Special test anchors

In cases where there is no prior experience of anchoring in a particular rock, special tests should be carried out to optimise check, design assumptions, and also to pinpoint any important practical considerations relating to construction and stressing. In rocks susceptible to creep, the duration of the test should be sufficient to establish a safe working load for minimal creep, or to permit assessment of an overload allowance or restressing programme to accommodate creep losses. In all rocks an attempt should be made to test these anchors to failure so that actual safety factors can be determined.

It is interesting to note that Stefanko and de la Cruz (1964) use the terms "Dynamic" and "Static" when summarising types of test, as follows:

(i) Dynamic: progressive and continual loading of the anchor until failure is induced. Such tests provide data on the ultimate capacity of certain elements e.g. grout/tendon bond or rock/grout bond, and usually these ultimate values are simply factored to provide suitable working parameters. In Europe such tests are referred to as "basic" or "suitability" tests, and they must be carried out on specially installed anchors which will not subsequently be employed in service.

(ii) Static: load-time relationships are determined to investigate the anchorage effectiveness. Such "decay" tests are more time-consuming and costly, and are not yet as widely conducted as would appear advisable. Anchors undergoing this type of test can be used as production anchors if required.

In Germany, the basic suitability of any ground anchor system is ascertained from basic tests on at least three anchors in recognised types of ground (DIN 4125: 1972). The construction, testing and subsequent excavation of the anchors must be monitored by a recognised professional institution which also classifies the ground. Approximately one week after grouting, stressing is carried out and top anchor displacements are measured from a remote datum for all loads above the initial seating load (T_s ≥ 0.1 T_u). Proceeding from this initial value T_s, load increments equivalent to 0.15 T_u are applied until failure, or until the yield stress of the tendon is reached (Fig. 18a). After the load increment equal to 0.3 T_u and thereafter at each successive higher load increment, the tendon is unloaded to T_s to provide data on permanent displacements, and to enable calculation of the effective free length of the tendon. The top anchorage displacements occurring at loads below T_s are not measured.

Before each unloading operation displacements are observed under constant load in non-cohesive soils until the movements stop, but for at least five minutes. At 0.6 T_u the load is held for 15 minutes and the associated displacement Δ_s is noted (Fig. 18b). At 0.9 T_u the observation time is increased to at least one hour (associated displacement = Δ_s). In cohesive soils the observations at 0.6 T_u and 0.9 T_u are continued until the displacement during the last two hours is less than 0.2mm. If the working load (T_w) is less than 0.6 T_u the maximum applied test load should be at least 1.5 T_w (observation time at least 1 hour), and the working load (T_w) should be applied for at least 15 minutes.

All applied loads should ideally be measured with the aid of load cells, and the displacements via dial gauges accurate to 0.01mm.

During the basic test the actual shape, length and character of the complete anchorage is determined by excavation after the stressing stage. Particular attention is paid to the grout-tendon interface and central position of the tendon in the grouted fixed anchor zone.

On plotting the load-displacement results, the measured displacements at the top anchorage are divided as for acceptance test analysis into elastic (Δ_e) and permanent (Δ_p) portions (Fig. 18). For a specific anchor load (Point X) as shown in Fig. 18a the total displacement is Δ, with an elastic component Δ_e, and permanent displacement Δ_p. In Fig. 18b the elastic and permanent components of displacement are plotted for each load increment, and the failure load is readily observed as being 0.94 T_u. However in this case, the upper load limit specified might be 0.9 T_u, if this was the maximum load step at which the displacements under constant load clearly stabilised during the observation period.

If the upper load limit is not reached in the basic test, the largest test load applied is taken as the upper limit, but never greater than T_u.

Following the basic test a report is produced which describes fully the ground conditions, anchor characteristics and stressing results. The upper load limit is quoted for the observed free and fixed anchor lengths. In the case of the observed free length, the curve of the elastic displacement Δ_e (Fig. 18b) should lie between the boundary lines (a) and (b) (see "Acceptance testing").

It is noteworthy that any anchor system

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†This is the third and final section of this article on rock anchor stressing and testing: the first, which appeared in our March issue, pp. 20-29, covered stressing and precontract component testing. The second, published in April, pp. 55-66, dealt with acceptance testing of production anchors, and long-term monitoring of selected production anchors.

These three articles together form the third and final part of a three-part series on rock anchors. The first two parts were published in May 1975. The series was published in two sections: the first, dealing with design considerations, was in September pp. 34-45 and November pp. 36-45.

