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DISCUSSION REPORT BY C TRUMAN DAVIES (Ground Anchors Ltd)

DESIGN

Overall stability

1. Dr. Ostermayer, introducing the Session, highlighted points from his pre-printed text and made a number of specific proposals relating to anchor design. His first proposal was that, in the case of multi-tied walls, the stability of the wall and the position of the fixed anchors should never be calculated only to take account of rotational sliding. The sliding surfaces for the individual anchors should also be calculated to ensure that the deepest anchors are long enough. The deformation of the whole anchored soil block and the horizontal compression of the soil below the excavation base should also be considered, as they can both produce large wall displacement.

2. Commenting on Dr. Ostermayer's design proposals Dr. Littlejohn raised the following questions:-

(a) How does one assess the spacing required between the fixed anchor zones to ensure that the anchors, the retained structure and the ground act as a composite unit in the field with the assumed mechanism of failure?

(b) What experimental or theoretical evidence is available to justify the upper limit of a 4.0m spacing, where the point of load transmission may be taken as the mid-point of the fixed anchor length?

(c) Is the Kranz definition of safety factor - that is, Fa = Maximum A/Working A - really relevant to the overall stability analysis?

Can engineers be blamed for inferring that only low capacity anchors should be employed since, according to the factor of safety definition, the higher the working A value, the lower the factor of safety. This may be misconstruing what Kranz originally meant, but it is the easy inference (ref.1). Dr. Littlejohn confirmed his preference for Brinch Hansen's concept of safety factor (ref.2).

3. Dr. Littlejohn then developed his own views on overall stability to illustrate the basic similarities in the methods employed in Germany and Britain (ref.3). Using Fig.1, minor differences were highlighted to emphasise the conservative approach in Britain. With regard to this figure, the passive resistance is ignored initially which gives an estimate on the safe side, cf. Dr. Ostermayer's Fig.2b, only when the factor of safety calculated is about unity or less, is the passive resistance included to give a more accurate analysis. The justification for

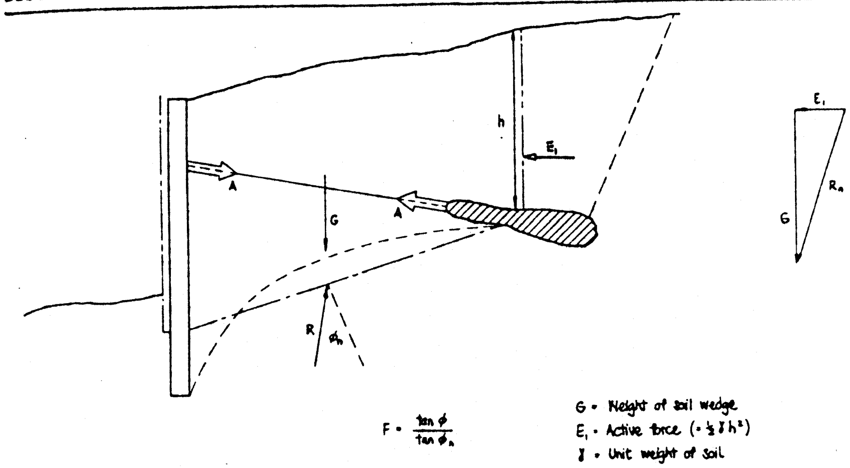

Fig.1. Stability of a wall with one row of anchors (modified Kranz method)

this simple approach is that anchors in the field are often long or deep seated because of adjacent foundations, and the calculations need only check that an adequate safety factor is present. Unlike Continental practice Dr. Littlejohn suggested that on completion of the calculation the fixed anchor should be re-positioned behind the wedge, in view of the limited data currently available on the mechanism of load transfer.

4. Dr. Littlejohn considered that many of the points raised by Dr. Ostermayer in relation to his Fig. 3 were important, and demanded serious consideration by practising engineers. Finally, Dr. Littlejohn emphasised that wedge mechanisms of failure were normally applied to frictional soils, and that it was incorrect to assume a wedge mechanism in cohesive soils, particularly for the analysis of short term stability.

Fixed anchor zones

5. In his written introduction Dr. Ostermayer had proposed that there should be a minimum anchor spacing of 1.5m and a separation of 1.5m between fixed anchor zones. Dr. Littlejohn considered that, if interference between anchors is the main concern, it should be sufficient simply to specify the minimum fixed anchor separation. In Britain a spacing of 0.70m between anchors has been used successfully in practice.

6. Whilst agreeing with Dr. Ostermayer that there should be a minimum distance of 3.0m between fixed anchors and adjacent foundations, Dr. Littlejohn noted that, for surface foundations it may be necessary to increase the depth of the fixed anchor to 5 - 6m, either to improve the resistance to withdrawal or perhaps guarantee local shear failure around the fixed anchor.

Fig.2. Reinforced concrete walings, Lake Binnenalster, Hamburg

Proof loading

7. Dr. Ostermayer declared that each anchor at a construction site must be tested because, even using the design charts in Conference Paper 18 and applying a factor of safety in the region of 2.0 to give an estimation of permissible working loads, actual loads will be dependent upon soil conditions and installation techniques (ref.4). Dr. Ostermayer showed how reinforced concrete waling beams were used in Germany to guard against the danger of failure of single anchors. Fig.2.

