

Exhumation and Design of Anchorages in Chalk.

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Synopsis

An anchorage test programme carried out on the site of deep basement construction in Norwich, UK, allowed determination of anchorage capacities, creep characteristic data and the exhumation of several anchor bodies in chalk.

This paper reports on the performance of the chalk based anchorages, the observation of exhumed grout body shapes and sizes, extent of grout penetration into dissolution cavities, pressure grouted fissures and the formation of grout cake around the grout body periphery.

Knowledge of construction techniques utilised for drilling, flushing, pre-grouting, re-drilling, pressure grouting and the general methods of control implemented to prevent dissolution cavity collapse, has been related to the characteristics of the exhumed anchor bodies. Analysis of these grout bodies has allowed the proposal of a failure mechanism, comprising shaft resistance and shear of grouted fissures.

Tests were carried out on both conventional and single bore multiple anchors (SBMA). Using short fixed length capacities with recently developed efficiency factors allowed accurate evaluation of the ultimate capacity of a typical 10m fixed length conventional anchorage. This design approach confirms that the SBMA system mobilised double the capacity of a conventional 10m fixed anchor. Advancements in SMBA technology has enabled designs of 20m fixed lengths, safely supporting working loads of 3000 kN.

INTRODUCTION

Although the installation of anchorages in chalk is not a new technology, there is a dearth of published literature adequately relating the constructed product and the load transfer mechanisms to the design approach. Most developments in ground anchorage technology over the past 30 years have been driven by the sound application of reliable field data from anchorage practitioners. One benefit of the rigorous construction and testing requirements for ground anchorages is that analysis can be carried out on the field data acquired during installation and testing. However, when the opportunity arises to exhume a tested anchorage the observations from such operations are of immense value and allow correlation with the associated ground conditions, construction technique, grout penetration and design.

This paper describes the testing, exhumation, inspection and analyses of anchorages installed in chalk during the Castle Mall Development in Norwich, UK. The results have been used to present a novel approach to design of the fixed anchor for an anchorage installed in fissured chalk.

DESCRIPTION OF SITE

The Castle Mall development is a two-story shopping centre with a basement up to 18m below ground level adjacent to a five story underground car park in the centre of Norwich. The permanent works comprised a 900mm diameter contiguous bored pile perimeter retaining wall propped by the floor slabs. Temporary support of the perimeter wall during construction was achieved using 1120 ground anchorages. Overall stability analysis determined that a limited number of fixed anchor lengths would be located in the Norwich Crag (granular materials) but that the majority would be founded in the underlying chalk stratum (Grose and Toone, 1992).

The site topography varies widely from 17 to 28 m OD resulting in retained heights of 7 to 18m. A five-phase site investigation comprised three inspection shafts up to 28m deep and 65 shell and auger boreholes up to 50m deep at the locations shown in Figure 1. The general findings of these investigations are summarised in Table 1.

Description	Level (m OD)	Depth (m)	Thickness (m)
MADE GROUND	17 to 29	0	0.1 to 11
Silty SANDS and GRAVELS with lenses and pockets of silty clay (NORWICH CRAG)	14 to 26	0 to 5 (av. 3m)	0 to 10
Very weak to moderately weak partly weathered to weathered white CHALK with flint bands (UPPER CHALK).	12 to 18	3 to 15 (av. 8m)	Proved to 45m

Table 1 Summary of ground conditions at trial anchorage locations

The groundwater table was at about 1m OD, some 8m below basement level. As part of the site investigation 24 boreholes were put down outside the site to establish the stratigraphy and confirm the Crag and Chalk were similar to the materials investigated in more detail throughout the site. Within the fixed anchor zones, chalk with Munford (Ward *et al*, 1968) grades IV to VI were encountered with SPT values ranging from 5 to 15 (Barley *et al*, 1992).

The presence of dissolution features within chalk is a well-documented phenomenon and they were anticipated on this site. Not surprisingly, drilling and sampling anomalies, attributed to dissolution features, were recorded to depths of 30m (-3m OD) in eight of the 65 boreholes. These sub-vertical, steep sided features tended to be loosely infilled and sometimes associated with voids. The anchorage design and construction techniques had to overcome the difficulties that these dissolution features presented. Furthermore their location could not be predicted so they would only be discovered and accommodated during anchorage construction.

