	SS:T to 130	140	6	0.1	20.5	47.5	87	0.16	130	NONE
32	SS	160	6	0.1	17	76	87	0.16	106	FRICTION
22	SS	160	6	0.1	20.5	63	87	0.16	90	FRICTION
28	SS:T	160	6	0.1	20.5	63	87	0.16	80	FRICTION
34	SS	160	6	0.1	25	52	87	0.16	80	FRICTION
23	SS:T	160	6	0.1	20.5	76	87	0.16	40	FRICTION
31	SS	200	6	0.1	20.5	95	87	0.16	33	FRICTION
17	SS	100	6	0.05	20.5	47.5	43	0.16	22	FRICTION
16	SS	100	6	0.05	20.5	47.5	43	0.16	20	FRICTION
21	SS	160	6	0.1	37	55	87	0.16	18	FRICTION
3	AL	200	15	0.05	20	40	5.6	0.5	85	NONE
13	AL	145	7.5	0.05	20	31	2.8	0.5	110	TENSION
9	AL	160	7.5	0.05	20	45	2.8	0.5	85	TENSION
7	AL	297	7.5	0.05	20	41	2.8	0.5	82	TENSION
6	AL	297	7.5	0.05	20	47.5	2.8	0.5	82	TENSION
18	AL to 100	150	7.5	0.05	20	47.5	2.8	0.5	63	TENSION
8	AL	297	5	0.05	21	40	1.8	0.5	57	TENSION
30	AL	160	7.5	0.05	20	47.5	2.8	0.5	50	SKIN
14	AL	100	7.5	0.05	20	47.5	2.8	0.5	43	FRICTION/ TENSION

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