Professional Papers

DESIGN ESTIMATION OF THE ULTIMATE LOAD-HOLDING CAPACITY OF GROUND ANCHORS*

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Following a brief description of the four major types of cement grout injection anchor used in current practice, empirical design methods for the estimation of the ultimate pull-out capacity of the grouted fixed anchor zone are presented.

The design rules which have been created solely through systematic full scale testing and from general field experience are discussed in relation to rocks, cohesionless soils and cohesive soils.

Topics for further investigation are highlighted such as load transfer mechanisms, grout pressure limits, fixed anchor load/displacement relationships and serviceability safety factors.

The importance of construction technique and quality of workmanship are emphasised since they influence pull-out capacity and limit the designer's ability to make accurate predictions.

Introduction

CALCULATIONS ARE ESSENTIAL in designing ground anchors in order to judge in advance the technical and economic feasibility of a proposed anchorage solution. In retaining wall tie-backs, for example, anchor dimensions can be varied in the calculations to optimise such factors as anchor load and spacing in relation to wall design and cost considerations. Design rules also permit assessment of the sensitivity of the load-holding capacity to variations in anchor dimensions and ground properties, the results of which may dictate working loads, choice of safety factors, and possibly the extent and intensity of a supplementary site investigation.

The purpose of this Paper is to describe current design procedures for cement grout injection anchors, with particular reference to estimation of the ultimate resistance to withdrawal of the grouted fixed anchor zone (Fig. 1). Bearing in mind the wide variety of theoretical and empirical equations which have been proposed to date, the text concentrates on design rules created through field experience and systematic full-scale testing.

Design rule predictions of ultimate load-holding capacity are invariably created by assuming that the ground has failed along slip lines (shear planes), postulating a failure mechanism and then examining the relevant forces in a stability analysis. Using simple practical terms there are basically two load transfer mechanisms by which ground restraint is mobilised locally as the fixed anchor is withdrawn, namely end-bearing and side shear.

Anchors fail in local shear via one of these mechanisms or by a combination of both, provided that sufficient constraint is available from the surrounding ground. In this context general failure is defined as the full mobilisation of slip lines or the generation of significant deformations, extending to ground surface. Field experience indicates that general failure does not occur for slenderness ratios§ in excess of 15, and for the small diameters

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§ Slenderness ratio = depth to top of fixed anchor/effective diameter of fixed anchor.
involved, the top of the fixed anchor is usually founded at depths in excess of 5m. In such circumstances the ultimate load-holding capacity of the anchor ($T_f$) is dependent on the following factors, although due to lack of knowledge item 5 is not generally isolated in design calculations:

1. Definition of failure,
2. Mechanism of failure,
3. Area of failure interface,
4. Soil properties mobilised at the failure interface, and
5. Stress conditions acting on the failure interface at the moment of failure.

It should be emphasised that the design rules described herein for rocks and soils apply to individual anchors and no allowance is made for group effects or interference. Accordingly, it is assumed that the fixed anchor spacing is not less than four times the effective diameter ($D$), which usually means a spacing of not less than 1.5-2m. It is also noteworthy that field testing has been carried out on fixed anchor lengths ($L$) ranging from about 1 to 16m in order to create and check the design rules, but in current commercial practice a minimum fixed anchor length of 3m is considered prudent.

**Anchor types**

Anchor pull-out capacity for a given ground condition is dictated by anchor geometry but the transfer of stresses from the fixed anchor to the surrounding ground is also influenced by construction technique, particularly the grouting procedure, and to a lesser extent drilling technique where choice and method of

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**Fig. 1. Ground anchor nomenclature**
Fig. 2. Main types of cement grout injection anchor

Fig. 3. Detail of tube à manchette for pressure grouting control
flush are important. Accordingly, the types of anchor to which the design rules are applicable are now described. The four types are illustrated in Fig. 2. These comprise:

Type A: Tremie-grouted straight shaft borehole, which may be lined or unlined depending on hole stability. This type is most commonly employed in rock, and very stiff to hard cohesive deposits. Resistance to withdrawal is dependent on side shear at the ground/grout interface.

Type B: Low-pressure grouted borehole via a lining tube or in situ packer, where the effective diameter of the fixed anchor is increased with minimal disturbance as the grout permeates through the pores or natural fractures of the ground. Low pressure normally implies injection at pressures not exceeding total overburden pressure. This type of anchor is most commonly employed in soft fissured rocks and coarse alluvium, but the method is also popular in fine grained cohesionless soils. Here the cement particles cannot permeate the small pores but under pressure the grout compacts the soil locally to increase the effective diameter. Resistance to withdrawal is dependent primarily on side shear in practice, but an end-bearing component may be included when calculating the pull-out capacity.

Type C: High-pressure grouted borehole via a lining tube or in situ packer, where the grouted fixed anchor is enlarged via hydrofracturing of the ground mass to give a grout root or fissure system beyond the core diameter of the borehole. Where stage grouting along the fixed anchor or regrouting are envisaged a tube-a-manchette system can be incorporated as shown in Fig. 3. This anchor type is employed primarily in cohesionless soils although some success has also been achieved in stiff cohesive deposits. Design is based on the assumption of uniform shear along the fixed anchor.

Type D: Tremie-grouted borehole in which a series of enlargements (bells or under-reams) have previously been formed mechanically. This type is employed most commonly in stiff to hard cohesive deposits. Resistance to withdrawal is dependent primarily on side shear with an end-bearing component, although for single or widely spaced under-reams the ground restraint may be mobilised primarily by end-bearing.

Rock
The earliest reports of anchoring bars into rock to secure a roof date from 1918 in the Mir Mine of Upper Silesia in Poland, and by 1926 faces of an inclined shaft, in Chustenice shales in Czechoslovakia, were secured against caving by grouted bars installed in a fan pattern. In the field of civil engineering the history of rock anchors dates from 1934 when Coyne pioneered their use during the raising of Cheurfas Dam in Algeria. On this project 37 anchors were constructed in sandstone, fixed with the aid of double under-reams, and then tensioned individually to 1,000 tonnes.

Whilst all anchor types A - D are applicable to rock, the straight shaft tremie-grouted Type A is the more popular in current practice on the basis of cost and simplicity of construction. For such anchors designs are based on the assumption of uniform bond distribution. Thus the pull-out capacity is estimated from eqn. 1.

\[ T_f = \pi DL \tau_{ult} \quad (1) \]

where \( \tau_{ult} \) = ultimate bond or skin friction at rock/grout interface.

This approach is used in many countries such as France, Italy, Switzerland, Britain, Australia, Canada and USA, although it is just as common to use \( \tau_{working} \) in place of \( \tau_{ult} \) where a safety factor has been incorporated.