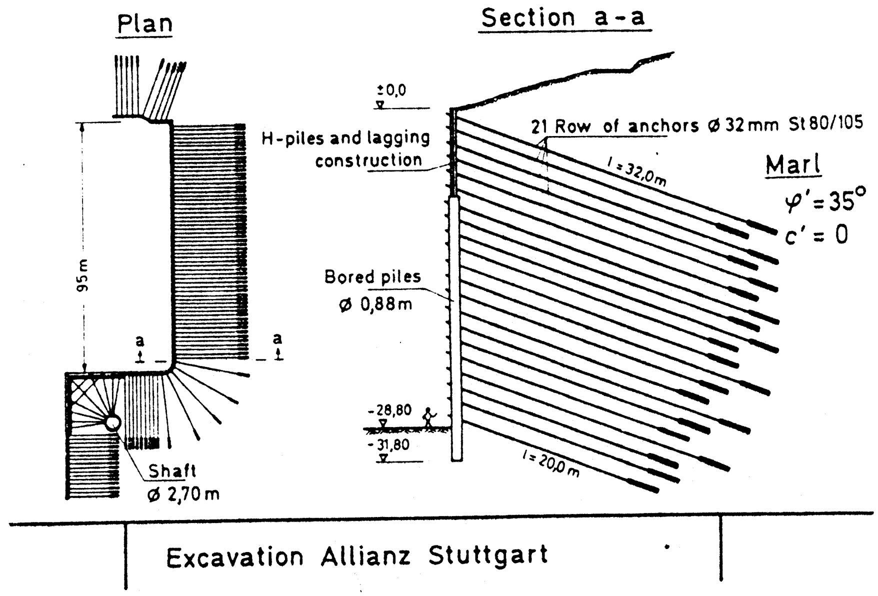

Fig.3. Anchor arrangement around re-entrant corner, Stuttgart

8. The point made by Dr. Ostermayer and other Introducers, that the overall design of the anchored system should ensure against failure of a single anchor leading, progressively, to serious damage to the anchored structure was taken up by Mr. Truman Davies. He was inclined to the opinion that taking advantage of the unique feature of anchors, in comparison with other geotechnical processes, it should be possible to eliminate individual anchor failures by proper proof loading. Proof loading each anchor as early as possible after installation can be carried out fairly cheaply and should protect against deficiencies of anchor design and workmanship.

9. Mr. MacFarlane agreed with Mr. Truman Davies that every anchor should be proof loaded but warned against the adoption of rigidly applied generalised overloads. He felt that the additional material costs were small compared with the benefits of allowing the designer to choose an appropriate measured factor of safety to take account of the probable behaviour of the structure in the soil. He hoped that the new code would give guidance in this respect.

Factors of safety

10. Having referred back to the written introduction, concerning the choice of a suitable anchor system, Dr. Ostermayer went on to recommend a uniform factor of safety of 1.75 against tendon failure. Mr. Truman

Davies queried this recommendation, which appeared to give a lower margin of safety for permanent anchors than the U.K. practice of adopting a factor of safety of 2.0. It was shown later, by Dr. Littlejohn, that there is very little divergence of opinion between U.K. practice and Dr. Ostermayer's recommendation, because Dr. Ostermayer's figure is related to the yield point (83.5% fpu) and U.K. engineers base their calculations on the characteristic strength (100% fpu) of the tendon.

11. To conclude his remarks Dr. Ostermayer impressively illustrated how the problems associated with re-entrant corners were solved in Stuttgart using steel ties between concrete waling beams and a 2.7 metre diameter shaft. Fig.3.

CYCLIC LOADING

12. Professor Hanna reported that whilst involved in giving design guidance on tying down permanent dry dock floor slabs he became interested in the effects of the tidal repeated loading that these anchors would be subjected to during a life cycle of 150 years or more. A search of the available literature showed that very little is known about slow repeated loading of foundations, either in tension or in compression, and consequently it was decided that laboratory tests might give an indication as to how such foundations might behave under this severe form of loading. To this end, model tests were carried out in dry sands subjected to different over-consolidation ratios and the anchors were subjected to different repeated loads and alternating loads.

13. Results of these tests have shown beyond doubt that repeated uplift loading leads to an accumulation of large displacements far in excess of those developed with corresponding static loads. The detrimental effect of alternating loads is to lead to failure of the anchor in the soil mass. The real problem is to be in a position to decide when an anchor is going to fail and as yet there is no simple and reliable means of predicting this. Test data show that an anchor may appear to be totally satisfactory and suddenly it will start to fail. This is clearly related to the dilatancy effects going on in the soil in the vicinity of the anchor and consequently is an effective stress problem. It is clear that much fundamental work must be done in this field with not only saturated sand but also with clays, and this work is now in progress.

14. Professor Hanna drew attention to the problem because he believed that a number of practical engineers are looking at the possibility of using anchorages and piles to support structures subjected to repeated loading, and he warned that in these situations great care must be exercised otherwise failure may result.

15. The questions he suggested which should receive attention are: What effect has proof loading an anchor on subsequent repeated loading behaviour? What form of field testing is practicable for the assessment of repeated loading behaviour of anchors? What scale effects, if any, exist, and how can these be overcome with respect to the rate at which the loading tests are performed?

