
British Standard Code of practice for

Ground anchorages

Code de bonne pratique des ancrages au sol

Leitfaden für Bodenanker

Foreword

This British Standard has been prepared under the direction of the Civil Engineering and Building Structures Standards Policy Committee. The code supersedes DD 81 : 1982 which is withdrawn.

The code contains material which is both for the information and guidance of engineers and material which forms recommendations on good practice. As such conformity with its recommendations is not obligatory and variations from its recommendations may well be justified in special circumstances and engineering judgement should be applied to determine when the recommendations of the code should be followed and when they should not.

A code of practice is intended for the use of engineers having some knowledge of the subject. It embodies the experience of engineers successfully engaged on the design and construction of the particular class of works so that other reasonably qualified engineers may use it as a basis for the design of similar works.

It is not intended that it should be used by engineers who have no knowledge of the subject nor that it should be used by persons who are not engineers.

A code of practice represents good practice at the time it is written and inevitably, technical developments may render parts of it obsolescent in time. It is the responsibility of engineers concerned with the design and construction of works to remain conversant with developments in good practice, which have taken place since publication of the code.

Ground anchorage techniques have developed dramatically over the past twenty years, to such an extent that the use of anchorages is widespread for both temporary and permanent applications throughout Europe and overseas. Not only has the number of anchorage installations increased but the range of applications has widened considerably, ranging from dam stressing and reinforcement of underground excavations in rock to tying back earth retaining structures and holding down tower and bridge foundations. For these applications, anchorages can be employed to solve problems involving direct tension, sliding, overturning, dynamic loading and ground pre-stressing, which in turn demand a variety of design and testing requirements.

Although ground anchorage technology continues to develop there has been a marked standardization of practice since 1982. In that year DD 81 was published and following a period of two years when the Draft for Development was employed in industry, the BSI Committee

CSB/22 was reconstituted to review the Draft for Development together with comments from users. In converting DD 81 into this British Standard the Committee has shortened the main text for simplicity, guidelines on the responsibilities and duties for anchorage designers and contractors are introduced, new classes of corrosion protection are proposed and testing procedures and criteria have been simplified. Rock bolts are also included following publication of the Construction Industry Research and Information Association (CIRIA) report *A guide to the use of rock reinforcement in underground excavations* in 1983. Anchorage design clauses in the main text retain recommendations on traditional safety factors, since more limit state design experience is required before optimum partial safety factors can be determined. SI units are used throughout.

The full list of organizations that have taken part in the work of the Technical Committee is given on the back cover. The constitution of the committee is unusual in that it includes an exceptionally large number of coopted members. The chairman of the committee is Professor G S Littlejohn and the personal contribution of the following is particularly acknowledged:

Mr P A G Andrews
Mr L J Arthur
Mr A D Barley
Mr J Burch
Dr D A Greenwood
Mr M G Hubbard
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Mr M S Wright

It has been assumed in the drafting of this British Standard that the execution of its provisions is entrusted to appropriately qualified and experienced people for whose guidance it has been prepared.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

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Code of practice

0 Introduction

In recent years, new applications for ground anchorage systems have continued to appear regularly and today anchorages may be associated with retaining walls, dry docks, cofferdams, storm-water tanks, concrete gravity dams, tall buildings, suspension or arch bridges, tension roofs, pile and plate loading tests, transmission towers, radio masts, ski jumps, cliff stabilization, open mine pits, shafts, tunnels, underground caverns, pipelines and oil platforms (Littlejohn 1982b).

Equally significant but perhaps more striking have been the developments in anchorage construction and descriptive terms such as multi-underream, lost point, straight shaft, compression tube, end plate, rotating plate, multi-helix, inflatable membrane, expandable wedge or plug, continuous auger, tube à manchette, resin capsule and resin injection indicate the number of techniques now available.

It is considered that if reliable performances are to be maintained, a technical appraisal of anchorage systems is required by the practising engineer, in addition to the routine comparisons on the basis of cost and duration of the contract. There is a need for a detailed knowledge of the ground, which may demand a supplementary investigation and a proper design related to static and dynamic loads, location of anchorages, load transfer lengths and overall stability. Bearing in mind the current trend towards the use of limit state design, account has to be taken of loads and accompanying deformations under service conditions, as well as the deformation mechanisms developed at failure. For temporary or permanent works, the corrosion protection applied to the anchorage has to be adequate for the circumstances.

With regard to anchorage construction, the importance of skilled operatives cannot be over-emphasized, since the quality of workmanship greatly influences subsequent performance. This workmanship factor limits the ability to predict anchorage performance accurately solely on the basis of empirical rules and ground investigation data.

Quality controls and record keeping are therefore strongly recommended during the construction stage and further, each anchorage, once installed, should be subjected to an initial proof load greater than the required working load followed by a check on the residual load after a short period of service. In this way, the safety and satisfactory performance of each anchorage should be ensured. For the recommendations on performance of anchorages and anchorage components related to their use, reference should be made to clause 11 of this standard.

1 Scope

This British Standard code of practice provides recommendations and guidance for soil and rock anchorage systems of the grouted or mechanical type and whilst emphasis is placed on the post-tensioned anchorages to reflect current trends and usage in the construction industry, many of the recommendations apply to passive anchorages.

Tension piles, deadmen, reinforced soil and soil nailing are excluded from this standard, since these subjects are covered by BS 8004, Civil Engineering Code of Practice No. 2 (1951)*† and the scope of BSI Committee CSB/56. In addition, driven anchorages of the duck-bill earth type, whilst used for temporary purposes are not addressed in this standard.

In relation to the ground anchorage systems covered, this code of practice also provides recommendations and guidance for site and ground investigation requirements, design methods and philosophy, corrosion hazards and protective measures, construction techniques and quality controls, stressing procedures, testing of anchorage components and complete installations, acceptance criteria, maintenance during service and legal aspects. In addition, comments on potential health and safety hazards have been incorporated throughout.

This code of practice attempts to deal adequately with all important and relevant aspects of the subject but where there are shortcomings in our present knowledge and no firm guidance can be given on a particular section, this situation is highlighted in the hope that it will be drawn to the attention of research institutions (see appendix A).

A bibliography, containing details of both the references in the text and other relevant publications, is given in appendix P.

NOTE 1. Three review publications by Littlejohn and Bruce (1977) CIRIA (1980) and Hobst and Zajic (1983) are recommended for study.

NOTE 2. The titles of the publications referred to in this standard are listed on the inside back cover.

2 Definitions

For the purposes of this code of practice the definitions given in BS 6100 : Section 1.6 apply together with the following.

2.1 accuracy

2.1.1 absolute accuracy. The deviation from the true value, i.e. where the measuring instruments have been calibrated against dead weight apparatus or loading machines and the accuracy is known.

2.1.2 relative accuracy. The deviation from the measured value, i.e. the error in measurement where small changes in load or displacement are monitored against time.

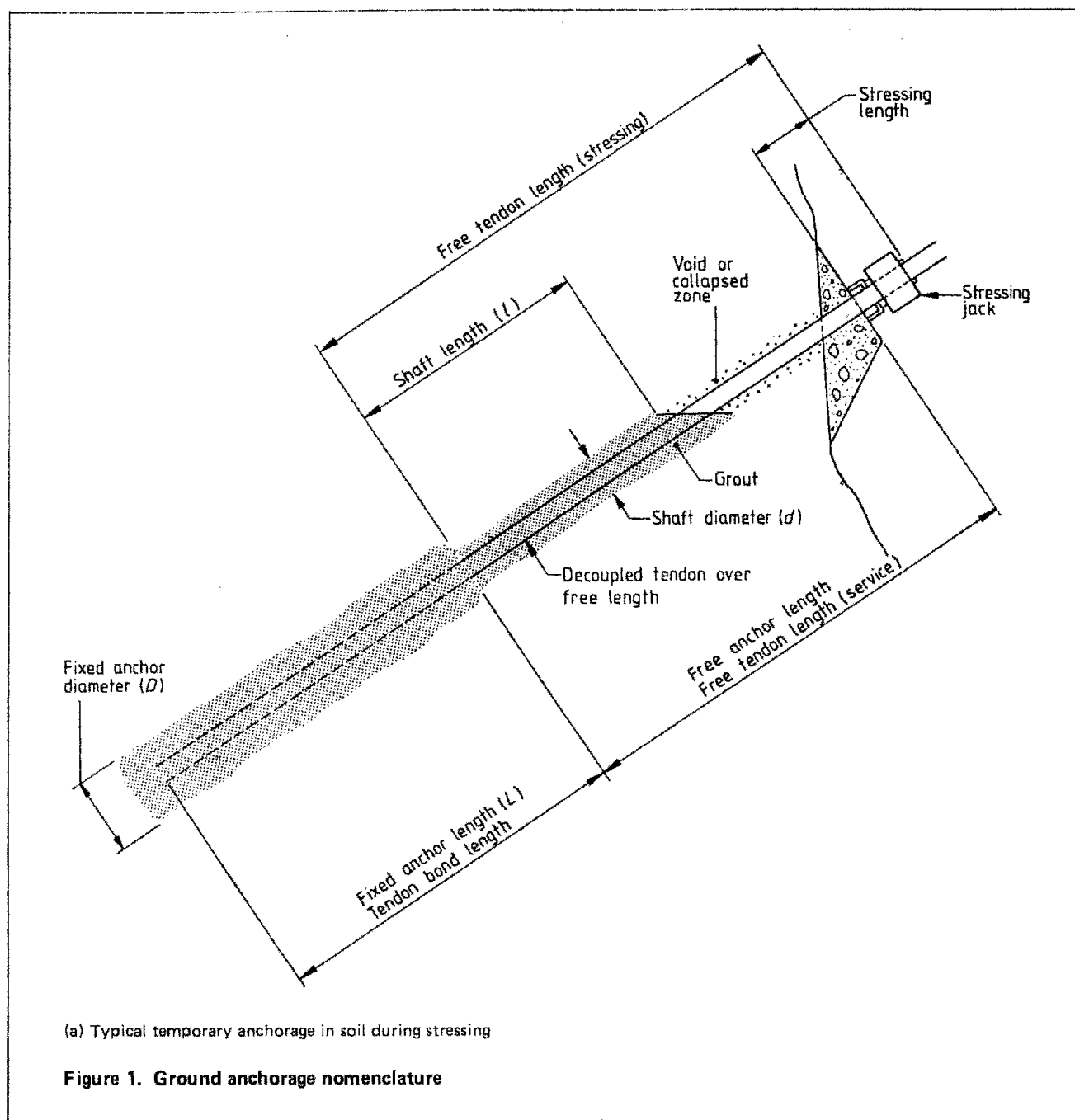
2.2 anchorage

2.2.1 ground anchorage. An installation that is capable of transmitting an applied tensile load to a load bearing stratum. The installation consists basically of an anchor head, free anchor length and fixed anchor (see figure 1).

2.2.2 permanent anchorage. An installation that is required to ensure the stability and satisfactory service performance of the permanent structure or excavation being supported (see table 2).

* Under revision.

† Available from the Institution of Structural Engineers, 11 Upper Belgrave Street, London SW1.



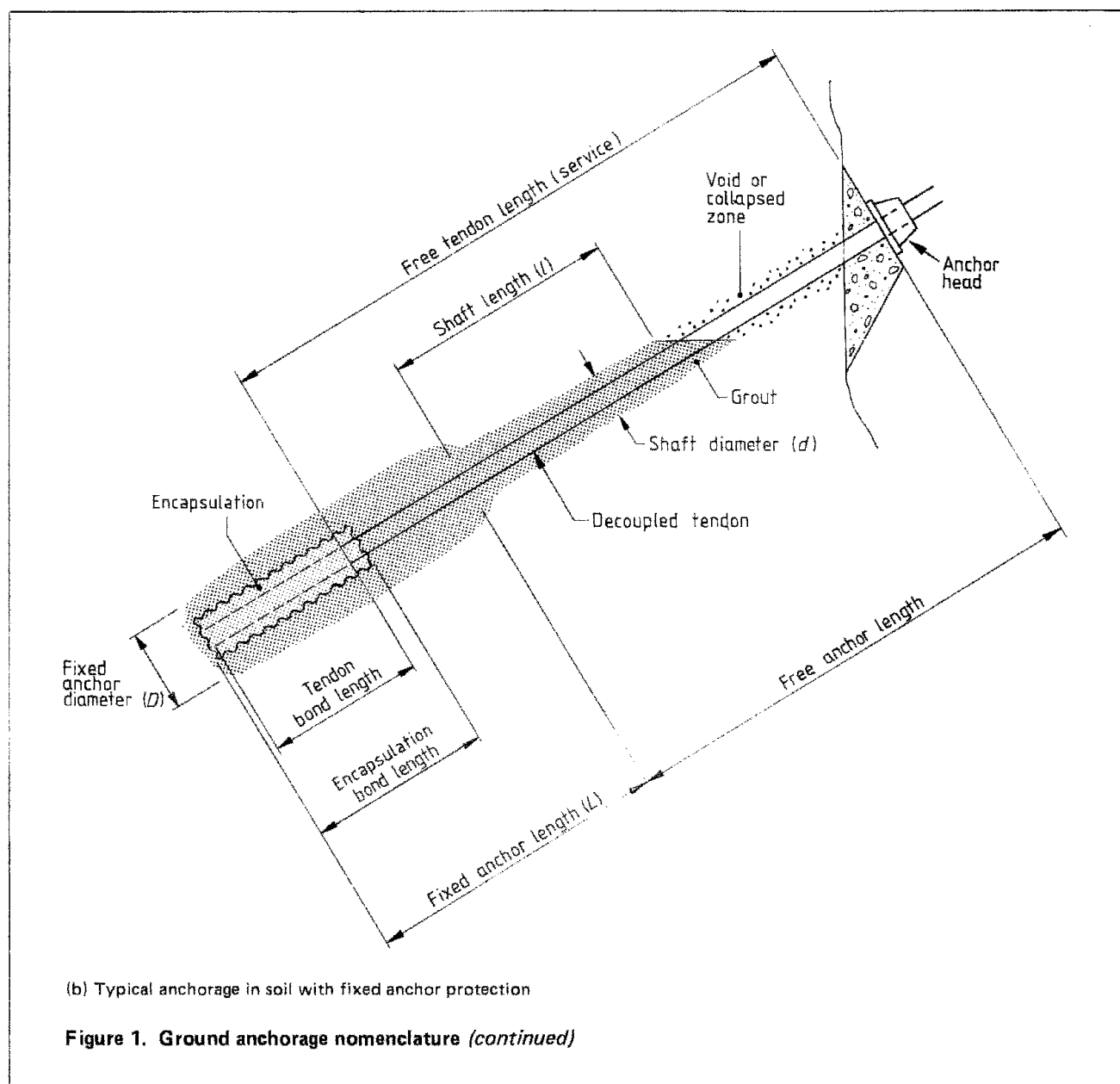
2.2.3 temporary anchorage. An installation that is often used during the construction phase of a project to withstand forces for a known short period of time, usually less than 2 years, (see table 2).

2.2.4 anchor. This term is used exclusively in this standard to denote a component of the anchorage i.e. anchor head and free anchor length.

2.2.5 anchor head. The component of a ground anchorage that is capable of transmitting the tensile load from the tendon to the surface of the ground or structure requiring support, (see 7.4).

2.2.6 detensionable type anchor head. An anchor head that has all the properties of the restressable head (see 2.2.8) and in addition permits the tendon to be detensioned in a controlled way at any time during the life of the structure.

2.2.7 normal type anchor head. An anchor head that is designed to permit the load in the tendon to be raised or lowered within the limits $0\% f_{pu}$ to $80\% f_{pu}$ (see 2.8) and measured, when necessary to comply with the requirements of acceptance testing. This facility is available during the initial stressing phase. If the tendon beyond the anchor head is cut, further measurement or adjustment is not possible.



2.2.8 restressable type anchor head. An anchor head that has all the properties of the normal head (see 2.2.7) and in addition permits the tendon, throughout the life of the structure, to be measured by check lifting and small losses, up to 10 % of working load, to be recovered by shimming or thread-turning, (see figures 24, 25, 37 and 38).

2.3 bearing plate. A bearing plate under the stressing head distributes the tendon force into the main structure (see 7.4.3).

2.4 bond (see 6.3)

2.4.1 adhesion bond. The initial bond before slip that arises mainly from the physical interlocking, i.e. gluing, of the microscopically rough steel and the surrounding grout.

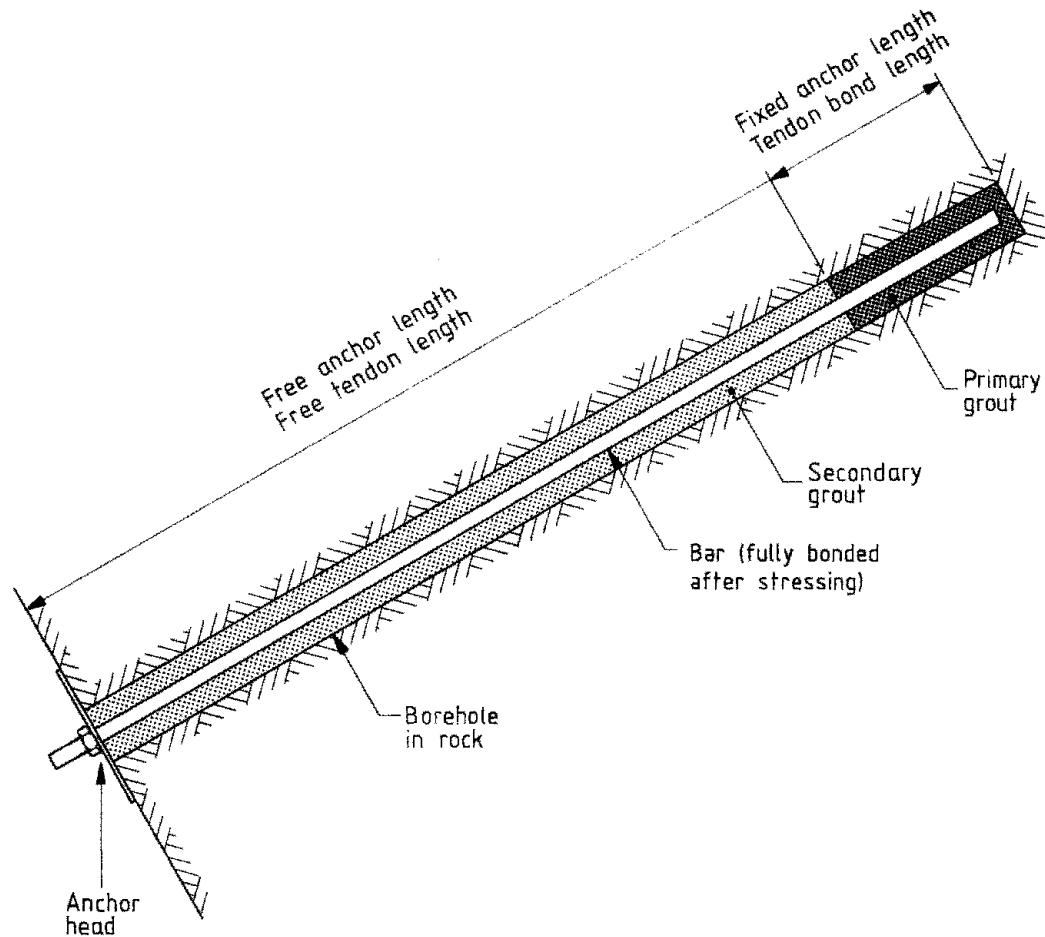
NOTE 1. Molecular attraction is also thought to act.

NOTE 2. Adhesion bond is considered to disappear when slip comparable with the size of the micro-indentations on the steel occurs.

2.4.2 friction bond. The bond that depends on the confining pressure, the surface characteristics of the steel and the amount of slip but is largely independent of the magnitude of the tendon stress. The phenomena of dilatancy and wedge action also contribute to this frictional resistance as radial strains are mobilized where the longitudinal strain changes.

2.5 borehole diameter. The diameter of the borehole assumed in design for the free length and fixed length prior to forming any enlargement.

2.6 capsule. A prepackaged grout for rock bolts. Capsules may be of various sizes, one or two component, and be resin or cement based.



(c) Typical rock bolt fully bonded over free tendon length

Figure 1. Ground anchorage nomenclature (continued)

2.7 centralizers. Devices to maintain the tendon centrally in the borehole (see 7.3.5 and figures 13 and 14) or within the encapsulation.

2.8 characteristic strength. The value of the cube strength of grout or concrete (f_{cu}) or the ultimate load of a prestressing tendon (f_{pu}) below which not more than 5 % of the test results fall.

2.9 compression anchor. A fixed anchor in which the tensile force is transferred to the distal end of the fixed length.

NOTE. The transfer device may take a number of forms of which the most common are either a plate washer at the distal end, or a concentric tube restrained similarly at the distal end. This is sometimes referred to as a compression tube anchor.

2.10 creep. Creep is the movement of any part of the fixed anchor with time under constant tendon load.

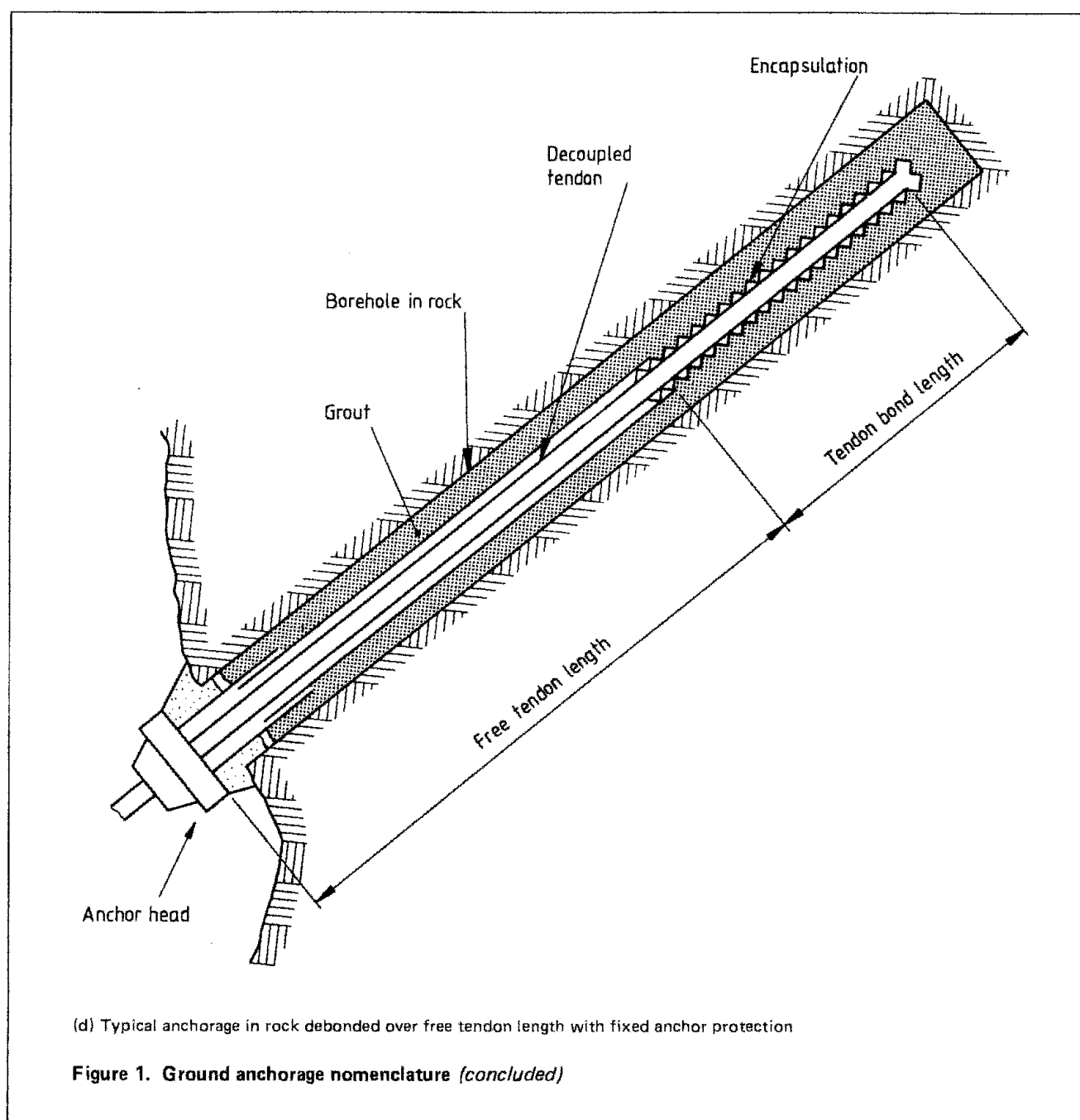
NOTE. Creep can occur at both grout/tendon and the ground/grout interfaces.