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Fig. 18. Stressing programme for basic or suitability tests.

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chosen for a contract must also be subjected to three suitability tests at the construction site, if the local ground is different to that of the basic test, or if the drilling procedure or borehole diameter is substantially different from the basic test. In contrast to the basic tests however, the anchors in suitability tests are not excavated after stressing.

For permanent soil anchors in Germany (Draft DIN 4125: 1974) the basic tests are similar to those already described for temporary anchors with the following variations.

The tensile load is applied in the stages specified in Table VI commencing at $T_w$. When each stage of loading has been reached, the load is subsequently reduced to $T_w$, so that elastic and permanent displacements can be judged.

The anchors should be stressed to 0.9 $T_w$ if the failure load of the grouted fixed anchor is not reached at an earlier stage.

In order to determine the limit load for minimal or acceptable creep ($T_x$), the displacement must be measured under constant loading prior to the removal of each load e.g. after 1, 3, 5, 10 and 30 minutes, and recorded as shown in Fig. 18c. The required minimum observation periods are shown in Table VI but these periods can be extended if necessary until the trends are clear and the creep $\Delta$ related to the displacement of the anchor, can be determined. In addition, it is recommended that if the creep is greater than 1mm for a course grained soil, then the longer minimum observation periods for fine grained soils should be adopted.

In accordance with Fig. 19, the creep $K$ is calculated as follows

$$K = \frac{\Delta - \Delta_i}{\log t/t_i} \ldots (1)$$

The values of $K$ are evaluated at different stages of loading and recorded as shown in Fig. 20, and by definition the limit force $T_x$ corresponds to a creep $K\Delta$ of 2mm. After this stage of the test, the anchor is subjected to twenty load cycles (range——0.3 $T_x$ to 0.6 $T_x$) and the extension at the maximum and minimum loads must be measured at least after every five cycles. Pauses for observation of extensions should not be included for intermediate cycles. Subsequently, the load is reduced to $T_w$ then increased to 0.6 $T_s$ with an appropriate observation period.

A similar approach is applied to the suitability tests on the construction site, where it is specified that the tests should be carried out in the most unfavourable soil conditions. The loading stages are shown in Table VI with the basic observation periods. Subsequently, twenty load cycles (range——0.5 $T_s$ to 1.0 $T_s$) are carried out. Only when these rigorous tests have been completed satisfactorily, is the permanent service load locked-off.

In both the basic and suitability tests the maximum permissible load specified for the anchor is the smallest of the following values:

(i) $T_s/1.75$ ($T_s$ = guaranteed yield strength of the tendon),
(ii) $T_s/1.75$ ($T_s$ = failure of the bonded fixed anchor), and
(iii) $T_s/1.50$ ($T_s$ = limit force for creep > 2mm according to equation (1) above.

In France, basic test anchors as detailed by Bureau Securitas (1972) are categorised by geometry and ground type, and the minimum number of test anchors is related to the number of production anchors in one category, as shown in Table VII.
TABLE VI. LOAD STAGES AND OBSERVATION PERIODS FOR BASIC AND CONSTRUCTION SITE SUITABILITY TESTS (after Draft DIN 4125: 1974)

<table>
<thead>
<tr>
<th>Stage of loading</th>
<th>Minimum period of observation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic test $T &gt; 0.1 T_v$</td>
<td></td>
</tr>
<tr>
<td>Suitability tests* $T &gt; 0.2 T_v$</td>
<td></td>
</tr>
<tr>
<td>Coarse grained soils</td>
<td>Fine grained soils</td>
</tr>
<tr>
<td>0.30 $T_v$</td>
<td>0.40 $T_v$</td>
</tr>
<tr>
<td>0.45 $T_v$</td>
<td>0.80 $T_v$</td>
</tr>
<tr>
<td>0.60 $T_v$</td>
<td>1.00 $T_v$</td>
</tr>
<tr>
<td>0.75 $T_v$</td>
<td>1.20 $T_v$</td>
</tr>
<tr>
<td>0.90 $T_v$</td>
<td>1.50 $T_v$</td>
</tr>
</tbody>
</table>

*If the working load is not known at the time of the test or the upper limit load is uncertain, it is recommended that smaller load stages should be selected.