ANCHORAGE CONSTRUCTION AND DESIGN

All anchorages involved the use of duplex drilling and water flush techniques. Drill casing of 114mm diameter with a 120mm casing shoe was advanced to full depth of all anchors. During drilling the majority of the chalk matrix was dissolved in the flush water and removed as a milky solution. Drillers reported extensive bands of flint beds, plus randomly occurring flints that were subsequently identified during anchorage exhumation as ranging in thickness from 20mm to over 1 m.

On this busy city site, where anchors passed below main streets, the risk of collapse of existing cavities induced by anchoring operations caused great concern. Strict controls of drilling, flushing, flush loss and precise rules for borehole termination and pre-grouting, or borehole completion and pre-grouting were applied. Where pregrouting was required then a grout mix

containing sand filler was used in order to reduce unnecessary penetration into fissures and cavities remote from the borehole. This also reduced grout flow characteristics and utilised a more economic grout as a filler. The implementation of controlled ground improvement as an integral element of the anchoring operations was specified in the contract. However, the contract also ensured appropriate itemisation for reimbursement for materials used and operations executed. Many of the 800 anchorages installed in the chalks did encounter full loss of flush within cavities or flint beds. Pregrouting was therefore essential to mitigate the risk of sink hole collapse. All anchorages were finally constructed using end of casing pressure-grouting techniques utilising pressure of up to 6 bar and a 0.45 water/cement ratio grout.

For design purposes sacrificial test anchorages installed in similar chalk conditions some 1.5 km from Castle Mall provided values of ultimate bond stress of normal anchors with long fixed lengths. However, such anchorages were unlikely to provide the ultimate anchor capacity with the specified factor of safety of 2.5. As a result anchorages were designed as Single Bore Multiple Anchor (SBMA) type, mobilising 525kN working load in the very weak chalk utilising short efficient unit fixed lengths.

PROVING TESTS

As part of the contract requirements, proving tests were specified in advance of the main contract in order to demonstrate the designed anchorages performance in relation to the weak ground conditions, workmanship and materials. Eighteen proving test anchorages were required of which eleven were installed with fixed anchor lengths located in the underlying chalks. The construction and performance of these test anchorages is fully described by Barley et al (1992).

Having successfully tested the anchorages up to 2.4 times working load (1260kN), it was established that during the site excavation at one location, it would be possible to exhume three of the proving test anchorages installed in the chalk (see Table 2).

Anchorage	Fixed Anchor Length (m)	Maximum Test Load (kN)	Maximum ** Bond Stress (kN/m²)	Type of Anchorage
4M7	12	1260	283	Four unit SBMA – single corrosion protection
5M7*	12	-		Four unit SBMA – single corrosion protection
6M7	2	538	751**	Single unit SBMA – single corrosion protection.

SBMA = Single Bore Multiple Anchor

* On completion of pressure grouting and withdrawal of casing, top of tendon not accessible for testing

** Other short anchorages achieved 849 kN/m²

Note: Original results from Barley et al (1992)

Table 2 Chalk Anchorages Exhumed at Castle Mall, Norwich

ANCHORAGE EXHUMATION

The investigation, carried out by the Authors, involved the following.

- Monitor the extent of the grout penetration into the chalk surrounding the anchor borehole.
- Monitor and photograph the size and shape of the full grouted anchor body and adjacent grouted fissures.
- Comment on any relevant geological features that may be apparent during the exhumation.
- Assess the integrity of the plastic sheathing and corrosion protection after testing.

Exhumation and inspection involved four stages –

1. At general site excavation level the anchorages were identified by tendon inspection (Plate 1)
2. A trench was excavated either side of the anchorage using a backacter machine, care being taken to reduce any disturbance (see Plates 2 and 3).
3. Pick axes and shovels were used to break the chalk away from the grout and to clear the underside of the anchorage grout column.
4. Finally, during the photograph and logging stage, wire brushes were used to clean the surface of the exhumed anchorage.

Certain sections broke during the above procedure but generally intact sections were cut in a controlled manner using a Stihl Saw, then lifted and placed aside for observation and logging.

A valuable consideration for any future exhumations would be a staged approach whereby finite lengths (1 to 1.5m) of fixed anchor are exposed, then cut and removed. By adopting this method, the process of handling long slender sections of the anchor grout would be avoided and facilitate easier inspection.