Eqn. 1 is based on the following simple assumptions:
(i) Transfer of the load from the fixed anchor to the rock occurs by a uniformly distributed stress acting over the whole of the perimeter of the fixed anchor,
(ii) The diameter of the borehole and the fixed anchor are identical,
(iii) Failure takes place by sliding at the rock/grout interface (smooth borehole) or by shearing adjacent to the rock/grout interface in weaker medium (rough borehole),
(iv) There are no discontinuities or inherent weakness planes along which failure can be induced, and
(v) There is no local debonding at the grout/rock interface.

Where shear strength tests are carried out on representative samples of the rock mass, the maximum average working
**TABLE I. ROCK/GROUT BOND VALUES WHICH HAVE BEEN RECOMMENDED FOR DESIGN**

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Working bond</th>
<th>Ultimate bond</th>
<th>Factor of safety</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Igneous</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Medium hard basalt</td>
<td>5.73</td>
<td>1.50-2.50</td>
<td>3-4</td>
<td>India—Rao (1964)</td>
</tr>
<tr>
<td>Weathered granite</td>
<td>1.21-1.38</td>
<td>3.86</td>
<td>2.8-3.2</td>
<td>Japan—Suzuki et al (1972)</td>
</tr>
<tr>
<td>Serpentine</td>
<td>0.45-0.59</td>
<td>1.55</td>
<td>2.6-3.5</td>
<td>Britain—Wycliffe-Jones (1974)</td>
</tr>
<tr>
<td>Granite &amp; basalt</td>
<td></td>
<td></td>
<td></td>
<td>USA—PCI (1974)</td>
</tr>
<tr>
<td><strong>Metamorphic</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Manhattan schist</td>
<td>0.70</td>
<td>2.80</td>
<td>4.0</td>
<td>USA—White (1973)</td>
</tr>
<tr>
<td>Slate &amp; hard shale</td>
<td>0.83-1.38</td>
<td>1.5-2.5</td>
<td>USA—PCI (1974)</td>
<td></td>
</tr>
<tr>
<td><strong>Carbonates</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Limestone</td>
<td>1.00</td>
<td>2.83</td>
<td>2.8</td>
<td>Switzerland—Losinger (1966)</td>
</tr>
<tr>
<td>Chalk—Grades I-III</td>
<td>0.005N</td>
<td>0.22-1.07</td>
<td>2.0</td>
<td>Britain—Littlejohn (1970)</td>
</tr>
<tr>
<td>(N=SPT in blows/0.3m)</td>
<td>0.01N</td>
<td>0.10-0.40</td>
<td>(Temporary)</td>
<td></td>
</tr>
<tr>
<td>Tertiary limestone</td>
<td>0.82-0.97</td>
<td>2.76</td>
<td>2.9-3.3</td>
<td>Britain—Wycliffe-Jones (1974)</td>
</tr>
<tr>
<td>Chalk</td>
<td>0.96-1.00</td>
<td>2.12</td>
<td>2.8-3.2</td>
<td>Britain—Wycliffe-Jones (1974)</td>
</tr>
<tr>
<td>Soft limestone</td>
<td>1.03-1.52</td>
<td>1.5-2.5</td>
<td>USA—PCI (1974)</td>
<td></td>
</tr>
<tr>
<td>Dolomitic limestone</td>
<td>1.38-2.07</td>
<td>1.5-2.5</td>
<td>USA—PCI (1974)</td>
<td></td>
</tr>
<tr>
<td><strong>Arenaceous sediments</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hard coarse-grained sandstone</td>
<td>2.45</td>
<td>1.75</td>
<td>Canada—Coates (1970)</td>
<td></td>
</tr>
<tr>
<td>Weathered sandstone</td>
<td>0.69-0.85</td>
<td>2.0-2.5</td>
<td>New Zealand—Irwin (1971)</td>
<td></td>
</tr>
<tr>
<td>Well-cemented mudstones</td>
<td>0.69</td>
<td>2.0-2.5</td>
<td>New Zealand—Irwin (1971)</td>
<td></td>
</tr>
<tr>
<td>Bunter sandstone</td>
<td>0.60</td>
<td>3.0</td>
<td>Britain—Littlejohn (1973)</td>
<td></td>
</tr>
<tr>
<td>Bunter sandstone (UCS&gt;2N/mm²)</td>
<td>0.69-0.83</td>
<td>2.24</td>
<td>2.7-3.3</td>
<td>Britain—Wycliffe-Jones (1974)</td>
</tr>
<tr>
<td>Hard fine sandstone</td>
<td>0.83-1.73</td>
<td>1.5-2.5</td>
<td>USA—PCI (1974)</td>
<td></td>
</tr>
<tr>
<td>Sandstone</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Argillaceous sediments</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Keuper marl</td>
<td>0.17-0.25</td>
<td>3.0</td>
<td>Britain—Littlejohn (1970)</td>
<td></td>
</tr>
<tr>
<td>(0.45 c_u)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weak shale</td>
<td>0.10-0.14</td>
<td>0.37</td>
<td>2.7-3.7</td>
<td>Canada—Golder Hrawner (1973)</td>
</tr>
<tr>
<td>Soft sandstone &amp; shale</td>
<td>0.21-0.83</td>
<td>1.5-2.5</td>
<td>USA—PCI (1974)</td>
<td></td>
</tr>
<tr>
<td>Soft shale</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>General</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Competent rock (where UCS&gt;2N/mm²)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uniaxial compressive strength—30 (up to a maximum value of 1.4N/mm²)</td>
<td>3</td>
<td></td>
<td>Britain—Littlejohn (1972)</td>
<td></td>
</tr>
<tr>
<td>Uniaxial compressive strength—10 (up to a maximum value of 4.2N/mm²)</td>
<td>3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weak rock</td>
<td>0.35-0.70</td>
<td></td>
<td></td>
<td>Australia—Koch (1972)</td>
</tr>
<tr>
<td>Medium rock</td>
<td>0.70-1.05</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strong rock</td>
<td>1.05-1.40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Wide variety of igneous and metamorphic rocks</strong></td>
<td>1.05</td>
<td>2</td>
<td>Australia—Standard CA35 (1973)</td>
<td></td>
</tr>
<tr>
<td><strong>Wide variety of rocks</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>France—Fargeot (1972)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Switzerland—Walther (1959)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Switzerland—Comte (1965)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Switzerland—Comte (1971)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Italy—Mascardi (1973)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.89</td>
<td>2.76</td>
<td>4</td>
<td>Canada—Golder Hrawner (1973)</td>
<td></td>
</tr>
<tr>
<td>0.69</td>
<td>4.2</td>
<td>3</td>
<td>USA—White (1973)</td>
<td></td>
</tr>
<tr>
<td>1.4</td>
<td>15-20 per cent of grout crushing strength</td>
<td>3</td>
<td>Australia—Longworth (1971)</td>
<td></td>
</tr>
<tr>
<td>Concrete</td>
<td>1.38-2.78</td>
<td>1.5-2.5</td>
<td>USA—PCI (1974)</td>
<td></td>
</tr>
</tbody>
</table>
core recovery) up to a maximum value $\tau_{ult}$ of 4.2N/mm$^2$. As confirmation $\tau_{ult} = 4.3N/mm^2$ is indicated for design in hard coarse grained sandstone by Canadian research.