TABLE VII. MINIMUM NUMBER OF TEST ANCHORS RELATED TO NUMBER OF PRODUCTION ANCHORS (after Bureau Securitas, 1972)

<table>
<thead>
<tr>
<th>No. of test anchors</th>
<th>No. of production anchors</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1— 200</td>
</tr>
<tr>
<td>3</td>
<td>201— 500</td>
</tr>
<tr>
<td>4</td>
<td>501—1,000</td>
</tr>
<tr>
<td>5</td>
<td>1,001—2,000</td>
</tr>
<tr>
<td>6</td>
<td>2,001—4,000</td>
</tr>
<tr>
<td>7</td>
<td>4,001—8,000</td>
</tr>
</tbody>
</table>

As an example, if a project involves 500 anchors, of which 300 are inclined and 200 are vertical, then two categories are present, based on geometry. If, in addition it is known that 200 are inclined into gravel, 100 are inclined into clay, and all the vertical anchors are installed in clay, then a total of three categories must be recognised as follows:

- 200 inclined/gravel — 2 test anchors
- 100 inclined/clay — 2 test anchors
- 200 vertical/clay — 2 test anchors

Bureau Securitas states that the test anchors must be similar to the categories of the production anchors envisaged. This requirement concerns the method of construction and anchor geometry although it is accepted that the tendon can be of larger capacity to permit a high test load to verify a high safety factor or possibly induce failure of the grouted fixed anchor.

For ground where previous anchoring knowledge is available and there is no risk of creep, the Bureau states that it is possible with confidence to load the test anchor up to the anticipated working load of 0.75 $T_v$ and 0.60 $T_v$ for temporary and permanent anchors, respectively (Figs. 21a & b). $T_v$ is the elastic limit of the tendon and equivalent to 83.5% fpu.

In order to eliminate from the start parasitical movements such as tendon slack and plate "bedding-in", two successive load cycles are recommended (Table VIII) with pauses only to record the extensions. On completion of the second loading cycle, stressing is carried out in stages, with observation periods under constant load at each stage to permit creep observations.

At each of the stages, displacement measurements are taken every 30 seconds during the first two minutes, every 5 minutes between the second and tenth minutes, and every two minutes thereafter. After the one hour observation period at 0.9 $T_v$, the load is removed completely in stages and then reapplied in stages up to the lock-off load with pauses only for displacement readings. Allowing for lock-off losses, the initial residual load must not be lower than 0.80 $T_v$ and 0.65 $T_v$ for temporary and permanent anchors, respectively, to accommodate tendon relaxation and ground creep. After 72 hours the load is reapplied and the increment of top anchorage displacement to regain the initial residual load is monitored. This displacement should be less than 4mm.

The anchor is then unloaded completely prior to a final stressing operation where the load is increased in load increments as before until failure occurs or the extension of the steel tendon is equal to 150% of the extension at the 0.1% proof stress (Fig. 22). The test is now complete and the load is reduced to zero before the anchor is abandoned.

Where the ground conditions are not known, or prior experience of anchoring in the ground does not exist failure may occur at a load below 0.9 $T_v$. In these circumstances the maximum test loads for the first three load cycles which are carried out without pauses are lower (see Figs. 23a & b). During these three cycles, displacement measurements are taken each time the load is changed by 0.05 $T_v$. With regard to creep or relaxation losses measured over 72 hours, the initial residual loads locked-off are 0.85 $T_v$ and 0.7 $T_v$ for temporary and permanent anchors, respectively. If the displacement required to regain the initial residual load is less than 4mm, then the test proceeds as already described. If however, the displacement is greater than 4mm indicating creep of the grouted fixed anchor, a second 72 hour check is carried out (Fig. 23c). If the displacement now required to regain the initial residual load is less than 1mm, the test may proceed as already described. If however, the creep displacement exceeds 1mm, the Engineer may continue the present test or order a second test anchor and repeat the test but with a lock-off load at least 30% lower. It is important to note that the Bureau Securitas recognises that the figures of 4mm and 1mm are rather arbitrary and should be regarded as provisional values only.

If failure of the first test anchor occurs at load $T_v$ during one of the intermediate test stages, tensioning of the second or subsequent anchors should follow the principal illustrated in Fig. 24 for temporary anchors. The basic approach is identical to that already described in Figs. 21a & 23 but this time the load increments are related to $T_v$ and not $T_v$.

With regard to the scatter of results, if all test anchors fail in the fixed anchor zone or the test is stopped due to excessive extension, the ultimate loads should not differ by more than 30%, with respect to the smallest ultimate load. Where the scatter is above this figure, a rigorous analysis of the reasons is necessary.

