RECENT ADVANCES IN GROUND ANCHOR AND GROUND REINFORCEMENT TECHNOLOGY WITH REFERENCE TO THE DEVELOPMENT OF THE ART

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ABSTRACT

The recent advances in ground anchor technology and the related techniques of cable bolting and rock bolting are reviewed. Collectively the technology associated with the three techniques enable design engineers to address stability problems over a range of scales and in a range of geomechanical environments. The techniques have similar aims but have developed into separate disciplines with unique attributes. The procedures for designing and creating ground anchors that meet the stringent requirements associated with modern civil infrastructure are discussed. This contrasts the very different design approaches being developed in rock bolting and cable bolting where standards are less exacting but the design problems can sometimes be more complex.

1.0 INTRODUCTION

Excavations and other engineering constructions in the ground are central to many civil and mining projects. For both economic and safety reasons ground reinforcement is often a key component in the successful completion of these projects. Ground reinforcement includes, amongst other methods, the techniques of ground anchoring, cable bolting and rock bolting. Basically, all of these techniques seek to assure the stability of an artificial structure constructed within or on a soil or rock mass by the installation of structural elements within the ground.

The differences between the three techniques are predominantly associated with scale and the standards of design and installation. Ground anchors tend to be longer with the highest capacity, rock bolts tend to be shorter with the lowest capacities and cable bolts have evolved to address stability problems that lie between the two. Ground anchors are usually associated with civil infrastructure projects which demand exacting standards of design and installation. Cable bolts are most commonly used in mining engineering and attract less formal standards whereas rock bolts are used in both branches of engineering.

These differences have allowed the three techniques to evolve into separate disciplines resulting in a wide range of different reinforcement devices and methodologies. The large number of options now collectively available from ground reinforcement technology allow engineers to satisfy most geomechanics stability requirements. This is despite the fact that engineering projects are now often more adventurous and are being conducted in progressively more difficult geomechanical environments.

This review will explore recent developments in the more advanced discipline of ground anchoring. This will be followed by a discussion on some of the unique attributes that characterise and differentiate the less developed disciplines of rock bolting and cable bolting.

2.0 GENERAL ANCHOR CAPACITY DEVELOPMENTS:

Vertical holding down anchors at Cheurfas Dam, Algeria, in 1934 marked the advent of modern prestressed anchor technology. Prestressed tensile members, complete with a debonded free length and cement grouted fixed length in rock increased the stability of the structure. Within the fixed anchor “anchorage chambers” were constructed to enhance bond over that provided in a parallel side borehole.

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The utilisation of a free length with fixed length founded in soil commenced in Europe (Germany and France) in the late 1950s, albeit passive underreamed tension piles were being utilised in Texas and India to resist ground heave at that time. There may be truth in the story that the inability to recover a temporary drill casing, inadvertently grouted into the granular overburden whilst constructing a rock anchor, instigated the initial German research and development of soil anchor technology.

UK anchor technology in the 1960's was influenced by the systems developed in both Germany (end of casing pressure grouting) and France (post-grouting) even though the successful development of the multi underreaming systems associated with an anchor size bore diameter, appears to have been UK lead. In the US underreamed “tie backs” were of a larger diameter (305mm bore) developed as an extension of tension pile concepts.

In the early 1960s, high capacity dam anchors were generally constructed utilising simple two stage grouting of the fixed and free length supplemented by simple methods of corrosion protection. Rock anchor technology advanced gradually with anchor capacities increasing, generally with borehole size, to accommodate an increasing number of high capacity prestressing strand.

Australia probably leads the way with the most extensive use of long ultra high capacity dam anchors (12,000kN working load, up to 120m long) complete with modern corrosion protection systems. (Cavill 1997). The conference city, Melbourne itself boasts intense application of anchor technology with some 7000 anchors, each with a 120 year life span requirement, to stabilise the tunnel beneath the River Yarra.

Generally, the working capacities of soil anchors increased little through the 1970s and 1980s (Littlejohn 1970, Ostermayer 1974; Barley 1987);
- In gravels working loads up to 800kN using grout “injection” techniques
- In sands working loads up to 500kN using grout "compaction" techniques
- In stiff to very stiff clays working loads up to 600kN by use of postgrouting or underreaming techniques.