Anchorage 4M7

On recovery the 4680mm of fixed anchor grout was cut into four shorter lengths. Each length was visually inspected and key physical dimensions recorded. Figure 2 graphically presents the field measurements together with the original drilled diameter shown as a dashed line. The Figure shows longitudinal sections (two axes) and one cross section through the grouted end bulb.

Photographs corresponding to the sections in Figure 2 are presented in Plates 4, 5 and 6.

Over the last 520mm of the fixed anchor length a large bulbous formation of grout was observed (Plate 7 and 8). Beyond the fixed anchor lay a separate mass of grout (1.2m x 1m x 0.45m) (Plate 9)

Anchorage 5M7

A 4990mm grout column was recovered from the lower free length as one section. In addition a horizontal irregular 'block' of grout spanning either side of and perpendicular to the bore axis was encountered. Figures 3 and 4 presents a graphical representation of the field measurements for the free tendon length grout recovered and the large grout bulb. This is also shown in Plates 10 and 11.

Anchorage 6M7

A 770mm length of cylindrical grout column from the lower fixed length was recovered. No bulbs or major sectional changes were observed.

OBSERVATIONS AND DISCUSSIONS

Grouts and Grouting

Pebble sized flints were observed at the contact interface between the grout and the chalk, mainly at the underside but others randomly located. Larger, more irregular shaped flints were also unevenly distributed. These flints, together with the grouted voids and fissures in the chalk, accounted for the local variations in the surface roughness at the grout-chalk interface.

The presence of flints was observed over the free length of anchorage 5M7 and the bore diameter was reasonably uniform. The exception to this was that large irregular shaped block of grout attached to a section of the tendon free length 1500mm beneath the main section that was logged. This block, 1840mm by 790mm, was considered to be an existing cavity that may have been expanded during flushing prior to filling with by neat grout or the sand /cement grout (Figure 4 and Plate 11).

At four locations in the fixed length of anchorage 4M7, localised increases in diameter were observed. (see Figure 2 and Plate 4). The shape of these grout enlargements bore no resemblance to the original drilled hole. The orientation of these enlargements was random, and shapes varied from thick to thin fin-like protrusions on one side of the fixed anchor grout to the bulbous shapes seen in Plate 4. It is considered that the general shape of the grout bulbs reflect the grouting of existing voids or voids created by the water flushing operations and pressure grouting. The larger grout bulb at the distal end of the fixed anchor 4M7 (Figure 2 and Plates 7 and 8), is considered to have been formed as a result of extended flushing operation carried out on the completion of the hole in an effort to clear loose spoil.

In 4M7 the grout was observed to be either a neat cement grout, or a sanded cement grout surrounding a neat cement grout. The sanded grout is known to be from the pregrouting operations, whilst the neat cement grout is the grout used during the final pressure grouting carried out whilst withdrawing the drill casing

Besides the presence of the grouted voids the recovered bodies showed evidence that local fissures in the chalk had been grouted along the length of the anchorage. This was evident from thin 1 mm to 30mm wide broken fissure grout that protruded perpendicular to the circumference of the grout shaft (Plate 12). The clean dark fracture of these grouted fissures indicated the grout had penetrated into the fissures to some depth in the chalk mass, and that during the process of exhumation, some of these thin protrusions, associated with fissure

grouting, had been severed from the grout mass. Other protrusions remained intact and resulted in withdrawal of a fissure grouted chalk matrix. Removal of the chalk provided evidence of an irregular skeletal grout body (Plate 13).

These observations of the grouted fissures within the chalk mass are consistent with those recorded by Newman & Ingle (2002) in their inspection of exposed lance-grouted tunnel faces in chalk of similar grading. Plate 14 illustrates the existence of “grout patches” and “grout vein” adjacent to the grouted lanced hole.

Parts of the grout column exhumed from pressure grouted anchorage 4M7 (Plate 15), highlights the presence of a dense, dark grout (grout “cake”) of thickness from zero to 20mm around the grout body periphery (typical grout pressure between 5 and 10 bar). It is known that during pressure application rapid squeezing of the excess water from within the grout body into the permeable chalk can occur. As the grout density increase, reducing permeability further, water cannot flow from the inner grout body and further squeezing can only influence the peripheral zones. Hence, when observed, the colour change in the peripheral zones marking the dried cake boundary is well defined. Littlejohn (1970) observed similar characteristics in fine to medium sized sand and stated “In fine soil the value of A [locked in pressure] depends primarily on the residual grout pressure at the fixed anchor/soil interface, since during the injection stage the cement forms a filter cake at the interface through which only water travels”.