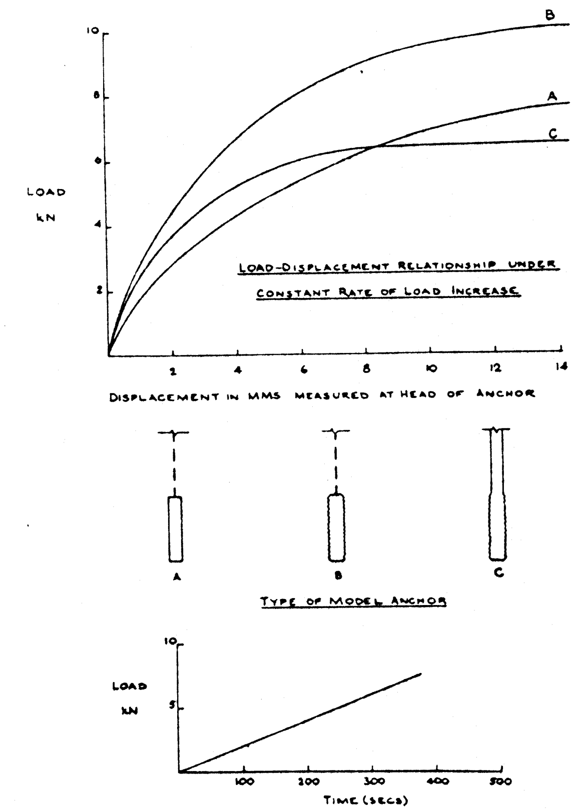

Fig.4. Load displacement curves for three types of model anchor

ANCHOR BEHAVIOUR

Model tests

16. Mr. Maddocks described a series of model tests being carried out a the Department of Civil Engineering, University of Bristol, into behaviour of model ground anchors in cohesionless soils, using a specia developed test rig to carry out plane strain and axi-symmetric tests in a narrow tank filled with a dense uniform sand. Photographic technique were used to observe displacements and strain fields throughout the loa ing with the aid of a stereo comparator. Two series of tests were carried out. First, the behaviour of different models under constantly increasing loads was established. Secondly, two of the models were sub mitted to repeated load tests.

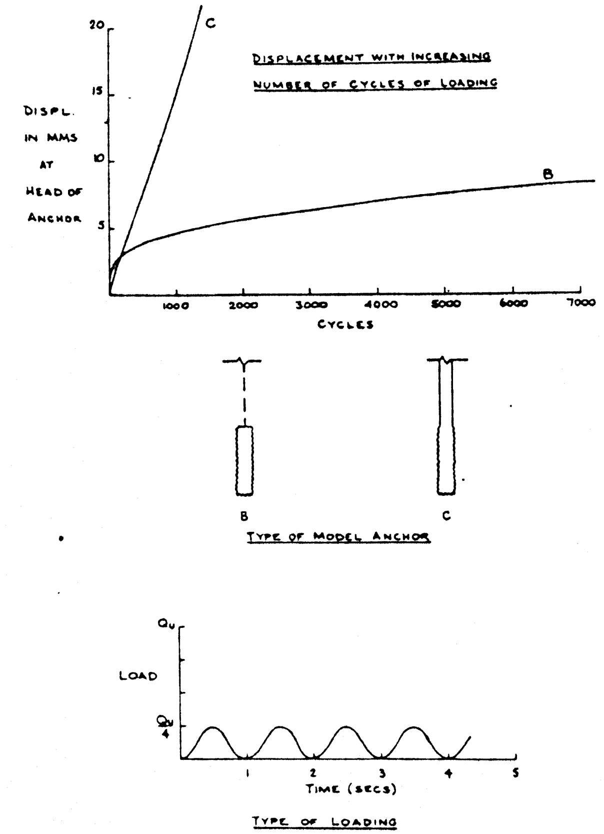

Fig.5. Displacement curves, repeated loading tests

17. The behaviour of three different types of model was investigated at a constant "depth of anchor body" to "anchor body width" ratio of 40, to ensure a "deep" mode of failure. The 25mm wide and 125mm long anchor bodies were made of steel and attached to the jack by two steel rods. Type A anchors had smooth steel surfaces, Types B and C had rough surfaces, achieved by sticking sand to the anchor body, and Type C had smooth wooden formers around the anchor rods, to model a continuous shaft to the surface.

18. The curve for Type A represents resistance to loading developed at the front end of the anchor only. Fig.4. The curve for Type B shows how side friction combines with the front end resistance to create a