However, construction techniques did improve particularly due to the gross advancement in performance of drilling rigs and equipment; increased power, higher torque and the introduction of top drive rotary percussion. Generally, factors of safety increased and as a result, confidence in the use of anchors. Ground conditions certainly in the UK which still caused most concern, due to higher risk of failure, or utilisation of only low working loads, were weathered mudstones and silty soils.

Throughout these periods, research work investigating the performance of anchors in various soils and rocks and the associated load transfer mechanism, was carried out at numerous locations by a multiple of researchers. The vast majority of results supported the concept of the non linear relationship between load and fixed length owing to the phenomenon known as “progressive debonding.” However, it was further investigation and analysis of the distribution of load along the length of the anchor borehole in the late 1980s that allowed evolution and advancement of a new anchor concept. This “multiple” anchor system has lead to the attainment of 3000 to 4000 kN capacity in anchors installed in soils and weak rock and the frequent installation of soil anchors with working loads in the 800 to 2000 kN range.

3.0 DRILLING, REAMING AND GROUTING METHODS FOR ANCHOR CONSTRUCTION

3.1 Anchors in Rock
In the majority of moderately weak to strong rocks, rotary or rotary percussive open hole drilling with air flush, followed by normal tremie grouting techniques, will achieve the required grout/rock bond capacity to satisfy most anchor loading conditions. Where fissures or voids are detected by loss of flush, by water ingress, by water testing, or by inability to maintain a head of grout within the bore, then pregrouting operations, or alternatively pressure grouting operations may be required. Normally neat cement grouts are injected, but where fissures are known to be wide, sanded mixes may be used.

In coarse grained weak rocks (sandstones in particular) similar techniques or alternatively rotary water flush drilling can be used and in most conditions a reasonable anchorage capacity can be attained. However, difficulty in achieving capacity is often encountered in weak to very weak fine grained rocks of limestone, chalk, mudstone and siltstone, where coarse particles are not present to provide enhancement in grout/rock bond. In these conditions, where the bedrock is dry, air flush is still most appropriate, but in wet fissured mudrocks there is a high risk of the borehole becoming smeared with wet sticky airlifted particles if lifting wet or damp spoil. This causes loss of bond due to wall smearing or caking.
If the weak rock is in a highly weathered and fissured condition, (particularly in chalk) advancement of drill casing into part or all of the fixed length may be necessary to support the bore and ensure a clear passage for flush recovery. In such cases, the use of water flush may be preferred if using top drive techniques whilst eccentric bit down-the-hole hammer systems, complete with casing, are generally accompanied by air flush. However, in these conditions, when DTH drilling, the risk of blockage of the spoil recovery slots alongside the hammer body should not be overlooked since lost flush may rupture or damage adjacent strata.

On drill string removal and after tendon installation application of pressure (in the order of 1MPa – 145 psi) during end-of-casing grouting frequently provides considerable improvement in rock/grout bond and enhances local rockmass strength.

Pre-grouting (before tendon installation) or post-grouting, both via manchette systems, can also contribute to bond enhancement in many weak rock conditions. When used in weak chalk, there are indications that the capacity achieved by post grouting is in the same order as that attained by end of casing pressure grouting. Underreaming, although employed infrequently, will also improve anchor capacity, but attention must be paid to spoil removal from the underreams themselves, the risk of underream collapse and particular attention must be paid to spoil removal. In highly weathered and/or extensively fractured and broken bedrock, the converse of the conditions requiring forcible introduction of grout in the rock may occur. In such situations where neat grout tends to flow away from the bore, grouting with a sanded mix might help to stem this flow and hence control grouting. In certain conditions, after achieving fissure blockage, the attainment of backpressure may provide evidence of subsequent non-leakage from the fixed anchor and override the necessity to carry out pre-grouting and re-drilling.