The maximum working load is specified equivalent to 0.67 $T_{min}$ and 0.50 $T_{min}$ for temporary and permanent anchors, respectively ($T_{min}$ = minimum ultimate load for test anchors). If none of the test anchors fails, the maximum working load must not exceed 0.75 $T_v$ and 0.60 $T_v$ for temporary and permanent anchors, respectively. These working loads can only be applied of course to test anchor results where the creep displacement criteria already described have also been satisfied.

The Czech Draft Code (1974) relates to...
both DIN 4125 (1972) and Bureau Securitas (1972). A basic anchor test is recommended for each type of anchor which includes subsequent excavation. No details are provided however on acceptance criteria related to test load or creep displacement. It is noteworthy however that a prime objective of the basic tests is to confirm design safety factors of 1.5 and 1.6 for temporary and permanent anchors respectively.

In the case of ground where anchor behaviour is unknown, the FIP Draft Recommendations (1973) suggest special long-term tests using restressable top anchorage heads. Where it is necessary to observe the variation of load over a period of time, lift-off checks or the use of load cells is an acceptable practice but monitoring the displacements of the fixed anchor and the top anchorage is also recommended to facilitate analysis of anchor behaviour. No specific guidance is provided by FIP on acceptance criteria in relation to these long-term tests.

In order to optimise the design and construction of anchors in a particular type of ground, a minimum of three test anchors has been recommended in Britain (Littlejohn 1970). The fixed anchor length is varied, and for a particular ground condition and anchor position an estimate of the magnitude of the side shear and end-bearing component of the ultimate load is ascertained, if failure is achieved at the ground/grout interface, by plotting the failure load against fixed anchor length. In addition to establishing actual factors of safety, the validity of empirical design rules can be checked.

When assessing the suitability of a proposed anchor system for a contract the minimum data required from test anchors on the construction site are shown in Fig. 25. In current practice the number of test anchors usually ranges from one to three.

Assuming that the basis of the production anchor design is to be checked before the contract, then the tendon strength at 80% fpu should be sufficient to test the anchor to give a measured safety factor of 2 or in the case of ground susceptible to creep, the safety factor may be in the range 2.5 to 4.0 depending on duration of service.

The anchor is first loaded incrementally up to 1.25 \( T_T \) or 1.5 \( T_T \) depending on whether the production anchors are temporary or permanent, respectively, since this will represent the normal load test \( (T_T) \) in practice. After an observation period of five minutes the anchor is de-stressed, the load-extension graph being plotted for the full cycle. On restressing to \( T_T \), the load at the cross-over point \( T_T \) is noted if additional fixed anchor displacement is required to mobilise \( T_T \). In this situation it is considered that for the value \( T_T \) shown, \( T_T \) should have a value less than \( T_T \) in order to minimise loss of prestress particularly if the production anchors are subjected to cyclic loading.

The test anchor is then locked-off at \( T_T \), and left for at least 24 hours to measure loss of prestress. Thereafter, the duration of the test should be as long as possible since it serves to indicate whether creep of the anchor is likely to be serious during service. The test anchor is finally stressed to failure, or 80% fpu, in an attempt to...
establish the actual factor of safety of the anchor. In addition, the ultimate bond values attained at the ground/grout and grout/tendon interfaces, respectively, are compared with the values assumed in design.

During the second loading cycle up to \( T \), the load-extension curve should compare closely with the theoretical extension due to the bending moment from the anchor in mind the known sources of error in materials and measurements (see Part 3—Stressing), British engineers normally accept a discrepancy of \( \pm 5\% \) between observed and calculated results. Where discrepancies approach \( \pm 10\% \), a detailed examination of the results is undertaken to more fully interpret and explain the observed behaviour.

**Remark**

The major advantages of test anchors may not be fully appreciated at present, but it is important to note that these tests can provide:

(i) confirmation of specified safety factors (in the case of test anchors taken to failure, the validity of empirical design rules can be verified since ultimate values are determined);

(ii) verification of the reliability of the proposed anchor system for the construction site.

(iii) advance warning of construction difficulties, and

(iv) predictive capacity concerning time-dependent phenomena, where the test loading is observed over a significant period of time.

A survey of the most influential recommendations reveals no general agreement on the number of anchors to be tested, but it would appear that a minimum of three precontract anchors should be tested for each geotechnically distinct rock type likely to be encountered on site. One test anchor in each group should have sufficient strength of tendon to fail, or at least test severely, the bond at the rock/grout interface.