In some rocks, particularly granular weathered varieties with a relatively low $r_p$ value, the assumption that $\tau_{ult}$ equals $10\%$ UCS may lead to an artificially low estimate of shear strength (Fig. 4). In such cases, the assumption that $\tau_{ult}$ equals 20-35\% UCS may be justified.

Bond values which have been recommended for a wide range of igneous, metamorphic and sedimentary rocks, are presented in Table I. Where included, the factor of safety relates to the ultimate and working bond values, calculated assuming uniform bond distribution. It is common to find that the magnitude of bond is simply assessed by experienced engineers and the value adopted for working bond stress often lies in the range 0.35 - 1.4N/mm$^2$.

The Australian Code states that whilst a value of 1.05N/mm$^2$ has been used in a wide range of igneous and sedimentary rocks, site testing has permitted bond values of up to 2.1N/mm$^2$ to be employed.

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**TABLE II. FIXED ANCHOR LENGTHS FOR CEMENT GROUTED ROCK ANCHORS WHICH HAVE BEEN EMPLOYED OR RECOMMENDED IN PRACTICE**

<table>
<thead>
<tr>
<th>Fixed anchor length (metres)</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Range</td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td>Sweden—Nordin (1966)</td>
</tr>
<tr>
<td></td>
<td>Italy—Berardi (1967)</td>
</tr>
<tr>
<td>3.0</td>
<td>Canada—Hanna &amp; Seaton (1967)</td>
</tr>
<tr>
<td>3.0</td>
<td>Britain—Littlejohn (1972)</td>
</tr>
<tr>
<td>3.0</td>
<td>France—Fenoux et al (1972)</td>
</tr>
<tr>
<td>3.0</td>
<td>Italy—Conti (1972)</td>
</tr>
<tr>
<td>4.0</td>
<td>South Africa—Code of Practice (1972)</td>
</tr>
<tr>
<td>6.0</td>
<td>South Africa—Code of Practice (1972)</td>
</tr>
<tr>
<td>5.0</td>
<td>France—Bureau Securitas (1972)</td>
</tr>
<tr>
<td>5.0</td>
<td>USA—White (1973)</td>
</tr>
<tr>
<td>3.0</td>
<td>Germany—Stocker (1973)</td>
</tr>
<tr>
<td>3.0</td>
<td>Italy—Mascardi (1973)</td>
</tr>
<tr>
<td>3.0</td>
<td>Britain—Universal Anchorage Co. Ltd. (1972)</td>
</tr>
<tr>
<td>3.0</td>
<td>Britain—Ground Anchors Ltd. (1974)</td>
</tr>
<tr>
<td>3.5</td>
<td>Britain—Associated Tunnelling Co. Ltd. (1973)</td>
</tr>
<tr>
<td>(very hard rock)</td>
<td></td>
</tr>
<tr>
<td>(soft rock)</td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td></td>
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<tr>
<td>3.0</td>
<td></td>
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<td>3.0</td>
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<td>3.0</td>
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<td>3.0</td>
<td></td>
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<tr>
<td>3.0</td>
<td></td>
</tr>
<tr>
<td>3.5</td>
<td></td>
</tr>
<tr>
<td>(chalk)</td>
<td></td>
</tr>
</tbody>
</table>
In this connection the draft Czech Standard\(^8\) concludes that since the estimation of bond magnitude and distribution is a complex problem, field anchor tests should always be conducted to confirm bond values in design, as there is no efficient or reliable alternative. Certainly, a common procedure amongst anchor designers is to arrive at estimates of permissible working bond values by factoring the value of the average ultimate bond calculated from test anchors.

In general, there is a scarcity of empirical design rules for the various categories of rocks, and as shown in Table I too often bond values are quoted without provision of strength data, or a proper classification of the rock and cement grout.

The degree of weathering of the rock is a major factor which affects not only the ultimate bond but also the load-deflection characteristics. Degree of weathering is seldom quantified but for design in soft or weathered rocks there are signs that the standard penetration test is being further exploited. For example, in weathered granite in Japan the magnitude of the ultimate bond has been determined\(^9\) from eqn. 2.

\[
\tau_{\text{ult}} = 0.007N + 0.12 \text{ (N/mm}\^2\text{)} \quad \text{... (2)}
\]

where \(N\) = number of blows per 0.3m

Similarly, eqn. 3 has been established for stiff/hard chalk\(^10\)

\[
\tau_{\text{ult}} = 0.01N \text{ (N/mm}\^2\text{)} \quad \text{... (3)}
\]

**Fixed anchor length**

The recommendations made by various engineers with respect to length of fixed anchor\(^5\) are presented in Table II. Under certain conditions it is recognised that much shorter lengths would suffice, even after the application of a generous factor of safety. However, for a very short anchor the effect of any sudden drop in rock quality along the fixed anchor zone, and/or constructional errors or inefficiencies could induce a serious decrease in that anchor's capacity. As a result a minimum length of 3m is often specified.

In Italy, much valuable experimental research\(^11\) has been conducted into the distribution of stresses both along the fixed anchor and into the rock. From this work it is concluded that the active portion of the anchor is independent of the total fixed anchor length, but dependent on its diameter and the mechanical properties of the surrounding rock, especially its modulus of elasticity.

Fig. 5 shows typical diagrams\(^11\) which illustrate the uneven bond distribution as calculated from strain gauge data. Both anchors were installed in 120mm diameter boreholes in marly limestone \((E = 3 \times 10^4 \text{kN/m}^2; \ \text{UCS} = 100 \text{N/mm}^2 \text{ approx.})\). Other results show that the bond distri-

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Fig. 5. Distribution of bond along fixed anchor length
butions are more uniform for high values of $E_{\text{grout}}/E_{\text{rock}}$, non-uniform for low values of this ratio, i.e. for rock of high elastic modulus. These findings have also been predicted by Canadian researchers (Fig. 6).

**Remarks**

It may be concluded that the distribution of the bond mobilised at the rock/grout interface is unlikely to be uniform unless the rock is "soft". It appears that non-uniformity applies to most rocks where $E_{\text{grout}}/E_{\text{rock}}$ is less than 10.

It is realised that the determination of the modulus of elasticity is rather involved and expensive, particularly for rock masses. However, as the influence of this parameter on anchor performance has already been demonstrated, efforts should be made whenever possible to obtain a realistic value in order to advance our understanding.