From exhumation of an end casing pressure grouted anchors founded in granular materials in Ealing, London in 1983, Barley (1997)¹ recorded his observations and findings. The anchorage was installed in soils with a high proportion of gravels but it was the presence of fine to medium sand that prevented the cement particles penetrating the soil (as concluded by Littlejohn 1970). However chemical analysis on a sample of fixed anchor grout demonstrated that the water:cement ratio had been reduced from 0.5 to 0.34 fully confirming that excess water had been squeezed from the grout into the surrounding soil.

Chemical analysis of the exhumed grout at Castle Mall revealed that the water cement ratio of the installed grout had reduced from 0.45 to as low as 0.29 and 0.261 (established by silicate analysis,) or 0.27 to 0.31 (established by calcium analysis)(Barley 1997)².

Experimental research work carried out by McKinley (1993) and by Kleyner and Krizek (1995) have investigated the practice of application of pressure to grouts contained in permeable soils and have produced valuable data on the change in the grout body. It was explained that changes in grout body are controlled by initial grout water/cement ratio, applied pressure, application time, soil permeability and locked in pressure.

The formation of an observed dense filter cake of grout around the borehole periphery is considered to be a major contributory factor in ensuring that a proportion of the applied grout pressure is maintained in the surrounding soil thereby enhancing pull out capacity. However the quantification of this phenomena is beyond the scope of this study and it is suggested that this could form the basis of future investigation in chalk anchorages.

Shaft Roughness Classification based on Past Practice and Influence of Construction Techniques.

The surface roughness of each exhumed section was classified using a system similar to that proposed by Weerasinghe & Littlejohn (1997) for anchorages exhumed in weak mudstone. The classification in Table 3 presents roughness values of R1 to R6 specific to chalk anchorages.

Chalk Roughness Number	Description
R1	No obvious irregularities; roughness varies from smooth to a feint imprint of the rock matrix. Uniform diameter.
R2	Protrusions/deformations on grout surface due to flint or grout penetration causing localised and general diameter increases up to 20mm.
R3	Larger protrusions/deformations on grout surface due to flint or grout penetration causing localised and general diameter increases greater than 20mm and up to 30mm.
R4	Very large protrusions/deformations on grout surface, distinct changes in diameter and localised blocks or fissures of grout stemming from main grout shaft. Average and localised diameter increases greater than 30mm and up to 80mm.
R5	Very large protrusions/deformations on grout surface, distinct changes in diameter and localised blocks or grouted fissures of grout stemming from main grout shaft. Average diameter increase greater than 80mm and up to 200mm.
R6	Extremely large protrusions/deformations on grout surface, distinct changes in diameter and localised blocks or grouted fissures of grout stemming from main grout shaft. Average diameter increases greater than 200mm.

(R1 to R4) = equivalent roughness number observed by Weerasinghe & Littlejohn (1997) in Mudstone
(R5 and R6) added for chalk classification purposes

Table 3 Classification of surface roughness for chalk anchorages

The above approach has been applied and the extent of each roughness classification calculated and converted into a percentage of the exhumed length. The average exhumed diameter has also been calculated for each anchorage and converted to a percentage increase on the original drilled 120mm hole. These figures are presented in Table 4.

Roughness Classification	Roughness classification as percentage of recovered length.		
	Anchor 4M7	Anchor 5M7	Anchor 6M7
R1	0	0	0
R2	6	66	100
R3	8	8	0
R4	63	17	0
R5	11	9	0
R6	11	0	0

Table 4 Summary of roughness classifications measured after exhumation

The results presented in table 4 show a noticeable difference in the shaft diameters in anchorage 4M7, compared to those measured in 5M7 and 6M7. In anchorage 4M7, a much greater percentage of the recovered grout body exhibited an increase in diameter (above the original cased diameter) of between 30mm and 80mm (R4), and up to and above 200mm (R5 & R6). In anchorages 5M7 and 6M7, the greater percentage of recovered anchor grout exhibited increases in diameter up to 20mm (R2). However, the recovered length of anchorage 4M7 was from the *fixed* anchor length where intense grouting had taken place under pressure. In anchorages 5M7 and 6M7, the recovered lengths were both from the tendon *free* length where grouting under gravity with no additional pressure grouting had taken place. Since increased shaft diameter is indicative of increase in capacity, these results support the benefits of pressure grouting of the fixed anchor zone of anchorages installed in chalk.