The time and expense involved in test anchor programmes warrants careful planning, execution and analysis, otherwise the potential advantages above will not be fully realised. In this respect the value of practical, on-site, agreed nationally or internationally, cannot be over-emphasised.

**Monitoring of the overall anchor rock structure system**

Monitoring the complete anchor/rock/structure system can improve basic understanding of anchor behaviour and act as a quality control by checking that the overall engineering solution adopted is satisfactory during service. This form of monitoring covers the behaviour of the structure, rock mass and anchors, whether individually or in groups, and facilitates study of the short and long-term interaction between different components of the complete system.

This type of monitoring is particularly important in excavation engineering e.g. stabilisation of opencast pit slopes, where it is impossible to observe overall behaviour of the anchored slope as excavation proceeds. Clearly, monitoring of overall behaviour is expensive and time consuming and in practice may be restricted to major mining operations or prestigious civil engineering projects. Nevertheless, only by such studies in the field can important concepts relating to overall stability and group effects be verified.

**Service behaviour of production anchors**

**Introduction**

This final section deals with the long-term behaviour of rock anchors in service, with particular reference to the load-retaining characteristics of anchors for periods in excess of 10 years following stressing. Disproportionately little field research has been conducted into this aspect of rock anchors, despite its important bearing on various fundamental aspects of design, stressing and testing. Therefore, data including attempts to correlate anchor performance up to, and after, the first 24 hours of service—due partly to the fact that the potential yield of such results is not fully appreciated, and partly to the time and expense required to set up and pursue a programme of long-term monitoring.

This lack of knowledge exists despite the fact that all engineers associated with anchor contracts have a responsibility to be concerned with long-term behaviour and would benefit from such information. For example, the designer would be able to design and build a rock/anchor structure into future designs and thereby optimise such parameters as over-load allowances and safety factors. Likewise a prospective client could be accurately and directly informed by the consulting engineer of how the anchors installed at his expense would perform after installation. Furthermore the presence of a comprehensive “data bank” would permit engineers to judge the stage at which anchors were being monitored were, in fact, acting satisfactorily or in a potentially dangerous manner. Long-term monitoring also permits correlation of anchor load fluctuation with time and structural movement e.g. the performance of a diaphragm wall tied at several levels (Saxena, 1974; Littlenjohn and MacFarlane, 1974; and Ostermayer, 1974).

In the following review the authors firstly discuss information relevant to the relaxation and creep properties of steel tendons, since tendon characteristics alone can be assessed accurately under controlled test conditions in the laboratory. In analysing subsequent data relating with rock anchors this knowledge can be used to isolate and recognise other time-dependent variables influencing the service behaviour of full-scale anchors. Finally, a limited number of case records is presented to illustrate different aspects of field anchor performance.

**Time-dependent behaviour of steel tendons**

Assuming that no structural movement occurs, relaxation or creep of the tendon will result in loss of prestress during service. In practice relaxation is regarded as the decrease of stress with time while the tendon is held under constant strain, whereas creep is the change in strain of the tendon with time under constant stress.

**Relaxation**

According to Antill (1965), both relaxation and creep lead to approximately the same loss of prestress in practice for a given tendon under constant temperature, but the magnitude of such loss from relaxation characteristics of the steel is preferred by steel manufacturers because of its closer simulation of actual working conditions in the field. If prestressed concrete is in connection, prestressed rock anchors may be regarded as a similar application and long-term relaxation properties for the tendon permit prestress losses, and therefore residual loads, to be determined in practice.

Details of tendon relaxation have already been presented in Part 1—Design, of this review. However, it is relevant at this point to consider the major conclusions reached by Antill (1964), Bannister (1962), and Mihajlov (1968):

(i) Early conceptions that relaxation values at 1000 hours are equivalent to ultimate values are completely erroneous. Currently, long-term relaxation values are in accord with the stress loss after 100,000 hours, and Antill (1965) suggests that the ultimate loss of stress is about twice the loss at 1000 hours at 20°C, for all common values of initial stress. In fact, the loss at 1000 hours is twice that at 1 hour, 80% of that at 1000 hours and 40% of the loss at 30 years, according to long-term tests on various types of steel.

(ii) The introduction of “stabilised” wire has reduced load losses from 5-10% in ordinary stress relieved steel, to 1.5% at 75% GUTS (= guaranteed ultimate tensile strength) and 20°C.

(iii) The rate of load relaxation increases with temperature to 20°C.