Although it would appear from evidence presented that the assumptions made in relation to uniform bond distribution are not strictly accurate, it is noteworthy that few failures are encountered at the rock/grout interface and new designs are often based on the successful completion of former projects; that is, former "working" bond values are re-employed or slightly modified depending on the judgement of the designer.

**Cohesionless soils**

It was in Germany in 1958 that Bauer for the first time demonstrated that a bar could be anchored into gravels through a 150mm diameter borehole with the aid of cement grout injection under pressure. Since then the development of grouted anchors in frictional soils has steadily gained momentum, particularly in Europe, the Americas and South Africa.

For low pressure grouted anchors of Type B the ultimate load holding capacity $T_f$ is most simply estimated from eqn. 4.

$$T_f = L n \tan \phi$$ 

where $L = \text{fixed anchor length (m)}$

$\phi = \text{angle for internal friction}$

$n = \text{factor which apparently takes account of the drilling technique (rotary-percussive with water flush), depth of overburden and fixed anchor diameter, grouting pressure in the range 30 - 1000kN/m}^2, \text{insula stress field and dilation characteristics.}$

Field experience indicates that for coarse sands and gravels ($k_v > 10^{-4} \text{m/sec}$), $n$ ranges from 400 - 600kN/m, whilst in fine to medium sands ($k_v = 10^{-4}$ to $10^{-6} \text{m/sec}$) $n$ reduces to 130-165kN/m.

Eqn. 4 is simple but crude and is used mainly by specialist contractors familiar with their own particular anchorage sys-
The rule tends to be conservative in view of the limited use of information concerning anchor dimensions and ground parameters, and the underestimate can be significant if the rule is applied to dense "over-consolidated" alluvium where the $n$ values were initially established in "normally consolidated" materials. In this regard the over-consolidation ratio (OCR) should be quantified in ground investigation reports, to permit more field studies into the effect of OCR and relative density on pull-out capacity. For more general use eqn. 5 is recommended since it relates anchor pull-out capacity to anchor dimensions and soil properties.

\[
T_f = A \sigma'_v \pi D L \tan \phi + B \gamma h \left( D^2 - d^2 \right) \frac{\pi}{4} \quad \text{(side-shear) + (end-bearing)} \quad \cdots (5)
\]

where

- $A =$ ratio of contact pressure at the fixed anchor/soil interface to the average effective overburden pressure,
- $\gamma =$ unit weight of soil overburden (submerged unit weight beneath the water table),
- $h =$ depth of overburden to top of fixed anchor,
- $L =$ length of fixed anchor,
- $\sigma'_v =$ average effective overburden pressure adjacent to the fixed anchor (equivalent to $\gamma (h+L/2)$ for a vertical anchor in ref. 10),
- $D =$ effective diameter of fixed anchor,
- $\phi =$ angle of internal friction,
- $B =$ bearing capacity factor, and
- $d =$ effective diameter of grout shaft above fixed anchor.

In practice the fixed anchor diameter ($D$) is rarely assessable with any accuracy, but approximate estimates can be made from grout takes in conjunction with ground porosity. For boreholes of 100 to 150mm, $D$ values of 400 - 500mm can be attained in coarse sands and gravels, say $3 - 4d$. Where grout permeation is not possible and only local compaction is achieved, $D$ values for the above borehole diameters and an applied pressure up to 1000kN/m$^2$, may range from 200 - 250mm for medium dense sand\textsuperscript{10}, say 1.5 - 2$d$. For very dense sand $D$ values of 180 - 200mm have been attained\textsuperscript{14}, say 1.2 - 1.5$d$.

The value of $B$ depends on the angle of shearing resistance of the soil adjacent to the top of the fixed anchor, and slenderness ratio ($h/D$). Based on Russian research\textsuperscript{15}, the relationship between the conventional bearing capacity factor ($N_q$) and $\phi$ is shown in Fig. 7 for slender piles. Up to a value of 15, $h/D$ can influence $N_q$ significantly, but for increasing slenderness ratios the effect becomes progressively less significant (Table III). A complimentary study\textsuperscript{16} has also indicated that $N_q/B$ equals 1.3-1.4, and this combined information is used in current practice to estimate $B$. For compact sandy gravel ($\phi = 40^\circ$) at Vauxhall Bridge, London, and compact dune sand ($\phi = 35^\circ$) at Ardeer, Scotland, values of $B$ equal to 101 and 31 have been measured in the field\textsuperscript{10}, which are in good agreement with respective values of (99-106) and (35-38) estimated via Fig. 7.

The value of $A$ depends to a large extent on construction technique and for the Type B anchor relevant to eqn. 4, values of 1.7 and 1.4 have been recorded in compact sandy gravel ($\phi = 40^\circ$) and compact dune sand ($\phi = 35^\circ$) respectively\textsuperscript{10}.

The end-bearing component of eqn. 5 is occasionally omitted by anchor specialists, perhaps on the basis that anchor yield can be recognised at relatively small fixed anchor displacements, which do not permit full mobilisation of the end-bearing resistance. In this regard eqn. 6 has been produced in British Columbia\textsuperscript{17} for grouted bar anchors installed in medium to dense sandy gravel with some cobbles ($\phi = 35^\circ$-$42^\circ$)

\[
T_f = K' \pi D L \sigma'_v \tan \phi \quad \cdots (6)
\]

where $K'$, coefficient of earth pressure, varies from 1.4 to 2.3 with no grout injection pressure.

For fine sands and silts recommended values for $K'$ are 1.0 and 0.5 for high and low relative densities, respectively\textsuperscript{18}, although it is recognised that $K'$ is probably dependent on injection pressure\textsuperscript{10}. For dense sands in Boston, Massachusetts\textsuperscript{20}, $K' = 1.4$ has been obtained for the Bauer anchor. Bearing in mind the difficulty in assessing the effective fixed anchor diameter ($D$), eqn. 7 using the shaft or borehole diameter ($d$) has been sug-
TABLE III. APPROXIMATE RELATIONSHIP BETWEEN $N_q$ AND SLENDERNESS RATIO

<table>
<thead>
<tr>
<th>$h$</th>
<th>$\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>26°</td>
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<tr>
<td>15</td>
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<tr>
<td>20</td>
<td>9</td>
</tr>
<tr>
<td>25</td>
<td>8</td>
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</table>

This rule has been tested recently for injection pressures of 1000-2000kN/m² in dense fine uniform sand ($\phi = 40°$) at Kükük Çekmece Lake in Turkey. In such soil the rule is shown to overestimate pull-out capacity and a modified version (eqn. 9) is recommended.

$$T_f = \frac{2}{3} P_i \pi D L \tan \phi \quad \ldots (9)$$

The overestimate of eqn. 8 has been further highlighted for the very dense shelly sand at Orford Ness, England and injection pressures of 1000 to 1400 kN/m², where the residual pressure approximated to $1/3 P_i$. It is considered that as the in situ permeability of the soil increases, filter cake formation becomes more difficult and hence more of the injection pressure is dissipated during the plastic stiffening stage, as the grout slowly permeates through the soil. In this regard the stiffening time and shrinkage of the grout, together with the load/deformation properties of the soil may also be influential. In spite of this apparent restriction design curves, based on the work of Jorge, have been published relating grouting pressure directly to ultimate load capacity per metre of fixed anchor for major classes of ground (Fig. 8). These curves are used primarily for Type C anchors where the injection pressures usually exceed 1000kN/m².

