SLOPE STABILIZATION BY NEW GROUND ANCHORAGE SYSTEMS

IN ROCKS AND SOILS

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48. Slope stabilization by new ground anchorage systems in rocks and soils

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The advent of anchoring techniques some two and a half decades ago offered new solutions for soil retention and slope stabilisation. Their use is now well accepted and proven. This paper describes five recent case histories where anchorages were founded in difficult ground or rock conditions and required development of new techniques to achieve the most effective and economic solutions.

LANDSLIP STABILISATION AT RISCA BY-PASS

Introduction

1. During the construction of a dual carriageway by-pass at Risca in South Wales, a landslide some 60 m in plan diameter occurred in the steep (1:3) wooded slope above and through the site of the proposed carriageway embankment (See Fig.1). Information presented in the subsequent site investigation report suggested that the depth of the slip was in the order of 12 m, probably at an interface between clayey fragmented mudstones and the underlying strong sandstones. It was proposed to stabilise the landslide area by constructing a 6m high gabion wall along the line of the embankment and approximately at the toe of the slip and also install two rows of anchored precast plank walls approximately 15m and 25m up the regraded slope from the gabion wall. (See Fig. 2 and 3). Each inclined plank (2.5m x 2m) was to be retained by two 1900 kN working load anchors, one above the other, at inclinations of 32° and 56° below the horizontal.

Anchor and Soil Details

2. The materials within the body of the slip varied considerably, but may be summarised as loose to dense silty, clayey, sandy gravels, with occasional cobble and boulders of mudstones or sandstones. Below this debris was a general transition through very weak shales, mudstones and clays to moderately strong sandstones. During anchor installations, it was found that the sandstones themselves varied enormously in degree of weathering and fissuring - the sandstones were highly fragmented at some locations. Due to varying depths to the stronger sandstones, the envisaged anchor lengths ranged from 20 to 38 m. It was proposed in design to penetrate 8m into the stronger sandstone rock, and mobilise a working load stress of 560 kN/m² in the sandstone/gabion interface.

3. It was intended that anchors would be drilled using duplex drilling and water flushing to allow casing to the sandstone, then open-hole drilled with water flush into the fixed length in the rock. The hole would then be grout tested, pre-grouted and re-drilled if necessary, all in accordance with BSI Publication 0091. (Ref 1).

Preliminary Test Anchorages

4. Proceeding with the above construction techniques, two 22 m long preliminary anchorages were installed and tested to 2500 kN and proved a factor of safety of 2.5 at the rock/gabion interface, without failure. However, the installations of these anchorages
highlighted several problems:

i) the continual collapse of the open bore in the rock,

ii) the continual loss of flush returns during fixed length drilling,

iii) the very high volumes of grout necessary for pre-grouting (approaching 10 times the borehole volume),

iv) the failure of a single pre-grouting and re-drilling operation to seal adequately and stabilise the bore and prevent further loss of flush and high grout take.

Revised Construction Method for Production Anchorage

6. The above problems demanded a reconsideration of the proposed method adopted from the recommendations in DD61. It was, after test anchorage installation, considered that the "rock" formation in the proposed fixed length was more akin to dense gravel and rock fragments with a high void ratio, than to strong intact rock. Thus, the construction technique was adjusted accordingly and the rotary percussive drill casing was advanced to almost the full depth of the fixed anchor length. The casing was trimmed full of neat cement grout and providing the grout could be contained the tendon installed. The grouting mix was then changed to 1:1 sand cement to stem the flow of grout where possible into the fractures, fissures and voids. Grouting was carried out using end of casting grouting techniques as it was withdrawn over the fixed length. The grout was injected until a nominal pressure was attained to confirm that grout was being contained within and adjacent to the bore.

6. In situations where the casing could not be filled with grout, a sand-cement mix was tremied in and the casing withdrawn over the fixed length pre-grouting the fixed length only. On re-drilling the procedure was repeated. It was appreciated that where grout tightness was checked by the attainment of nominal back pressure only, there could be some element of risk with this system in the event of possible leakage of grout from the bore (after completion of grouting) and subsequently, failure during testing. However, the efforts to reduce anchor installation to a single or double phase operation in lieu of a multiple of pre-grouting and re-drilling operations was considered financially beneficial and the potential risk worthwhile.

The actual number of pre-grout and re-drills during construction of the 82 anchors amounted to only 80, whereas the initial proposed system would have involved well over 100. All 82 anchors achieved the full 1500 kN total load without failure and total grout take amounted to an average of 3 times the volume of the bore using the cheaper sand cement grout, whereas early operations indicated that the loss of neat cement grout would have been double that volume.