In a situation where the drilling system with support from the flushing technique can ensure direct exposure of the open fissures prior to and during grouting operations and the grout has adequate penetrability and is subject to applied pressure, then an effective fissure grouting process will take place. It is possible that the fissures may be actually enlarged by the flush system. The grout pressure may also contribute by increasing the fissure width and the borehole diameter. Since the strength of the injected cement grout, which penetrates the fissure is well in excess of that of the chalk mass, increased bond will be derived.

However there are drilling and grouting or concreting systems carried out in chalks that may not offer such benefits -

- auguring, which smears the entrance to chalk fissures that intersect the borehole
- the use of a “grout” (or concrete) which will not adequately penetrate the fissures
- the use of a grouting method that does not ensure some fissure penetration .

In such circumstances there is no or little enhancement from the shear of the grout and the capacity is only available from chalk/grout bond or from smeared chalk to grout bond.

This is consistent with observation of anchor grout bodies exhumed from weak mudstones. Here the grain structure is finer, there are no flints or cavities in the rock matrix and no pressure grouting was applied, Weerasinghe and Littlejohn (1997) observed no obvious irregularities in the fixed anchor surface. The exhumed anchorages were found to exhibit a fine imprint of the rock and diametric variations were generally below 10mm (R2); the exhumed diameters varied between -8% (reduced) and + 11 % of the original shaft diameter.

In chalk anchorages where grout take of fissures and voids may be high there is effectively a balance between the volume of grout which penetrates the chalk mass (associated with the enhancement of the anchor capacity), and the volume of grout which travels away from the anchor bore and is associated with ground improvement. The balance between these volumes is difficult to achieve. For a contract measurement guideline BS8081:1989 advises that in addition to the grouting of the drilled borehole, a supplementary grout volume of twice the borehole volume may be associated with ground strengthening adjacent to the bore to provide this enhancement in anchor capacity i.e. a total grout take of three times the volume of the fixed length bore. At Castle Mall total grout takes, even utilising stiff sanded mix, were considerably higher.

The Integrity of the Corrosion Protection System in Exhumed Anchor Bodies

The integrity of the corrosion protection applied to any anchor of permanent or semi permanent usage is critical to the success of the performance of the anchor (European Standard EN1537:2000).

At Castle Mall, although the anchors were in principle “temporary” it was considered that there was a possibility of construction interruption such that the anchor support may be required for

a lengthy period. Such occurrences due to change of ownership or financial difficulties have unfortunately been encountered on other deep basement sites. The production anchors were therefore required to be double protected whilst test anchors were required to be single protected. This would allow two strands to be installed in each unit SBMA encapsulations in lieu of one, resulting in the testing to a much higher load, particularly appropriate in trial anchor works.

Free length protection consisted of a greased strand surrounded by a 1mm thick close fitting plastic sheath, then contained within a loose fitting plastic sheath (often considered to be sacrificial). Damage to this loose fitting sheath was apparent but generally attributable to the excavator works during exhumation and anchor body withdrawal. There was however no evidence of the perforation or splitting of the tight fitting sheath around the individual strands. The bond lengths of the unit anchors were contained and grouted within a 50mm diameter corrugated plastic duct of the form generally used in the anchor industry. Close inspection revealed that where ducts were contained within the grout bodies, undamaged by exhumation, the ducts were fully intact with no cracks to allow entry of corrosive elements (Plate 16).

THE CONSTRUCTED ANCHOR BODY AND SHORT LENGTH LOAD TRANSFER MECHANISM

The photographs of the exhumed anchorages highlight the distribution of grout, despatched under pressure, from the anchor bore into the chalk mass to form a black irregular “skeleton” of grout within the chalk mass itself; this grout being many times greater in strength than that of the chalk material.