3.2 Anchors in Clay

In order to enhance the capacity of the anchorage within the normal range of fixed lengths, either underreaming or soil fracturing systems have been employed. More recently, the single bore multiple anchor system has been allowed efficient use of non-enhanced boreholes and attained loads of 3500 kN. Where required the majority of clay anchor underreaming tools associated with anchor borehole diameters of 200mm have been developed in the UK, for use in London Clay. Development of tools which ream the clay is relatively straightforward but, as several anchor contractors have found out, the efficient removal of drill spoil and its replacement with grout in all the underreamed cavities is considerably more difficult. The largest anchor underreaming tool which has been developed forms up to five 700mm diameter bell shaped underreams simultaneously from a 200mm bore, using an efficient high pressure water flushing system. Underreamed anchors have been constructed in stiff London clay and tested to 2300kN without failure.

Underreaming in these circumstances can normally be expected to provide an enhancement of between 4 and 6 times the capacity of a normal shaft anchor but less capacity than the new multiple anchor system. Soil fracturing systems were generally developed in continental Europe, but with the major advancement in special geotechnical operations such as Compensation Grouting systems have advanced on a world-wide basis. The fracturing of the soil, and the grouting prior to tendon installation, generally involves a large diameter steel manchette, which, after treatment, remains insitu. Treatment may be carried out over a 2 or 3 day period prior to tendon installation, by repeatedly injecting grout through manchette valves at ½ m centres in the fixed length. Although injection through the single large tube may prove more efficient in ground strengthening than the use of a single or a multiple of grout tubes, normally used for post-grouted anchors, the manchette tube itself must fulfil a number of other vital requirements. The anchor tendon is, after pregrouting treatment, installed and grouted within the large tube. The tube must efficiently transfer the entire load from the anchor tendon and internal grout to the external grout and then into the ground. In doing so it must provide safety factors no less than those provided at other bonded interfaces. It must not exhibit creep losses and, most importantly, it must not degrade (by corrosion) in any way such that there is a reduction in bond capacity or performance within the grout body, which may reduce the capacity of the anchor within its intended lifetime. With these factors in mind, the use of post grouting system, with smaller adjacent plastic tubes or manchette (or underreaming), may be considered to be more appropriate for permanent anchor works. It should also be borne in mind that it may not be possible to contain the high pressure associated with soil fracturing and subsequent grout treatment solely to within the intended area. Such pressure may influence adjacent foundations or activate potential slip surfaces.
3.3 Anchors in Granular Materials

Anchors are, in the majority of instances installed in granular deposits by drive drilling with a knock-off bit or by use of duplex drilling techniques, unless the soil is weakly cemented or contained within a cohesive matrix to allow open hole drilling. Drive drilling involves the percussion driving of a strong casing with a conical lead bit resulting lateral soil displacement and no flush recovery. The lead bit is knocked off the casing allowing tendon installation and pressure grouting during casing withdrawal. There are limitations in the depth penetrable.

Duplex drilling involves the advancement of both drill rods and drill casing, utilising casing sizes of the 80 to 150mm ranges. Either air or water flush or augers can be used, although bit wear and casing wear may well be considerably higher without the lubrications and cooling provided by water.

Where uncemented granular deposits are drilled without the use of drill casing, then alternative flushing medium are generally required to support the bore. Over recent years there has been an increase in the use of cement grout as a flushing medium albeit it is then essential to balance recirculation of the grout with costs of its disposal. In the past, other flushing mediums in the form of water with additives, which when in soil contact temporarily bind the granular particles together, or may be of drilling muds consistency, or of foam have been used. When any of these latter methods are employed, the flushing medium may strongly influence the effectiveness of the soil grouting and the available bond at the grout/soil interface. In these instances, the influence of the flushing medium and the mode of application of grouting pressure must be fully investigated by preliminary tests.

During normal pressure grouting operations via end of casing grouting, the penetration, or containment, of the grout by the granular soil is controlled by the grading size of the lower 5-15% soil content. When using normal Ordinary Portland cement, the grout will penetrate into coarse sands and gravels to form “injection” anchors, whilst the use of same pressure grouting techniques in medium to fine sands and silts will form compaction anchors. During construction of the latter, there may appear to be identical characteristics of grout take under pressure as with the injection anchorage, but in fact only the bleed water filters off to penetrate the soils. The remaining very dense, pressurised grout within the anchor body compacts the surrounding soil and increases the bore diameter by typically 10 to 20% (depending on time period of compaction). Observation of such exhumed anchors portrays very clear boundaries, where the cement particles have failed to penetrate the soil but high capacities have been satisfactorily achieved as a result of soil strengthening by compaction. (Photo 1). In instances where large diameters of grout have been observed in anchors exhumed from fine sands, they have resulted from over flushing, which can obviously be beneficial to anchor capacity when controlled and fully grouted.