It is a feature of Type C anchors that calculations are based on design curves created from field experience in a range of soils rather than relying on a theoretical or empirical equation using the mechanical properties of a particular soil. In alluvium for example, test results in medium sand in Brussels, alluvium at Marcoule, sands and gravels at St-Jean-de-Luz, and Seine alluvium at Bercy have indicated for 100-150mm diameter boreholes ultimate load-holding capacities of 90-130kN/m of fixed anchor at $P_i$ of 1000kN/m², and 190-240kN/m at $P_i$ of 2500kN/m².

In more recent years design curves for Type C anchors have been extended through basic tests in Germany and for sandy gravels and gravelly sands. Fig. 9 shows how the ultimate load increases with density and coefficient of uniformity. Compared with these two soil properties, increases in grouting pressure over the range 500-5000kN/m², and fixed anchor diameter (100-150mm) are found...
to have little influence on pull-out capacity which contrasts with the French observations. In this regard the particular use of the tube-à-manchette system in the French tests to provide a secondary stage of grouting at high pressure may explain the different emphasis on injection pressure.

For the German design curves average skin frictions can be as high as 500\(\text{kN/m}^2\) for sand, and 1000\(\text{kN/m}^2\) for sandy gravel. Since these skin frictions are much higher than would normally be predicted by conventional soil mechanics theory, the values attained in ground anchors are explained by an interlocking or wedging effect due to dilation of the soil as the fixed anchor is withdrawn. The effect is an increase in radial or normal stress at the ground/grout interface, and values of 2-10 times the effective overburden pressure have been noted. For very dense fine to coarse gravelly sand at National Capital Bank in Washington DC, \((P_i = 2800-3100\text{kN/m}^2)\), radial stresses of approximately 20\(\times\) the overburden pressure have been deduced.

In practice density is commonly measured indirectly by in situ penetrometer tests, and Fig. 10 illustrates how penetration resistance can be used to provide a rough estimation of ultimate load holding capacity for 3m 6m and 9m fixed anchor lengths. The authors emphasise,
Fig. 9. Ultimate load-holding capacity of anchors in sandy gravel and gravelly sand showing influence of soil type, density and fixed anchor length.

Fig. 10. Relationship between ultimate load-holding capacity, fixed anchor length and dynamic penetration resistance for two types of frictional soil.
However, that certain fluctuations in test results are possible due to the soil inhomogeneity even when anchors have been properly installed. Japanese investigators have also provided a relationship between maximum skin friction and mean N value (Fig. 11).

The most sophisticated attempt to calculate accurately load-holding capacity is provided by an evaluation of test anchors in Hannover, West Germany using statistical methods, specifically a linear multiple regression analysis. For frictional soils eqn. 10 is recommended:

\[ T_f = a_0 + a_1 \pi DL + a_2 D_5 + a_3 D_6 + a_4 D_7 + a_5 D_8 + a_6 k + a_7 \tau \]  

where \( \tau = \frac{2 \sin \phi'}{2} \gamma h_m \tan \phi' \) (kN/m²)

\[ a_j = \text{regression constants}, \]
\[ D = \text{effective fixed anchor diameter (cm)}, \]
\[ L = \text{fixed anchor length (m)}, \]
\[ D_5 = \% \text{soil grains with diameters} \]
\[ < 0.2 \text{mm}, \]
\[ D_6 = \% \text{soil grains} 0.2 \text{mm} < \text{dia.} < 0.6 \text{mm}, \]
\[ D_7 = \% \text{soil grains} 0.6 \text{mm} < \text{dia.} < 2.0 \text{mm}, \]
\[ D_8 = \% \text{soil grains} \text{dia.} > 2.0 \text{mm}, \]
\[ k = \text{coefficient of permeability (cm/sec)}, \]
\[ \gamma = \text{unit weight} \text{ (kN/m}^3) \text{, and} \]
\[ h_m = \text{depth of overburden to mid-point of fixed anchor (m)} \]

The correlation analysis yielded a multiple correlation coefficient of 0.96 and the following values for the constants of eqn. 10:

\[ a_0 = -2679.36 \quad a_1 = +20.63 \]
\[ a_2 = +34.12 \quad a_5 = +31.92 \]
\[ a_3 = +32.90 \quad a_6 = -2051.48 \]
\[ a_4 = +30.94 \quad a_7 = +9.73 \]

Even with this mathematical sophistication however, there is no possibility of taking into account different construction procedures, and this rule applies solely to anchors of Type C.

Using eqn. 10 to estimate the pull-out capacity it must be observed that the grain size curve lies within the boundaries of Fig. 12 and that the values of the influence factors do not exceed the following limits:

\[ 0.98 \text{m}^2 \leq \pi DL \leq 3.61 \text{m}^2 \]
\[ 7.40 \text{cm} \leq D \leq 11.50 \text{cm} \]
\[ 4.10 \text{m} \leq L \leq 15.00 \text{m} \]
\[ 0\% \leq D_5 \leq 86\% \]
\[ 10\% \leq D_7 \leq 78\% \]
\[ 0\% \leq D_8 \leq 17\% \]
\[ 0\% \leq D_6 \leq 77\% \]
\[ 0.122 \text{10}^{-2} \text{cm/s} \leq k \leq 25.2 \text{10}^{-2} \text{cm/s} \]
\[ 31.7 \text{kN/m}^2 \leq \tau \leq 95.6 \text{kN/m}^2 \]

The importance of these limits and boundaries cannot be overemphasised as field experience indicates that use of one parameter outside the stipulated range e.g. \( k \) which may then be incompatible with the grain size, can produce...
### Hydrometer Analysis

<table>
<thead>
<tr>
<th></th>
<th>Fine</th>
<th>Medium</th>
<th>Coarse</th>
</tr>
</thead>
<tbody>
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<td></td>
<td></td>
</tr>
<tr>
<td>Silt</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td></td>
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<td>Gravel</td>
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### Sieve Analysis

<table>
<thead>
<tr>
<th></th>
<th>Fine</th>
<th>Medium</th>
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</thead>
<tbody>
<tr>
<td>Sand</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Gravel</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 12. Boundaries of the grain size distribution of the investigated frictional soil

---

**Anomalous results.**

**Distribution of skin friction**

Designs are normally based on the assumption of an equivalent uniform skin friction; actual field values are rare and even then are estimated from bond stresses at the grout/tendon interface. For the last loading step before failure is reached, Fig. 13 shows for instrumented anchors the distribution of skin friction on fixed anchors ranging from 2 to 4.5m in length.