Over a one metre length of bore it is reasonable to assume that the elasticity of the anchor tendon and the progressive debonding mechanism does not influence pull out, i.e. the tendon and grout body move as one unit within the chalk mass. Thus, for this one metre long cylindrical body to pull-out from the chalk mass the following must occur:

1. failure of bond over the area of the grout/chalk interface approximately to the original bore **and**
2. failure of the grout in shear at the intersection of the grouted fissures within the failure cylinder **or**
3. failure at chalk/chalk and/or grout/chalk interface along a larger diameter irregular cylindrical shape than the original bore

Failure mechanisms (1) and (2) along a cylindrical failure plane are presented in Figure 5 (a). The surface area of the cylindrical failure plane has been projected in order to illustrate the area

of the grout/chalk interface and that of grout/grout shear (Figure 5b). A 5% area of grouted fissures is estimated from the exhumation of the grouted bodies (plates 12 & 13, anchor 4M7).

It is then possible to estimate the pull-out capacity (T_m) of the one metre length of borehole:-

$$T_m = A_{gg} \times \tau_{gg} + A_{gc} \times \tau_{gc} \quad (1)$$

Where

- τ_{gg} = shear capacity of confined grout
- A_{gg} = Area of grout/grout contact
- τ_{gc} = bond capacity of grout/chalk
- A_{gc} = Area of grout/chalk contact

Barley (1978), while researching the capacity of underreamed anchors in weak rocks, investigated the shear capacity of cement grout, over a range of shear lengths, along an induced cylindrical failure plane. After further work, Barley (1997)² concluded that in a confined state shear stresses at failure ranged from 13 to 20 N/mm² which compared favourably with a single test result of 14 N/mm² established by Raison (1987).

In order to assess the bond stress available at a grout/chalk interface where the benefit of fissure grouting is omitted but the potential benefit from the presence of flints is acknowledged, it is appropriate to consider bond capacity exhibited by bored piles. Hobbs and Heally (1979) researched published data and indicated a general range of ultimate bond stress of 100 to 300 kN/m². Considering a mid-range grout/chalk bond stress of 200 kN/m² and a grout/chalk shear area totalling 95% of the available area on a 120mm diameter borehole, then the grout/chalk bond capacity of a one metre length can be calculated;

$$P_{gc} = 0.95 \times \pi \times 0.12 \times 1.0 \times 200 = 72\text{kN} \quad (2)$$

Considering a grouted fissure area totalling 5% of the available area and utilising a lower bound value of grout shear stress of 13000 kN/m² then the bond capacity of the grouted fissures over a one metre length would equate to

$$P_{gc} = 0.05 \times \pi \times 0.12 \times 1.0 \times 13000 = 245\text{kN} \quad (3)$$

This would indicate that the enhancement resulting from fissure grouting with a 5% contact area may be as much as four times the capacity achieved without fissure grouting. Reference to Figure 6 which allows comparison of the bond capacity achieved in piling with that achieved in pressure grouted anchor supports this statement. If these two contributory bond components

are then considered together, the overall bond stress, averaged over a metre length at failure would be

$$\tau = \frac{72 + 245}{\pi \times 0.12 \times 1.0} = 840 \text{ kN/m}^2 \quad (4)$$

This design approach, evolved from Castle Mall exhumation, is supported by the results achieved over short fixed length tests. Several pull out tests on 2m fixed lengths exhibited bond stress values of 849 kN/m², some without failure (Table 2).

However there is the potential alternative failure mechanism 3 (Figure 5a) in which pull-out of a large diameter cylinder containing grouted fissures and voids could occur. When a failure bond stress of 840 kN/m² is mobilised along the 120mm diameter grouted bore then a bond stress of 200 kN/m² is being mobilised at the grout/chalk interface on a 500mm diameter cylinder. Exhumation of pressure grouted fixed anchors did suggest that such overall diameters of grout and grouted bulbs did exist.

PROGRESSIVE DEBONDING, EFFICIENCY FACTORS AND ANCHOR DESIGN

When considering long fixed lengths it is fully acknowledged by numerous researchers who have investigated the load transfer from anchor tendon to the ground via grout, that the distribution of stress along fixed lengths greater than one or two metres is non-uniform. This results from the general incompatibility between the elastic modulus of the anchor tendon, of the anchor grout and of the grouted ground. Typically, an anchor tendon with a 6m fixed length will at proof load need to extend some 15 to 20mm at the proximal end of the fixed length before any load will be transferred by the tendon to the distal end.