2.11 debonding. The breakdown of adhesion bond and friction bond at the grout/tendon or ground/grout interfaces.

2.12 decoupling. The separation of components when tendons are separated from the surrounding ground by grease and sheathing. The term also relates to the breakdown into subdivisions of the grouted fixed anchor or encapsulation due to the presence of spacers or centralizers that inhibit the flow of grout during the injection process or create artificial boundaries, thereby producing a discontinuous medium.

2.13 distal. Situated furthest from the anchor head.

2.14 distributed stress transfer (dts) anchor. A fixed anchor comprising several decoupled strands of differing lengths each coupled only at its distal end such that the tendon load is constrained to be transmitted to the ground through discrete lengths uniformly distributed at intervals along the fixed anchor length.



2.15 duct. A tube or covering used in corrosion protection, which in current practice is usually of semi-rigid plastics and may be smooth or corrugated.

A preformed hole.

2.16 encapsulation. A preformed corrosion protection applied to the bond length of the tendon.

2.17 enlargement. A concentric local increase in the borehole diameter within the fixed length of the anchorage (see 6.2.2.5).

NOTE. This is also called underream or bell.

2.18 grout

2.18.1 cementitious grout. Grout made from cement products as defined in 7.1.1.

2.18.2 primary grout. The grout placed or injected before or after tendon homing and prior to stressing, to secure the fixed anchor to the surrounding ground.

2.18.3 resinous grout. Grout made from materials complying with the requirements of 7.2.1.

2.18.4 secondary grout. The grout injected after stressing to bond and/or protect the free length of the tendon.

NOTE. With the development of restressable anchorages where the free length is decoupled, e.g. greased and sheathed, the complete injection may be carried out in a single stage using primary grout.

2.19 hydrofracturing. The process of high pressure injection of grout in the borehole such as to cause fracturing of the ground adjacent to the hole, so that grout travels extensively in the fissures in a non-uniform way (see 6.2.2.4 and 10.4.4).

2.20 length

2.20.1 fixed anchor length. The designed length of the anchorage over which the tensile load is capable of being transmitted to the surrounding ground, (see figures 19 and 22).

NOTE. This may be the same as the tendon bond length.

2.20.2 free anchor length. The distance between the proximal end of the fixed anchor and the anchor head.

2.20.3 apparent free tendon length, is calculated from the load/elastic displacement data following testing, (see 11.2.9) to indicate the length of tendon which is apparently fully decoupled from the surrounding ground or grout.

2.20.4 encapsulation bond length. The overall length of the outer perimeter of the encapsulation, which is bonded to the surrounding grout in the case of a protected anchorage.

2.20.5 shaft length. The length of grout filled hole in front of the proximal end of the fixed anchor. The grout is often placed prior to stressing and therefore is capable of mobilizing resistance to withdrawal.

2.20.6 free tendon length. The designed length of the tendon that is decoupled from the surrounding ground or grout during stressing.

NOTE. During the initial stressing operation, the free tendon length may be extended by the stressing length, depending on the type of stressing system employed (see figure 1(a)).

2.20.7 tendon bond length. The length of tendon that is bonded directly to the grout and capable of transmitting the applied tensile load.

2.21 load

2.21.1 lift-off load. The minimum load monitored during a restressing operation that permits a locking nut to turn on a bar tendon or provides a clearance or lift (see 10.6.3.2) in the case of a wire or strand tendon.

2.21.2 lock-off load. The load transferred to the anchor head immediately on completion of a stressing operation.

2.21.3 proof load. The maximum test load to which the anchorage is subjected during the initial stressing phase.

2.21.4 proof load factor. The ratio of proof load to working load (see table 2).

2.21.5 residual load. The load remaining in the anchorage at any time during service.

2.21.6 ultimate load holding capacity. The minimum load (T_f) which causes loss of equilibrium in a part or whole of the ground anchorage (see 6.2.1).

NOTE. Ultimate load holding capacity is also referred to as pull out capacity.

2.21.7 working load. The safe load (T_w) of the anchorage.

2.22 mechanical anchor. A mechanical device attached to the distal end of a rock bolt which, when expanded against the sides of the borehole, generates friction to provide restraint for the tensile load.

2.23 mechanical interlock. The bond similar to micro-mechanical locking but on a much larger scale, as the shear strength of the grout is mobilized against major tendon irregularities, e.g. ribs or twists (see 6.3.2).

2.24 noding.

2.24.1 tendon noding. Where a tendon comprises a number of strands or wires, and the clearance is controlled by spacers, the spacing between the tendon units may be reduced at an intermediate point between the spacers along the tendon bond length by the use of smaller spacers or by banding the units together. The reduction in tendon diameter at intervening points creates a wave effect or general noding which improves mechanical interlock.

2.24.2 strand noding. A strand node is formed by unravelling the helical wires, fitting a small steel washer around the core or king wire, so that when the helical wires are returned to their former lay, these wires remain apart. Strand or local noding is formed at regular intervals along the tendon bond length to improve mechanical interlock.

2.25 pitting. The condition where the rate of corrosion into a metal from the surface is significantly greater than the rate of attack parallel to the metal surface. A width/depth ratio of one is commonly recognized for a pit (see J.2.3).

2.26 protection. Components of the anchorage may be protected physically by one or more continuous impermeable barriers to corrosion. These are often described as single or double protection, (see 7.3.4), implying one or two physical barriers, respectively.

2.27 proximal. Situated nearest to the anchor head.

2.28 reinforcement

2.28.1 primary reinforcement. A term used for rock anchorages installed in underground excavations to maintain overall stability of the excavation.

2.28.2 secondary reinforcement. A term used for rock anchorages (generally low capacity rock bolts) installed in underground excavations to overcome localized stability problems, e.g. spalling or surface instability. Secondary reinforcement may be used as the sole reinforcement or in conjunction with primary reinforcement.

2.29 relaxation. The decrease of stress or load with time while the tendon is held under constant strain (see 7.3.2.4.2).

2.30 resin

2.30.1 resin anchorage. An installation where resin, generally polyester, is used to transfer the load from the tendon to the ground, particularly when rapid stressing is required.

2.30.2 full column single speed resin type anchorage. A resin anchorage that has the bar fully bonded and is not suitable for subsequent stressing.

2.30.3 two-speed resin type anchorage. A resin anchorage that utilizes fast and slow setting resin capsules that allow the bar to be stressed against the fast setting fixed anchor zone and subsequently fully bonded by the slow setting resin filling the remainder of the hole.

2.31 rock bolt. A specific form of ground anchorage tensioned during installation, where a bar of steel or other appropriate material is fixed in rock (see figure 1(c)).

2.32 rock dowel. A specific form of untensioned ground anchorage where a bar of steel or other appropriate material is fixed in rock.

2.33 safety factor. The ratio of the ultimate capacity to the working load used for design (see table 2).

2.34 sheath. A flexible plastics covering to the tendon.

2.35 skin friction. A term commonly used in practice to define the ultimate bond value at the ground/grout interface of the fixed anchor (see 6.2).

2.36 slenderness ratio. The vertical depth (or normal distance for steep slopes) from the ground surface to the top of the fixed anchor divided by the diameter of the fixed anchor for downward inclined anchorages.

2.37 spacers. In multi-unit tendons spacers ensure separation between individual components of the tendon (see 7.3.6).

2.38 stressing head. The component of the anchor head assembly that attaches the tendon to the anchor head, e.g. nut for a rock bolt or steel plate with tapered holes to accommodate wedges in the case of a wire or strand tendon (see 7.4.2).

2.39 tendon. The part of an anchorage that is capable of transmitting the tensile load from the fixed anchor to the anchor head (see 7.3).

2.40 tube à manchette. Equipment used to permit multi-phase high pressure grouting or regrouting along the fixed anchor length (see 6.2.2.4 and 10.4.2.2).

2.41 uniformity coefficient. The ratio of D_{60} to D_{10} , where D_{10} is the size such that 10 % (by weight) of the sample consists of particles having a smaller diameter and D_{60} is the equivalent size for 60 % of the sample.

3 Symbols

The following symbols are used in this code of practice.

Symbol	Description	Unit
A	Ratio of contact pressure at the fixed anchor/soil interface to average effective overburden pressure	—
A_t	Cross-sectional area of tendon	mm ²
B	Bearing capacity factor equivalent to $N_{q/1.4}$	—
C_a	Shaft adhesion	kN/m ²
C_p	Reaction due to wall adhesion	kN
C_u	Undrained shear strength	kN/m ²
C_{ub}	Undrained shear strength at proximal end of fixed anchor	kN/m ²
C_{us}	Average undrained shear strength over the shaft length	kN/m ²
c	Cohesion of ground	kN/m ²
c'	Effective cohesion of ground	kN/m ²
D	Diameter of fixed anchor or enlargement	m
D_r	Relative density	%
d	Diameter of shaft or neck between enlargements	m
E	Elastic modulus	kN/m ²
E_c	Elastic modulus of concrete or grout	kN/m ²
E_R	Elastic modulus of rock	kN/m ²
E_s	Elastic modulus of steel strand	kN/mm ²
E_t	Elastic modulus of steel tendon	kN/mm ²
f	Friction coefficient ($f = \tan \phi'$)	—
f_{cu}	Characteristic compressive strength of concrete or grout	N/mm ²
f_{pu}	Characteristic tensile strength of tendon	kN
f_s	Efficiency factor for undrained shear strength along the shaft	—
f_u	Efficiency factor for undrained shear strength along underreamed fixed anchor	—
H	Depth of excavation	m
h	Depth to top of fixed anchor	m
h_r	Depth of rock to fixed anchor (measured along the axis of the anchorage when inclined)	m

Symbol	Description	Unit	Symbol	Description	Unit
I_c	Consistency index	—	T_x	Load in anchorage at which tendon displacement (measured at the anchor head) on restressing crosses the previous distressing displacement curve	kN
K	Coefficient of earth pressure	—	T_y	Yield strength of tendon	kN
K_O	Coefficient of earth pressure at rest	—	t	Time	minutes
K_Δ	Creep displacement	mm	V_c	Volume of rock cone	m ³
k_w	Coefficient of permeability of the ground	m/s	W	Weight of sliding wedge or block	kN
L	Length of fixed anchor	m	W_n	Component of self weight acting normal to failure surface	kN
L_L	Liquid limit	%	W_O	Overturning self weight	kN
l	Length of shaft	m	W_s	Stabilizing self weight	kN
m	Natural moisture content of the ground	%	W_t	Component of self weight acting tangentially to sliding surface	kN
N	Number of blows per 0.3 m or force acting normally to failure surface kN	—	α	Adhesion factor	—
N_x, N_y	Number of anchorages	—	β	Angle of inclination of potential plane of failure	degrees
N_c	Bearing capacity factor	—	γ	Unit weight of ground	kN/m ³
N_q	Bearing capacity factor	—	γ_{fl}	Partial safety factor for loads	—
n	Empirical factor related to anchorage installation technique and ground conditions	—	γ_m	Partial safety factor for strength of materials	—
q	Surcharge	kN/m ²	γ_w	Unit weight of water	kN/m ³
P_a	Active earth force	kN	δ	Angle of wall friction	degrees
P_L	Plastic limit	%	δ_u	Spacing of underreams	m
P_n	Nominal active earth force	kN	δ_w	Settlement behind wall	mm
P_p	Passive earth force	kN	Δ	Crest displacement or fixed anchor displacement	mm
pH	Degree of acidity or alkalinity	—	ΔL	Width of ground strip over sliding surface	m
p_i	Grout injection pressure	kN/m ²	ΔW	Self weight of ground strip over sliding surface	kN
R	Resisting force acting on failure surface	kN	ΔW_n	Component of self weight of ground strip normal to sliding surface	kN
R_n	Nominal resisting force acting on failure surface	kN	ΔW_t	Component of self weight of ground strip tangential to sliding surface	kN
S	Shearing resistance acting tangentially to failure surface	kN	$\Delta V_x, \Delta V_y$	Volume of overlapping cones	m ³
S_f	Load factor of safety	—	Δ_c	Volume of single rock cone	m ³
T	Anchorage force	kN	Δ_e	Elastic displacement of tendon at initial residual load	mm
T_f	Ultimate load capacity of anchorage	kN	Δ_x	Displacement of tendon at anchor head at load X	mm
T_k	Limit load for acceptable creep	kN	Δ_{xe}	Elastic displacement of tendon at anchor head at load X	mm
T_{lo}	Load in anchorage at lock-off	kN	Δ_{xp}	Permanent displacement of tendon at anchor head at load X	mm
T_{lt}	Long term residual load in anchorage	kN	θ	Angle of inclination of anchorage	degrees
T_n	Component of anchorage force acting normal to failure surface	kN	τ	Shear strength of rock	kN/m ²
T_t	Proof load to which anchorage tested or component of anchorage force acting tangentially to failure surface	kN			
T_w	Working load of anchorage	kN			

Symbol	Description	Unit
τ_M	Theoretical skin friction	kN/m ²
τ_s	Long term skin friction	kN/m ²
τ_{ult}	Ultimate bond or skin friction at rock/grout interface	kN/m ²
τ_x	Bond stress or skin friction mobilized at rock/grout interface at distance x from proximal end of fixed anchor	kN/m ²
σ'	Average effective overburden pressure adjacent to fixed anchor	kN/m ²
σ'_E	Effective stress normal to proximal end of fixed anchor	kN/m ²
σ_{gw}	Hydrostatic ground water pressure	kN/m ²
σ_{sr}	Stress due to soil reaction	kN/m ²
σ_{tw}	Stress due to working load of anchorage	kN/m ²
σ'_v	Average effective overburden pressure adjacent to fixed anchor	kN/m ²
ϕ'	Effective angle of shearing resistance	degrees
ϕ'_n	Nominal effective angle of shearing resistance	degrees

4 Responsibilities

To ensure that ground anchorages perform satisfactorily during their intended life, the degree of safety and deformations under appropriate loading conditions should be assessed in accordance with design criteria set out within this code of practice. To ensure satisfactory performance relating to durability, serviceability and safety the qualities of materials and workmanship should be verified by inspection and testing. This approach is similar to BS 8110 : 1985.

Ground anchorages form only one part of the design process for an anchored structure. It is therefore essential that the design responsibilities of the designer and specialist anchorage contractor are defined properly in the contract documents. Table 1 shows an appropriate separation of duties. The table has been prepared based on the assumption that it is only the designer who is in a position to judge how the ground anchorage will interact with the ground/structure system. The designer is also the most appropriate person for judging the degree of risk in the use of ground anchorages and is the person who obtains agreement from the client to employ ground anchorages. This is very important bearing in mind the continuing rapid development and innovation in anchorage installation technology which is dominated by empirical design methods.

Table 1. Recommended design and construction duties

Designer*	Contractor
(1) Site investigation data for ground anchorages (borings near fixed anchor locations and outside the site working area if necessary). (2) Decision to use ground anchorages, required trials and testing and provision of a specification (Risk assessment). (3) Overall design of anchored structure, calculations of anchorage force required. Definition of safety factors to be employed. (4) Definition of anchorage life (permanent/temporary) and requirement for corrosion protection. (5) Anchorage spacing and orientation. Free anchor length and anchorage load. (6) Anchorage behaviour monitoring system, (structure/anchorages) and interpretation of results. (7) Supervision of the works (inspection by testing and sampling). (8) Maintenance specification for anchorages. (9) Instruction of all contracting parties of key items within design philosophy to which special attention should be directed.	(1) Anchorage components and details (2) Determination of fixed anchor dimensions (3) Anchorage spacing/orientation free anchor length and anchorage loads (instead of by the designer) (4) Detailing of the corrosion protection system for anchorage (5) Supply and installation of anchorage monitoring system (6) Quality control of works (7) Anchorage maintenance as directed by the designer
* The designer may be employed by the client, the main contractor, a specialist subcontractor or by a consultant. Contractual arrangements between the parties will vary with each contract and be specified in the contract document.	

The designer should also define the design life of corrosion protection required. This will help to ensure consistency in the proposals from contractors tendering for this work.

Should the designer consider that all these considerations are beyond his or her experience then the designer is strongly advised to seek the advice of a specialist consultant.

Alternative arrangements to those recommended in table 1 are possible but it cannot be emphasized too strongly that designer and contractor should be clear what their respective responsibilities and duties are and that these should be defined in the contract documents.

5 Site investigation

5.1 General

A ground anchorage system may be required:

- (a) as an integral part of the design concept of a project;
- (b) to solve a problem revealed by a ground investigation;
- (c) as a remedial or improvement measure to an existing structure.

The information on ground conditions available at the stage of considering the feasibility of a ground anchorage system will therefore depend on the initial requirement for the installation. Whilst there may be adequate data to indicate both the feasibility and advantages of an anchorage system, there may well be insufficient detailed information to permit its safe and economic design or construction. The geometry of a ground anchorage and its mode of operation requires, in particular, a detailed knowledge of ground conditions local to the fixed anchor. Minor variations in ground conditions can therefore assume a greater degree of importance than, for example, in foundation design.

The ground is one structural component of the ground anchorage system, therefore the importance of a good quality site investigation cannot be over-emphasized. Lack of adequate information on the ground remains the most common cause of individual anchorage failures at the proof loading stage.

Investigations are most satisfactorily undertaken in a number of stages (see 4.1 of BS 5930 : 1981). These can be summarized broadly as:

- (a) initial desk and field study;
- (b) main field and laboratory investigation;
- (c) investigation during construction.

The point of entry into such a programme of investigation will clearly depend upon the data available at the time of considering the feasibility of a ground anchorage system. Similarly, the work required in any one stage will be dependent on the nature of the project. For example, the consideration of a rock bolt system for a limited depth rock cut might be based predominantly on a visual field study, whilst that for a major retaining wall could require that the main field and laboratory investigation stage be split into preliminary and detailed phases.

The adoption of a stage programme of investigation also provides significant advantages where the constraints on the project are predominantly those of economy rather than time. Each stage can be undertaken after due consideration of the available data and following a clear commitment to proceed further with the study.

It should be noted that, with only a few exceptions, the data required for the safe design of temporary works is similar to that necessary for an installation that forms part of the permanent structure.

5.2 Initial desk and field study (see 4.2 and 4.3 of BS 5930 : 1981)

Any previous desk or field study should be checked to ensure that it has considered in adequate depth those details which might be useful for anchorage design.

Prior to the start of anchorage work, a pre-construction site report should be drawn up on the condition of the surrounding land, including buildings, streets and underground services, as appropriate. This report should be agreed by all parties concerned, who may include the client, designer, contractor and adjacent landowners or tenants.

5.3 Ground investigation

5.3.1 Extent and intensity of investigation (see clause 10 of BS 5930). By its very geometry, the behaviour of an anchorage is extremely sensitive to local ground conditions. The effect of an unexpected change of ground in the fixed anchor zone can be serious in terms of time and cost.

The aim of the investigation is therefore to determine, by the most economic means, the nature of the block of ground that is influenced by, or influences, the installation and behaviour of ground anchorages. Because inclined ground anchorages are installed as commonly as vertical anchorages, lateral variations in ground properties should be investigated as thoroughly as the more easily investigated vertical variations. Since investigation methods are often vertical, it may follow that an inclined anchorage system requires rather more boreholes than is usual for other structures, particularly in soils.

Where the fixed anchor length lies beneath an existing structure, the drilling of inclined boreholes may be possible. Whilst such a technique is within standard practice for rocks, practical difficulties generally preclude the boring of inclined investigation boreholes in soil. In this case, it may be necessary to employ supplementary investigation ahead of anchorage construction during the contract period (see 5.8.1).

The number of locations investigated and the depths to which the investigation has to extend, will be dependent on the type of project, the plan dimensions of the site (where this is defined so as to include the probable fixed anchor zone) and the data available from previous investigations or from the initial desk and field study (see 5.2). Whilst these variables preclude general rules, it is to be recommended that a sequential investigation is made under the supervision of an experienced geotechnical engineer or engineering geologist. Thus, for example, on a site for which little previous data are available, a number

of key boreholes should be sunk to identify the geological sequence and those strata that are of particular interest, either from the point of view of overall stability or for the design of the fixed anchor length. These key boreholes should be located at site extremities, so that the strata profile over the relevant zone can be initially interpolated between the boreholes rather than extrapolated outside the area investigated. The depth of these boreholes should be adequate to ensure that either:

- (a) a known geological formation is proved; or
- (b) no underlying stratum will affect design.

This preliminary work can then be followed by further boreholes and/or in situ tests, as applicable, to confirm the predicted strata sequence and obtain more detailed data on the strength and deformation characteristics of relevant strata. The number of locations investigated in this stage will depend upon the confidence that can be placed on the uniformity of the strata. Thus, for example, a greater number of locations would be required in glacial drift than would be necessary to confirm the level and properties of a well known solid geological formation.

Notwithstanding these considerations, it has to be emphasized that minor structural or fabric changes can occur in a known geological formation that can significantly affect the behaviour of a ground anchorage. It is therefore recommended that, for anchorages in soil, the maximum centres at which the ground is investigated in detail should not exceed 20 m, unless the geology and the characteristics of the relevant strata are well known. For anchorages in rock, the spacing of the locations requiring detailed investigation will depend on the nature of the problem and the available geological data. For example, assessment of the stability of rock faces in slopes and tunnels will demand a detailed geological investigation involving the location of almost every anchorage or group of anchorages. Where rock does not outcrop, the necessary spacing of the investigation locations will depend on the knowledge of the geology of the formation.