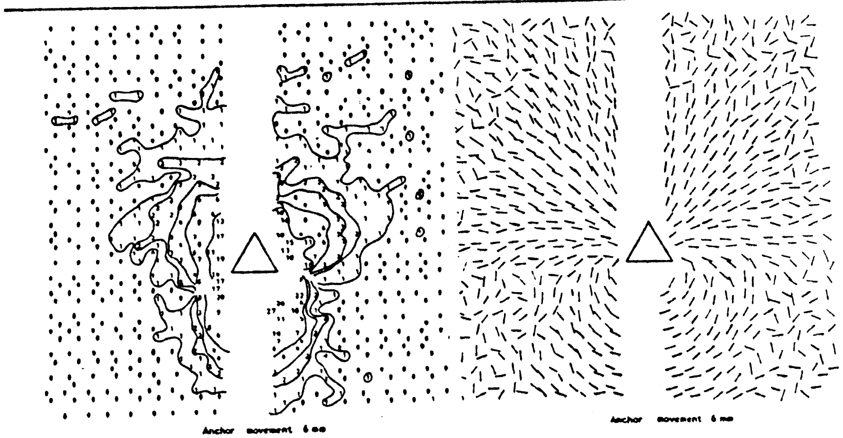

Fig.6. Shear and compressive strains around a single anchor

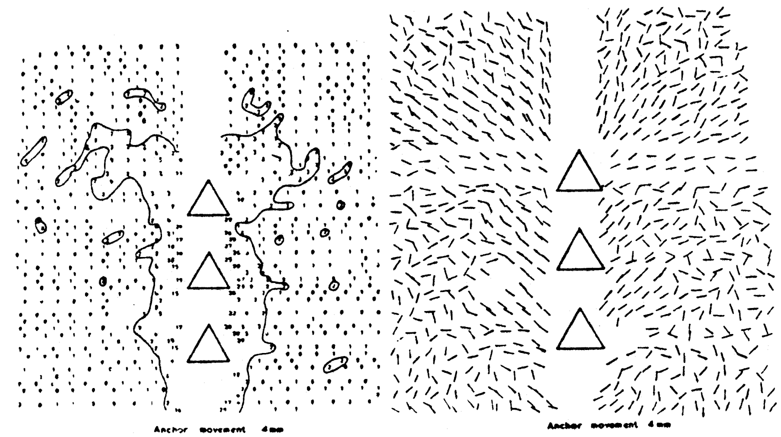

Fig.7. Shear and compressive strains around a triple underream anchor

larger displacement field and a higher ultimate load. The Type C curve relates more to the full scale situation, where displacements at the front end of the anchor are limited by the presence of the shaft.

19. A series of slides showed the development of the displacement fiel of a Type B anchor and the formation of a cavity below the anchor until a point is reached, at a figure equivalent to over half the ultimate lo capacity, when the sand starts flowing into the cavity from the sides o the anchor, reducing side friction. This reduction continues and as ultimate load is approached all resistance is due to front end displace ments. Stress measurements on the vertical face of the anchor indicate

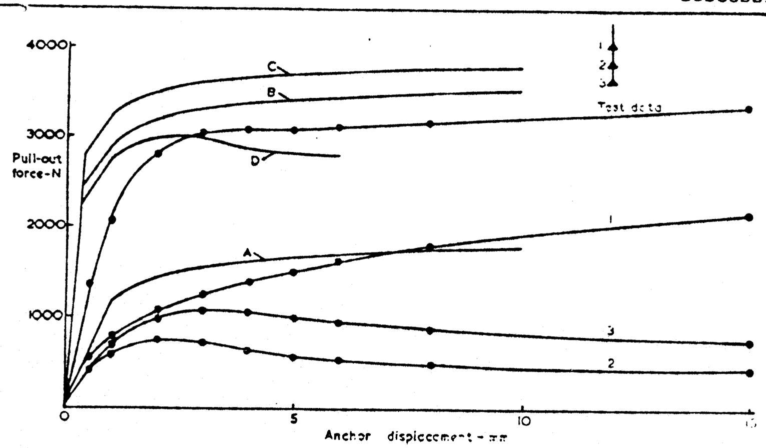

Fig.8. Loads carried by the three underreams

that the maximum frictional resistance is developed before the applied load has reached half its ultimate value.

20. The results of the repeated loading tests, of sinusoidal form with a frequency of 1 Hertz and an amplitude of 25% of the ultimate load, determined in the constant rate of load tests, are shown in Fig.5. With the Type B anchor, because the model allows front end displacement of the sand, the rate of anchor displacement is substantially reduced after the first few loading cycles. Sand moves from the sides of anchor into the cavity developing beneath it and the resistance due to side friction diminishes quickly until, by the 5000th cycle all resistance to loading is developed by displacements at the front end of the anchor. The Type C anchor where, because of its construction, end displacement fields are unable to develop exhibits a rapid decrease in side friction and, by the 800th cycle resistance to loading is mainly due to side friction at the top end of the anchor only.

21. Dr. Ostermayer enquired whether it had been possible to separate the proportions of total load provided by the side friction and end resistance components and pointed out that it is well known that only a small displacement is required to mobilise side friction whilst end resistance is only mobilised at large displacements.

22. Mr. Maddocks replied that they had not been able to separate the individual components in the work done to date but that his work will continue with axi-symmetric tests and further load tests introducing other parameters.

23. Following on from his contribution to the discussion at the 1974 Conference Dr. Swain described further plane strain results obtained from his model tests at Cambridge (ref.5).