Summary

7. The availability of rotary percussive drilling technique, marketed in the early 80's, allowed the use of end of casting pressure grouting systems, normally associated with coarse granular materials, to be applied to highly fractured and fissured weathered bedrock. This was supplemented by the use of sand cement grouts to enhance the rock strength local to the bore, and yet to stem the flow away from the bore. The development of this system at Rissa contributed greatly to the major amendments made to the grouting recommendations detailed in DD61 for those now adopted in the new Code of Practice for Ground Anchorage. BS 8081 (Ref. 2). It should be noted that the number of working anchors retaining the slope are annually checked for load change. Information received over the four year period to date, indicates load losses in the anchors, due to consolidation of the slip material and plank settlement, to be in the order of 10 to 25%. Despite the occurrence of the expected settlement, there is every indication that the dual row of anchored planks and the ganion wall are maintaining a satisfactory and stable situation (see Fig. 4).

ALEXANDER OVAY, SOUTHAMPTON

Introduction

8. At the Southampton site where a new anchored sheet pile wall was required to support and protect the residential buildings of the new Ocean Village, an opportunity was provided to compare the behaviour of two anchorage systems. Several enquiries - some relating to stabilisation of slopes - had in...
previous years requested anchorages founded on the mixed granular/cohesive strata of the Bracklesham Beds. Some ten years earlier, temporary anchorages had been constructed in the beds using normal cement grout injection techniques, but prudently using low loads in the order of 250 kN. During those intervening years the largest anchor clay underreaming system available in the U.K. had been developed. The underreaming tool could simultaneously form a number of 700m diameter underreams at 1.5 m centres from a 160 mm diameter bore and was activated by a hydraulic ram at the drill head (Fig 5). More important than the actual size and the ability to ream simultaneously was the high capacity water flushing system which jetted water through portholes into the underreams and ensured fast efficient removal of the reamed spoil. Several systems developed by anchorage contractors in the 1970's had failed, not from the problem of reaming but from that of spoil removal. However, the major concern with underreaming the Bracklesham Beds lay in the potential collapse of the underreams constructed within the bands where the soil was more granular and less cohesive. In this situation it was questionable whether the head of flush water would provide adequate soil support to prevent collapse prior to grouting. Thus it was elected to investigate at trial anchor stage an alternative anchor system which involved the full advancement of drill casing to provide ground support to the end of the bore yet enhance the anchor capacity using a refined but untried system of load transfer.

9. The Single Bore Multiple Anchor system (SBMA) involves the installation of a multiple of "Unit Anchors" within a single borehole. Each unit anchor has its own tendon, and its own bond length in the form of an "encapsulation". (An encapsulation is a prefabricated bond length of tendon incorporating a corrosion protection system. The encapsulation of each anchor is located at a staggered depth within the borehole so that each anchor, loaded with its own stressing jack, transfers its load to a discreet length of the anchor bore. This system almost eliminates the effect of the progressive failure mechanism present in normal anchor systems and allows the simultaneous mobilisation of almost the entire ground strength throughout the full fixed length in the borehole (Fig 6). Furthermore, it also allows the utilisation of the ground strength over fixed lengths much greater than previously considered useful.

Anchorages and Soil Details

10. Both trial anchorages were installed to a depth of 34 m at an inclination of 30°, the fixed anchor length being founded within "firm to stiff silty clay with laminations and small lenses of carbonate fragments" (Bracklesham Beds). The presence of some silty clayey fine sand bands was also described. During site investigation work SPT values ranging from 20-44 had been reported at the fixed anchor depth in addition to ci' and ci values of 10-15 kN/m² and 24° to 26° respectively and also Cv values of 152 to 200 kN/m².

11. The underream anchor bore was used to 250 mm dia and drilled on with water flush to 24 m prior to simultaneously forming four underreams within a total period of approximately 2½ hours (underream installation, ramming and removal). Grout consisted of neat cement grout with w/c ratio of 0.85 and the total of 44 (50 kg) bags amounted to 1/³ of the holes and underream volume. Some grout may have penetrated the fill, but no collapse of the underream was apparent during the tremie tube or tendon installation. The double protected multistrand anchor tendon consisted of 6 no. 18.2 mm dystrands with an ultimate capacity of 1900 kN. The fixed length encapsulation was 10.5m long, 90 mm diameter.