For retaining structures, the locations should be sited both along the line of the structure and, where practicable, along the line of the probable fixed anchor zone. Where long anchorages may be required, a number of locations should be sited at intermediate points to investigate the strata that will be penetrated by the drilling operations. Whilst it is not practicable to ensure the discovery of minor geological anomalies, the locations should so cover the site as to limit the possibility that a major geological feature, e.g. buried stream bed, can lie undetected between investigation positions.

In particularly difficult strata conditions, e.g. variable glacial drift, additional investigation may be required during the contract period at locations ahead of the construction (see 5.8.1).

Where rock is encountered during the investigation, the depth of penetration into this stratum should not be less than 5 m to ensure that it is bedrock and not a large boulder. In general, however, the encountering of bedrock at a shallow depth will imply the adoption of a rock rather than soil anchorage. It will then be necessary to extend the investigation to depths sufficient to prove rock of the

quality necessary to give adequate rock/grout bond strengths (see 6.2).

5.3.2 Methods of investigation. The most economical and effective method of obtaining the required data on ground conditions will depend on factors relating to the type of structure, the geological conditions and the information gained from the initial desk and field study.

The available techniques are detailed in BS 5930.

5.3.3 Sampling (see clauses 19 and 20 of BS 5930 : 1981). Whilst the available sampling techniques are described in BS 5930, particular emphasis should be placed on obtaining samples that can identify the fabric/structure of the stratum in which the fixed anchor length may be installed. In view of the relatively small diameter of most anchorage systems, minor differences in ground characteristics can be of significant influence in the behaviour of the anchorage/ground complex (see 5.1, 5.3.1, 5.7.1).

In soils, samples of a size and type suitable for examination and laboratory tests relevant to the nature of the soil should be taken from each stratum and at maximum intervals of 1.5 m in thick strata. Intermediate disturbed samples, suitable for simple classification tests, should also be obtained, so providing a specimen of the ground at a maximum of 0.75 m intervals of depth. In variable strata, continuous undisturbed samples may be necessary in the probable vicinity of the fixed anchor zone.

In rocks, emphasis should be placed on obtaining maximum continuous core recovery. In most rocks, this generally implies the use of proper equipment to obtain cores of not less than Nx size (55 mm) and, in weaker rocks, of larger diameter (see 19.8 of BS 5930 : 1981).

Samples of ground water should always be obtained in clean containers for subsequent chemical analysis (see 20.3 of BS 5930 : 1981).

5.3.4 Ground water (see clause 20 of BS 5930 : 1981). Determination of the ground water conditions on the site will be essential for the overall design of the project, particularly where excavations are proposed, and is of equal importance in the design of anchorage systems. This area of investigation, particularly the recording of long term ground water conditions, is too often given scant attention during routine site investigation works.

It has to be appreciated that the speed of boring, together with the addition of water to stabilize a borehole or as circulation fluid during drilling, usually precludes the measurement of equilibrium ground water conditions during the investigation period. Nevertheless, all observations of water conditions during boring or drilling should be carefully recorded throughout the progress of the investigation, as these often permit an initial assessment of true ground water conditions. For example, the percentage of circulation water return during rotary drilling may assist in a first appreciation of ground water level in a rock.

The long term ground water conditions can only be measured satisfactorily by the installation of standpipes or piezometers (see 20.2 of BS 5930 : 1981). Where ground water may be contained in several aquifers, particularly in impervious strata, it may be necessary to install piezometers at different levels to record the head in each aquifer.

5.4 Field testing

5.4.1 Tests from surface

5.4.1.1 Static cone penetrometer (see 23.3 of BS 5930 : 1981). When carried out under careful supervision this test can provide an accurate indication of the in situ properties of granular soils. The tests can be made quickly and the costs can be offset on a large site by using the equipment between a reduced number of boreholes (Meigh 1987).

The electric (standard or reference) cone is particularly useful in determining the presence of minor anomalies in apparently uniform soils, e.g. thin clay seams in a granular deposit or sand partings in a stratum of clay. Since these details are essential to anchorage design and construction, the use of the test is to be recommended whenever such problems are suspected.

5.4.2 Tests in boreholes

5.4.2.1 Standard penetration test (see 21.2 of BS 5930 : 1981). Standard penetration tests (SPT) should be made at regular intervals of depth in all granular strata in order to obtain an indication of their in situ density classification. In the probable zone of the fixed anchor length, tests should be made at least at 1 m intervals of depth in each borehole.

The in situ strength of cohesive soils can also be assessed by the test. The results provide a valuable check on laboratory data when made alternately with undisturbed sampling intervals (Stroud 1974; Meigh 1979; Stroud and Butler 1975). The test is particularly applicable where the soils possess a structure or fabric that may preclude representative undisturbed sampling e.g. clays containing partings of water-bearing sand.

The value of any empirical in situ test depends critically on its being performed and interpreted in a standard manner as prescribed by its originator. Specialist engineering supervision is essential to obtain reliable results on which technical and commercial decisions are to be based. When making standard penetration tests, particular attention must be paid to maintaining stability of the bottom of the borehole by attention to size of bore, boring methods, and hydrostatic balance.

The test is also of value in obtaining a relative measure of the in situ quality of weak rocks. Although experience is required in the interpretation of such tests, they have been found to be of value in materials that are too weakly cemented to allow the satisfactory recovery of rotary drill cores. In such tests, it is valuable to modify the standard procedure to allow the recording of the penetration of the probe under the influence of, for example, four sets of 20 blows. This technique is to be preferred to the measurement of penetration under the single set of, for example, 50 blows, since it allows the disturbance created by previous chiselling or other boring/drilling operations to be identified.

5.4.2.2 Radial load tests (see 21.7 of BS 5930 : 1981).

The radial stress/strain characteristics of the ground mass can be obtained in granular and cohesive soils, as well as in soft rocks, by a pressuremeter test. Recent developments in pressuremeter design include a self boring probe (Windle and Wroth 1977), (Hughes et al. 1977) and (Mair and Wood 1987).

For strong rocks, the Goodman jack may be appropriate (Goodman 1968), but the loaded area is small and correction procedures may be necessary. If results are difficult to obtain, it may be necessary to resort to estimating the in situ characteristics from the results of tests on core samples that relate only to the properties of the intact rock material.

In strong rocks, if the failure parameters are not relevant and if deformation characteristics are required at loads much less than the ultimate strength of the rock, then the rock need only be tested over the load range considered appropriate in the design of the anchorage. In such cases use of a pressuremeter may be appropriate (Hughes et al. 1977).

5.4.2.3 Permeability tests (see 21.4 of BS 5930 : 1981). Where the ground investigation proves strata which may lead to potential grout loss, then permeability tests supplemented by fabric description may be required to quantify this problem.

In fractured ground masses, a knowledge of the size and frequency of fractures may be more important than mass permeability, since the former may dictate the need for pregrouting (Littlejohn 1975) and the choice of grout. Permeability may also determine the appropriate corrosion protection (see 8.2.1).

5.4.2.4 Borehole logging techniques. Although at the time of publication of this code of practice borehole logging techniques cannot provide a substitute for a comprehensive core log of rock, additional valuable information on the in situ conditions of the rock can sometimes be obtained (Burwell and Nesbitt 1954) and (Krebs 1967).

Provided that clear vision is not obscured by ground water or drilling fluid, features on the sides of boreholes may be viewed by television or photographic camera. The nature of the materials in the borehole may be determined less directly by measuring the diameter of the borehole by caliper logging. Such measurements may show variations in borehole diameter due to preferential erosion of weaker rocks by the drilling fluid. Successive measurements over a period of time may reveal deformation of the hole due to swelling. Impression packers are also available for borehole wall logging (Hinds 1974).

In-hole geophysical logging by electrical methods, i.e. resistivity, radioactive methods or seismic methods may be undertaken (Carroll 1966).

5.5 Laboratory testing

5.5.1 Soils

5.5.1.1 Classification. Tests for the classification of soils are described in BS 1377. It is recommended that the grading of granular soils and the liquid and plastic limits of cohesive soils be determined for every stratum encountered in the investigation.

The determination of low plasticity indices in a cohesive soil, even in very localized zones, can influence both the type of anchorage and the method of drilling. For example, the drilling technique adopted may have to be that used for sands rather than clays, i.e. cased as opposed to uncased.

5.5.1.2 Shear strength. For granular soils, it is only possible to estimate the in situ shear strength characteristics on the basis of in situ tests, combined with grading and particle shape assessments from laboratory classification tests.

In granular soils of mixed grading, it may be justifiable to determine the peak shear strengths of samples prepared to a series of densities from loose to dense. The in situ strength may then be inferred by interpolation at the density estimated from field tests. It should be noted that the shear strength is dependent on stress level. Hence, all laboratory tests should be made at a stress level approximating to that in situ.

For cohesive soils, the shear strengths should be obtained by triaxial compression tests. The type of test to be employed will depend on the design method, the mass permeability of the soil and the probable rate of stressing of the anchorage.

Where stressing of the anchorage is rapid, the undrained shear parameters should be used in the necessary total stress analysis. For a slow rate of stressing or where the long term anchorage behaviour is required, the shear strength in terms of effective stresses should be obtained from a consolidated drained test. Alternatively, and particularly where the behaviour of the pore water pressure during shear is of interest, a consolidated undrained test with pore water pressure measurement should be made. The rate of strain employed should be sufficiently slow to ensure equalization of pore water pressure throughout the specimen (Bishop and Henkel 1962).

Where a cohesive soil possesses a relatively high mass permeability, e.g. clayey silts, clay possessing a permeable fabric, chalk and marl, both undrained and effective shear parameters should be determined to permit a study of the influence of this property under different rates of loading of the anchorage.

In all tests, the stress/strain characteristics of the soil during shear should be recorded. In soils exhibiting a marked structure or fabric, e.g. laminated clays, it may be necessary to study the influence of these features on the shear strength of the soil in the plane of shear induced around the fixed anchor length.

5.5.1.3 Compressibility. For systems that will apply a high average stress to a clay lying between the fixed anchor length and the structure, the compressibility characteristics should be determined by testing in a suitable type of oedometer. The rate of consolidation determined by such tests will also assist in assessing the mass permeability of the soil, particularly where tests can be made on large diameter specimens in a hydraulic oedometer (Rowe 1972). Such data may influence the choice of design method, i.e. whether a total or effective strength analysis is relevant, and also provide guidance as to the probable loss of prestress through case history comparisons.

5.5.2 Rocks

5.5.2.1 Index tests

5.5.2.1.1 General. Index tests are cheap and easily undertaken, the results often correlate approximately with other parameters and provide indications of rock quality. Although correlations of general application are available it is recommended that site specific correlations are developed for large projects.

5.5.2.1.2 Point load tests. Point load tests are a form of strength test that may be conducted on cores or irregular fragments of rock. The results may be related to unconfined compressive strength (Broch and Franklin 1972; Beinowski 1975), provided that the size of specimen is not less than 25 mm. The test apparatus is portable and can be used in the field as an aid to core logging.

5.5.2.1.3 Schmidt hammer tests. The apparatus used was developed for concrete testing. The L design hammer is often considered most appropriate to rock testing. Results may be correlated to compressive strength (Martin and Miller 1974) but these are best conducted on outcrops or large blocks of rock rather than cores.

5.5.2.1.4 Sonic velocity. The velocity of ultrasonic pulses is measured usually with instruments developed for concrete testing. The velocities measured may be related to strength and deformability properties of the rock (ISRM 1977a). Tests are most easily conducted across core diameters. Such tests are necessary if velocity index on rock masses is to be established.

5.5.2.2 Strength and deformability tests

5.5.2.2.1 Shear strength. The shear strength of a rock mass is commonly estimated from a detailed study of joint geometry and roughness together with a knowledge of the material characteristics (Hoek and Bray 1977) and (Hoek and Brown 1981). The design of ground anchorages is, however, usually based on borehole information and is often dependent upon a confining stress system local to the fixed anchor. The influence of joint systems may be less marked than, for example, in a problem involving rock slope stability. The results of laboratory tests on intact rock cores can therefore be useful, if carefully interpreted.

5.5.2.2.2 Unconfined compressive strength. This is determined on specially prepared test cylinders using laboratory compression test apparatus (ISRM 1978a).

5.5.2.2.3 Unconfined tensile strength. This may be determined by direct pull, which involves quite complex test arrangements, or indirectly by the Brazilian test or the pierced disc test (ISRM 1977b).

5.5.2.2.4 Deformability. This is determined commonly by measuring stress/strain relationships in the unconfined compression test. Poisson's ratio may be measured if suitable strain gauges are installed. It should be noted that such tests provide a measure of the deformability characteristics of the rock material. The characteristics of the rock mass can only be determined by in situ tests (see 5.4.2.2).

5.5.2.3 Durability

5.5.2.3.1 Slake durability. The slake durability test apparatus (ISRM 1978b) assesses the susceptibility of weak rock to weathering or softening in the presence of water. This test gives an indication of the potential loss of strength of the rock around the borehole and the period which a hole may be left open prior to installation of the anchorage. The results of the test may have a considerable influence on the design of the fixed anchor length.

5.5.2.3.2 Microscopic examination of minerals. Microscopic examination of the rock will determine the nature of the minerals of which it is composed (Shelley 1975). When this is known, the possibility of mineral reaction with grout or ground water may be assessed.

5.5.2.3.3 Swelling. The results of swelling tests are pertinent in determining the influence of anchorage construction methods such as flushing.

5.5.2.3.4 Drillability. A guide to drillability may be obtained by assessing in situ strength and structure (Blindheim 1979). Wear on drilling bits may be related to quartz content of the rock, which may be determined by microscopic examination (see 5.5.2.3.2).

5.6 Chemical testing

5.6.1 Aim of chemical analysis. The purpose of any chemical analysis of the ground or ground water is to determine its aggressivity to the constituents of the anchorage system and in particular the tendon and its grout surround (see 8.1.3).

5.6.2 Aggressivity towards metals

5.6.2.1 General. Some generalized measure of redox potential and soil resistivity can be applied to the assessment of potential ground corrosiveness to embedded metals (see 8.1.3.1). If the site is adjacent to an electrical installation the ground should be checked for stray electrical currents (see 8.1.4 and J.2.7).

5.6.2.2 Redox potential. The quantitative assessment of redox potential (see also 8.1.3 and table 9) provides guidance on the risk of microbiological corrosion, frequently characterized by pitting attack and most commonly encountered in heavy clay soils (see J.2.4). At the time of preparation of this code no standard test for redox potential is available. Present methods (King 1977; Constanzo and McVey 1957) may be difficult to perform and to interpret, but see also Allen, Grimshaw, Parkinson and Quarmby (1974), Mueller, Stolzy and Fick (1985) and Department of Transport (1986).

5.6.2.3 Resistivity. Methods of measurement vary but all involve passing a known current through the ground and measuring the voltage drop along the line of current flow (Palmer 1974). For soils that are well graded and homogeneous, the resistivity values should adequately predict the corrosion hazard. Non-cohesive soils are the most reliable to measure, i.e. there is less scatter of results.

5.6.3 Aggressivity towards cementitious grout or concrete. Ground aggressivity towards cement based grout or

concrete structural elements is partly dependent upon the sulphate content of the ground and ground water and should be determined chemically (BRE Digest 250, 1981) (BRE/CP 2: 1979). Soil aggressivity may be classified according to such determination (see 8.1.3.2).

5.7 Presentation of data

5.7.1 Description of strata (see clauses 41 and 44 of BS 5930 : 1981). All strata should be described in accordance with BS 5930 and the Working Party Report on the logging of rock cores for engineering purposes (QJEG 1970).

Of particular importance in investigations for soil anchorages is the recording of the structure or fabric of the soil. Relatively minor features in the strata can have a significant influence on anchorage behaviour in view of the limited volume of the stressed zone. For example, the presence of thin, even single grain, partings of silt or sand within a clay can have a marked effect on the behaviour of the soil and, in particular, on the action of drilling water on the soil prior to grouting. This, in turn, can severely limit the load that can be placed on clay anchorages, particularly of the underreamed type. Similarly, the presence of even small pockets of granular material within a generally cohesive glacial drift can have a major influence on the installation technique that it is necessary to employ and, in extreme cases, may preclude the economic use of an anchorage system. Certainly such minor features within an apparently uniform soil can seriously affect the cost of a project.

In the case of rock anchorages, knowledge of the continuity of geological structures is important. Discontinuity orientation data are also valuable and may be recorded in various ways, but the most convenient is by use of stereographic projection. This information can be vital in determining the size and shape of a rock mass liable to fail in service and is therefore critical in any overall stability analysis. The presentation of data in this way may facilitate analysis of slope stability problems (Hoek and Bray 1977) and (Matheson 1983). Such methods yield the trends of the various joint sets within the rock mass. Attention is drawn to the fact that it is the specific location of the joints and their continuity which will dictate the number and location of anchorages installed. Where a face has yet to be exposed, the orientation of the joint sets and their continuity should be assessed, if possible, from the nearest available rock exposure. The discontinuity data should be continually re-evaluated during excavation of the face. Discontinuity frequency data may be given by reference to various parameters such as rock quality designation, which may be obtained from the examination of rock cores. The frequency of fractures, for example, can be invaluable when back-analysing water test data to determine the need for pregrouting (see 10.4.2).

These careful fabric/structure observations are rarely made in routine investigations for the overall project.

This recommendation will therefore frequently provide the major justification for a detailed investigation during the feasibility study for an anchorage system.

5.7.2 Field data (see clause 39 and 40.2.6 to 40.2.8 of BS 5930 : 1981). All field data should be presented in a clear form following the recommendations of BS 5930.

5.7.3 Laboratory test results. All laboratory test results should be reported in accordance with BS 1377 or, for other tests, in such a form as to ensure that complete data are available for consideration during anchorage design.

5.8 Investigation during construction

5.8.1 Supplementary investigation. Where the initial and any detailed investigations show that ground conditions are liable to random variations, e.g. glacial drift, then it is desirable to infill the borehole information early in the contract period with probes or in situ tests. This work should provide immediate data at site level without the necessity of any but routine laboratory tests, which can conveniently be carried out in a site laboratory. Whilst the best techniques will depend on circumstances, the necessity for in situ results suggests the use of penetrometers or pressuremeters for the strength/deformation characteristics of soils. A simple boring/probing technique can be employed where the problem involves general variations in lithology, e.g. rotary percussive drilling to establish rockhead.

5.8.2 Field records. During the course of construction of the anchorage installation, further evidence on the nature of the ground conditions is revealed by excavation, by clearing the site or in the boreholes of the individual anchorages. This information should be recorded and appraised as an extension of the engineering geology work that formed part of the earlier investigation. No site investigation can explore ground conditions as frequently as the installation process can and any major variations in the ground conditions from those anticipated have to be recorded and their significance assessed. It should be emphasized, however, that production drilling associated with anchorage installation is not geared to investigate the ground in detail. For example, production drilling cannot be expected to highlight local zones where silt or sand lenses occur in clay or variations in density in gravelly sand. It is recommended that all ground data obtained during anchorage drilling be recorded on suitable proformas (see appendix C) and subjected to daily analysis. Such a system can act as an early warning device, should variation in strata levels or ground type require changes in design or installation method.

5.8.3 Anchorage behaviour during tests. The behaviour of anchorages subjected to proving or on-site suitability tests (see 11.2 and 11.3) should be considered as part of the ground investigation and carefully compared with all field and laboratory data. In particular, the behaviour of an anchorage in a semi-permeable soil, e.g. clay possessing a permeable fabric, silt, chalk and some marls, will be

dependent on the ratio between the rate of dissipation of pore water pressure and the rate of loading. Thus, it is possible that the available anchorage load may well be increased if stressing can be applied slowly or in a series of stage increments. Similarly, the behaviour of a clay soil between the fixed anchor length and the anchor head may well depend on the stressing procedure.

On a major project, where the costs can be justified by possible design economies, anchorages of specific types can be installed and tested as part of the field testing stage of a precontract investigation programme. Whilst much valuable data can be obtained from such a programme, care should be taken that data determined from the behaviour of a specific type of anchorage is not unjustifiably applied to other systems of different design or installation technique (see 11.2 and 11.3).

5.8.4 Adjacent activities. Any activities on the site that may influence anchorage behaviour should be carefully monitored, recorded and their possible influence assessed at an early stage in the work. Such activities include local excavation, ground water lowering, piling, blasting, freezing and mining subsidence.

5.9 Health hazards and safety

Environmental hazards may be encountered when working in and around existing factory or chemical premises, or where industrial waste has been accumulated. Liaison is essential with the owners or occupiers of such premises or with other competent persons to determine the hazards and the type of protection needed.

6 Design

6.1 General

The design method recommended by this standard is based on safety factor methods. Limit state design may be applied as described in appendix B, but this does not reflect practice at the time of publication of this code. Suitable safety factors are listed in table 2 (see also Mitchell 1987).

The design of ground anchorages requires consideration of the following:

- (a) overall stability (see appendix D);
- (b) depth of embedment (see appendix D);
- (c) group effects (see 6.2.6);
- (d) fixed anchor dimensions (see 6.2).

To assist users of this standard, flow charts (figures 2 and 3) are included covering the preliminaries to design, detailed design and construction, and maintenance. Where it is appropriate, key steps are cross referenced to sections or clauses in the standard so that the user may consult the detailed recommendations

Table 2. Minimum safety factors recommended for design of individual anchorages

Anchorage category	Minimum safety factor			Proof load factor
	Tendon	Ground/grout interface	Grout/tendon or grout/encapsulation interface	
Temporary anchorages where a service life is less than six months and failure would have no serious consequences and would not endanger public safety, e.g. short term pile test loading using anchorages as a reaction system.	1.40	2.0	2.0	1.10
Temporary anchorages with a service life of say up to two 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-back.	1.60	2.5*	2.5*	1.25
Permanent anchorages and temporary anchorages where corrosion risk is high and/or the consequences of failure are serious, e.g. main cables of a suspension bridge or as a reaction for lifting heavy structural members.	2.00	3.0†	3.0*	1.50

* Minimum value of 2.0 may be used if full scale field tests are available.

† May need to be raised to 4.0 to limit ground creep.

NOTE 1. In current practice the safety factor of an anchorage is the ratio of the ultimate load to design load. Table 2 above defines minimum safety factors at all the major component interfaces of an anchorage system.

NOTE 2. Minimum safety factors for the ground/grout interface generally lie between 2.5 and 4.0. However, it is permissible to vary these, should full scale field tests (trial anchorage tests) provide sufficient additional information to permit a reduction.

NOTE 3. The safety factors applied to the ground/grout interface are invariably higher compared with the tendon values, the additional magnitude representing a margin of uncertainty.

6.2 Ground/grout interface

6.2.1 General. Design rule predictions of ultimate load holding capacity (see F.1) are invariably created by assuming that the ground has failed along a shear surface, 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 mobilized locally as the fixed anchor is withdrawn, namely end bearing and side shear. Fixed 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 mobilization of slip lines or the generation of significant deformations extending into the exposed surface.

The ultimate load holding capacity of the anchorage (T_f) is dependent on the following factors, although, due to lack of knowledge, item (e) is not generally isolated in design calculations and the design model is therefore imperfect and does not represent accurately physical states at the ground/grout interface:

- definition of failure;
- mechanism of failure;
- area of failure interface;
- ground properties mobilized at the failure interface;
- stress conditions acting on the failure interface at the moment of failure.