(iv) The rate of relaxation varies with the initial stress, the actual rate being a function of the type of steel. Relaxation from initial stresses up to 50% GUTS may be considered negligible in practice.

(v) For initial stresses greater than 0.55 %fy the relationship is

\[
\frac{f_1}{f_t} = 1 \log \frac{t}{t_1} - 0.55
\]

where \( f_1 \) = residual stress after time \( t_1 \)

\( f_t \) = initial stress

\( f_t = 0.1 \% \) proof stress at working conditions and temperatures,

\( t \) = time in hours after application of initial stress.

(vi) With initial stresses of 70% GUTS, restressing at 1000 hours reduces the amount of ultimate relaxation to almost one-quarter of its normal value and for initial stresses of 80% GUTS the reduction is about one half. Insufficient information is available at present to permit firm conclusions to be drawn with regard to the effect of restressing at 1000 hours.

(vii) An unduly high order of accuracy in determining relaxation losses is often not warranted since the significant parameter in practice is the residual stress in the tendon.

(viii) Deliberate temporary overloading of the tendons (for a short period of time e.g. 2-10 min.) at the time of initial stressing, is claimed to reduce relaxation losses by disposing of the rapid initial relaxation, is thought to be generally beneficial and a particular advantage in the case of strand. However, the reduction is of little consequence in strands where the long term relaxation loss is not appreciable in any case.

(viii) A feature of importance in the field is the effect of the design of strand jacks upon relaxation, which is closely related to the pre-stressed strand. The tendency of strand to “unwind” under load has been discussed by Bannister (1959); it arises from the presence of a torsional component approximating to 10% of the load application losses. The presence of this component would appear to have a marked effect upon relaxation losses and in tests on 12.7mm strand (Duckfield, 1964), the relaxation at 1000 hours was found to be of the order of about 5% and 8% with and without

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torsional restraint, respectively. Hence, for practical purposes, those jacks designed with a key way or other device to prevent rotation during stressing may be preferred. Creep

Creep is intrinsically more difficult to theorise upon, or measure experimentally in the field. The phenomenon of creep (flauge) in steel is, however, discussed by Fenoux and Portier (1972).

As a result of precise experiments augmented by the findings of other authors, they conclude that:

(a) The creep rate \( a (F) \) increases over the range 0-30% GUTS, is constant at the limit of proportionality (88% GUTS in the case studied), and then increases rapidly at higher loads.

(b) The amount of creep can be represented by an equation of the form: creep at time \( t \) after lock-off = \( a (F) \times t \).

Fenoux & Portier point out that creep does not terminate with time, but no indication of a practical time limit for prestress loss or negligible creep is provided.

(c) Values of \( a (F) \) appear independent of steel type for stresses less than the limit of proportionality.

To illustrate the importance of creep for a tendon, near the limit of proportionality, Fenoux & Portier have stated that the creep in 2 minutes is 0.22m/m of free length.

It is further shown that the relation between creep and relaxation rates, under identical conditions, is of the form

\[
\beta(F) = \frac{E \times a(F)}{E}
\]

where \( \beta(F) \) is the rate of relaxation, and \( E \) is the elastic modulus of the tendon.

Field observations

To illustrate the importance of the phenomena causing load loss, the authors have assembled some of the better documented case histories. Generally, however, the type and quality of the scanty data published to date relating to long-term behaviour are disappointing. For instance, it is intuitive to suppose that rock type is a major influence on anchor performance, yet the information on relevant rock properties, other than the geological name, is commonly supplied in case histories. For example, Schwarz (1972) monitored the behaviour of many anchors at frequent intervals over seven months in Stuttgart but although he presented comparisons of anchor performance in lithologically distinct horizons, no relevant rock properties were detailed.

It would appear that little guidance is available at Code level. PCI (1974) affirms that for most rock anchor applications, the primary-time-dependent loss is steel relaxation—up to 3% in seven days dependent on the type of steel, and the South African Code (1972) recommends locking-off an overload of 10% as "an allowance for relaxation and creep" similar to British practice.

In the following examples, the relevance of such allowances may be readily judged.

Much of the early published data relates to the prestressing of dams and in the particular case of raising existing dams founded on good quality rock, where the structure is "old and worked", Parker (1958) advises that no allowance is necessary for creep and shrinkage in the concrete. Loss of prestress with time, therefore, is only due to tendon relaxation. In this connection, Walther (1959) describes the performance of VSL anchors at the Luzzo Dam. In particular, for a 1 000kN test anchor (fixed anchor length = 3.20m, diameter = 90mm), the loss in prestress over 3 500 hours was 4%—"virtually exactly the load loss which had been anticipated from relaxation losses".