12. The SBMA anchor was drilled and cased (153 mm O.D.) to full depth using water flush. Each of the 4 double protected unit anchor tendons consisted of a single 18 mm diameter dystrand with a fixed anchor length encapsulation 1.5 m long, 50 mm diameter. The fixed anchor length was 3.5 m thus the encapsulations were staggered at 3.5 m centres from the base of hole and total fixed anchor length of 17.5 m was mobilised.

13. On completion of the tendon installation, the casing was withdrawn over the fixed length and pressure grouting carried out using pressures up to 7 bar. Grout was augmented to 30 (50 kg) bags equating to almost twice the

![Fig 5 Profile of the underream and details of the reported ground conditions.](image1)

![Fig 6 Load distribution along a normal and an SBMA anchor system.](image2)
bore volume.

Testing and Test Results

14. Owing to the designed variation in free length of the unit anchors in the SBMA, it is necessary to load each unit anchor with a monostrand jack and record the total load on the unit anchors with an annular load cell. Although it is possible to multistrand the underreamed anchor it was also loaded by monostand system and monitored with load cell. Loading was carried out in cycles up to 80% UTS of the tendon, in accordance with DD81 (then current BS publication for Ground Anchored).

15. The underreamed anchor was successfully loaded to 1520 kN (80% UTS of the tendon) without indication of failure. The SBMA achieved a total load of 1397 kN and failure was observed on the proximal (shallowest) unit anchor at 135 kN. Other unit anchors achieved 284 to 300 kN (almost 80% UTS of tendon) without failure (see Fig 7). Extension characteristics of all tendons lay within the 90 to 110% of theoretical even at maximum load and thus conformed with DD81 acceptance criteria.

Summary of Test Results and Production Anchor System

16. The 20 production anchors required a working load of 388 kN. The underreamed anchor demonstrated a Factor of Safety in excess of 3.4 on working load and mobilised a clay shear strength of 130 kN/m² in cylindrical shear and end bearing.

17. The 5 unit SBMA demonstrated a Factor of Safety of approximately 3.4 on the working load and mobilised grout to ground bond stresses of 115 kN/m² on the proximal unit anchor, and 205 kN/m² without failure on the lower four unit anchors. Considering the lower four unit anchors alone, with a total capacity of 1189 kN, a Factor of Safety of approximately 3.0 was proven.

18. The choice of anchorage system for the production anchor was the four unit SBMA system installed in 34m long anchorages. It was considered that the proven Factor of Safety was more than adequate and that the smaller diameter drilling system would allow more rapid and economical penetration of the obstructions.

5 UNIT ANCHOR SBMA

![Fig 7 Final load cycle of each "unit" anchor.](image)

known to be present at certain locations in the quay.

19. Both the underreamed and the SBMA anchorage system proved that high capacity anchorages can be successfully constructed in the Bracklesham Beds as encountered at Alexandra Quay for retaining walls or slope stability. Furthermore, the underreaming system has now achieved test loads in excess of 2000 kN in clays but even more outstanding is the performance of the SBMA system in soils ranging from Boulder Clays (1500 kN without failure), London Clay (1450 kN), Alluvium (1200 kN without failure) and more recently weak Chalk (1216 kN without failure). The efficiency of the load transfer mechanism in the SBMA has demonstrated at six anchor sites that the system enhances anchorage capacity by between 50 and 150% over that of normal systems.

SLOPE STABILISATION AT BARTON-ON-SEA

Introduction

20. Continued erosion and land slippage occurring on the coastal slope at Barton-on-Sea were jeopardising the existence of several properties. A system incorporating deep sheet piles and an anchored R.C. embankment beam was proposed to stabilise the slope on one of these locations (Fig. 10). Both anchors and sheet piles were to achieve fixity below slip planes identified in the Site Investigation Report.

Anchors and Soil Details

21. Anchors were to be founded in the fissured Barton Clays with a minimum free length of some 25 metres. Triaxial tests on clay samples (38 mm and 100mm dia.) indicated intact clay strengths in the order of 120 to 200 kN/m². Recommendations made by Karsland & Butler (Ref.3) stated that the due allowance should be made for the presence of fissuring, thus exhibited Cu values from triaxial tests should be factored by 1.25 to 1.5. In order to conform with this and incorporate some allowance for disturbance during underreaming, the average exhibited Cu value (160) was multiplied by a reduction coefficient of 0.67 and thus an ultimate shear strength along with the underream of 104 kN/m² considered.

Utilising the large diameter (700mm) simultaneous multi-underreaming system with 5 underreams, an ultimate capacity of 1842 kN was estimated (Formula 5 in BS 8081).