The design rules described in 6.2.3 to 6.2.5 for rocks and soils apply to individual anchorages and no allowance is made for group effects or interference (see 6.2.6 and 11.4.10).

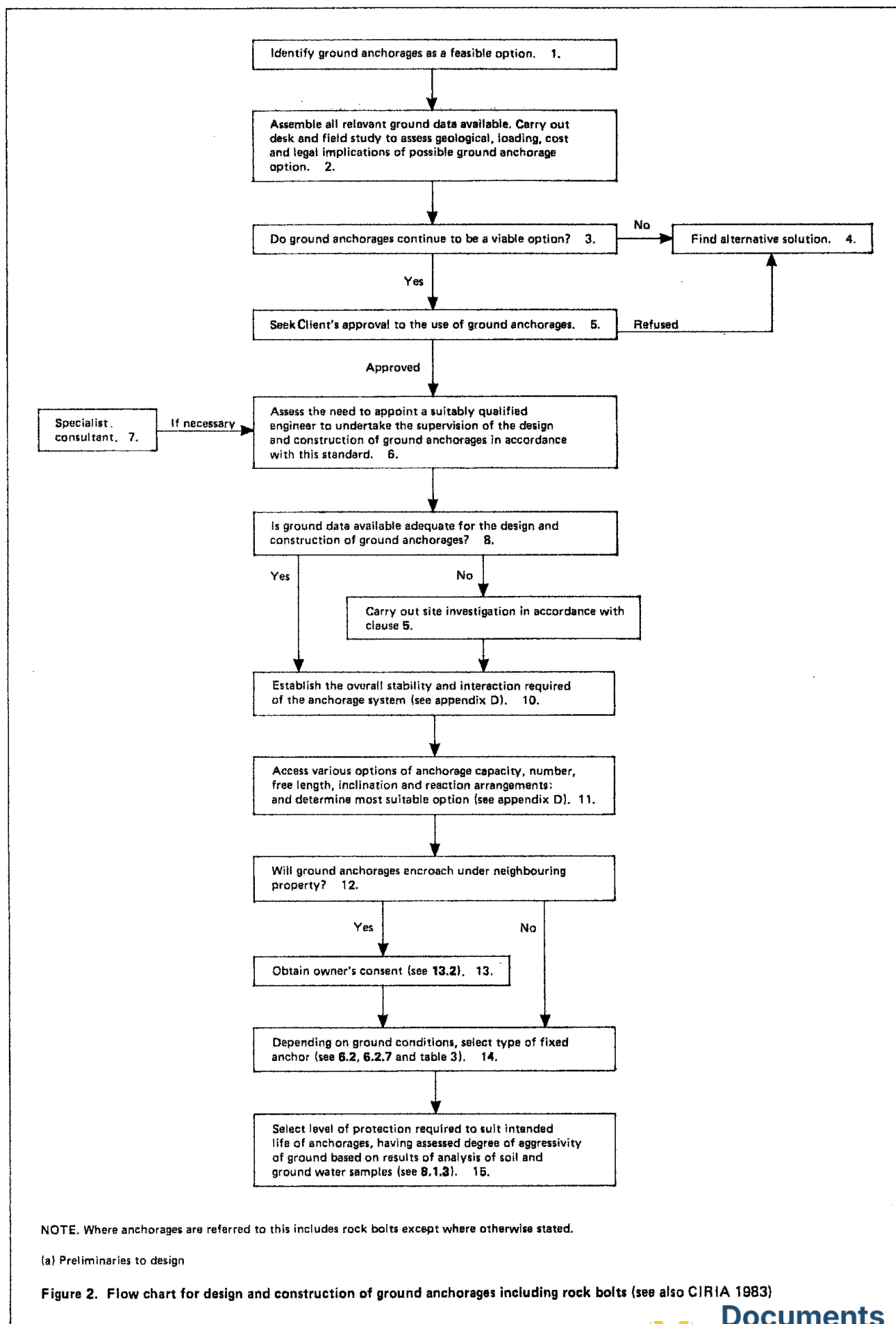
A general review of fixed anchor design rules that have evolved as a result of full scale testing is provided by Littlejohn (1979b).

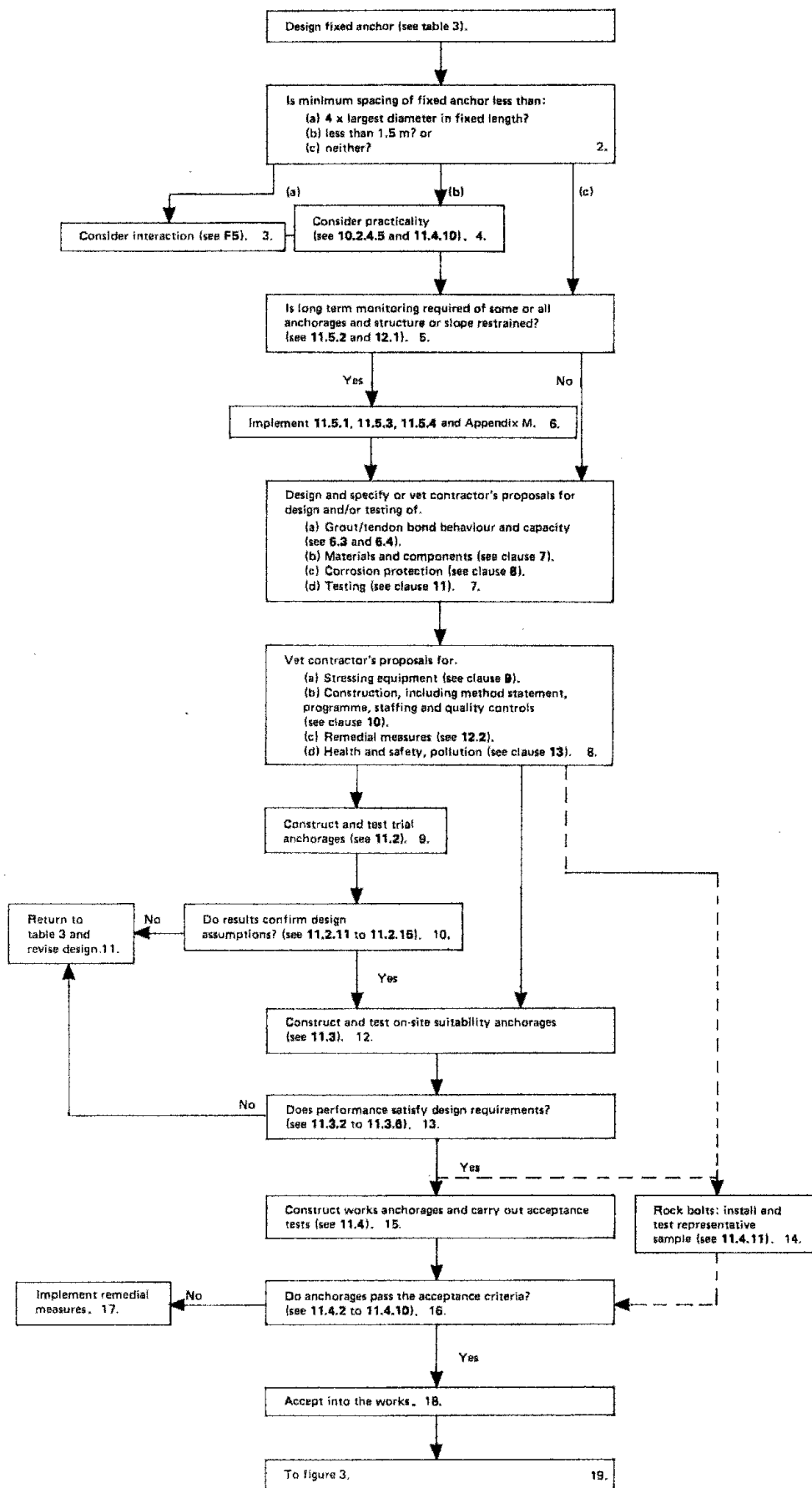
It should be noted that all anchorages should be subject to a testing procedure as detailed in clause 11. The anchorage ultimate load holding capacities calculated from 6.2.3, 6.2.4 and 6.2.5 are therefore for guidance only and may have to be modified depending upon the results of such testing.

6.2.2 Anchorage types

6.2.2.1 General. Anchorage pull-out capacity for a given ground condition is dictated by anchorage geometry. 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 the methods of drilling and flushing. Accordingly, the types of anchorage to which the design rules are applicable are described in 6.2.2.2 to 6.2.2.5 (see figure 4).

6.2.2.2 Type A anchorages. Type A anchorages consist of tremie (gravity displacement), packer or cartridge grouted straight shaft boreholes, which may be temporarily 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.





NOTE: Where anchorages are referred to this includes rock bolts, except where otherwise stated.

(b) Detailed design and construction

Figure 2 (concluded)

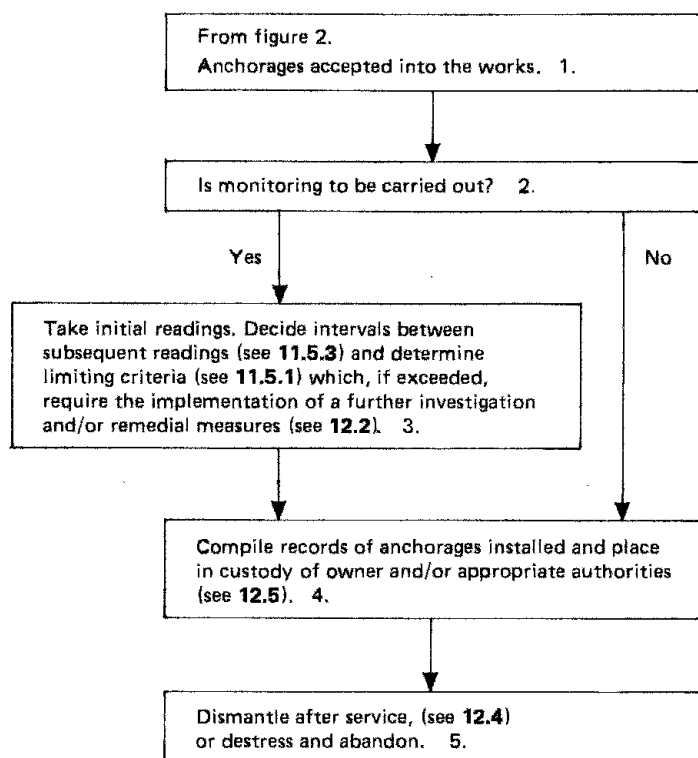


Figure 3. Flow chart for maintenance of ground anchorages including rock bolts

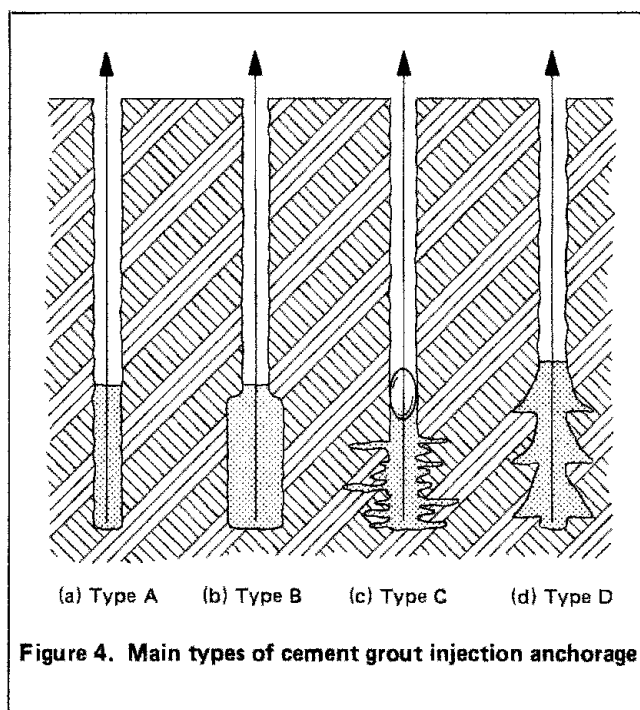


Figure 4. Main types of cement grout injection anchorage

6.2.2.3 Type B anchorages. Type B anchorages consist of low pressure (typically grout injection pressure $p_i < 1000 \text{ kN/m}^2$) grouted boreholes, via a lining tube or in situ packer, where the diameter of the fixed anchor is increased with minimal disturbance as the grout permeates through the pores or natural fractures of the ground. This type is

most commonly employed in weak fissured rocks and coarse granular alluvium, but the method is also popular in fine grained cohesionless soils. Here cement based grouts cannot permeate the small pores but under pressure the grout compacts the soil locally after boring to increase the effective diameter and enhance the shearing resistance. Resistance to withdrawal is dependent primarily on side shear in practice, but an end bearing component may be included when calculating the ultimate capacity.

6.2.2.4 Type C anchorages. Type C anchorages consist of boreholes grouted to high pressure (typically $p_i > 2000 \text{ kN/m}^2$), via a lining tube or in situ packer. The fixed anchor length is enlarged by hydrofracturing of the ground mass to give a grout root or fissure system beyond the core diameter of the borehole. Often pressure is applied during a secondary injection after initial stiffening of primary grout placed as for Type B anchorages. Secondary injections are usually made via either a tube à manchette system (figure 5) or miniature grout tubes incorporated within the fixed anchor length: the former is advantageous if several injections are envisaged. A relatively small quantity of secondary grout is needed. Continuous flow or a sudden drop in initial injection pressure might indicate hydrofracture after which only relatively limited pressures can be achieved.

Whilst this anchorage type is commonly applied in fine cohesionless soils, some success has also been achieved in stiff cohesive deposits. Design is based on the assumption of uniform shear along the fixed anchor.

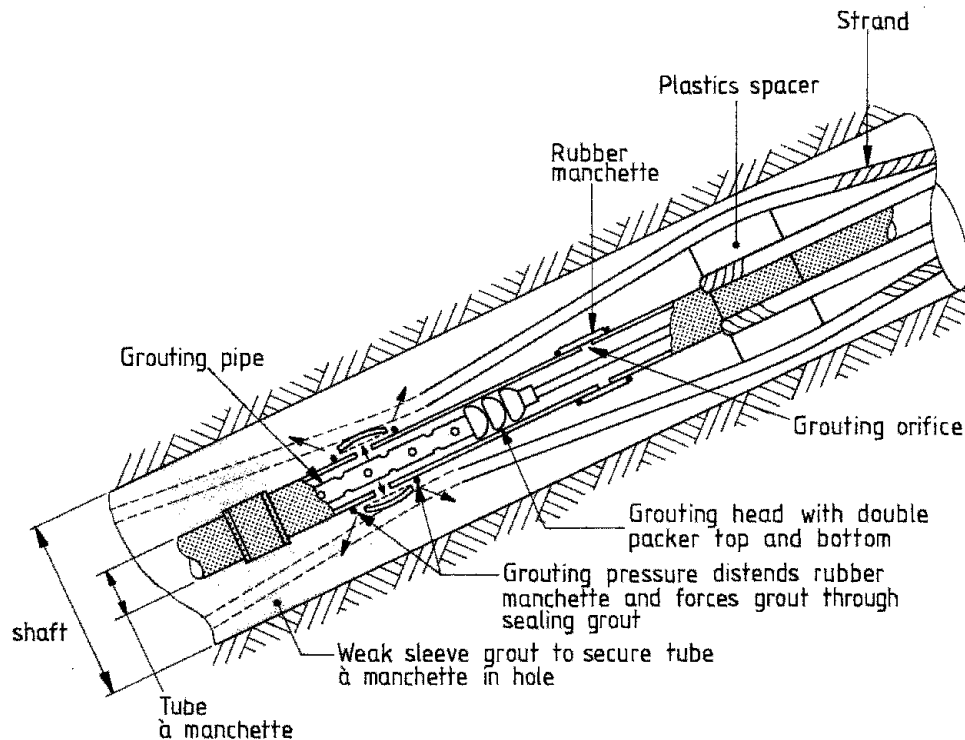


Figure 5. Detail of tube à manchette for pressure grouting control (after Mitchell 1975)

6.2.2.5 Type D anchorages. Type D anchorages consist of tremie grouted boreholes in which a series of enlargements, either bells or underreams, have previously been formed. This type is employed most commonly in firm to hard cohesive deposits. Resistance to withdrawal is dependent on side shear and end bearing, although, for single or widely spaced underreams (see equation 6), the ground restraint may be mobilized primarily by end bearing.

Although not common this type can be used in cohesionless soils in conjunction with some form of side wall stabilization over the enlargement length. Typically this may be by pre-injection of cement or chemical grout in the ground around the fixed anchor or by pumping polymer drilling fluid into the borehole during the drilling and underreaming operation.

6.2.2.6 Other types of anchorage. These anchorages do not readily fall into types A to D and are formed by newly developed techniques. Such anchorages currently include those which utilize expander systems within the fixed length or are formed by jet grouting techniques.

In the absence of proven design methods for these anchorages, it is recommended that proving tests should be undertaken. Furthermore the influence of corrosion on anchorage performance should be assessed fully.

6.2.3 Fixed anchor design in rock

6.2.3.1 General. Whilst anchorage types A to D can all be applied in rock, the straight shaft tremie or packer grouted

type A is the more popular in current practice, on the basis of cost and simplicity of construction.

In type B anchorages low pressure grouting is used to enhance rock/grout bond and/or to increase the rock/grout interface area in weak or fissured rock. When enhanced bonds are employed proving and on-site suitability testing are required to verify the design method.

For type C anchorages proving and on-site suitability testing to determine ultimate load holding capacity are required to establish ultimate or adequate factors of safety.

In type D anchorages underreaming may be used to increase the diameter of the fixed anchor in weak rock. Proving and on-site suitability testing are required to verify the design method.

6.2.3.2 Type A anchorages. For such anchorages, designs are based on the assumption of uniform bond distribution (Coates 1970), (Fargeot 1972), (Littlejohn 1972), (Mascardi 1973) and (White 1973). Thus the pull-out capacity of the fixed anchor T_f , in kN, is estimated from equation (1):

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

where:

τ_{ult} is the ultimate bond or skin friction at rock/grout interface (in kN/m²);

D is the diameter of fixed anchor (in m);

L is the length of fixed anchor (in m).

Equation (1) is based on the following assumptions.

- (a) 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 (see F.2.1).
- (b) The diameters of the borehole and the fixed anchor are identical.
- (c) Failure takes place by sliding at the rock/grout interface (smooth borehole) or by shearing adjacent to the rock/grout interface in the weaker medium (rough borehole).
- (d) There are no discontinuities or inherent weakness planes along which failure can be induced.
- (e) There is no local debonding at the rock/grout interface.

The assumption of a uniformly distributed stress along the fixed anchor may require careful consideration in terms of the likely stress concentrations at the proximal end of the fixed anchor in weak, deformable rock (Littlejohn 1979a). As a result of stress concentrations in the tendon/grout interface, failure in the grout/rock interface may be initiated even though average stress/strength conditions have been satisfied. Under such conditions it may be necessary to base the design directly on proving test results.

6.2.3.3 Skin friction values. For weak rocks where the unconfined compressive strength is less than 7 N/mm^2 , shear tests on representative samples should be carried out. In such cases, the ultimate skin friction proposed for design should not exceed the minimum shear strength. For strong rocks where there is an absence of shear strength data or field pull-out tests, the ultimate skin friction may be taken as 10 % of the unconfined compressive strength of the rock up to a maximum value τ_{ult} of 4.0 N/mm^2 (see F.2.2). The maximum value of τ_{ult} should not exceed 4.0 N/mm^2 for any rock, assuming that the design unconfined compressive strength of the grout is equal to or greater than 40 N/mm^2 . Increasing grout strength beyond 40 N/mm^2 will not lead to a significant increase in rock/grout bond.

In chalk and clay filled fissured rock, the value of friction may be affected by the presence of borehole smear.

For guidance, ground/grout bond (skin friction) values that have been recommended for design are included in F.2.

6.2.3.4 Fixed anchor length. With the exception of rock bolts, the fixed anchor length should not be less than 3 m (2 m in rock if working load $< 200 \text{ kN}$).

Under certain conditions, it is recognized that much shorter lengths than 3 m would suffice, even after the application of a generous factor of safety. However, for a very short fixed anchor, any sudden drop in rock quality along the fixed anchor length can induce a serious decrease in ultimate load holding capacity (see F.3).

The fixed anchor length for rock bolts may be derived from the data presented in 6.2.3.2, F.2 and CIRIA (1983). For higher capacity anchorages, the fixed anchor length may be derived from equation (1) using appropriate factors of safety.

The possibility of rock blasting in the vicinity of anchorages does not appear to require any increase in fixed anchor length (Littlejohn et al. 1977).

Where limited information is available proving tests are recommended, especially in weak, deformable rocks (see 11.2). This will confirm the adequacy of the adopted fixed anchor lengths in the prevailing conditions.

The fixed anchor length should not exceed 10 m.

6.2.3.5 Fixed anchor design: mechanically anchored rock bolts. No design rules are given for mechanically anchored rock bolts. The working or ultimate loads are generally given by the manufacturer; such values are usually based on the result of pull-out tests. As the conditions under which such tests are performed are not normally available, the working load should be ascertained on site from tests. These should investigate, in particular, the effect of changes in borehole orientation with respect to natural features in the rock and the effect of vibrations (Littlejohn et al. 1987).

6.2.4 Fixed anchor design in cohesionless soils

6.2.4.1 General. Type A anchorages are not used generally in cohesionless soils except where they are cemented. The most popular form of anchorage is type B with type C being more usual amongst continental European contractors. The formation of enlargements as in type D anchorages is rare and will require special measures to stabilize the region of the fixed length prior to and during the formation of the enlargement.

6.2.4.2 Type B anchorages. For low pressure grouted anchorages of type B, design equations for the estimation of the ultimate load holding capacity are based primarily on piling design technology (Lundahl and Adding 1966; Robinson 1969; Bassett 1970; Littlejohn 1970; and Oosterbaan and Gifford 1972). For guidance, T_f in kN, may be estimated from equation (2) (see also F.3):

$$T_f = Ln \tan \phi' \quad (2)$$

where:

L is the fixed anchor length (in m);

ϕ' is the effective angle of shearing resistance (in degrees);

n is the factor that takes account of the drilling technique (rotary percussive with water flush), depth of overburden, fixed anchor diameter, grouting pressure in the range 30 kN/m^2 to 1000 kN/m^2 , in situ stress field and dilation characteristics.

Field experience (Littlejohn 1970) indicates that for coarse sands and gravels (permeability of the ground $k_w > 10^{-4} \text{ m/s}$), n ranges from 400 kN/m to 600 kN/m, whilst in fine to medium sands ($k_w = 10^{-4} \text{ m/s}$ to 10^{-6} m/s), n reduces to 130 kN/m to 165 kN/m. These figures were initially measured in normally consolidated materials for borehole anchor diameters of 0.1 m approximately and where the enlarged diameter D varies significantly, n is modified in the same proportion.