Thus, in the vast majority of anchors, when applying the initial load, the bond stress is concentrated over the proximal length of the fixed anchor and at that time the distal component of the fixed anchor is unstressed and redundant. As load is increased in the anchor, the ultimate bond stress at either (or both) the tendon grout and grout ground interface (chalk) is exceeded. After interfacial movement the residual bond stress at that location is generally of a lower order. For example Weerasinghe and Littlejohn (1997) report ultimate bond stresses in mudstone of 520kN/m² and residual bond stresses of 425kN/m².

As load in the overall anchor is further increased the bond stress concentration zone progresses further along the fixed anchor. Prior to failure of the anchor the load concentration zone approaches the distal end of the anchor. Figure 7a typifies the distribution of bond stress along a normal fixed anchor during initial loading and when approaching failure, albeit the relationship between the ultimate bond stress and the residual bond stress will vary with ground conditions and grouting techniques.

Barley (1990) demonstrated qualitatively how anchorages in chalk (and other soils) much potential bond capacity was lost before failure as a consequence of progressive load transfer.

In 1995 Barley compared the ultimate tensile capacity of anchorages of the same diameter but with different fixed lengths; generally in stiff to very stiff clayey soils. Based on this data and with consideration of the Norwich chalk test anchorage which also investigated anchor capacities with different fixed lengths Barley introduced an efficiency factor, itself a function of the fixed anchor length L , to take account of the occurrence of progressive debonding such that the ultimate anchor load T_{ult} is given by:

$$T_{ult} = \tau_{avg} \times \pi \times D \times L$$

$$\text{and } \tau_{avg} = f_{eff} \times \tau_{ult}$$

T_{ult} = Ultimate load (kN)

D = borehole diameter (m)

L = fixed length (m)

τ_{ult} = ultimate bond stress over a short fixed length (kN/m²)

τ_{avg} = average value of ultimate bond stress over a long length (kN/m²)

By normalising the results from anchors of different lengths in clayey soils Barley (1995) empirically obtained:

$$f_{eff} = 1.6 L^{-0.57}$$

From Figure 8 it may be seen that an anchor 2.3m long would have an efficiency factor of 100%. If such an anchor developed an ultimate bond stress τ_{ult} , then for a fixed anchor of length L

$$(\tau_{avg})_L = 1.6 L^{-0.57} \times \tau_{ult}$$

If an ultimate average bond stress of $(\tau_{avg})_l$ was measured on a trial anchor of fixed length l it can be shown that for a longer contract anchor of length L the ultimate average bond stress is given by:

$$(\tau_{avg})_l = (l/L)^{0.57} (\tau_{avg})_L$$

Thus an 8m long fixed anchor would be expected to develop an ultimate average bond stress which was 67% of that for a 4m long fixed anchor.

Barley and Windsor (2000) reported on the investigation of the validity of the use of the value of the efficiency factor in a wide range of soils. They concluded that the curve defined by $1.6L^{-0.57}$ when imposed on similar graphical plots (defining reduction in bond stress with increase in fixed length) closely reflected the boundaries selected by Ostermayer (1974) in his extensive studies. This applied to anchors founded in stiff to very stiff clays, postgrouted or non-postgrouted and in sands and sandy gravels. Anchors founded in chalk which achieve grout penetration of open discontinuities may compare closely in finished form with post grouted anchors in clay or with pressure grouted anchors in gravels. On that basis Barley and Windsor suggested that provided the design formulae for fixed anchors in soils and weak rocks always incorporate an efficiency factor, test anchors 2.5 to 5m in length may easily be taken to failure to establish the average bond stress of that length, and then the fixed length of production anchorages may be accurately designed to provide the required factor of safety using the relationship between $(\tau_{avg})_l$ and $(\tau_{avg})_L$ given above.

In the modern application of this principle to Castle Mall Test Anchors, a 2m fixed anchor with an ultimate bond capacity of 840 kN/m² would provide a production anchor design for a normal 8m fixed length with an average bond capacity at failure load of 411 kN/m². Employing a factor of safety of 2.5 the maximum working load would be 496 kN.

THE SINGLE BORE MULTIPLE ANCHOR

It was realised in the late 1970's but not developed until the late 1980's that the combination of a number of "unit" anchors, each with a short fixed anchor length, could be founded at staggered depths in a single borehole and provide an exceptionally high capacity multiple anchor (Barley et al 1992, Barley 1995). It was this development and research work that allowed evolution of the efficiency factor concept (Figure 8) and a design formula that accommodated the progressive debonding phenomenon.