Preliminary Anchor Trials

22. Unfortunately, despite the lack of anchoring experience in this fissured clay stratum, it was not considered feasible to follow the guidelines of BS 8081 and construct three preliminary trial anchorages. It was in fact required that one production anchor be subjected to a preliminary test to 1.2 x working load (2 x 600 kN) and would suffice. The anchor was installed with the five no. 700 mm diameter underreams and was satisfactorily loaded to twice the working load. Results conformed with the BS 8081 acceptance criteria for both load(extension and creep) characteristics (10 day test).
irregular load extension characteristics were exhibited and some further down rating of anchor working load was required. Subsequently, after a number of stressing crew visits to investigate load holding capacity, the total of 32 anchors provided the total restraint required to be applied to the capping beam and the works completed.

Summary

24. A number of 600 kN working load anchors constructed with simultaneous formed multiple underreams could not achieve the full design capacity in the Barton Clay and necessitated down rating. Back analysis using the underreamed anchor design formula indicated that at failure load of certain anchors, clay shear strength along the cylindrical surface joining the tips of the underreams were as low as 23 kN/m². Analysis of observations and assessment of ground takes could not fault the underreaming system or the workmanship. It was finally concluded that the very wide range of anchor strength values exhibited by the Barton Clay in the underreamed anchors resulted from the presence of excessive and multi directional fissures within the clay which had been encountered in a host of permutations of fissure sizes and fissure inclinations. Some spontaneous load loss during testing associated with a ground vibration probably resulted from the brittle failure of bond capacity within the mudstone bands.

25. After the installation of a number of additional down rated anchors, the full load application to stabilise the slip was provided by the anchors. It is recommended that further investigatory trial anchors are installed in Barton Clays prior to their usage for high capacity [greater than 300 kN].

REMOVING OF LANDSLIP DEBRIS DURING CONSTRUCTION OF PALM BEACH FLATS, SANDGATE, KENT

Introduction

26. The coastal area being developed just east of Sandgate is typical of a slope subjected to continual landslips and sea erosion over the centuries. At the site the depth of landslip debris, consisting of loose sandy silts and occasional sandstone boulders, ranged from 2 to 8m. The debris overlay Sandgate Beds which in turn overlay Hythe Beds. The development required the stabilisation of the existing landslip debris and the cutting into the slope for construction of the basement of a new block of flats. This was achieved by installation of an anchored bored pile wall across the slope in which the 700 mm dia piles passed through the Sandgate Beds to penetrate the stronger Hythe Beds (Fig 8). It was not possible, however, to found the anchors in the Hythe Beds nor was it possible to install the anchorages at the normal optimum inclination of 30° in order to achieve a good length of penetration of the lower, more sandy stratum of the Sandgate Beds. These controls were due to the severe boundary constraints imposed on the developer and the contractor.
required by anchor practice. On completion of jetting and grouting, the drill casing was re-advanced into the fixed length and the double protected tendon installed. End of casing pressure grouting was then carried over the fixed length at pressures of approximately 7 bars in order to recompact the soils around the jet flushed bore. Total grout takes were in the order of 1 tonne.

Testing and Test Results
30. Prior to the commencement of production anchors, three preliminary test anchors were installed at locations distributed around the site. This was necessary to confirm that capacities previously achieved (greater than 1000 kN) on an earlier investigation programme in Portmouth could be attained on this site. Testing was carried out using a multistrand jack to apply the cyclic loading operation while the reaction was provided by a R.C. waling beam constructed in front of the bored pile wall. An ultimate capacity of 1178 kN had been estimated and all three anchorages exhibited capacities remarkably close to the estimate: 1265 kN, 1064 kN and 1215 kN. These capacities demonstrated adequate factors of safety in relation to maximum anchor working loads of 390 kN.

Summary
31. Where 50 production anchors were required to stabilise a bored pile wall in a landslide area anchors were constructed using a new anchor system of jet grouting. This allowed anchors to be founded in the mixed soils of the Sandgate Beds and achieve the required capacity over short fixed lengths demanded by the anchor length limitations imposed by the site boundary.

SUMMARY OF PAPER
32. One particular soil condition required anchor load reduction due to the nature and extent of fissuring, despite the use of the latest refined anchoring equipment. This emphasised the need for the installation and testing of preliminary proving anchors in improved ground conditions prior to finalising a retention system.

However, new anchoring techniques have allowed the safe use of relatively high capacity anchorages in mixed soils and broken rocks. Prior to these developments, the ground conditions might well have been considered unsuitable as an anchoring founding medium or suitable only for low capacity anchorages. These new techniques have been employed for both soil retention and for stabilising slopes and landslips in a wide spectrum of conditions.

REFERENCES