As an alternative, equation (3) may be used where anchorage ultimate load holding capacity is related to anchorage dimensions and soil properties (Littlejohn 1970):

$$T_f = A\sigma_v' \pi DL \tan \phi' + B\gamma h \frac{\pi}{4} (D^2 - d^2) \quad (3)$$

= (side shear) + (end bearing)

where:

- A** is the ratio of contact pressure at the fixed anchor/soil interface to the average effective overburden pressure (see F.3);
- γ is the unit weight of soil overburden (submerged unit weight beneath the water table (in kN/m³));
- h** is the depth of overburden to top of fixed anchor (in m);
- L** is the length of fixed anchor (in m);
- σ_v' is the average effective overburden pressure adjacent to the fixed anchor (in kN/m²) equivalent to $\gamma \{h + (L/2)\}$ for a vertical anchorage (Littlejohn 1970);
- D** is the diameter of fixed anchor (in m);
- ϕ' is the effective angle of shearing resistance (in degrees);
- B** is the bearing capacity factor equivalent to $N_q/1.4$ (see figure 6);
- d** is the diameter of grout shaft above fixed anchor (in m).

6.2.4.3 Type C anchorages. It is a feature of type C anchorages 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 (Jorge 1969) have indicated for 0.1 m to 0.15 m diameter boreholes, ultimate load holding capacities of 90 kN/m to 130 kN/m of fixed anchor at a grouting pressure p_i of 1000 kN/m², and 190 kN/m to 240 kN/m at a p_i of 2500 kN/m² where p_i is the grout injection pressure.

In more recent years, design curves for type C anchorages have been extended through proving tests in Germany. For sandy gravels and gravelly sands figure 7 shows that the ultimate load increases with density and uniformity coefficient (see F.3.2).

6.2.4.4 Type D anchorages. Some development work has been carried out on type D anchorages in cohesionless soils (Baker et al. 1966) but there is a dearth of published data. If the enlargements are required to take all the load in the fixed length then shearing across the interface of the nominal borehole diameter and the enlargement will be an important factor to consider. This is in respect of the number of enlargements that are required and the subsequent effect this has on the transfer of the load into the tendons in the fixed length. (Barley 1978).

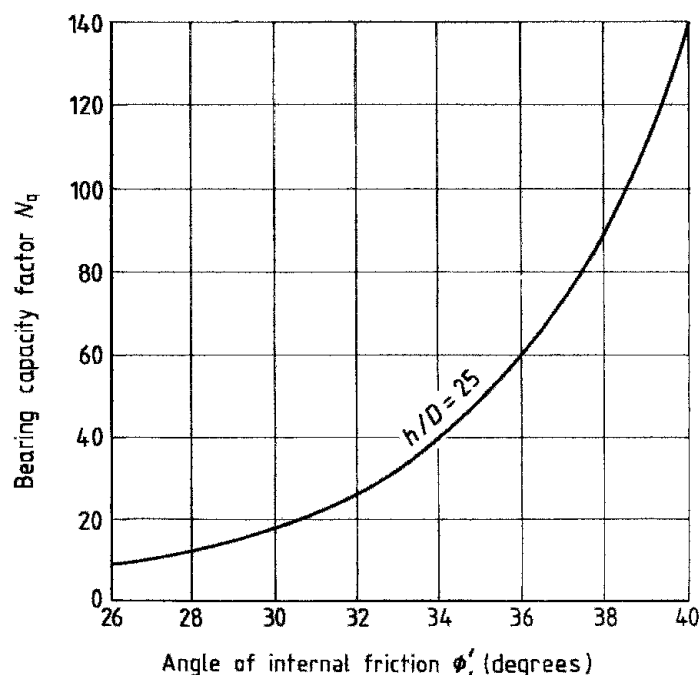
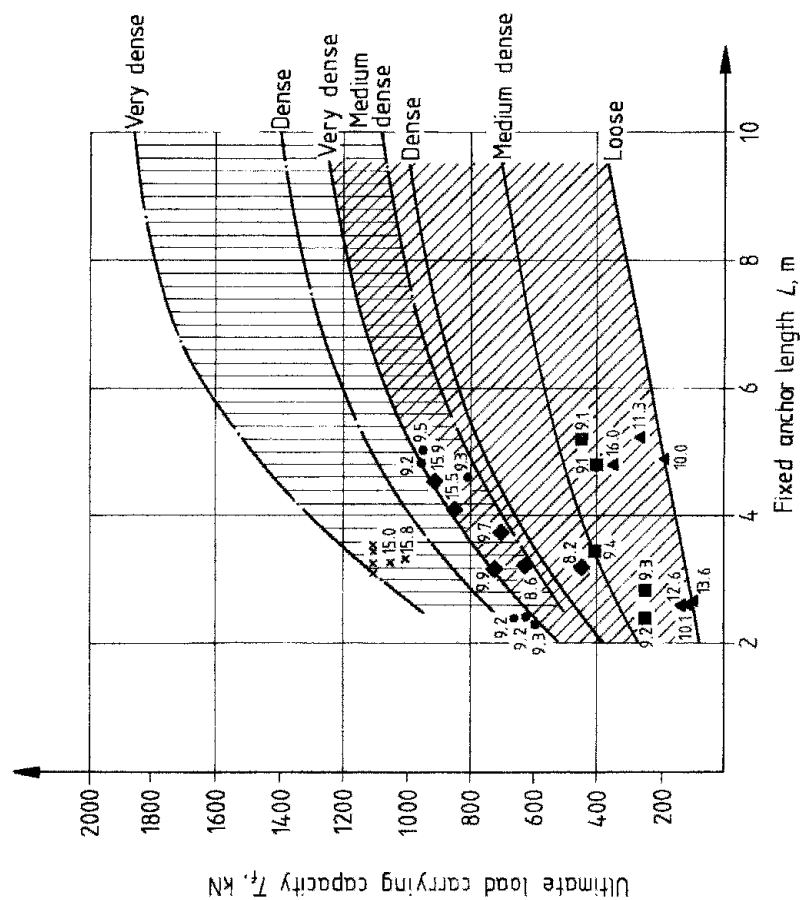


Figure 6. Relationship between bearing capacity factor N_q and effective angle of shearing resistance ϕ' (after Berezantzev et al. 1961)



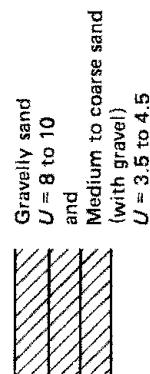
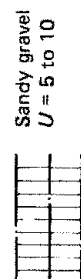
NOTE 1. Field evidence is limited to a fixed anchor range of 2 m to 5 m approximately.

NOTE 2. The relationships between soil density and standard penetration test (SPT) values are not in accordance with BS 5930.

Figure 7. Ultimate load-holding capacity of anchorages in sandy gravels and gravelly sand showing influence of soil type, density and fixed anchor length (after Oostermayer and Scheele (1978))

Type of soil	Density	Standard penetration test (SPT) N (bl/0.3 m)
Gravelly sand	Very dense	120
$U = 1.6/0.16$	Dense	60
	Medium dense	43
	Loose	11
Sandy gravel $U = 15/0.3$	Very dense	>130

where
 U Uniformity coefficient (D_{60}/D_{10})
 D_{60} Maximum particle size of the smallest 60 %
 D_{10} Maximum particle size of the smallest 10 %



Diameter of grouted bodies
 $D = 0.1$ m to 0.15 m

6.2.4.5 Fixed anchor length. The fixed anchor length for anchorages types A to D should normally not be less than 3 m, nor more than 10 m (see F.3.4).

6.2.5 Fixed anchor design in cohesive soils

6.2.5.1 General. The application of low grouting pressure and in certain instances the penetration of drill casing into the soil may be beneficial to the capacity of the anchorage. Even without the presence of hydrofracture the fissure penetration of the grout can enhance the skin friction values. This is particularly true in cohesive strata interbedded with weak mudstones or siltstones. Adhesion factors in excess of 0.45 can be mobilized, but these should be verified by proving tests (see 11.2).

6.2.5.2 Type A anchorages. For tremie or gravity grouted straight shaft anchorages of type A, design rules are similar to those developed for bored piles (Littlejohn 1970; Neely and Montague-Jones 1974; and Sapio 1975) and are based on the use of undrained shear strengths.

For guidance, the ultimate load holding capacity T_f in kN, may be estimated from equation (4) (see also F.4):

$$T_f = \pi DL \alpha C_u \quad (4)$$

where:

C_u is the average undrained shear strength over the fixed anchor length (in kN/m²);

α is the adhesion factor;

D is the diameter of the fixed anchor (borehole diameter) (in m);

L is the length of the fixed anchor (in m).

The actions of drilling and grouting cause stress changes within the ground which cannot be accurately modelled by either an effective stress or total stress analysis. An effective stress analysis will indicate a higher calculated ultimate load holding capacity, whilst a total stress analysis yields results more closely resembling actual ultimate capacities. Therefore, bearing in mind the short duration of the test on an anchorage and the fact that effective stress analysis implies deformation that is accompanied by loss of prestress, total stress analysis is considered more appropriate.

6.2.5.3 Type C anchorages. Where high grout pressures can be safely permitted, type C anchorages, with or without post-grouting, can be used.

The results of a large number of fundamental tests are shown in figure 8, which can be used as a design guide for borehole diameters of 0.08 m to 0.16 m. Skin friction (τ_M) increases with increasing consistency* and decreasing plasticity. In stiff clays (i.e. a consistency index, $I_c = 0.8$ to 1.0) with medium to high plasticity, skin frictions of 30 kN/m² to 80 kN/m² are the lowest recorded, whilst the highest values ($\tau_M > 400$ kN/m²) are obtained in sandy silts of medium plasticity and very stiff to hard consistency ($I_c = 1.25$). The technique of post-grouting is also shown to generally increase the skin friction of very stiff clays by some 25 % to 50 %, although greater improvements (from 120 kN/m² up to about 300 kN/m²) are claimed for stiff

clay of medium to high plasticity. From figure 9, the influence of post-grouting pressure on skin friction is quantified showing a steady increase in τ_M with increase in post-grouting pressure.

6.2.5.4 Type D anchorages. For multi-underreamed anchorages of type D, design rules (Littlejohn 1970; Bassett 1970; Bastaple 1974; Neely and Montague-Jones 1974) are available. For guidance, the ultimate load holding capacity T_f , in kN, may be estimated from equation (5).

$$T_f = \pi DL C_u + \frac{\pi}{4} (D^2 - d^2) N_c C_{ub} + \pi dl C_a \quad (5)$$

= (side shear) + (end bearing) + (shaft resistance)

where:

D is the diameter of underream (see figure 10) (in m);

L is the length of fixed anchor (in m);

C_u is the average undrained shear strength over fixed anchor length (in kN/m²);

d is the diameter of shaft (in m);

N_c is the bearing capacity factor (value of 9 commonly assumed);

C_{ub} is the undrained shear strength at proximal end of fixed anchor (in kN/m²);

l is the length of shaft (in m);

C_a is the shaft adhesion (a value for C_a of $0.3 C_u$ to $0.35 C_u$ is commonly assumed) (in kN/m²).

In the absence of results from trial anchorages in the field, empirical multiplier reduction coefficients ranging from 0.75 to 0.95 are sometimes applied to the side shear and end bearing components of equation (5) to allow for factors such as construction techniques and underream geometry (Bassett 1970; Buttlings 1977).

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 (Littlejohn 1970).

Of vital importance in cohesive deposits is the time during which drilling, underreaming and grouting take place. This should be kept to a minimum in view of the softening effect of water on the clay. The consequences of delays of only a few hours include reduced load holding capacity and significant short term losses of prestress. In the case of sand filled fissures, for example, where water flushing is employed, a period of only 3 h to 4 h may be sufficient to reduce the C_u strength to near the fully softened value.

Underreaming is ideally suited to clays of C_u greater than 90 kN/m². Some difficulties in the form of local collapse or breakdown of the neck portion between the underreams should be expected where C_u values of 60 kN/m² to 70 kN/m² are recorded. Underreaming is virtually impracticable below a C_u of 50 kN/m². Underreaming is also difficult in soils of low plasticity, e.g. plasticity index < 20.

* The consistency index, I_c , is given by the following equation:

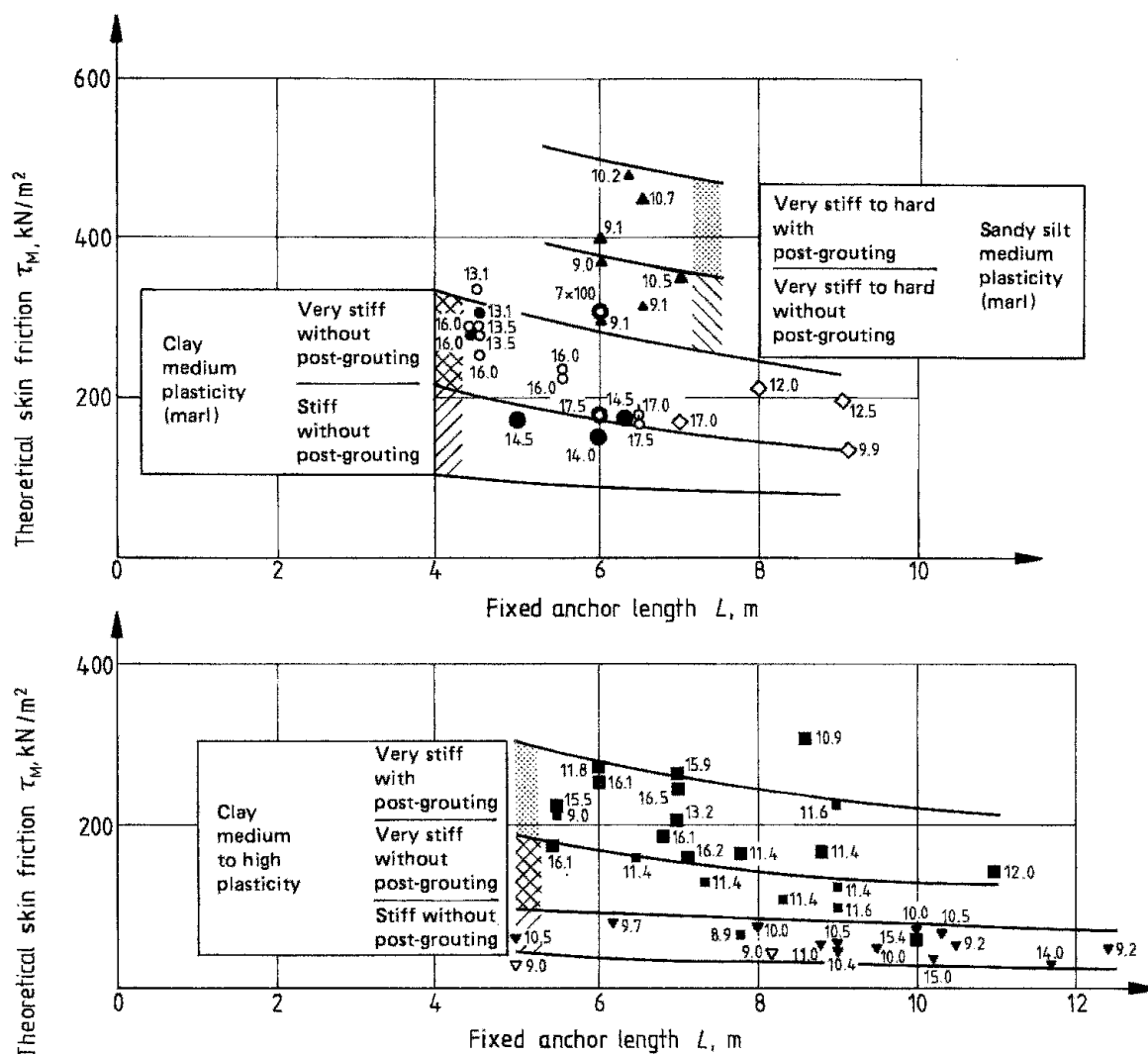
$$I_c = \frac{L_L - m}{L_L - P_L}$$

where:

L_L is the liquid limit (in %);

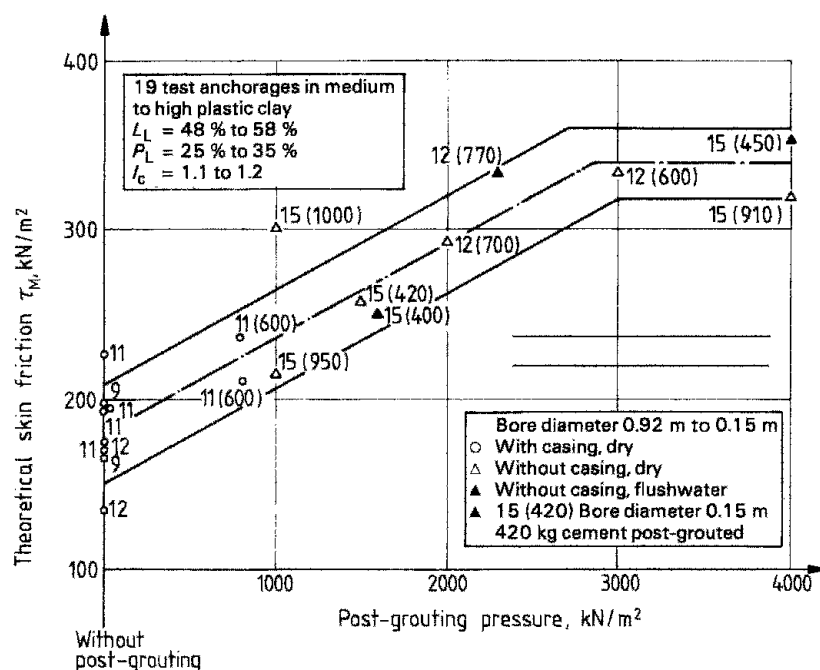
P_L is the plastic limit (in %);

m is the natural moisture content (in %).



Failure load was reached	Failure load was not reached	Post-grouting	Type of soil	L_L %	P_L %	I_c %
▲	△	Without	Silt, very sandy (marl) medium plasticity	~45	~22	~1.25
●	○	Without	Clay (marl) medium plasticity	32 to 45	14 to 25	1.03 to 1.14
●	○	With				
●	○	Without		36 to 45	14 to 17	1.3 to 1.5
	◇	Without	Silt medium plasticity	23 to 28	5 to 11	0.7 to 0.85
■		Without	Clay medium to high plasticity	48 to 58	23 to 35	1.1 to 1.2
■		With				
▼		Without		45 to 59	16 to 32	0.8 to 1.0

Figure 8. Skin friction in cohesive soils for various fixed anchor lengths, with and without post-grouting (after Ostermayer (1974))



NOTE. The theoretical skin friction is calculated from the borehole diameter and designed fixed anchor length.

Figure 9. Influence of post-grouting pressure on skin friction in a cohesive soil (after Ostermayer (1974))

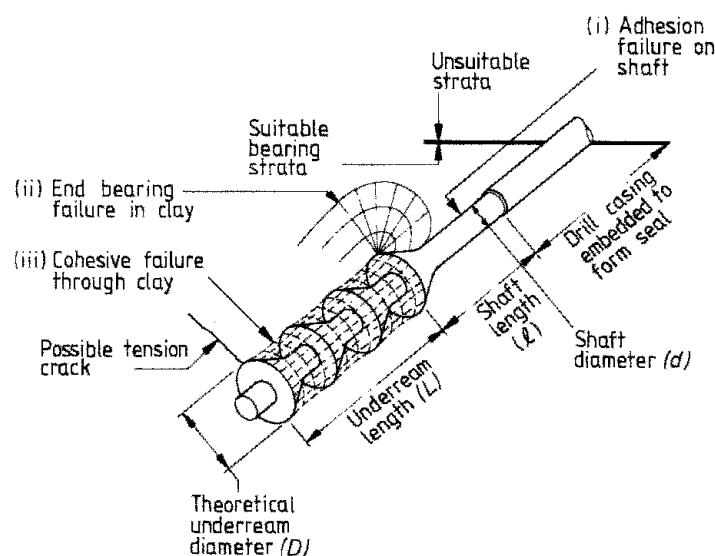


Figure 10. Diagram of multi-underream anchorage at ultimate capacity (after Bassett (1970))

With regard to spacing of underreams (δ_u) in type D anchorages, equations (6) and (7) can be used to estimate the maximum allowable spacing to give failure along a cylindrical surface, which requires less fixed anchor displacement to develop load. Where fixed anchor displacement is unimportant, more widely spaced underreams may be employed when they will act independently.

$$\pi D \delta_u C_u < \frac{\pi}{4} (D^2 - d^2) N_c C_u \quad (6)$$

from which

$$\delta_u < \frac{(D^2 - d^2)}{4D} N_c \quad (7)$$

6.2.5.5 Fixed anchor length in cohesive soils. The fixed anchor length should not normally be less than 3 m, nor more than 10 m.

6.2.6 Fixed anchor interaction. To limit interaction between fixed anchors, a spacing of not less than four times the diameter (D) centre to centre is assumed (where D is the largest diameter within the fixed anchor) and in practice a minimum spacing in the range 1.5 m to 2 m is commonly adopted (see F.5 and 11.4.10).

The distance between a fixed anchor and adjacent foundation or underground service should exceed 3 m, bearing in mind that for surface foundations it may be necessary to increase the fixed anchor depth to 5 m or more, either to improve the ultimate resistance to withdrawal or to encourage the possibility of local shear failure around the fixed anchor, as opposed to general shear failure, under ultimate conditions.

In locating the fixed anchors, the ability to form boreholes with the necessary accuracy should be taken into account.

6.2.7 Summary of design methods. To assist users of this standard, means of deriving the fixed anchor length of anchorage types A to D (see 6.2.2 and figure 4) in rock, cohesionless soils and cohesive soils are summarized in table 3.

6.3 Grout/tendon interface

6.3.1 General. While there is an appreciable amount of information available concerning the mechanism of bond transfer in the field of reinforced and prestressed concrete, it is considered that much more study is required in the field of ground anchorages. The mode of failure of a tendon bonded into the grout of an in situ ground anchorage may be dissimilar to that of the tendon pull-out test used in concrete technology and from which most bond data are obtained. In the former case, the grout is usually in tension, whereas during a standard bond test, part at least, of the surrounding concrete is in compression. In ground anchorages therefore, the mechanism of bond action depends on the respective elastic moduli of the steel and grout. Cement grout/tendon bond values which have been employed or recommended in practice are given in tables 28 to 30 for bar, wire and strand tendon (see G.5).

6.3.2 Magnitude of bond. The ultimate bond stress assumed to be uniform over the tendon bond length should not exceed:

- (a) 1.0 N/mm² for clean plain wire or plain bar;

- (b) 1.5 N/mm² for clean crimped wire;

- (c) 2.0 N/mm² for clean strand or deformed bar;

- (d) 3.0 N/mm² for locally noded strands (Barley, 1978).

The above values are based on a minimum grout compressive strength of 30 N/mm² prior to stressing. They may be applied to single unit tendons and to parallel multi-unit tendons, provided that the clear spacing is not less than 5 mm (Bruce 1976; Barley 1978). For noded strands and tendons that can mobilize mechanical interlock or the shear strength of the grout, the minimum spacing criterion does not apply.

For resinous grouts ultimate bond values should be obtained from proving tests in the absence of relevant documented test data.