For new dams, Zienkiewicz and Gerstner (1961) have estimated that load loss is primarily due to creep in the concrete of the dam and only secondarily to tendon losses. They computed that an ultimate prestress loss of 9% was possible—compared to an allowance of 10% at the Alt- na-Laigre Dam, where the anchors were installed in fissured granite. Eberhardt & Veltrup (1965) conclude the 24 hour load check is much too soon to check "one significant possible source of stress loss; namely shrinkage and creep of the concrete". They estimate ultimate load losses to be of the order Concrete-creep 2.0% 
Concrete shrinkage 3.6%
Steel creep 1.0%
but overload by 10% to cover the worst possible case.

Thompson (1969) describes six test BBRV anchors (fixed anchor length = 9m, diameter = 152mm) as detailed in Table IX, at the John Hollis Bankhead Dam, Alabama.

The relatively high load loss in anchor 6 is ascribed to its shorter length causing the fixed anchor to intersect the lower of two 0.6m thick coal seams in the sandstone—shale sequence. Thompson claims that the intersection of either coal seam could account for the higher loss.

The longest record of prestress loss available is that from Cheurfas Dam, the salient points of which are summarised in Table X. The fixed anchor zone, consisting of a grouted borehole (250mm dia.) with two under-reamers (370mm dia.) was formed in 10m of yellow sandstone, overlain by about 4m of fossiliferous limestone and underlain by marl.

A claim by Khavaa et al (1969) that the long-term load loss was due principally to corrosion of the tendons has proved unfounded (Portier, 1974). Gosschalk & Taylor (1970) describe various aspects of 2 740kN anchors (fixed anchor length 5.6-5.9m, diameter = 140mm) installed in quartzite at Muda Dam, Malaysia. The stressing procedure involved stressing to 3 030kN, followed by two complete loads unload cycles. The residual load was measured at seven days, and was found to have dropped by up to 450kN. Restressing to 3 030kN resulted in all loads being above 2 887kN three days later. Subsequently 25% of the anchors were monitored, and were found to have "remained fairly steady" as shown in Fig. 26. Measured settlements of the anchorage blocks at service were considered negligible.

The long-term performance of anchors designed for service in other applications has also been briefly recorded. Comte (1965) describes 1 250kN BBRV anchors in very fissured argillaceous schist in the Pendaz Cavern and recorded losses of 4-8%—notably less than the 10% margin allowed. The greater part of this loss was found to occur in the very early stages of a five year period of observation.

In the course of stressing two test anchors (fixed anchor length = 9m, diameter = 99mm), Barron et al (1971) subjected one to three loading cycles prior to lock-off, whereas the other was loaded directly to the lock-off load. Both were installed in jointed granite, the elastic modulus of which was 40-50 times less for the mass (0.15 ± 0.04 × 10^6 N/mm^2) than for the material 6.3 × 10^6 N/mm^2).

The load on the first anchor remained stable throughout the observation period, whereas this stable state was only achieved in the second anchor after marked loss in the first week (Fig. 27). This difference in behaviour was ascribed to "time-dependent behaviour of the rock under load, causing closing of fissures etc". They concluded that it is advisable to preclude the load up

### TABLE IX. LOSS OF ANCHOR LOAD WITH TIME FOR SELECTED ANCHORS AT THE JOHN HOLLIS BANKHEAD DAM, ALABAMA (After Thompson, 1969)

| Anchor No. | Free Initial Total Residual Time Load % Load (m) load (kN) | (m) load (kN) | elapsed load (kN) | (kN) |
|------------|---------------|---------------|-----------------|-----------------|-------|
| 1          | 35            | 3336          | 206mm           | 3336            | 16 hrs | 0     | 0     | 0     |
| 2          | 35            | 3363          | 206mm           | 3278            | 18 hrs | 85    | 2.5   |
| 3          | 35            | 3278          | 206mm           | 3220            | 19 hrs | 58    | 1.8   |
| 4          | 35            | 3363          | 214mm           | 3278            | 31 hrs | 58    | 1.7   |
| 5          | 35            | 3363          | 210mm           | 3336            | 5 & 10 days | 27    | 0.8   |
| 6          | 29            | 3363          | 217mm           | 3163            | 5 & 10 days | 200   | 6.0   |
to its maximum level for several cycles, in order to minimise load loss after lock-off. There would appear to be a temperature effect on the apparent load—but this could be due to the susceptibility of the load cells to temperature variation.