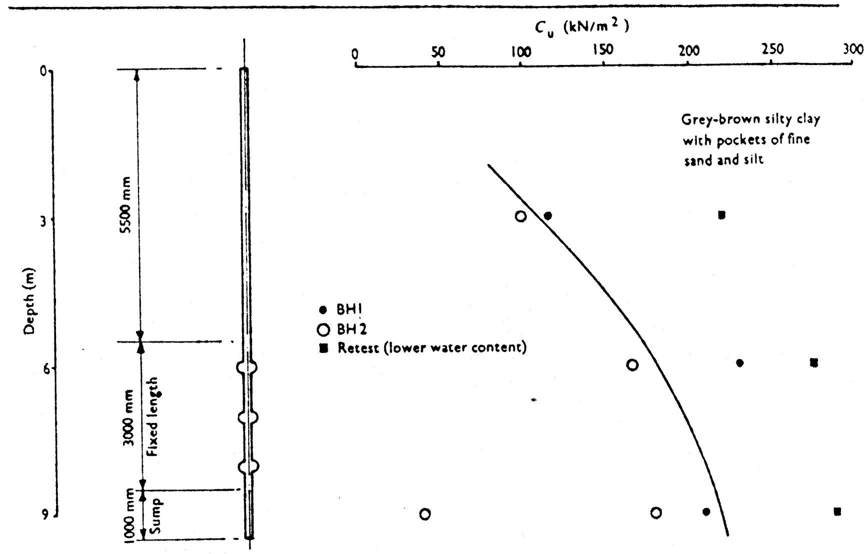

Fig.9. Strength depth relationship and anchor geometry

24. Figure 6 shows the cumulative shear strains around a single underream. One can observe "fingers" of shear strain developing where crack are beginning to occur. The cracks at the base start simultaneously an join up with those at front eventually leading to rotation. Even more significant, they are radial in front but circumferential behind with a expanding cylindrical cavity in front and a collapsing cylindrical cavi behind. Using this observed behaviour to modify available theory he wa able to predict, to within 4% the actual test data results. The actual displacements and principal strain directions were very clearly of a compressive nature in front, contrary to the bearing capacity theories generally used for anchor design.

25. A triple underream anchor is shown in Fig.7, with very localised shear strains on what turns out to be a shear or slip plane down the side of the anchor, small shear strains occurring behind. The principa strain directions are again radial in front and circumferential behind. In this case they are at approximately 45°, indicating underream shear down the sides.

26. The loads carried by the three underreams are shown in Fig.8. Curve A is that predicted for the front part of the top underream, assuming only an expanding cavity. It over predicts at a very early stage with respect to the elastic part of the expanding cavity theory. Curve B is the actual summation of the forces on the three underreams and is the predicted forces, using the expanding cavity theory in the front and rear plus the effect of shear on the slip plane down the side

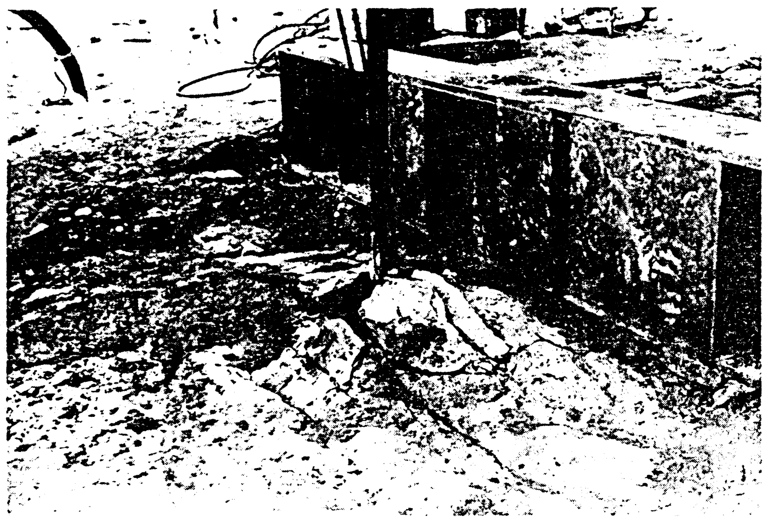

Fig.10. Failure of 1.5m long fully embedded rock anchor

taking a constant Cu throughout. This being the one case in the prediction where a value can be obtained for Cu. Curve C was predicted taking the lateral pressures on load cells on the sides of the apparatus. The actual prediction, with Cu as a constant, is very reasonable when compared with the actual data. Using this method Dr. Swain is predicting what is occurring with a method of analysis based on what was observed.

27. In reply to a point made in an earlier Session about full-scale testing Dr. Swain felt that he would not have obtained the data from a full-scale test even assuming sufficient funds were available.

Statistical analysis

28. Dr. Bassett then posed the question, what reliance do Consultants expect from design formulae for anchor capacities? He claimed that most design formulae were nothing more than intelligent guesses and considered it unjust that a specialist contractor was criticised, for the inaccuracy of his original estimates of capacity, when the first load test results appeared.

29. In support of this he referred back to the statistics on anchor tests at Orfordness which he mentioned at the 1974 Conference (ref.6). With 600 - 700 tests results he had been depressed to find that with a mean capacity of around 50 tons ultimate loads ranged from 35 to 80 tons. Although the model tests described by Mr. Maddocks and Dr. Swain

Fig.11. Anchor strands with strain gauges attached

have provided a little more insight into the mechanism of behaviour, he was not aware of any other publication feeding back data on test anchors to enable statistical estimates of the correctness of design formulae to be made. Taking up this point Dr. Littlejohn thought that Consultant in this country would not accept predictions of anchor capacity which although statistically correct resulted in a reduced factor of safety. In practice, Consultants require a design approach that guarantees a minimum ultimate load holding capacity, to which their stated factor of safety can be applied. He did not think that more sophisticated rules were necessary, provided that this minimum failure load could be guaranteed. He agreed with Dr. Bassett that Consultants should provide more data on the full scale performance of anchored systems, so that a statistical back-analysis could be used to judge the relative merits of the various empirical design rules.

30. Dr. Buttling gave information about a small-scale test programme recently undertaken on underreamed double protected anchors in the London Clay, where his experience had been rather different from that of Dr. Bassett. In Dr. Buttling's tests 12 anchors were installed, of four different geometries, and they were looking for failure within the clay or within the cement grout. Eight of the twelve were judged, by comparison with their design formula, to have failed within the clay at loads between 53 and 60 tons. Three of one particularly geometry failing with encouraging consistency at 59, 59 and 60 tons. One "rogue" anchor failed at 72 tons contrary to expectation and the other three were judged to have failed within the grout at loads between 38 and 44 tons. Dr. Buttling's point was that the spread of failure loads within the grout was proportionally greater than the spread of failure loads within the clay. Because this was a test programme he felt that the grout was likely to have been of more uniform consistency than grout mixed on a production site.