6.3.3 Tendon bond length. For cement or resin grouted anchorages (except rock bolts), the bond length should not be less than the following, unless full scale tests confirm that shorter bond lengths are acceptable:

- (a) 3.0 m where the tendon is homed and bonded in situ; and

- (b) 2.0 m where the tendon is bonded under factory controlled conditions (see 6.4).

For rock bolts shorter tendon bond lengths may be used but subject to the recommendations of 6.3.2.

Where doubt exists concerning the adequacy of tendon bond length, the proposed design should be confirmed by trial anchorage tests (see 11.2.3).

6.3.4 Effects of surface condition on bond. Bond strength can be significantly affected by the surface condition of the tendon, particularly when loose or lubricant materials are present at the interface. The surface of the tendons should be free from loose rust, soil, paint, grease, soap or other lubricants. Where the tendon has been supplied coated with protective oils or greases, these should be thoroughly removed using steam or approved chemical solvents for which purpose it may be necessary to temporarily unravel strands over the length to be bonded.

A film of rust on the tendon as shown in figures 15(a) and (b) is not necessarily harmful and may improve the bond (see 8.1.1 and Federation Internationale de la Precontrainte (1986)).

Tendons showing signs of pitting should not be used.

6.3.5 Concentration of steel tendon in anchorage borehole. For rock bolts, the ratio of tendon area to the area of the borehole should be in accordance with the manufacturer's recommendations subject to previous usage in similar conditions. In the case of capsule usage the annular space between the bar and the borehole is critical to allow efficient mixing of the capsule contents.

For all other anchorages unless full scale tests confirm satisfactory performance in accordance with agreed acceptance criteria (see 11.2 and 11.3), the cross-sectional area of the steel tendon should not exceed 15 % of the borehole area for parallel multi-unit tendons and 20 % of the borehole area for single-unit tendons or suitably noded multistrand tendons, in order to minimize debonding (Bruce 1976, Barley 1978.)

Table 3. Design methods for fixed anchors				
Ground	Rock	Cohesionless soil	Cohesive soil	
Anchorage type (see figure 2)	Subclause 6.2.3	Subclause 6.2.4	Subclause 6.2.5	
A	Subclauses 6.2.2.2 and 6.2.3 and F.2 $L = \frac{T_w \times S_f}{\pi D \tau_{ult}} \quad (1A)$	Not applicable, except where soil is cemented and may behave as soft rock, see type A for rock	Subclause 6.2.5.2 and F.4 $L = \frac{T_w \times S_f}{\pi D \alpha C_u} \quad (4A)$	
B	See type A for rock	Subclause 6.2.4.2, figure 6 and F.3 $L = \frac{T_w \times S_f - B \gamma h \pi/4 (D^2 - d^2)}{A \sigma_v' \pi D \tan \phi'}$ (3A) or $= \frac{T_w \times S_f}{n \tan \phi'} \quad (2A)$	Not applicable	
C	Proving or on-site suitability tests required	Subclauses 6.2.4.3 and F.3.2 and figure 7 Experience based on field tests	Subclause 6.2.5.3 and figures 8 and 9	
D	Proving or on-site suitability tests required	Proving or on-site suitability tests required	Subclause 6.2.5.4 and figure 10 $L = \frac{T_w \times S_f - \pi/4 (D^2 - d^2) N C_u - d C_a \pi}{\pi D C_u} \quad (5A)$	
Overall limits max. min, see F.3	Subclauses 6.2.3.4, F.2.3 and table 26 10.0 m 3.0 m (except for rock bolts)	Subclauses 6.2.4.5, F.3.4 and figure 61 10.0 m 3.0 m	Subclause 6.2.5.5 10.0 m 3.0 m	

6.3.6 Resin and cement capsules. For fixed anchors formed using resin or cementitious grout capsules, full scale proving tests should be carried out unless relevant documented evidence is available. In order to judge satisfactory performance and degree of damage, if any, proving tests are required supplemented where necessary by creep monitoring and subsequent dissection of the fixed anchor.

There is a dearth of technical data on the design consideration and associated service performance for epoxy or polyester resin based grouts in the field of ground anchorages. The relative importance of the engineering characteristics are not fully understood, e.g. elastic modulus and creep behaviour under load are interrelated but in conflict, in that the lower the E value, which encourages uniform transfer of load and minimizes cracking, the higher the creep under service load. It is therefore difficult to optimize these parameters in a design calculation (see 7.2.3).

6.4 Encapsulations

6.4.1 General. Encapsulations generally take the form of single or multi-unit tendons grouted within a single corrugated duct or within two concentric corrugated ducts which effectively protect the tendon bond length against corrosion. The system allows transfer of load from the tendon across a number of material interfaces to the fixed anchor grout within the borehole. This load transfer mechanism is complex due to the different engineering properties of the various materials involved e.g. steel, grout, plastics.

Owing to the complexity of the encapsulation system only the limiting values of average bond stresses on the major interfaces which have been measured in large or full scale tests, should be used in design. In order to confirm that an encapsulation system can maintain its corrosion protection under load, proving tests should be carried out in accordance with 11.2.

Load transfer mechanisms for typical encapsulation systems are shown in figure 11.

6.4.2 Magnitude of bond. The ultimate bond stress assumed to be uniform over the external grout/encapsulation interface(s) should not exceed 3.0 N/mm^2 for cementitious grouts unless adequately proven. Attention should be paid to the pitch, amplitude and profile of the corrugations on the encapsulation surface (8.2.4.3.2). Ultimate bond stresses at the grout/tendon interfaces within the encapsulation should not exceed those specified in 6.3.2. For resinous grouts ultimate bond stresses should be obtained from proving tests in the absence of relevant documented test data.

6.4.3 Encapsulation length. Encapsulation should not be less than 2.0 m in length for straight shafted fixed anchors. Where underreams are used in clayey soils, encapsulation lengths should equate to that of the fixed anchor to avoid risk of crushing of the grout shaft between upper underreams.

6.4.4 Concentration of steel within encapsulation. The cross-sectional area of the steel within an encapsulation may be considerably greater than the acceptable cross-sectional area of steel in the borehole (see 6.3.5) due to the fabrication of the encapsulation under controlled conditions and the incorporation of high strength cementitious or resin grouts. Each encapsulation system should be checked to ensure adequate internal clearances for the passage of the grout and to ensure that the encapsulation capacity is equal to that of steel tendon. When tendons comprise a large number of strands particular attention should be given to the transfer of the tendon force from the inner layers of strand and the grout-encapsulation interface taking into account the location of any curtailment of strands. In these circumstances proving tests are required to confirm the adequacy of the proposed strand geometry.

6.5 Free anchor length

The determination of the free length in soils and rocks is discussed in D.2 and D.3, respectively.

7 Materials and components

7.1 Cementitious grouts

7.1.1 Cements. For ground anchorage construction in general, cement should be one of the following.

- (a) Ordinary Portland cement complying with BS 12.
- (b) Rapid hardening Portland cement complying with BS 12.
- (c) Portland blastfurnace cement complying with BS 146.
- (d) Low heat Portland cement complying with BS 1370.
- (e) Sulphate resisting Portland cement complying with BS 4027.
- (f) Low heat Portland blastfurnace cement complying with BS 4246.

The use of high alumina cement complying with BS 915 should be restricted to temporary anchorages with a service life not exceeding 6 months and to anchorages used for testing ground holding capacity.

As a prudent measure to minimize sulphide content, only cements complying with BS 12, BS 1370 or BS 4027 should be used for tendon bonding in permanent anchorages.

Cement in grout capsules should comply with BS 12 or BS 915.

7.1.1.1 Preblended cement replacement materials. The use of recognized cement replacement materials e.g. pulverized-fuel ash (p.f.a.) (see BS 3892), finely ground blastfurnace slag (see BS 6699) and finely divided silica, is acceptable for ground anchorages. Specialist advice should be sought before selecting cement replacement materials and the manufacturer's instructions should be strictly followed (8.2.4.2.2.1).

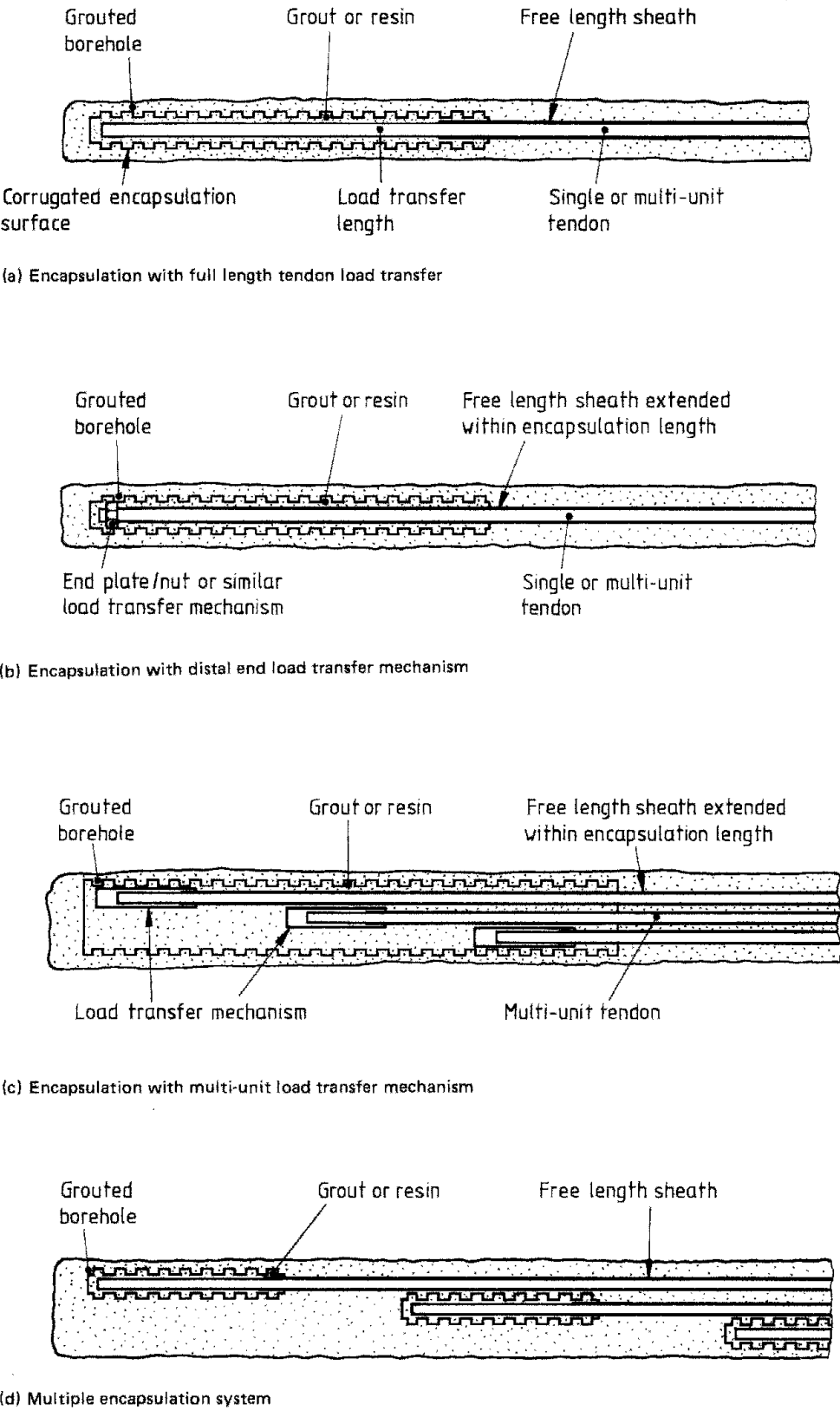


Figure 11. Load transfer mechanisms for typical encapsulation systems

7.1.1.2 Sulphate limits. The total sulphate content should not exceed 4 % (m/m) SO_3 of cement in the grout. For this purpose, the sulphate content, expressed as SO_3 , should be calculated from all sources as determined by the tests described in BS 1047, BS 3681, BS 3892 and BS 4550, where applicable. For hardened grout, sampling and chemical analysis should be carried out as described in BS 1881 : Part 6.

7.1.1.3 Chloride limits. The total chloride content of the grout derived from all sources should not exceed 0.1 % (m/m) of cement for stressed tendon bonding.

7.1.2 Fillers. Inert fillers such as sand or limestone dust are permissible in cement grouts. Use of fillers in tendon bonding grout is relatively rare and should be subject to 7.1.1.2 and 7.1.1.3.

7.1.3 Water. Water for mixing should not contain ingredients harmful to steel or cement grout. The water should not contain oil, organic matter or deleterious substances and should generally be in accordance with appendix A of BS 3148 : 1980. Water should not contain more than 500 mg of chloride ions per litre. If water is unfit to drink it should be subjected to chemical analysis.

7.1.4 Admixtures. Admixtures should only be used if tests have shown that their use improves the properties of the grout, e.g. by improving workability or durability, reducing bleed or shrinkage or increasing rate of strength development.

Admixtures should be free from any product liable to damage the steel or the grout itself. For example, no admixture that contains in total more than 0.1 % (m/m) of chlorides, sulphides or nitrates should be used.

Expanding agents containing aluminium to give grout expansion of up to 5 % may be used in accordance with the manufacturer's instructions. Wherever possible, admixtures should be used and tested in accordance with BS 5075.

Prior to the use of any admixture in the field, information related to the requirements of clause 7 of BS 5075 : Part 1 : 1982 and clause 7 of BS 5075 : Part 2 : 1982 should be supplied in writing by the manufacturer.

Admixtures to cementitious grouts in capsules should conform to the recommendations of this clause.

7.1.5 Grout properties. The design parameters of the grout should be in accordance with 6.2, 6.3, 7.1 and 8.2.

7.1.5.1 Mixes. Before grouting the following properties should be established:

- (a) type of cement;
- (b) water/cement ratio;
- (c) type and concentration of admixture and/or filler;
- (d) flow reading from viscometer, flow cone or flowmeter;
- (e) unconfined compressive strength development, e.g. at 3 days, 7 days, 14 days and 28 days; based on 100 mm cubes;
- (f) notes on amount of free expansion or shrinkage, bleed and setting time.

The mix should be retested if there is a change in the type or origin of the constituents.

7.1.5.2 Borehole anchorage grout. To ensure that the cement grout has good bond and shear strength, the mix should attain an unconfined compressive strength of 40 N/mm² minimum at 28 days, when the samples are manufactured, cured and tested in accordance with BS 1881.

The bleeding of the tendon bonding grout at 20 °C should generally not exceed 2 % of the volume 3 h after mixing and have a maximum of 4 % of the volume. Higher values may be permitted in the case of permeable ground where the bleed water is filtered from the grout during injection under pressure.

For anchorages the water/cement ratio of tendon bonding grouts generally lies in the range 0.35 to 0.60. In certain circumstances, it may be advantageous to use an initial grout mix of a higher water/cement ratio to facilitate tendon homing. The use of such mixes is permissible, provided that the dilute mix is displaced from the fixed anchor zone using a grout of suitable strength and consistency. For anchorages installed in low permeability ground, e.g. rock or clay, the water/cement ratio should not exceed 0.45. Guidance on the design of cement based grouts is provided by Littlejohn (1982b).

7.1.5.3 Encapsulation grout. Where tendon bonding cement grout acts as part of the corrosion protection, the maximum bleed should not exceed 0.5 % (see 8.2.4.2.2.1).

7.1.5.4 Capsule grout. The water/cement ratio in capsules with high alumina should not exceed 0.4.

7.1.5.5 Borehole pregrout. In pregrouting operations strength requirements are insignificant and need not be specified.

7.1.6 Health hazards and safety. Some chemicals used as admixtures in grouts are harmful and may also be severe skin irritants (CIRIA 1981, 1982). Barrier creams and gloves should always be worn by operatives and additional personal protection such as longsleeved overalls, goggles and respirator may be necessary, particularly if working in confined spaces. Manufacturer's safe handling instructions should always be followed. If no instructions are provided with an admixture, the user should seek advice from the supplier/manufacturer, who has a statutory duty to make available such information under Section six of the Health and Safety at Work etc. Act 1974.

7.2 Resinous grouts

7.2.1 Resins. Epoxy and polyester resins are most commonly used in capsules for rock bolting or fixed anchor protection encapsulations. Resins should be specifically designed and recommended for these anchorage applications by the manufacturer.

The selection of material performance properties should be made in conjunction with the manufacturer to suit the conditions of each particular application. Laboratory and field tests should be made to verify mix time, mixing efficiency, setting times and pull-out capacity.

In the absence of relevant British Standards, epoxy or polyester resins should be proved by tests (see BS 6319) and/or practical experience to have the following properties (see also 7.2.3.1):

- (a) adequate strength to transfer the applied stress safely;
- (b) suitably inert in the surrounding ground conditions;
- (c) where appropriate, capable of providing a barrier against corrosion of the steel tendon, e.g. no cracking under stressing and handling conditions;
- (d) adequate bond strength with tendon, adhesion in the case of smooth surfaces;
- (e) adequate ductility to ensure compatibility with load/extension characteristics of tendon up to 80 % f_{pu} ;
- (f) low post-gelation shrinkage;
- (g) sufficiently fluid, at installation temperature, prior to setting, to fill the void completely;
- (h) minimal creep under service load conditions.

Resins should be used strictly in accordance with the manufacturer's recommendations.

7.2.2 Fillers. Use of inert fillers may be beneficial technically and economically e.g. reduced exotherm and shrinkage. Many crushed minerals are suitable as fillers but to ensure compatibility of materials use of prepacked fillers supplied by the resin manufacturer is recommended. For capsule or encapsulation grouts, fillers should be graded with 100 % passing a 200 μ m sieve. For the rare case of resin injection of boreholes in situ, the maximum particle size may be increased to 1 mm.

7.2.3 Grout properties

7.2.3.1 General. The design parameters of the grout should be in accordance with 6.2, 6.3, 7.2 and 8.2.

7.2.3.2 Mixes. Resin systems are normally suitable for temperature conditions up to 40 °C. Where higher temperatures are envisaged special resin formulations should be used.

Before grouting, the following properties should be established:

- (a) type of resin;
- (b) type and concentration of filler;
- (c) flow reading (from viscometer, flow cone or flowmeter) where resin is to be injected;
- (d) compressive strength development with respect to time and temperature;
- (e) ultimate tensile strength;
- (f) elastic modulus and percentage extension at failure;
- (g) gel-time development with temperature;
- (h) bulk shrinkage after gelation;
- (i) exothermic value;
- (j) flash point;
- (k) shelf life.

The mix should be retested if there is a change in the type or origin of the constituents.

Post-gelation shrinkage should preferably be nil and not more than 5 %, otherwise debonding can occur, which in turn creates a potential corrosion hazard through the formation of leakage paths.

Traditionally, ultimate compressive and tensile strengths in excess of 75 N/mm² and 15 N/mm², respectively, are recommended for efficient load transfer, but elastic modulus and percentage extension at failure are just as important in relation to crack elimination for permanent anchorages, bearing in mind the ductility of steel tendons. With regard to this, it would appear that percentage extension at failure can be in the range 1.0 % to 1.5 % but currently a minimum elastic modulus that will guarantee negligible creep under service loading cannot be proposed. As a consequence where resin grout is proposed for tendon bonding, full scale tests (including sample sectioning) should be carried out prior to anchorage installation to prove the efficiency of the grout mix (see 6.3.6).

7.2.4 Health hazards and safety. For health hazards and safety pertaining to resinous grouts see 7.1.6.

7.3 Tendon

7.3.1 General. Tendons usually consist of steel bar, strand or wire either singly or in groups. Developments with other materials and material forms may find application in ground anchorages subject to satisfactory proving tests.

7.3.2 Prestressing steel

7.3.2.1 Specification. Non-alloy steel wire is covered by section two of BS 5896 : 1980, non-alloy 7-wire strand is covered by section three of BS 5896 : 1980 and low-alloy steel bar is covered by BS 4486.

At present there is no prestressing steel standard covering stainless steel wire, bar and stainless clad bar. The preferred composition of the steel is type 316S31 containing 0.15 % to 0.25 % nitrogen. The material may be either cold or warm worked.

7.3.2.2 Typical sizes and specified characteristic strengths. Table 4 includes typical data for prestressing steel that may be used in tendon design.

7.3.2.3 Load extension characteristics. For design purposes, the load-extension characteristics shown in figure 12 can be used. Values of γ_m can be found in BS 5400 : Part 4 or in BS 8110.

7.3.2.4 Relaxation

7.3.2.4.1 1000 h values. The maximum relaxation of force in the tendon at 20 °C after 1000 h, from an initial force of 70 % of the specified characteristic strength should comply with BS 4486 and BS 5896.

The 1000 h relaxation loss for the appropriate type of tendon should be taken from the manufacturer's UK Certificate of Approval. The information will normally be available for initial loads of 60 % f_{pu} , 70 % f_{pu} and 80 % f_{pu} and losses for intermediate jacking forces may be interpolated. For jacking forces of less than 60 % f_{pu} and where there is no experimental evidence, the relaxation at 1000 h may be assumed to decrease linearly, as follows:

(a) for normal relaxation (class 1) wire and strand: from 8 % of 70 % to 3 % of 50 % of specified characteristic strength;

(b) for low relaxation (class 2) wire and strand: from 2.5 % of 70 % to 1 % of 50 % of specified characteristic strength;

(c) for bar: from 4 % of 70 % to 1.5 % of 50 % of specified characteristic strength.

In the absence of a UK Certificate of Approval, the 1000 h relaxation should be taken as the maximum value for the appropriate initial force and type of tendon in the relevant British Standard for the product.

7.3.2.4.2 Loss of prestress due to relaxation of steel. The maximum loss of force in the tendon allowed for in design should be taken as the 1000 h relaxation loss, for an initial force equal to that imposed at transfer, times the appropriate multiplier coefficient given in table 5. Table 6 indicates typical relaxation figures with respect to time up to 1000 h for an ambient temperature of 20 °C.

Relaxation is temperature sensitive and in tropical climates, for example, the effect of higher ambient temperatures in the ground should be taken into account in the testing

Table 5. Recommended multiplier coefficients for estimating maximum relaxation loss in design

Anchorage category	Recommended multiplier		
	Normal relaxation (class 1)	Bar	Low relaxation (class 2)
Permanent	2.0	1.75	1.5
Temporary	1.75	1.50	1.25

programme, since relaxation of prestress increases significantly with increase in temperature.

Based on high temperature relaxation tests in the UK related to nuclear power applications, if the temperature exceeds 40 °C only low relaxation prestressing steel should be used. As a guide, the 1000 h relaxation loss from an initial stress of 70 % specified characteristic strength for low relaxation (class 2) wire or strand at slightly elevated temperatures, may be approximately related to the 1000 h loss at 20 °C (see table 6) using the multiplier coefficient given in table 7.

Table 4. Typical sizes and specified characteristic strengths for prestressing tendon design

Type of steel	Nominal diameter	Specified characteristic strength	Nominal steel area
Non-alloy steel Wire	mm	kN	mm ²
7-wire strand	7.0	60.4	38.5
	12.9	186	100
	15.2	232	139
	15.7	265	150
7-wire drawn strand	12.7	209	112
	15.2	300	165
	18.0	380	223
Low alloy steel bar Grade 1030/835	26.5	568	552
	32	830	804
	36	1048	1018
	40	1300	1257
Grade 1230/1080*	25	600	491
	32	990	804
	36	1252	1018
Stainless steel Wire	7	44.3	38.5
Bar	25	491	491
	32	804	804
	40	1257	1257

* This grade is not covered in BS 4486.