The decisive influence of soil density is clearly shown by the maximum $\tau_s$ values of 150, 300 and 800kN/m² for loose, medium dense and very dense gravelly sand, respectively. For the 4.5m long anchors in loose and medium dense gravelly sand, skin friction is more or less constant over the ground/grout interface. For dense and very dense sands the maximum values are effective along a relatively short length, and the location of this peak zone shifts distally as the test load increases. These observations for Type C anchors have been confirmed in similar very dense frictional soils in Washington DC, where it was also noted that fixed anchor displacements of only 2-3mm were required to mobilise high values of load transfer (150-370kN/m).

Assuming that the limit value or maximum $\tau_s$ is identical for different fixed anchor lengths, the mean values of $\tau_s$ for long anchors are smaller than for short anchors, a feature which is apparent in Fig. 9. Taken to the extreme, there exists a critical limit to the effective fixed anchor length beyond which there is no evident increase in load-holding capacity. Fig. 14 for dense frictional soil ($N=50$) indicates very small load increases for $L$ greater than 6.7m, which supports Ostermayer who concluded that 6-7m was optional from an economic point of view.

**Remarks**

For pressure-grouted anchors of Type B and C, two distinct design approaches have evolved—namely empirical equations and design envelopes, respectively. Since the main distinction between the two anchor types relates to the magnitude of grout injection pressure, more guidance is required on injection pressure limits which would determine if the ground is to be permeated or hydrofractured.

**Cohesive soils**

For tremie-grouted straight shaft anchors of Type A, the pull-out capacity is most conveniently estimated from eqn. 11,

$$T_f = \pi DL \alpha c_u$$  \hspace{1cm} (11)

where $c_u$ = average undrained shear strength over the fixed anchor length, and

$\alpha$ = adhesion factor.

In stiff London Clay ($c_u > 90kN/m²$) $\alpha$ values of 0.3-0.35 are common, bearing in mind the dilute cement grout ($w/c < 0.40$) usually employed. Type A anchors installed in stiff overconsolidated...
clay \( (c_u = 270 \text{kN/m}^2) \) at Taranta, Southern Italy\cite{22}, have indicated similar values of \( \alpha = 0.28-0.36 \). For stiff to very stiff marls \( (c_u = 287 \text{kN/m}^2) \), at Leicester in England, values of 0.48-0.60 have been monitored, although \( \alpha = 0.45 \) is suggested for design\cite{10}. A value of \( \alpha = 0.45 \) has also been confirmed for stiff clayey silt \( (c_u = 95 \text{kN/m}^2) \) in Johannesburg\cite{33}. Anchorages of Type A are generally of low capacity, and various construction methods have been attempted\cite{10,33}, including the use of explosives in London Clay at Herne Bay\cite{31} as early as 1955, in order to increase resistance to withdrawal. The most successful method to date in terms of ultimate load-holding capacity is the multi under-reamed Type D anchor which was developed from the field of piling.

Under-reaming of pile bases was pioneered in locations such as Texas, USA, the Orange Free State in South Africa\cite{34} and India\cite{35,36}, where severe foundation problems in expansive soils were experienced. Of particular note is the development of single, double and multi under-ream piles which has taken place at the Central Building Research Institute at Roorkee dating from 1955. In design terms the result of this work\cite{36} includes (i) development of equations for estimating ultimate bearing capacity, (ii) confirmation that under-reamed piles act similarly in tension or compression, and (iii) optimisation of the under-reamed spacing/

<table>
<thead>
<tr>
<th>Soil density</th>
<th>Bond length</th>
<th>( L_v )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very dense</td>
<td>2.0m</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4.5m</td>
<td></td>
</tr>
<tr>
<td>Dense</td>
<td>3.0m</td>
<td></td>
</tr>
<tr>
<td>Medium dense</td>
<td>2.0m</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4.5m</td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>2.0m</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 13. Distribution of long-term friction \( \tau_s \) at ultimate load in relation to tendon bond length and soil density \( (D = 91-126 \text{mm in gravelly sand}) \)
diameter ratio at 1.25-1.50.

Following the pioneering work in piling, retaining wall tie-backs in the form of single under-ream tension piles \((D = 600-900\text{mm}, d = 300\text{mm})\) were installed in soft shales and very stiff clays in the United States\(^{37}\) from 1961 and rapidly developed commercially\(^{38}\) from 1966. In the same year, small diameter under-reamed anchors \((D = 250\text{mm}, d = 75\text{mm})\), using a mechanical expanding flight under-reaming tool, were already being successfully installed in clay at Westfield Properties in Durban\(^{31, 39}\) to give safe working loads of up to 340kN with a 4m fixed anchor. In England high capacity multi-under-reamed anchors were extensively developed from 1967 in stiff clays and marls, which resulted in the use of eqn. 12 for design\(^10\):

\[
T_f = \pi D L c_u \left( \frac{\pi}{4} (D^2 - d^2) N_c c_u + \frac{\pi}{4} d l c_u \right)
\]

The rule was proved initially in London Clay at Lambeth \((c_u = 134-168\text{kN/m}^2)\), for the following dimensions, using a brush under-reamer:

- Diameter of under-ream \((D) = 350 - 400\text{mm}\)
- Diameter of shaft \((d) = 130 - 150\text{mm}\)
- Fixed anchor length \((L) = 3.1 - 7.6\text{m}\)
- Shaft length \((l) = 1.5 - 3\text{m}\)

In the absence of results from test anchors in the field, multiplier reduction coefficients ranging from 0.75 to 0.95 are commonly applied to the side shear and end-bearing components of eqn. 12 to allow for disturbance and softening of the soil which may occur during construction. In the particular case where the clay adjacent to the fixed anchor contains open or sand filled fissures, a reduction coefficient of 0.5 is recommended for the side shear and end-bearing components.

Of vital importance also in cohesive deposits is the time during which drilling, under-reaming and grouting take place. This should be kept to a minimum in view of the softening effect of water on the clay. The consequence of delays of only
a few hours include reduced load capacity and significant short-term losses of prestress.