36. This indicates a slight reduction in the undisturbed cohesion, which is reasonable as a result of:-

(a) soil relaxation due to hole formation;
(b) water used in forming underreams;
(c) the underreams themselves forming a proportion of the failure surface.

Considering the straight shaft compared to the underreams however, there is an increase in diameter of 363/140 which implies that, relative to a straight shafted anchor, the adhesion factor has been increased to $363 \div 140 \times 0.78 = 2.02$. This is an increase of nearly five times the normally assumed adhesion for grout in clay whereas Dr. Ostermayer's curves (Fig.12) only indicate increases in shaft friction in the region of 50%.

CORROSION PROTECTION

37. Dr. Bassett put the question, is double protection necessary? As most anchors are comparatively deep, in ground well below the oxygenated layer he believed that even unravelled strand was satisfactory for permanent anchors. He then raised the more important question, does a double protection system consisting of an Araldite block cast around the strands and having a corrugated outer surface offer the correct solution? This system has proved very satisfactory in rock but in clays, which can expand at relatively low loads, the outstands on these surfaces which are normally 2mm at 4mm spacing can overstress the grout so that it fails in tension. By his calculation, it only requires about 20 tons to split a 1.5 to 2.0m length of 150 - 175mm diameter grouted column. Dr. Bassett concluded that, for reasons other than corrosion protection this type of double protection system is unsuitable for clay anchors.

38. Mr. MacFarlane disagreed with Dr. Bassett and felt strongly that all permanent anchors should be double protected. His argument was that without a double protected bottom anchorage there was a danger of re-

juvenated oxygen and water travelling up the strands of a strand tendon which would lead to corrosion.

39. In response Dr. Bassett stated that the current form of double protection with a corrugated Araldite tube is not sufficient. He agree that the aim should be for double protection but felt that it should no be used unless its mode of operation was fully understood.

FULL SCALE TESTS

40. Mr. Bruce described in outline an elaborate series of full-scale anchor tests conducted by the Aberdeen University Geotechnics Research Group and Universal Anchorage Company Ltd. The fundamental theme of th programme was to investigate the influence of several parameters on anchor performance, particularly load transfer mechanisms. The anchors were installed in the Upper Carboniferous sandstones and shales at Withnell Quarry, Lancashire. The results of these investigations are still being analysed but a number of significant points that have already emerged, relating to design and construction methods, were explained by Mr. Bruce.

31. Following on with another question Dr. Bassett asked, how much responsibility should the Consultant or Main Contractor place upon the specialist contractor, and what should be the relationship between the various parties? It was his opinion that the responsibility for the overall design and the problem of wall/anchor interaction must lie with the Consultant. As part of the design process he felt that the Consult must seek the advice of the specialist about the strain displacement performance of the fixed anchor because different situations would brin into play different mechanisms.

Creep

32. To provoke discussion on the subject Dr. Buttling stated that cree is not really relevant to anchors because creep is something which occu at constant stress. He argued that below some critical level an equilibrium between stress and strain will be reached and above the critica level creep will continue until failure is reached. He contended that most anchoring situations represent a stress relaxation condition, because of the relative fixity of the two ends of the anchor.

33. With regard to permanent anchors Dr. Buttling considered that in time the wall and the anchored soil mass act in a composite manner due to stress relaxation. The original test and proof loads may well reduc with time but if other extraneous factors induce extra movements in the wall they will reproduce the anchor loads already tested.

Underreamed anchors

34. In a later written communication to Dr. Ostermayer in response to the statement made by Dr. Ostermayer in his paper to the 1974 Conferen when he reported that, in Germany, post-grouting had been found more

effective and cheaper than underreaming in increasing anchor carrying capacity, Dr. Buttling provided the following results (ref.4).

35. At the test site already mentioned, eight of the anchors had the same external geometry as indicated in Fig.9. The results of quick undrained triaxial tests to give Cu are also shown. The failure loads of these anchors were:-

59, 60, 59, 57, 55, 53, 52.5, 54 tonnes
Giving an average of 54.9 tonnes

The underreams were measured with calipers giving an average diameter of 363mm the straight shaft being 140mm diameter. Using a Littlejohn (1970) analysis but including adhesion factors for the "undisturbed" failure plane and also allowing for friction over half the free length gives:-

Average Cu 6-9m depth 190 kN/m^2
Average Cu 3-6m depth 140 kN/m^2

$$T = \beta(\pi \; DL \; Cu) + \pi (D^2 - d^2) 9 . Cu. + \pi \; dl \; \alpha \; Cu$$

$$\therefore 549 = \beta(433.4) + 126.9 + 83.1$$

$$\therefore \beta = 0.78$$

41. The conventional approach to overall stability based upon the weight of a theoretical cone of rock may be incorrect. Mr. Bruce's tests show that the insitu strengths of the rock, which are not normally calculated and are usually assumed to represent the "factor of safety", form by far the larger part of the ultimate resistance to rock mass pull-out. As an example he quoted one short anchor with a design capacity, by cone theory, of 190 kN which actually pulled out at nearly 1500 kN, Fig.10.