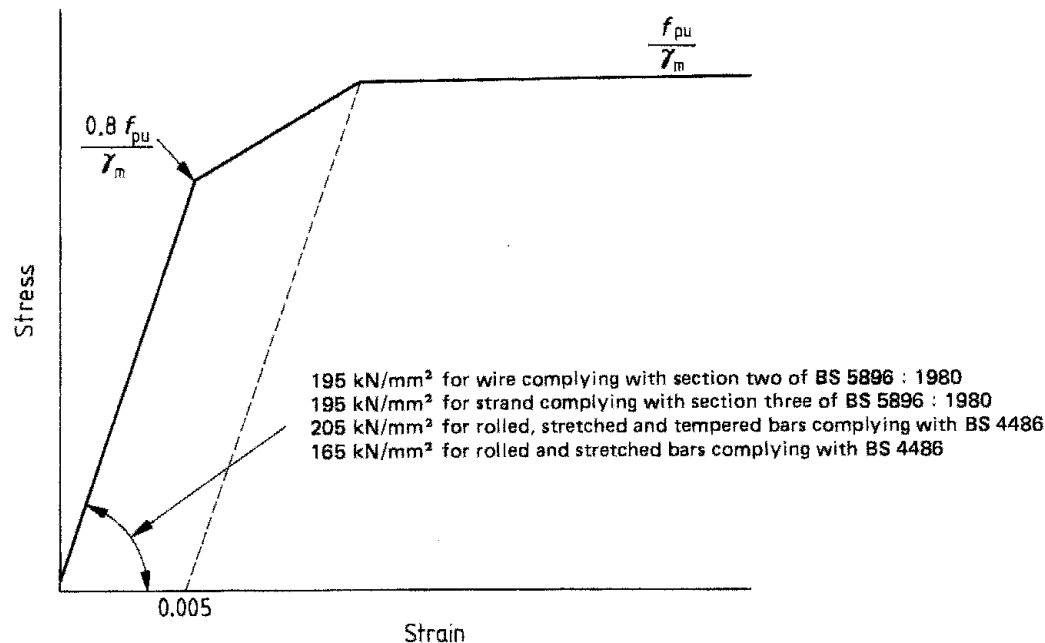


Figure 12. Short-term design stress-strain curve for normal and low relaxation products

Table 6. Typical relaxation of stress (as percentage of initial stress) from an initial stress of 70 % specified characteristic strength at a temperature of 20 °C

Tendon	Class of relaxation	Typical relaxation of stress					
		Elapsed time					
		0.1 h	1.0 h	10 h	100 h	500 h	1000 h
Prestraightened wire	Normal	%	%	%	%	%	%
	Low	0.25	0.80	1.90	2.90	4.00	4.50
7-wire strand	Normal	0.09	0.20	0.40	0.65	0.95	1.10
	Low	0.35	1.15	2.10	3.40	4.70	5.50
Low alloy bar	Normal	0.07	0.33	0.60	0.84	0.98	1.10
	Low	0.90	1.80	2.45	2.75	2.90	3.05
Stainless steel wire and bar*		1.0	3.0	4.5	5.1	5.9	6.3

* Relaxation figures are for wire only and are based on very limited information.

No reduction in the relaxation should be allowed for a tendon that has been subjected to a load in excess of jacking force for a short time before anchoring the tendon.

7.3.3 Reinforcement steel

7.3.3.1 Specification. For rock bolts or dowels not requiring high strength, steel of reinforcing quality in either smooth or deformed bar complying with BS 4449, BS 4461 or BS 6744 may be used as an alternative to prestressing steel.

For applications where corrosion resistance is required, galvanized reinforcement or, alternatively, austenitic stainless steel complying with BS 6744 may be used.

7.3.3.2 Typical sizes and specified characteristic strengths.

Table 8 includes typical data for UK manufactured reinforcing steel that may be used in bolt or dowel design:

7.3.3.3 Couplers. Tendons manufactured from bar may be extended using appropriate couplers. The strength of the coupled bars should not be less than that of the tendon.

Table 7. Recommended multiplier coefficients for estimating 1000 h relaxation loss (from 70 % specified characteristic strength) of low relaxation (class 2) wire or strand at elevated temperatures

Temperature	Multiplier coefficient
°C	
20	1
30	1.5
35	1.8
40	2
60	3
80	4

Couplers should be designed such that grouting of the anchorage is not seriously impeded. Care should be taken to ensure adequate corrosion protection of the tendon at the coupler and to ensure that the coupled section is free to move during stressing. Couplers should not be used in conjunction with grout in capsules as the presence of the couplers does not allow full mixing of the grout during spinning of the bar in the hole.

7.3.4 Protective coverings

7.3.4.1 Sheathing. The individual tendon may be covered with a protective oil or grease and an outer flexible plastics sheath, applied either during manufacture or when making up the complete tendon on site.

Manufacturer's recommendations should be sought about their specific product followed by pull-out tests on greased and sheathed tendon samples to confirm efficiency of decoupling.

Plastics sheathing should be in accordance with 8.2.1, 8.2.2 and 8.2.4.2.3.5.

7.3.4.2 Ducting. The tendon bond length requires the same degree of protection as the free length. Since the protective elements have to transmit high tendon stresses to the ground, semi-rigid corrugated ducting should be used where the strength and deformability characteristics have been checked structurally (see 6.3 and 11.2). BS 4962

provides guidance on dimensional tolerance and performance requirements for plastics pipe.

7.3.5 Centralizers. Centralizers should be provided on all tendons or encapsulations to ensure that the tendon or encapsulation is centred in the grout column. Centralizers should ensure within the borehole a minimum grout cover of 10 mm to the tendon or encapsulation at the centralizer and should be provided at centres according to the angle of inclination of the ground anchorage and the possible sag between points of support in order to provide a minimum grout cover of 5 mm to the tendon or encapsulation.

The design of centralizers within encapsulations should ensure a minimum 5 mm grout cover where such cover is deemed to be a physical protective barrier.

For rock reinforcement systems using a bar spun into resin, a centralizer, where required, may be used to retain the bar in the centre of the hole and to retain resin in the fixed anchor section in up hole configurations. Where this centralizer is also used as a spacer, it should ensure within the borehole the minimum specified grout cover.

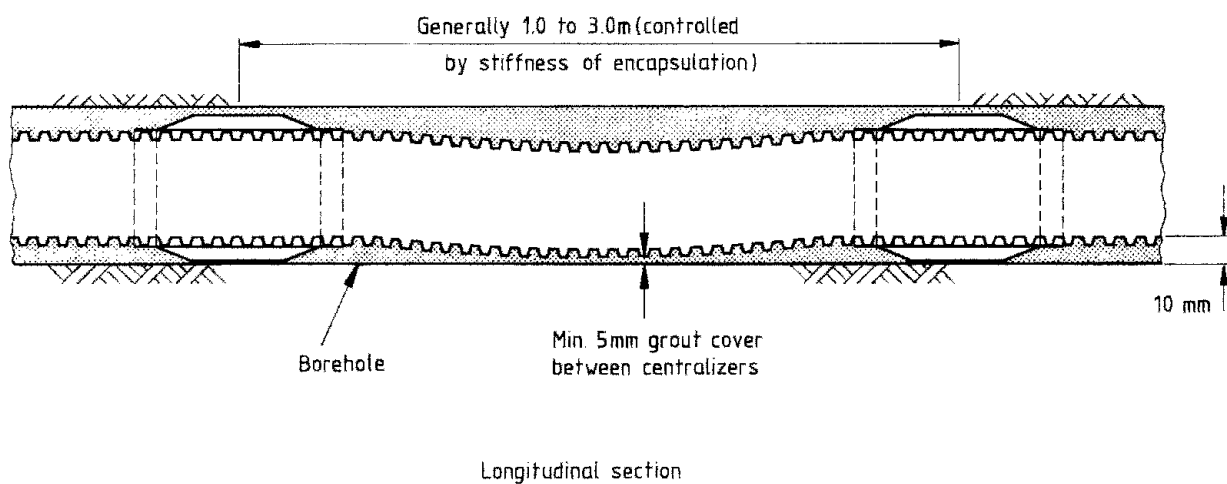
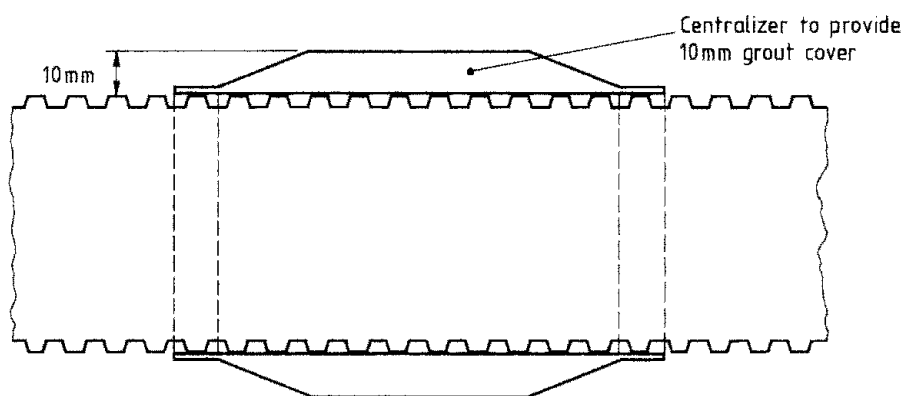
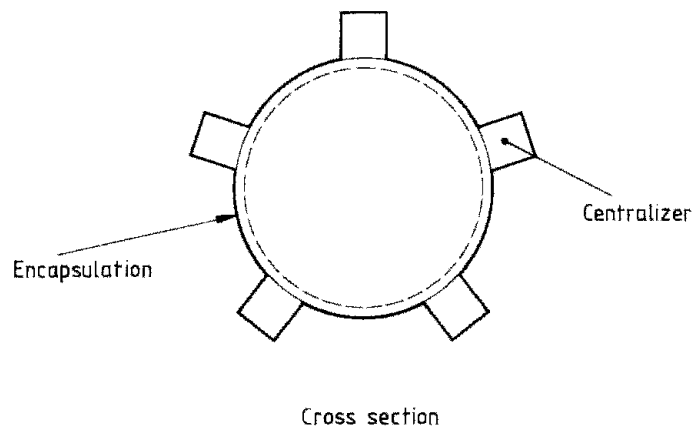
Centralizer design in soft cohesive ground, e.g. clay, requires special attention with respect to component dimensions, since thin sections may penetrate the clay.

7.3.6 Spacers (see figure 14). Spacers should be provided in the fixed anchor length of all multi-unit tendons to ensure separation between the individual components of the tendon and thus the effective penetration of grout to provide adequate bond. A minimum of three spacers should be provided in each fixed anchor length, but optimum distance between spacers may be dictated by potential sag of tendon units between points of support. The spacer should not be compressible nor cause decoupling of the fixed anchor grout (see appendix A). With regard to this, the spacer should not be bulky and it may be advantageous to keep the spacer arms of small dimension so as not to inhibit grout flow. The spacer should be designed to provide a clear spacing between tendon units of not less than 5 mm for parallel multi-unit tendons (see 6.3.2).

The practice of wire unravelling of strands to create a bushing effect for the complete tendon cannot guarantee spacing or cover and therefore should be restricted to temporary anchorages and where the corrosion risk is low.

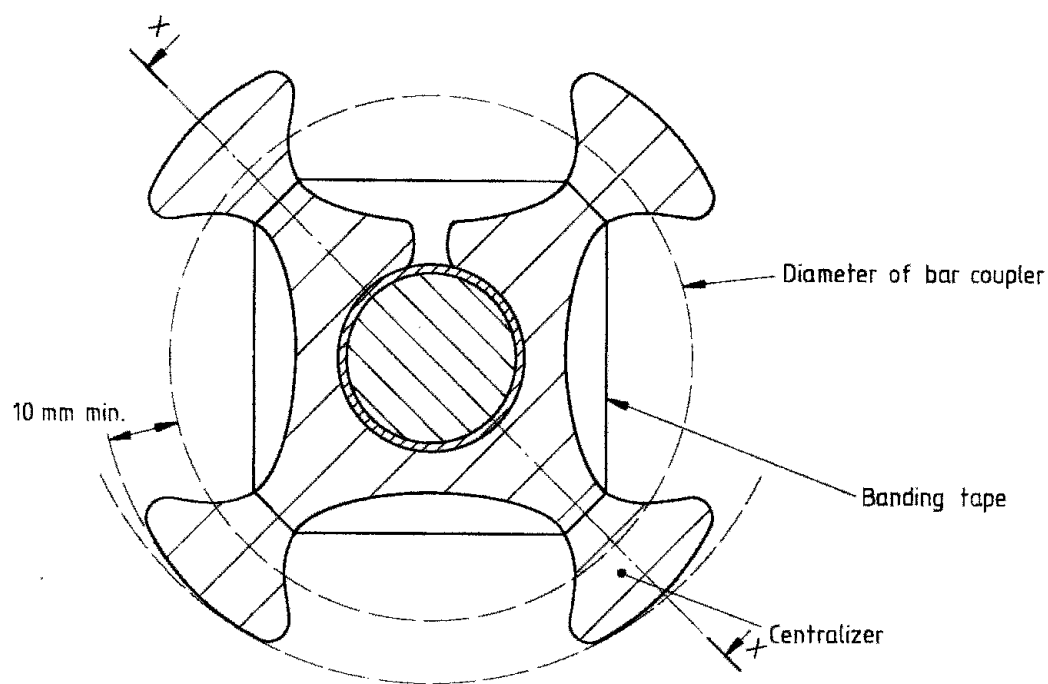
Table 8. Typical sizes and specified characteristic yield stresses for reinforcing bar

Type of steel bar	Nominal diameter	Characteristic stress
	mm	N/mm ²
Mild steel smooth (BS 4449)	20, 25, 32, 40, 50	250
High yield deformed (BS 4449)	20, 25, 32, 40, 50	460
Cold worked deformed (BS 4461)	20, 25, 32, 40, 50	460
Stainless steel deformed (BS 6744)	20, 25, 32, 40	460

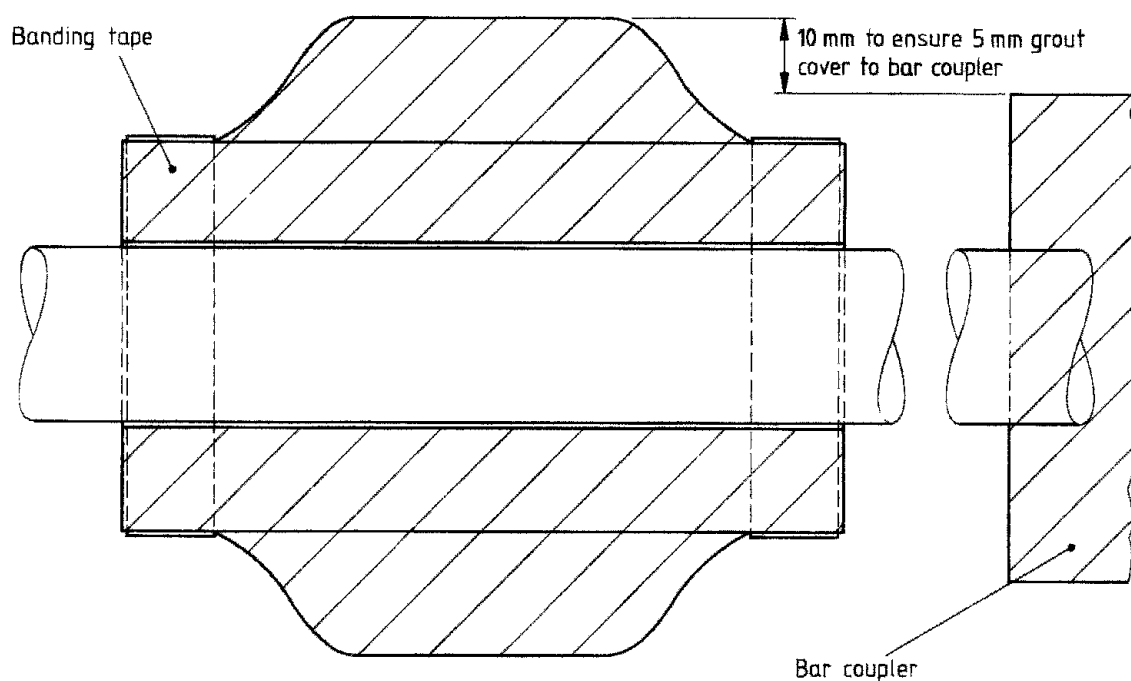


(a) Typical encapsulation centralizer

Figure 13. Centralizers



Cross section

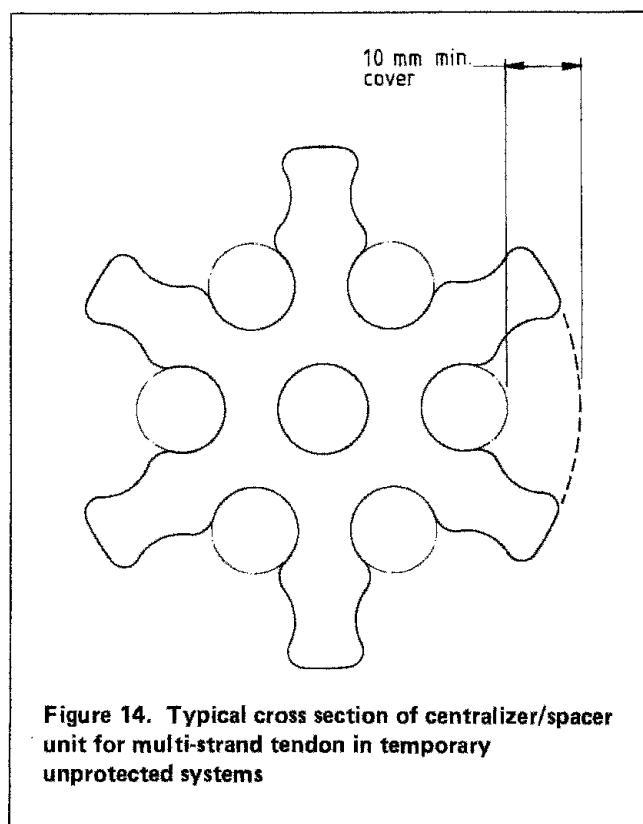


Longitudinal section X-X

NOTE. Position of centralizer is controlled by stiffness of bar and bar coupler location. Spacing of centralizers generally 2 m to 4 m.

(b) Typical bar centralizer for rock anchorages

Figure 13 (concluded)



7.3.7 Centralizer/spacer materials. Centralizers and spacers should be made of materials having no deleterious effect on the tendon itself. The use of metals dissimilar to the tendon should be avoided (see 8.1).

It is often convenient, particularly in temporary, unprotected systems, to use a combined centralizer/spacer unit having the characteristics of both for the whole of the tendon length (see figure 14).

7.3.8 Acceptable working loads. Working loads should not exceed 62.5 % and 50 % of the characteristic strength of the tendon for temporary and permanent works, respectively, under normal circumstances (see 6.1).

To distribute load to the ground more uniformly strands of differing length are sometimes used within the fixed anchor zone. When these are stressed simultaneously displacements at the anchor head are the same for all strands, and thus the strains and hence stresses differ in individual strands. In such cases the stress in the shortest strand should limit the acceptable working load. If the design requires uniform stresses within the tendon, mono-strand stressing is essential (see 10.6.3.4).

7.3.9 Fatigue. The tendon/anchor head system should comply with BS 4447 in relation to both static and dynamic considerations.

7.4 Anchor head

7.4.1 General. The anchor head normally consists of a stressing head in which the tendon is anchored and a bearing plate by which the tendon force is transferred to

the structure or excavation. Secondary distribution systems in the form of concrete blocks or steel walings then transfer the force to the main structure.

The components of the anchor head should be suitable for use with the tendon system proposed. The stressing head and bearing plate should be designed to carry the proof load of the anchorage in both the ultimate and service limit states. If necessary, the sleeve through the concrete anchor block, steel waling or other load distribution system should be of sufficient size to permit the removal and replacement of an anchorage at any time in the future. The protection system applied to the anchor head components should be consistent with the level of protection envisaged for the rest of the anchorage. In certain circumstances it will be necessary for the seals of the anchor head to be designed to be capable of resisting a hydrostatic pressure and tests may be needed to demonstrate the ability of the anchor head to sustain the design pressure. (Figures 25 and 27.)

The form of the anchor head system should comply with 8.7.5.2 of BS 8110 : Part 1 : 1985.

7.4.2 Stressing head. The stressing head should be designed to permit the tendon to be stressed and anchored at any force up to 80 % of the characteristic tendon force and should permit force adjustment up or down during the initial stressing phase. The performance of the stressing head should comply with BS 4447.

Monitoring requirements during service will dictate the need for a normal, restressable or detensionable head (see 2.2.5, 2.2.6 and 2.2.7, figures 37 and 38).

The stressing head should be designed to anchor the tendon without damaging it and also permit an angular deviation of $\pm 5^\circ$ from axial position of the tendon. It should be verified in a static tension test as described in BS 4447 that this angular deviation does not have an adverse influence on the ultimate load carrying capacity of the anchor head (see 10.5.1).

Heads for decoupled tendons should be equipped with a means to prevent any accident due to the energy of release of a tendon failure.

7.4.3 Bearing plate. The bearing plate is under the stressing head to distribute the tendon force into the main structure or excavation. It should be designed in accordance with either BS 5950 or BS 5400 : Parts 2, 3 and 6.

Where the bearing plate is bedded on to concrete the bedding mortar thickness should not exceed 10 % of the plate width nor 100 mm in thickness. The design bearing stress should not exceed 30 % f_{cu} (28 day strength) at the normal working force in the tendon.

7.4.4 Support plinth or pad in concrete. Where the bearing plate is bedded to the main structure through a concrete plinth or pad, i.e. where the depth of concrete exceeds 10 % of the bearing plate width or 100 mm, the plinths should be designed in accordance with either 4.11 of BS 8110 : Part 1 : 1985, or BS 5400 : Parts 2, 4 and 7 for the design of post-tensioned concrete end blocks. Anchorage plinths on retaining walls should be considered as part of the wall design.

For rock reinforcement applications, bearing plates may be bedded on to resin or mortar pads. For low capacity rock bolts and dowels, it is generally sufficient for the plate to bear directly onto the rock surface.

7.4.5 Steel grillage support. Where the bearing plate is attached to a steel structure, e.g. by a steel grillage, this should be designed in accordance with either BS 5950 or BS 5400 : Parts 2, 3 and 6.

7.4.6 Design of support structures for a group of anchor heads. In cases where part of the main structure contains a group of anchor heads, e.g. a waling of a tied wall, this part of the structure should be designed so that any one anchorage can fail without the load in other anchorages exceeding that proof load to which they were initially tested (see also D.2.1).

In addition the structure should be capable of sustaining the redistributed forces in the ultimate limit state with $\gamma_{fL} = 1$ (see also Stille (1976); Stille and Broms (1976)). This calculation should be made taking the load/extension characteristics of the tendons into consideration.