Techniques involving soil fracturing by post grouting to construct anchors in granular soils follow the systems highlighted in the clay anchorages. They are documented by Ostermayer and published results suggest a very close correlation with those capacities achieved by end-of-casing grouting methods. Clearly, in using all systems, capacities generally attained in coarse materials are higher than those achieved in finer materials. Soil density plays an equally important role but this can now be improved local to the bore by “compaction” grouting integral with anchor operations.

3.3 Anchors in Mixed Cohesive and Granular Materials

When the presence of granular materials in cohesive deposits presents a risk of borehole collapse, then the use of underreaming systems is inappropriate. In such situations, the pre-grouting or post grouting techniques highlighted above will prove
advantageous. Alternatively, by fully casing the borehole, normal end-of-casing grouting can be used to
direct the grout into the granular bands, which will in all cases enhance the capacity of the anchor.

In mixed clays and silty deposits the borehole diameter can be enhanced by the use of special jet grouting
techniques. Jet grouting followed by pressure grouting has increased the anchor capacities from 450kN into
the 1150kN range albeit by incorporating a modern multiple anchor system even greater capacity could be
achieved.

4.0 ANCHOR TENDONS AND MODES OF LOAD TRANSFER

4.1 General Development

4.1.1 Solid Bar Tendons

During initial development of soil anchor technology in the 1950-1960s, steel bars were frequently used,
generally smooth, but with threaded ends to enable transfer of load both at the anchorhead and within the
fixed length. As anchor bores became longer the extension of individual bars (up to 12m long) by use of
couplers was common. The bar systems were readily available having been predeveloped for tie rod and
other engineering works requiring tensile members. Acknowledgement of the basic requirement to prevent
tendon bonding in the "free" length demanded the placement of plastic sleeves around the free length bars
and in order to reduce friction bond the smearing of grease was carried out on the bar surface prior to
sleeving. Initially the debonding of the coupler from the free length grout column was effected by densotape
wrapping but only at a later date was the placement of a compressible packer above the coupler to allow its
free movement considered necessary. The unequal threading of the connected bars into the couplers and the
unthreading of the bars from the couplers as a consequence of drill casing rotation on withdrawal, were both
frequent and repeated experiences as new drilling contractors entered the anchor market. Systems such as
use of locking nuts or quickset epoxy resin or locking screws tapped into the couplers were all evolved to
improve the reliability of the finished product. Even as late as 1978 during construction of early anchors for
the Thames Bank Raising Scheme were these problems with bar anchors encountered such that the client
subsequently approved the use of strand anchors as an alternative.

The use of smooth steel bars is not normally conducive to effective load transfer from the tendon to the
cement grout since bond stresses at such an interface rarely exceed an average of one MPa.

Initially, it was common practice to utilise an end nut and a compression fitting above the nut (a number
of 300mm long hollow corrugated cast iron tubes could be stacked above the nut) Figure 1 illustrates this
system. However, although this load transfer mechanism is effective when the grout column is fully
confined, it has severe limitations, in soils and weak rocks where high bursting stresses, induced in the grout,
by the use of a short load transfer length, cannot be contained. Later a much more refined compression tube
system incorporating corrosion protective components was available in Germany. Enhancement of bond in a
"smooth" bar was generally achieved by the increase in the length of the threads turn on the bar in the fixed
length. This in turn lead to the provision of fully threaded bars of various fine and coarse thread patterns for
use in both anchors and minipiling in the Geotechnical Industry. A very broad selection of size and capacity
of fully threaded bars now exists, up to a maximum capacity of 4300kN. Such a high capacity system was
specifically developed in the UK for refitting operations associated with the Nuclear Submarine Complex at
Devonport.