42. Bond stress distribution and debonding were other mechanisms being investigated by means of 1400 variable resistance strain gauges attached to the strands on about half the anchors, Fig.11. Analysis of the results has shown that interfacial bond is not uniformly distributed but is concentrated at the proximal end. Debonding occurs from the lowest loads and is progressive distally.

43. Results show that underreaming increased the ultimate capacity of certain anchors by almost 50%, relative to straight shafted anchors in the sandstone. They also found that there were distinct advantages, in terms of grout steel bond by using spacers to ensure an inter-strand spacing of at least 5mm.

44. The full results of the test programme will be published in due course and will also contain information about the relative merits of multi- and mono-strand stressing, friction losses in tendon free lengths due to various protection media, the effects of inter-strand or tendon "noding", the influence of end plates on bar tendons, and the minimum slenderness ratio required to ensure localised shear failure within the rock mass.

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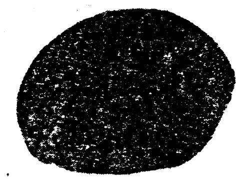

Fig.3. Withnell Quarry: detail of underream (pencil indicates scale)

26. Mr. J.T.C. Harvey discussed the stressing of ground anchors and felt that several undesirable features were being taken from prestressed concrete techniques. In his view it was essential that the stressing system was chosen to allow the load to be checked at any time on the upward cycle using the jack in conjunction with shims and a loading bridge (Ref. 9). He also felt that the pressure gauge on the pump for the jack was more reliable and more easily calibrated than load cells or dynamometers and should be used in preference to them. The bursting forces developed in any wall member during stressing should also be allowed for by providing suitable helical reinforcement.

27. In a written contribution Mr P. Robins described the programme of testing ground anchors undertaken by the Greater London Council in connection with the Thames Flood Prevention Works. Eighteen anchors installed in sand and gravels, Thanet Sand and Chalk were tested to destruction. On the basis of the ultimate values derived, 4 long term (up to 3 years) test anchors were designed.

For the contract anchors 40mm diameter Macalloy bars with working loads of 550 kN and lengths generally exceeding 20m were employed. The fixed anchor length was determined on the basis of the previous tests to give a factor of safety of 3.0. Each contract anchor was tested to 1.75 times working load.

CASE HISTORY OF INSTALLATION TECHNIQUES AND STRESSING OF ANCHORAGES AT WITHNELL QUARRY

28. Mr. A.D. Barley described the installation and testing methods used for the 84 anchors installed as a joint venture between his company and the

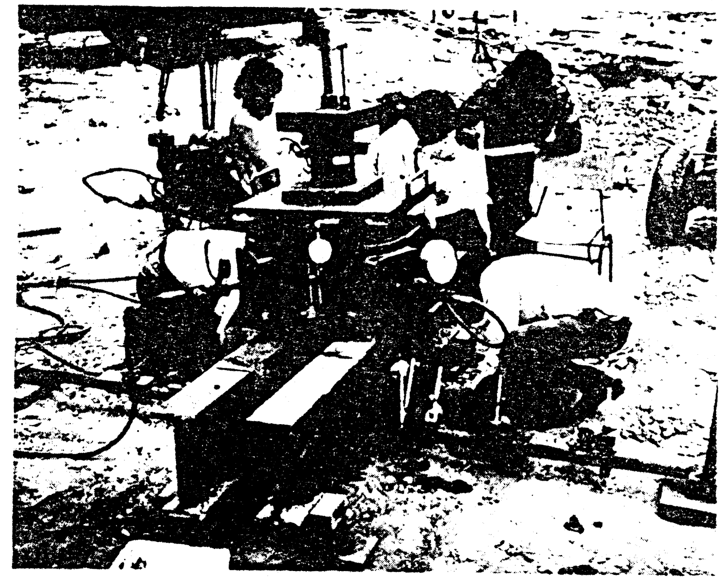

Fig.4. Withnell Quarry: anchor testing arrangem

University of Aberdeen. The scope of the investigation had alre described by Mr. D.A. Bruce in Session IV (40-44).

29. Drilling rates in the carboniferous sandstones and shales follows :-

(i) 114mm ($4\frac{1}{2}$") diameter holes using rotary percussi (down the hole hammer) 1.0m/10 mins.

(ii) 92mm ($3^5/8$") diameter holes using rotary methods flushing 1.0m/7 mins.

The nature and colour of rock cuttings, ground water level, drilli bit wear were logged throughout. Anchor drilling rates could als to the point load index test (Ref. 10).

30. Twenty four of the 84 anchors were under-reamed using th under-reaming tool. A pair of cutting flights were opened gradua advancement of the drill string during continuous rotation. In or assess the size of the cones formed by the flights two shallow hole under-reamed while monitoring drill string advancement. Fig. 3 typical under-ream. Under-reaming times varied from 15 to 45 Experiments were also carried out with cutting flights designed to ellipsoidal under-reams.