8 Corrosion and corrosion protection

8.1 Corrosion

8.1.1 General. The object of design against corrosion is to ensure that during the economic design life of the anchorage the probability of unacceptable corrosion occurring is small.

The time taken for a depassivating front to penetrate to the tendon steel will depend on the quality of the grout, whether or not the grout is cracked, the crack width, the cover and the nature of the environment (Beeby, 1978). Cracked grout will afford little protection in a marine environment. The problem is to define the rate of corrosion and what is unacceptable corrosion. A film of rust on the tendon is not necessarily harmful and may improve the bond, but any form of pitting is unacceptable (see 6.3.4). In terms of tendon supply, manufacturing defects such as longitudinal cracks or depressions in wire or bar that do not impair the specified physical properties are currently accepted, provided that these defects are less than 4 % of the nominal diameter of the tendon components or up to 1 mm maximum depth of depression, whichever is the lesser (see 7.3). These manufacturing defects should be borne in mind in any detailed assessment of corroded tendon. (Figure 15.)

Based on field experience anchorages, which are installed to deal with the construction phase requirements of a project, generally have a service life of up to 2 years. As a consequence a service life of 2 years is a reasonable period to define the limit for temporary anchorages. Where regular monitoring of anchorage performance is carried out (see 11.5) it may be possible to extend this service life. For all permanent ground anchorages and temporary anchorages in non-benign conditions corrosion protection appropriate for the circumstances should be provided (see 8.2.1).

Good durability of ground anchorages is vital in ensuring their structural effectiveness and performance over the long

term. In this respect, protection of the tendon against corrosion is essential and an understanding of possible corrosion hazards is of particular value when choosing protective systems. The main hazards arise as a result of ground aggressivity (see 5.6 and 8.1.3), differential embedment (see J.1.6) and bimetallic action (see J.1.2). Stray currents may also pose problems (see J.2.7). The results of such corrosive action can be manifest in generalized corrosion (see J.2.2), localized corrosion leading to pitting attack (see J.2.3) or highly localized corrosion in which cracking of the tendon occurs, this phenomenon being known as stress corrosion cracking (see J.2.5) and corrosion fatigue (see J.2.6).

8.1.2 Corrosion of steel in hydraulic cement

8.1.2.1 General. Steel is protected against corrosion when maintained in a high pH environment free of aggressive ions. Such an environment is provided by hydrated hydraulic cement, which will give excellent protection over the long term while the high level of alkalinity remains. However, loss of protection can occur as a result of lowering of alkalinity, through cracks or carbonation, or the presence of aggressive ions, especially chloride. It is therefore essential, if cement grouts or other hydraulic cement based materials are used as part of a double protection system, to exclude the possibility of alkalinity loss or aggressive ion ingress into the protective system (see J.3).

8.1.2.2 Influence of cracking. With regard to corrosion of steel in concrete and its relation to cracking (Beeby 1978), corrosion is likely to start first where a bar intersects a crack. In the short term, e.g. 2 years, there is a significant influence of crack width on the amount of corrosion found near a crack. In the long term, e.g. 10 years, however, based on observations of 0.05 mm to 1.5 mm cracks, the effect of differences in these widths of crack on the amount of corrosion reduces dramatically. Nevertheless, the smaller the crack the lower is the corrosion risk. Whilst there is little field evidence to indicate what crack widths are acceptable in a cementitious protective barrier, an upper limiting crack width of 0.1 mm is proposed for guidance, based on the work of Houston et al. (1972), Ryell and Richardson (1972), Schiessel (1975), Beeby (1978), Naus (1979) and O'Neil (1980).

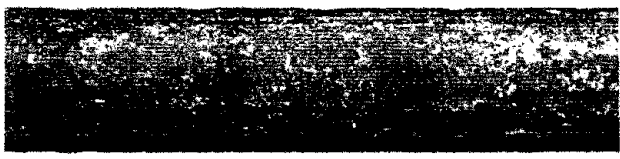
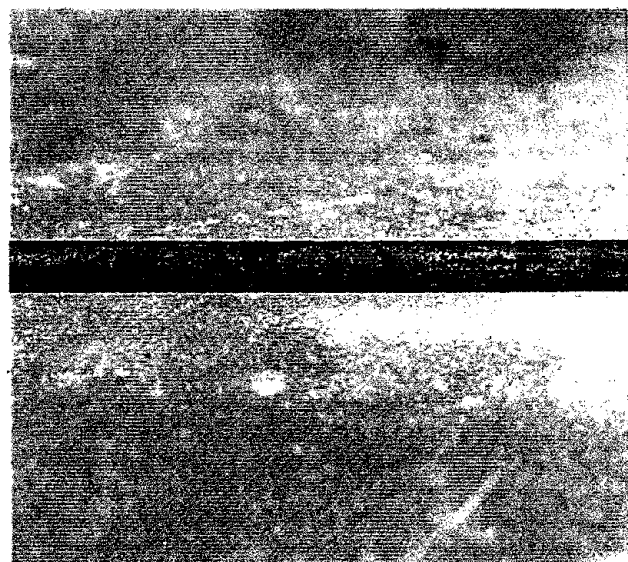
8.1.3 Ground aggressivity

8.1.3.1 Aggressivity towards metals. While water content, aggressive ion content, e.g. chloride and sulphate ions, and permeability of the ground all influence corrosion, it is now accepted that some generalized measure of redox potential and soil resistivity can be applied to the assessment of potential soil corrosiveness to embedded metals (King 1977).

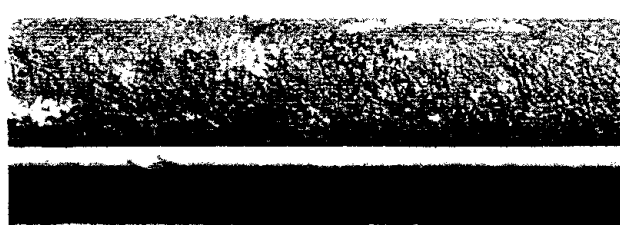
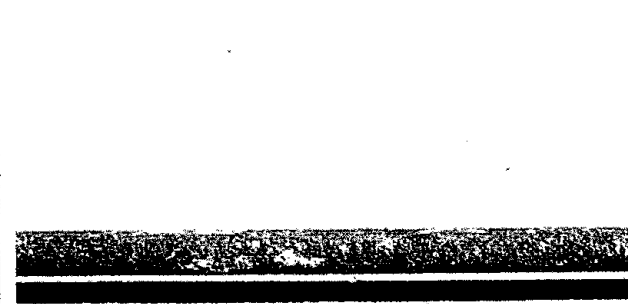
Table 9 provides guidance for soils of single composition and special precautions may be necessary where the anchorage passes through strata of differing composition to avoid the development of differential embedment cells (see J.1.6). ASTM report STP 741 (1979) on corrosion provides useful data on the processes and measurements involved underground. In general, fills and disturbed soils demand careful investigation.



(a) Normal acceptable condition



(b) Minimum acceptable condition



(c) Unacceptable condition

Caution. Visual appearance is a useful indicator for the rejection of tendon due to corrosion. Where doubt exists rejection should be based on tensile strength testing and/or examination of metallographic sections of samples taken to represent the worst cases of corrosion found.

Figure 15. Conditions of surface corrosion for steel tendon

Table 9. Corrosiveness of soils related to values of resistivity and redox potential (after King (1977))

Corrosiveness	Resistivity	Redox potential (corrected to pH = 7). Normal hydrogen electrode
	$\Omega \cdot \text{cm}$	mV
Very corrosive	< 700	< 100
Corrosive	700 to 2000	100 to 200
Moderately corrosive	2000 to 5000	200 to 400
Mildly corrosive or non-corrosive	> 5000	> 430 if clay soil

NOTE. In the absence of the above tests, ground and ground water samples should be taken for detailed chemical analysis e.g. chloride and sulphate ions, in order to judge aggressivity.

8.1.3.2 Aggressivity towards cementitious grout or concrete

8.1.3.2.1 General. Recommendations regarding grout durability are based on those already published for concrete and are not developed from data on grouts. The more important conditions are described below.

8.1.3.2.2 Sulphate-bearing ground and ground waters. Aqueous solutions of sulphates attack the set cement in concrete or grout, the chemical reactions occurring depending upon the kind of sulphate present and the nature of the cement. The rate of attack depends greatly on the permeability of the concrete or grout. The factors influencing attack are as follows:

- (a) amount and nature of the sulphate present;
- (b) level of the water table and its seasonal variation;
- (c) form of construction;
- (d) type and quality of the concrete or grout.

If sulphate conditions cannot be prevented from reaching the structure, the only defence against attack lies in the control of the fourth factor; it cannot be too strongly emphasized that dense concrete or grout of low permeability should be used (Littlejohn 1982a).

Whilst there is an absence of test data on cement grout, Table 10 (BRE Digest 250, 1981) gives recommendations for guidance on the requirements for concrete exposed to sulphate attack. In this regard where the ground or ground water contains chloride, nitrate or phosphate ions, BS 1377 : 1975 Test 10 is not appropriate and the gravimetric procedure should be used.

8.1.3.2.3 Acid-bearing ground and ground waters. Experience seems to indicate that, despite the vulnerability of calcareous cements to attack from acids, for most uses below ground level little erosion of well-made concrete takes place when the pH is above 5.5 and the water stagnant. Where the pH is between 5.5 and 3.5, a dense grout should be ensured and the use of pulverized-fuel ash (PFA) or ground granulated slag may be beneficial (Gutt and Harrison 1977). Values of pH below 3.5 are unusual and, unless in stagnant water, alternative grouts to those based on Portland cement should be considered.

8.1.4 Stray electrical currents. The presence in the ground of stray electrical currents arising adjacent to electrical plant, e.g. electrified railways, and cathodic protection systems can cause corrosion of a steel tendon if this becomes the anode in a galvanic process (J.2.7).

8.2 Corrosion protection

8.2.1 General recommendations. The safety of people and property in the event of anchorage failure should be balanced against the cost of providing protection. Since unprotected ground anchorages of steel will probably corrode in time, it is also necessary to decide whether the rate of corrosion merits the expense of protection. Corrosion rates vary enormously according to anchorage and working mode. Further, there is no certain way of identifying corrosive circumstances with sufficient precision to predict corrosion rates (see 8.1.2.2). Consequently, as a general rule, permanent anchorages should be protected.

In current practice the design solution ranges from double protection, implying two physical barriers against corrosion, in aggressive permeable soils, to simple grout cover (see figure 16) in the case of low permeability non-aggressive rock ($k_w < 10^{-10}$ m/s) or where low capacity rock bolts are used solely as secondary reinforcement. For high capacity permanent anchorages installed in low permeability rock, designers should insist on at least one physical barrier to protect the tendon against corrosion (single protection). Nevertheless, satisfactory performance has been observed where an alkaline environment is the only protection against corrosion (Federation Internationale de la Precontrainte 1986).

Temporary anchorages should normally be expected to last two years and this period is proposed as the limit for temporary anchorages in any circumstances. In very aggressive conditions (see table 9) or where there is a risk of local damage or corrosion by pitting, unprotected anchorages may last only a few weeks. The anticipated working life of the anchorage should include an allowance for any unforeseeable delay. Various degrees of protection are possible, but unless there is positive demonstration that anchorages are not at risk within their working life, protection graded according to severity of the problem should be provided.

In general, for corrosion resistance, the anchorage should be protected overall as partial protection of the tendon may only induce more severe corrosion on the unprotected part. Thus, the least protected zone of a ground anchorage defines the class of protection provided, e.g. single or double.

Choice of class protection (table 11) is the responsibility of the designer. The choice depends on such factors as consequence of failure, aggressivity of the environment and cost of protection.

By definition single protection implies that one physical barrier against corrosion is provided for the tendon prior to installation. Double protection implies the supply of two barriers where the purpose of the outer second barrier is to protect the inner barrier against the possibility of damage during tendon handling and placement. The second barrier therefore provides additional insurance, given the distinction between the degree of protection of the tendon once installed in the ground, and that of the tendon as supplied.

Table 10. Recommendations for concrete exposed to sulphate attack

Concentration of sulphates expressed as SO ₃				Type of cement	Requirements for dense, fully compacted concrete made with aggregates complying with BS 882 or BS 1047	
Class	In soil		In ground water		Minimum cement content	Maximum free water/cement* ratio
	Total SO ₃	SO ₃ in 2:1 water/soil extract				
1	% less than 0.2	g/L less than 1.0	g/L less than 0.3	Ordinary Portland cement (OPC) Rapid-hardening Portland cement (RHPC) or combinations of either cement, cement with slag† or pulverized-fuel ash (p.f.a.)‡ Portland-blastfurnace cement (PBFC)	kg/m ³ Plain concrete § 250 Reinforced concrete 300	0.70 0.60
2	0.2 to 0.5	1.0 to 1.9	0.3 to 1.2	OPC or RHPC or combinations of either cement with slag or p.f.a. PBFC	330	0.50
				OPC or RHPC combined with a minimum 70 % or maximum 90 % slag OPC or RHPC combined with minimum 25 % or maximum 40 % p.f.a.¶	310	0.55
				Sulphate-resisting Portland cement	290	0.55
3	0.5 to 1.0	1.9 to 3.1	1.2 to 2.5	OPC or RHPC combined with minimum 70 % or maximum 90 % slag OPC or RHPC combined with minimum 25 % or maximum 40 % p.f.a.	380	0.45
				SRPC	330	0.50
4	1.0 to 2.0	3.1 to 5.6	2.5 to 5.0	SRPC	370	0.45
5	Over 2	Over 5.6	Over 5.0	SRPC protective coating (see CP 102)	370	0.45

* Inclusive of content of p.f.a. or slag. These cement contents relate to 20 mm nominal maximum size aggregate. In order to maintain the cement content of the mortar fraction at similar values, the minimum contents given should be increased by 50 kg/m³ for 10 mm nominal maximum size aggregate and may be decreased by 40 kg/m³ for 40 mm nominal maximum size aggregate.

† Ground granulated blastfurnace slag BS 6699.

‡ Selected or classified pulverized-fuel ash complying with BS 3892.

§ When using strip foundations and trench fill for low-rise buildings in class 1 sulphate conditions further relaxation in the cement content and water/cement ratio is permissible.

|| Per cent by mass of slag/cement mixture.

¶ Per cent by mass of p.f.a./cement mixture.

Table 11. Proposed classes of protection for ground anchorages (see CESMM 1985; Federation Internationale de la Precontrainte 1986)

Anchorage category	Class of protection
Temporary	Temporary without protection Temporary with single protection Temporary with double protection
Permanent	Permanent with single protection Permanent with double protection

8.2.2 Properties of a protective system. A protective system should have the following properties:

- have an effective life at least equal to that required of the anchorages;
- not adversely affect the environment nor the efficiency of the protected anchorage;
- not restrict movement of the free tendon length, at least until after the anchorage has been locked off permanently, where restressability is not required;
- comprise materials that are mutually compatible with respect to deformability, permanence and avoidance of corrosive circumstances;
- allow for once only treatment because, with few exceptions, protective systems can be neither replaced nor maintained;
- not fail during stressing to proof load, especially at junctions between components of the protective system;
- be tough enough to withstand handling during manufacture, transport and installation in the ground;
- permit inspection prior to installation, where practicable.

8.2.3 Principles of protection. To fulfil the recommendations of 8.2.2, protective systems usually aim to exclude a moist gaseous atmosphere around the metal by totally enclosing it within an impervious covering or sheath.

There is a variety of protective coatings or coverings. The principles of protection are the same for all parts of the anchorage, but different detailed treatments are necessary for the tendon bond length, tendon free length and anchor head.

Protective measures include one or more coatings applied during manufacture of the tendon and essentially attached to it and also material introduced as fluids within the coating. Grout injected in situ to bond the tendon to the ground does not constitute a part of a protection system because the grout quality and integrity cannot be assured.

Injected fluid material should ideally remain ductile to constitute a physical barrier that does not crack or debond.

Fluid materials that become brittle on hardening crack in service as the structure suffers differential strains, the onset of cracking depending upon tensile strength and ductility.

Typically, when smooth bar, wire or strand tendons in cement fixed anchor grouts are stressed, cracks tend to occur at about 50 mm to 100 mm apart and of widths up to 1 mm or more (see also Graber (1981) and Meyer (1977)). Rate of tendon corrosion can be inhibited by reducing crack width to not greater than, for example 0.1 mm (see 8.1.2.2). This inhibition arises from fineness of cracks in conjunction with alkaline environment of cement. By contrast, although ductile, cracked resin does not provide protection, since the resin is inert. Crack width in cements may be controlled and reduced by incorporation of indented bar or spiral steel cages or meshes within the grout to reduce crack spacing, but there is currently a lack of field evidence to substantiate the design concept in practice (see appendix A).

In ribbed bar tendons, field evidence (figure 17) illustrates that the ribs can control the frequency of cracking within a corrugated duct encapsulation, to such an extent that the crack widths are less than 0.1 mm. For this situation, the inner cracked grout will provide a physical barrier to corrosion. The field work (Ostermayer and Scheele 1977) was carried out in compact gravelly sands ($D_r = 76\%$; $\phi = 42^\circ$) (but see also A.5).

If protective grout is injected in situ, e.g. where a long rigid pregouted capsule would be difficult to install, visual inspection of the grouted bond length is impossible and quality control of the highest standard is required. Some encapsulations can be water tested to check their integrity after tendon installation and prior to grouting, but accurate monitoring is essential and care is required in the interpretation of results.

Non-hardening fluid materials such as greases have limitations as corrosion protection media. Reasons include:

- fluids are susceptible to drying out, this is usually accompanied by shrinkage and change in chemical properties;
- fluids are liable to leakage if even slight damage is sustained by their containment sheaths;
- fluids having virtually no shear strength are easily displaced and removed from the metal they are meant to protect;
- even in ideal conditions their long term chemical stability, e.g. susceptibility to oxidation, is not known with confidence.

These aspects require that non-hardening materials are themselves protected or contained by a moisture proof, robust form of sheathing, which must itself be resistant to corrosion.

Whilst non-hardening fluids such as a grease fulfil an essential role in corrosion protection systems, in that they act as a filler to exclude the atmosphere from the surface of a steel tendon, create the correct electrochemical environment and reduce friction in the free length, they should not be regarded as a permanent physical barrier to corrosion.

Therefore, a layer of grease is not considered acceptable as one of the physical barriers required in the decoupled free length of a double corrosion protection system. Grease is acceptable, however, as a protective barrier in a restressable anchor head, since the grease can be replaced or replenished.

Use of thicker metal sections for the tendon, with sacrificial area in lieu of physical barriers, gives no effective protection, as corrosion is rarely uniform and extends most rapidly and preferentially at localized pits or surface irregularities. The presence of such corrosion pits cannot be rectified by brushing or covering.

There is a dearth of information on the performance of cathodic protection of prestressing steel tendons at the time of publication of this standard. If cathodic protection is considered for ground anchorages, some unattractive factors warrant attention (K.2) and for detailed design guidance reference should be made to CP 1021.

Non-corrodible metals may be used for anchorage components, subject to verifying their electrochemical behaviour relative to the other components, and stress corrosion characteristics in appropriate environments.

Non-metallic fibres may be used for tendons, subject to investigation of their effective life in stressed condition when exposed to potentially aggressive environments that may differ from those aggressive to steel. No further guidance can be given at the time of publication of this code.

8.2.4 Protective systems

8.2.4.1 General. Recommendations concerning some commonly used protective systems for anchorage components are given in 8.2.4.2 to 8.2.4.5. Designers should choose systems appropriate to their assessment of service conditions.

8.2.4.2 Free length of tendon

8.2.4.2.1 Methods of protection. Protection is achieved generally by either injection of solidifying fluids to enclose the tendon or by pre-applied coatings, or by a combination of both, depending on circumstances. Typical examples of protection for the free length are illustrated in figures 18(a), (b) and (c). In the free length, the protective system must permit reasonably uninhibited extension of the tendon during stressing, and thereafter, if the anchorage is restressable.

8.2.4.2.2 Injected fluids

8.2.4.2.2.1 Solidifying fluids. Solidifying fluids are injected round the steel tendon where such simple protection is judged appropriate and the anchorage is not restressable, or, in combination with a sheath (see 8.2.4.2.3.5) where a higher degree of protection and/or restressability is required. Injection should be by tremie (see 10.4.4) from the bottom of the hole, displacing any atmosphere or water out of the hole. Centralizers on the tendon and fixed anchor are essential to ensure uniform cover of injected materials in the annulus. Packers may be necessary to retain external grout placed inside holes angled upwards. Injection pressures up to 250 kN/m² may be needed to ensure full penetration of the grout and exclusion of fluids as far as practicable from any voids, subject to fracturing the ground.

Neat cement grouts are most frequently used, sometimes with bentonite or other admixtures (see 7.1.4). Minimum grout cover around the tendon should be 5 mm. For temporary anchorages, this cover does not require an outer sheath, unless the ground is aggressive.

If restressable anchorages are contemplated, the tendon should be surrounded by a sheath to prevent bonding with the grout. Light corrugated metal sheaths are not suitable for corrosion protection, since they are easily perforated by corrosion. Any metal used has to be compatible with the tendon, so as not to induce corrosion potentials between the differing metals (see J.1.2 and J.1.3).

Cement used as corrosion protection for steel tendon should not be of blastfurnace origin and otherwise should be checked to ensure no sulphides are present nor any other elements aggressive to any of the tendon components (see 7.1.1).

Cement based grouts are generally designed with minimal bleed (see 7.1.5.2). Where the cement grout is employed as an internal stage of permanent protection, e.g. annulus filling inside a sheath or capsule, the total bleed during simple sedimentation should not exceed 0.5 %. Where grout with a greater bleed capacity is employed, provision for removal of bleed water should be made unless load tests and sample sections confirm satisfactory performance.

8.2.4.2.2.2 Visco-elastic fluids. For corrosion protection, visco-elastic fluids are primarily bitumen based. It is essential that checks should be made to ascertain that the bitumen solvents contain no chlorides or sulphides and, if used in doubly protected systems, that they do not affect the properties of the tendon coating by dissolution, chemical attack or because of the temperature required to achieve fluidity for injection. The bitumens have to remain sufficiently viscous when cool to remain in place in pervious ground and it is prudent to provide a means of checking and topping up the bitumen levels, as necessary during the life of the anchorage.

8.2.4.2.2.3 Greases. Greases should be compounded to provide corrosion prevention properties for stressed high tensile steel tendons and manufacturers should be made fully aware of the requirements and service conditions to ensure an appropriate choice of grease. Little guidance is currently available on assessment of suitability of greases and, where possible, documentary evidence should be produced of successful use of the greases in similar applications, e.g. nuclear reactor vessels (see K.3).

Greases should not contain any substance that could provoke corrosion, e.g. unsaturated fatty acids and water, and individual contents of sulphides, nitrates and chlorides should not exceed 5×10^{-4} % (m/m). Anti-corrosion compounds should be described. Greases should be stable against water and oxygen and should not separate into soap and oil. Hydrophobic greases are preferable. Other important factors include bacterial and microbiological degradation resistance, low moisture vapour transmission and high electrical resistivity (see K.3).