With regard to spacing of under-reams \( (\delta l) \), eqn. 14 can be used to estimate the maximum allowable spacing to give failure along a cylindrical surface:

\[
\pi D \delta l c_u < \frac{\pi}{4} (D^2 - d^2) \frac{N_c}{c_u} \quad (13)
\]

i.e.

\[
\delta l < \frac{(D^2 - d^2)}{4D} \frac{N_c}{c_u} \quad (14)
\]

For example, if \( D = 400\text{mm}, d = 150\text{mm} \) and \( N_c = 9 \), then \( \delta l = 0.77\text{m} \). Quite independently, a similar design approach was developed in London Clay at Orford Ness \( (c_u = 54-72\text{kN/m}^2) \) where Type D anchors were constructed using a mechanically expanded double flight under-reamer (see eqn. 15):

\[
T_f = \pi D f_u c_u + \frac{\pi}{4} (D^2 - d^2) (N_c c_u \text{ end} + \sigma_u') + \pi d f_u c_u \text{ shaft}
\]

where \( f_u = 0.75 - 0.96 \)

\( f_u' = 0.3 - 0.6 \)

\( N_c = 6.5 \) (range 6 - 13 or greater)

\( \sigma_u' = \) effective stress normal to proximal end.

The anchor dimensions on site were \( D \) (460-530mm), \( d \) (140mm) \( L \) (3m) and \( l \) (7.8m). In regard to under-ream spacing it is stipulated that \( \delta l \geq (1.5 - 2) D \) and \( d \geq (0.6 - 0.7) D \) in order to ensure cylindrical shear failure. For stiff to very stiff fissured silty clay \( (c_u = 130-290\text{kN/m}^2) \) at Neasden Underpass, London, with a mean value of 175kN/m\(^2\) assumed for design, test results\(^{49} \) for a multi-flight mechanical under-reamer \( (D = 540\text{mm}, d = 175\text{mm}) \) have indicated an efficiency factor \( f_u = 0.75 \).

The success of multi under-reamed anchors over straight shafts can perhaps be illustrated best\(^{41} \) by reference to Fig. 15. Based on the same augered hole diameter of 150mm, the straight shaft Type A anchor with a fixed anchor length of 10.7m failed at 1000kN, whereas the under-reamed anchor of only 3m withstood, without any sign of failure, a load of 1500kN. The advantages have also been quantified for London Clay\(^{42} \) where measurements of brushed under-reams by borehole caliper indicate \( D \) (363mm) and \( d \) (140mm) i.e. an improvement of 2.59 and test anchor back-analysis gives an adhesion factor \( \alpha = 0.78 \) c.f. the straight shaft \( \alpha \) of 0.35 i.e. an improvement of 2.23. Consequently, an overall improvement of more than five times is confirmed by both examples.

As a result of tests of this type and accumulated field experience of commercial anchors, safe working loads of 500 - 1000kN can be obtained in stiff to hard clays using the multi under-reamed anchor Type D, compared with 300 - 400kN using straight Type A anchors. These figures are based on load safety factors of 2.5 - 3.5, which are considered necessary to minimise prestress losses due to consolidation of the clay.

In general, there is still a serious shortage of field performance data for anchors in cohesive soils, and little information is available on soil strength below which under-reaming is impracticable. In the writer's experience, under-reaming is ideally suited to clays of \( c_u \) greater than 90kN/m\(^2\), but some difficulties in the form of local collapse, or breakdown of the neck portion between the under-reams should be expected where \( c_u \) values of 60 - 70kN/m\(^2\) are recorded. Under-reaming is virtually impracticable below \( c_u \) of 50 kN/m\(^2\).

In such circumstances, use of the high pressure Type C anchor, with and without post-grouting, is worthy of study. The results of a large number of fundamental tests\(^{28} \) are shown in Fig. 16 which can be used as a design guide for borehole diameters of 80 - 160mm. Skin friction increases with increasing consistency and decreasing plasticity. In stiff clays \( (l_c = 0.8 - 1.0) \) with medium to high plasticity, skin frictions of 30-80 kN/m\(^2\) are the lowest recorded, whilst the highest values \( (\tau_w > 400\text{kN/m}^2) \) are obtained in sandy silts of medium plasticity and very stiff to hard consistency \( (l_c = 1.25) \). The technique of post-grouting is also shown to generally increase the skin friction of very stiff clays by some 25-50%, although greater improvements (from 120 up to about 300kN/m\(^2\)) are claimed for stiff clay of medium to high plasticity. From Fig. 17 the influence of post grouting pressure
Fig. 16. Skin friction in cohesive soils for various fixed anchor lengths, with and without post-grouting
19 test anchors in medium to high plastic clay

\( w_L = 48-58\% \)
\( \phi = 25-35\% \)
\( I_c = 1.1-1.2 \)

Fig. 17. Influence of post-grouting pressure on skin friction in a cohesive soil.

Fig. 18. Boundaries of the grain size distribution of the investigated cohesive soil.
on skin friction is quantified for clays of medium to high plasticity\textsuperscript{25}, showing a steady increase in $\tau_c$ with increase in $p_1$

For Type C anchors in cohesive soil, the Hannover analysis\textsuperscript{29} provides eqn. 16.

$$T_f = a_0 + a_1 \pi D L + a_2 D_1 + a_3 D_2$$
$$+ a_4 D_3 + a_5 D_4$$
$$+ a_6 I_c + a_7 \tau_c \ldots \ (16)$$

where $\tau_c = \gamma h_m \tan \phi \sqrt{\cos \alpha}$

$$+ \sin \alpha (1 + 2 \tan^2 \phi)$$
$$+ 2 \sin \alpha \cos \alpha$$
$$+ c' \cos 2 \tau_c$$

$a_0 = 721.51$ $a_1 = 71.84$ $a_2 = 9.81$ $a_3 = 1.99$

Estimating the carrying capacity of ground anchors in cohesive soil by using eqn. 16, the grain size curve must be within the boundaries of Fig. 18 and the values of the influence factors are not allowed to exceed the following limits:

$0.98m^2 \leq \pi DL \leq 6.48m^2$

$6.50cm \leq D \leq 16.80cm$

$4.10m \leq L \leq 15.00m$

$0.84 \leq I_c \leq 1.35$

$20\% \leq D_1 \leq 76\%$

$50.7kN/m^2 \leq \tau_c \leq 165.3kN/m^2$

**Distribution of shear stress**

As for strong rock and dense frictional soils, the variations in measured stress in grout bonded tendons in clay, and the calculated shear stresses at the clay/grout interface can be non-linear\textsuperscript{43,44} both at low stress levels and at failure.

For stiff overconsolidated clay at Taran-ta\textsuperscript{32} ($c_u = 270kN/m^2$ average), Fig. 19 illustrates the shear stress distribution at failure, where $E = 6.9 \times 10^4 kN/m^2$ was deduced\textsuperscript{44}.