Moschler and Matt (1972) presented data on the performance of a 1330kN VSL anchor (fixed anchor length 4.50m) after testing load to 1725kN in fractured calcareous schist in the Waldeck Cavern. This is shown in Fig. 28, in which the theoretical steel relaxation curve is also plotted. As noted previously, one of the largest scale anchor performance programmes described (McLeod & Hoadley, 1974) involved the placement of load cells under 100 anchors (diameter = 76mm) installed in Silurian mudstone in Melbourne. The maximum working load was about 900kN with most locked-off at 250-300kN, following a test load of 1.4T. Of the results considered satisfactory, the average load loss after 3-6 months was 9%, but 80% of the anchors had an average loss of only 5%. The rather higher apparent losses in the other anchors may have been due to instrument malfunction. On a second site where more care was taken with the load cells, the average loss after one month was only 1%, with no large losses recorded in that time. The authors concluded that in general load loss can be expected, normally 5-10%, but occasionally up to 20%.

One of the most informative case histories has been published by Hutchinson (1970). Six rows of anchors were installed into Upper Chalk on the Isle of Thanet to stabilise a cliff face (Fig. 29a). With a factor of safety on the ultimate chalk-grout bond (0.8N/mm²) of 3.75, the fixed anchor lengths ranged from 5-8m (hole diameter = 102mm) to provide working loads from 167 to 265kN.

The anchors were initially locked-off at 1.25 Tₚ and checked ten days later when they were restored to the designed initial values. The maximum recorded load in this time was 14% in one of the upper rows of anchors (in the poorest quality chalk). Load restoration was repeated three times on all anchors, after which all but one in each row were finally grouted up and locked off. The remaining six anchors were monitored over 1.1 years, and the results after that time are shown in Fig. 29b. A maximum loss of 16% was recorded in the uppermost anchor.

Hutchinson considers his data provide a good correlation between chalk quality and load loss, and it is noteworthy that in the good quality chalk, the interfacial safety factor employed in design was associated with insignificant creep loss.

**Remarks**

The quality and accuracy of information published on the time-dependent behaviour of steel tendons would appear to be wholly suitable for application to rock anchor systems. On the other hand, the authors find that too few long-term records of actual field behaviour provide sufficient data about anchor load and geometry, and rock classification. One important consequence is that optimum overload allowances cannot be determined to accommodate long-term losses.

However, it is evident that cyclic pre-loading may eliminate creep during service, choice of a large interfacial safety factor may inhibit creep, and restreassable anchor blocks can be used to compensate for creep.

**General conclusions**

In the field of rock anchors the quality of workmanship during construction greatly influences subsequent performance of the anchor. In addition, rock anchors are often spaced at close centres, and the normal initial investigation programme cannot highlight, on such a small scale, subtle variations in rock quality which will affect the behaviour of individual anchors.

As a consequence, it is strongly recommended that each anchor should be subjected to an initial proof loading stage. Whilst it is fully appreciated that stressing is a skilled operation, and that considerable judgement must be exercised when analysing the results of the operation, only in this way can the safety of each anchor be ensured.

Bearing in mind the rapid growth of ground anchor technology, specialists should be aware of possible conflicts between new design concepts and existing code recommendations. For example, BS 4447 stipulates a 92% efficiency for the head relative to the tendon GUTS, although the minimum load rating factor in current design is related directly to tendon f.p.u. As a result, BS 4447 may well be stipulating a lower rating factor than those actually specified (see Table XV, Part I).

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gravity foundation

Further settlement of the tower. This process was accelerated by a storm on Mar. 12 and 13, 1976 when southerly winds of 80-90 mph hit the tower producing waves of up to 6 m. Following this storm the tower was measured to be upright to within 12 m of arc and to have settled to about 1.4 m above the original level. Unfortunately the contractor's engineer does not afford sufficient freeboard under these conditions.

future programme

The tower is to be raised by buoyancy tanks. The tower is then set on piles to an undisturbed part of the seabed. At the new site an inverted filter of sand and gravel to the design of the Hydraulics Research Station will be placed around the piling to provide scour protection for the edge. Some of the piezometers in the sand and it will then be possible to put the data module on the tower. It is hoped that the next severe storm will not occur until after the data module is put down, to be able to record the behaviour of the tower.

A more detailed Paper, giving details of the measured sea state and probable forces on the tower, is being prepared.

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References