The method of calculation of capacity of short fixed anchors is summarised above. The ultimate capacity of a multiple of short "unit" anchors, the lengths of which themselves may vary (2.5 to 4m generally) is the summation of the capacity of each unit

i.e. SBMA capacity = $\sum T_{ult}$ (of a multiple of units)

$$= \pi \times D \times L_1 \times f_{eff} \times \tau_{ult} + \pi \times D \times L_2 \times f_{eff} \times \tau_{ult} + \dots + \pi \times D \times L_n \times f_{eff} \times \tau_{ult}$$

The comparison of capacity of an SBMA with a conventional anchor is best illustrated in Figure 7b: the shaded area under the lines equating to the anchor capacity. Hence the efficiency of ground strength mobilisation of a multiple of short anchors, say 4 x 2.5m units, is in the order of twice that of a normal 10m long fixed anchor. Furthermore for each additional unit anchor added beyond 10m (generally up to 7 in total) the SBMA capacity increases proportionally. Hence with efficient multiple fixed lengths up to 25m, capacities of up to 3 times that of normal anchors may be achieved. The system is particularly suited to soils and weak rocks such as chalk. The 800 chalk anchors installed at the Castle Mall site utilised this system and encountered no failures. Based on bond capacities previously demonstrated in grades 2 to 3 Chalk and application of the multiple anchor design principles, chalk anchor working capacities of up to 3000 kN have recently been proposed.

CONCLUSIONS

1. The installation, testing and subsequent exhumation of trial anchors, founded in fissured chalk using end of casing pressure grouting techniques, has allowed considerable advancement in the understanding of load transfer in chalk anchors. Gravity pregrouting of chalk cavities and wide fissures using a sanded mix stemmed the unnecessary flow of grout away from the bore. Subsequent pressure grouting with a neat grout resulted in the local grout penetration of fine but irregular fissures which were clearly observed on removal of the grouted anchor bodies from the chalk mass.
2. Pressure application on the neat grout also resulted in the forced filtration of water from grout into the chalk matrix leaving a dark dense annulus of grout up to 20 mm thick around the grouted bore. Such observations are comparable to the characteristics identified in exhumed injection anchors founded in fine granular material.
3. After removal of the chalk matrix from within the skeletal formation of grouted fissures two potential modes of load transfer from the grout body to the chalk are evident. The necessary shearing of the grout in the fissures prior to pull-out failure would provide a shear resistance of an order of magnitude greater than the grout/chalk bond resistance. Such fissure penetration is considered to be the major contributory factor in chalk injection anchors which allows mobilisation of average bond stresses of four times that obtained in piling practice.
4. Based on an assumed grouted fissure profile adjacent to the anchor body (from exhumation observations), and knowing from previous research the direct shear capacity of grout, it is possible to assess the pull-out capacity per metre length of grouted bore for design purposes.
5. Measurements of exhumed shaft diameters revealed that in the fixed anchor length where pressure grouting was used during construction, over 85% of the recovered length exhibited an increased diameter ranging from 30mm to in excess of 200mm (i.e. expanding from 120 mm bore to a 180 to 520 mm anchor body). In the free length, where the anchorage was grouted under gravity during construction, between 74% and 100% of the recovered length exhibited an increased diameter of less than 30mm. Since increase in diameter is likely to result in an increase in capacity, it is concluded that pressure grouting in the fixed anchor length is grossly beneficial to the enhancement of capacity of anchorages in chalk.

6. Acknowledging that the anchor capacity in soils and weak rocks is not proportional to the fixed anchor length, it is possible, using the recently developed “efficiency” factor theory, to estimate the capacity of normal long fixed length anchor, or of the single bore multiple anchor in which unit anchors utilise a multiple of short efficient fixed lengths. Using the latter techniques some ten years of research and practice has demonstrated ultimate anchor capacities of up to 5000kN. Working capacities of 2000 kN have been satisfactorily achieved, complete with proof loading. Working capacities of 3000 kN are currently proposed for multiple anchors founded in chalk.
7. Inspection of the tendons and encapsulation units over the fixed anchor length revealed that the corrosion protection system in the individual unit anchors had remained fully intact during anchor installation and during proof load testing to 2.4 times working load.

Information from exhumation and anchor performance has provided a much improved understanding of the benefits of appropriate methods of flushing and grouting, and of the ensuing load transfer mechanisms, within the anchor body.

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