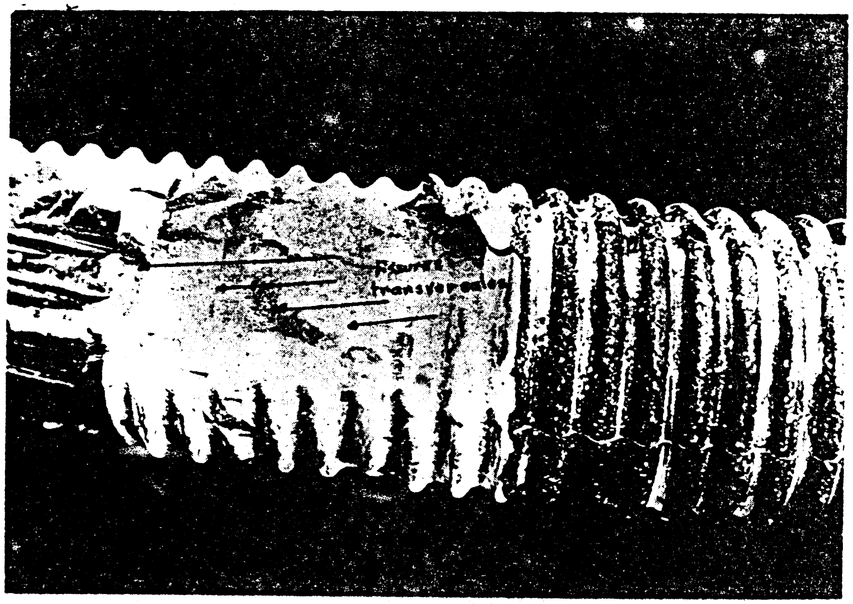

Fig.5. Transverse cracking of the fixed anchorage length

31. On completion of drilling or under-reaming each anchor hole a water test was carried out (Reference 2). Many anchor holes did not satisfy the water tests and were filled with grout based on a water/cement ratio of 0.45.

Most holes were redrilled within two days of grouting and drilling rates varied from $\frac{1}{4}$ of rock drilling rate with the grout one day old to $\frac{1}{2}$ at two days old. Waterproofing proved effective in all the straight shafted holes but water loss was still excessive in the under-reamed holes. Unless the redrilled holes were drilled to the exact depth the under-reaming tool exposed fresh fissured rock and a second grouting operation was necessary.

32. The grout was produced using a colloidal type mixer. Expansion bleed, and slump tests, were carried out regularly. Test cubes were taken from every batch of grout, and anchor testing was carried out after a minimum cube strength of 28 N/mm^2 had been reached.

33. Fig. 11 in Session IV gives details of the instrumented strands. Initially the tendons were placed by hand but a crane was used later to minimise the possibility of damaging the strain gauge installations.

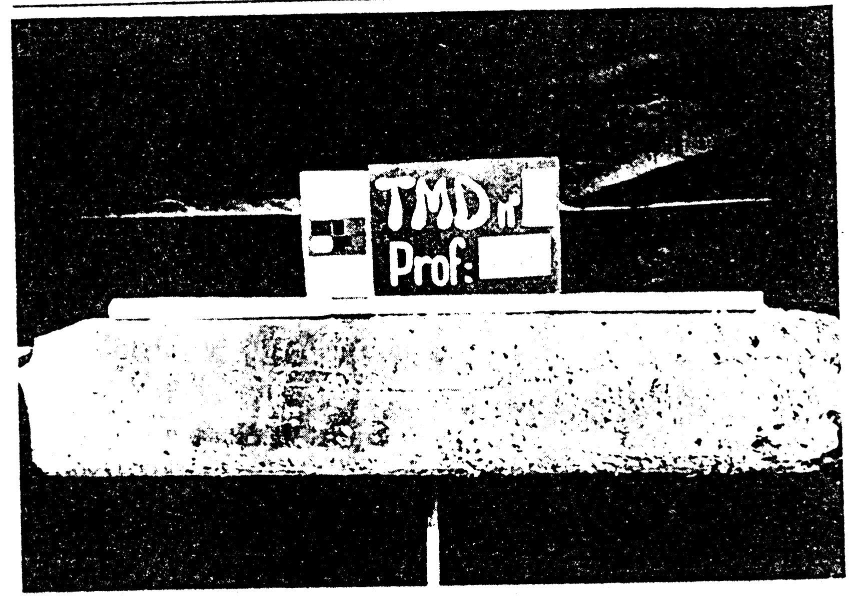

Fig.6. Longitudinal cracking of fixed anchorage length

34. Anchor testing was carried out using the arrangement shown in Fig. 4. Both multi-stressing (loading all strands simultaneously) and mono-stressing (loading strands individually) were employed. Anchor load was recorded from an annular load cell to within an error of 1%. During multi-stressing extensions were measured to 0.01mm using dial gauges. During mono-stressing extensions were measured to 0.5mm accuracy. The stressing procedures adopted were similar to those shown in Fig. 1 and allowed the separation of the permanent and elastic displacement during each cycle.

35. Although stressing the strands simultaneously permitted accurate extension readings to be taken, uneven loading of strands occurred regularly. At the end of a load cycle there was often slack in some strands, and this had to be corrected This effect was particularly frequent when loading "ferruled" anchor strands. These were formed by unravelling the strand and adding two 25mm long ferrules to the central wire of each strand. The remaining six wires of strand were then placed around the ferrules whose positions were staggered for each strand making up an anchor tendon. Despite this precaution there were instances where individual strands broke at high loads due to very uneven loading.

On this site this phenomenon was not found when each strand was loaded individually but it has been known to occur elsewhere (Ref. 9).

a review of diaphragm walls

a discussion of 'Diaphragm walls and anchorages' published by the Institution of Civil Engineers

INSTITUTION OF CIVIL ENGINEERS LONDON 1977