Bearing in mind that $E$ values for grout can be in the range $(1-2) \times 10^4 kN/m^2$, and that for rocks a uniform stress distribution is anticipated\textsuperscript{12} where $E_{\text{grout}}/E_{\text{rock}}$ exceeds 10, it is interesting to observe non-uniformity in Fig. 19, where the elastic modular ratio is well in excess of 100.

**Remarks**

The subject of load transfer with particular reference to the major parameters which influence stress distribution appears to warrant further study. Under failure conditions the results could indicate an upper limit to fixed anchor length ($L$). In current practice $L$ seldom exceeds 10m. Under service conditions a knowledge of the stresses imposed on the clay would assist calculation of the magnitude and rate of consolidation around the fixed anchor, and hopefully improve our predictive capacity concerning loss of prestress with time. The relative importance of the tendon type e.g. bar or strand, must also be ascertained in this respect bearing in mind the greater stiffness of bars which will magnify the prestress loss in any comparative study.

**Factors of safety**

When a grouted anchor fails, it must be by one of the following modes:

(a) Failure of the ground mass,

(b) Failure of the ground/grout bond,

(c) Failure of the grout/tendon bond, or

(d) Failure of the tendon or anchor head, and in order to determine the mechanism of failure and actual safety factor for the anchor, consideration must be given to all of these aspects.

The traditional aim in designing is to make a structure equally strong in all its parts, so that when purposely overloaded to cause failure each part will collapse simultaneously.

“Have you heard of the wonderful one-hoss shay,

That was built in such a logical way

It ran for a hundred years to a day,

And then, of a sudden it . . .

. . . went to pieces all at once, —

All at once, and NOTHING FIRST, —

Just as bubbles do when they burst.”

_The Deacon’s Masterpiece_, by

Dr. Oliver Wendell Holmes.

Thus for each potential failure mechanism a safety factor must be chosen hav-
ing regard to how accurately the relevant characteristics are known, whether the system is temporary or permanent, i.e. service life, and the consequences if failure occurs i.e. danger to public safety and cost of structural damage. Since the minimum safety factor is applied to those anchor components known with the greatest degree of accuracy, the values suggested in Table IV invariably apply to the characteristic strength of the tendon or anchor head.

In regard to the ground/grout interface of the fixed anchor upon which this Paper has concentrated, overall design load safety factors \( S_1 \) range from 2-4 generally, where \( S_1 \) is applied to the ultimate load-holding capacity \( T_f \). \( T_f \) may be defined as the constant load at which the fixed anchor can be withdrawn at a steady

**TABLE IV. SUGGESTED SAFETY FACTORS FOR ANCHOR DESIGN**

<table>
<thead>
<tr>
<th>Anchor category</th>
<th>Minimum safety factor</th>
<th>Proof load factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temporary anchors where the service life is less than 6 months and failure would have few serious consequences and would not endanger public safety e.g. short term pile test.</td>
<td>1.4</td>
<td>1.1</td>
</tr>
<tr>
<td>Temporary anchors with a service life of up to 2 years, where although the consequences of failure are quite serious, there is no danger to public safety without adequate warning e.g. retaining wall tie backs.</td>
<td>1.6</td>
<td>1.25</td>
</tr>
<tr>
<td>All permanent anchors. Temporary anchors in a highly aggressive environment, or where the consequences of failure are serious e.g. temporary anchors for main cables of a suspension bridge or as a reaction for lifting heavy structural members.</td>
<td>2.0</td>
<td>1.5</td>
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rate e.g. creep in cohesive soils, or the maximum load attained prior to a distinct failure involving a sudden loss of load e.g. breakdown of bond in rock. As more poor quality ground has been exploited by anchors, so safety factors have increased in value to take account of (i) larger fixed anchor displacements for given load increments (Fig. 20), or (ii) creep phenomena. In the case of (ii) for example, $S_1$ values of 2-2.5 for temporary anchors in clay where the service period is less than 2 years, rise to 3-3.5 for permanent anchors, in order to keep prestress fluctuations within acceptable limits. In other words designers are quietly building in Serviceability Factors.

To avoid the growing situation where engineers simply specify the latest and largest safety factors irrespective of the ground, there is a need for a more thorough investigation of load-displacement relationships for fixed anchors in different ground conditions, since these relationships influence choice of safety factors which should be related to permissible movement as well as ultimate load. For specific ground conditions it may be possible for example to establish a correlation between a yield load giving unacceptable movement, and the ultimate load-holding capacity. Thus, if an engineer wishes to specify a factor of safety ($S_u$) against a yield condition, it may be feasible then to apply a modifying factor to $S_y$ to provide an estimate of $S_1$ for the ultimate load-holding capacity ($T_1$) or vice versa when $T_1$ is estimated from an empirical equation or design envelope. This concept of safety factors may grow in importance with the advent of Limit State Codes. In an effort to encourage the analysis of test anchor results there is perhaps a case for two levels of safety factor depending on whether actual test results or calculated ultimate loads are used for design purposes.

**Conclusions**

Anchor construction technique and quality of workmanship greatly influence pull-out capacity, and the latter in particular limits the designer's ability to predict accurately solely on the basis of empirical rules. As a consequence the calculated figures should not be used too dogmatically in every case, since they often provide merely an indication of comparative values to the experienced designer. In anchor technology, practical knowledge is just as essential to a good design as ability to make calculations.

In 1969 at the Mexico Conference, Reporter Habib observed spectacular progress in anchoring in loose soils, but stated that it was rather odd to realise that the theories were still empirical in nature. Since empiricism in design is still prevalent today it might be argued that little progress has been made. In the author's view some sympathy must be expressed for the attitude that resists the creation and application of the more sophisticated theories, since they invariably demand accurate values of a multitude of ground and anchor parameters in order to attain the improved accuracy. In this regard, a

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*Fig. 20. Load/displacement relationship for compact fine/medium sand ($\phi = 35^\circ$)*
good example is short, low capacity rock bolts where the cost of investigating the detailed variation in a heterogeneous rock mass far outweighs the cost of installation and proof-loading additional bolts, in the event of unsatisfactory performance.

In reality a period of technical consolidation has taken place over the past decade in the form of standardisation of practice combined with a steady collection of short-term test data. At the same time the world anchor market has continued to expand dramatically, and forced designers into a wider range of ground conditions, particularly poor quality materials. Design rules have been created, employed and confirmed to be satisfactory in the main over this period, and significantly most attention has been directed towards simple pull-out tests. Routine tests of this kind are of paramount importance, since the results can be used to optimise the design and construction of the anchors on a particular site, in addition to establishing actual factors of safety. In this way the validity of empirical design rules can also be checked for the different ground conditions encountered in anchorage work. In the future, more attention should be directed towards monitoring load displacement relationships and service behaviour with particular regard to loss of prestress with time in order that more confidence can be established for permanent anchors in soils and weathered rocks.

References