Figure 1 : Simple End Bearing Fittings Used on Bar Anchors circa 1970
(The Consulting Engineer, May 1970)
4.1.2 Prestressing Strand Tendons

Early in the use of anchor systems, the difficulties of handling and coupling of long steel bars was realised and the alternative by use of a multiple of prestressing strands was introduced. The presence of V-grooves between the helically wound peripheral wires provided surface deformation to enhance the bond between the strand and the grout and achieved average bond stress in the 1 to 2 MPa range when the strands were appropriately spaced (>5mm). (Bruce 1976). However, it was realised that the capacity of an upstand deformation is greater than that of an indent (Barley, 1997A) and the surface indent alone was not adequate to ensure that the full capacity of each strand could be mobilised. It was quickly appreciated that the unravelling and rewinding of the strand wires grossly enhanced bond (Photo 2). This operation being known as “caging” or “bushing”; or if a ferrule was placed on the kingwire to result in a small controlled deformation then “strand noding”. At this location, bond stresses as high as 12MPa have been established (Barley, 1978). Development in the rock bolting industry some 15 years after initial use in anchor technology also realised the bond enhancement benefit, the system being referred to as “birdcaging” and now used extensively in that industry.

Throughout the period, an alternative mode of enhancement of strand to grout bond was developed in the form of “tendon noding”: the peripheral strands being spaced between 15 and 20mm apart at one location and tightly banded together about 1.5m above and below the spacer. This provides the multiple strand arrangement with a deformed outer profile and ensures an inlocking effect, all of which enhances grout to tendon bond (Bruce 1976, Adams and Littlejohn 1997).

All these load transfer systems are well proven and now well documented (Anchor Conference 1997) albeit it should be realised that efficiency of tendon bond may fall with increase in number of strands and increase in steel density within the bore. All internal strands contained within an outer periphery of strands must transfer their load capacity across that outer periphery. Consequently, the spacing between the peripheral strands should increase as the number of strands contained within increases. (Barley 1997A)

With regard to free length debonding, this was initially effected by two stage grouting, i.e. grouting the fixed length then stressing the anchor, followed by free length grouting. However, such precise control of the grout level in soil anchoring is difficult to achieve and this method was replaced by tape-wrapping of the group of strands. By 1970, the provision to the Industry of factory applied pregreased and plastic coated strand was available in the UK. This, however, necessitated the development of an efficient system for removal of the plastic coating and the degreasing of the strand wires over the bond length.

At a similar time, systems were being developed to effect the greasing and sheathing of selected sections of pre-cut strand tendons, which although discontinuous as in operation, had distinct advantage in avoidance of application followed by removal (Photo 3).

The development of these refined systems did in the view of certain specialist contractors make use of prestressing strand as the anchor tendon more favourable than the use of bars.

4.1.3 Hollow Steel Bar Tendons

As soil anchor technology developed, the distinct benefit of drilling the borehole with the same steel member as to be incorporated in the anchor body was realised. Initially, a simple drill bit was screwed into the lead thread length of the bar, an air or water flush system pressure injected through the drill head or via a simple side swivel system to allow its passage down the core of the “drill rod”. Its emission at the bit
ensured drill spoil removal and on completion of drilling a precalculated quantity of grout was simply injected down the core to effect fixed length grouting. Secondary grouting via a tremie could be carried out after stressing, or a debonding sleeve could be placed on the free length bar. This system could be practical and economical when used up to depths of 12m, but in its original form it presented severe limitation in control of the borehole size and prevention of drill flush penetration of adjacent strata, and also the difficulties and effects of the use of couplers. For these reasons its use in the simple form did not comply with the requirements of modern codes certainly for permanent work and only for temporary usage to limited depths.

However, the concept of the use of the hollow bar as both the drill rod and as the tensile member of an anchor (or minipile or soil nail) has since the early days been considerably developed and refined in an effort to control the drilling, the flushing, the grouting and the quality of the finished product. The presence of a continual thread along the external face generally eliminates bond capacity problems at the steel grout interface. The presence of plastic sleeving between couplers in the free anchor length may provide adequate debonding facilities to ensure completion with code minimum free length requirement, albeit grout bond at the coupler may still occur.

4.1.4 Special Systems

A number of specially developed systems have been evolved to allow high production low load anchoring methods. One system involves the drill installation of metal plate coupled to wire rope, which on removal of drill rod and during the tensioning of the wire rope causes the eccentrically loaded plate to rotate and be located perpendicular to the bore. The system may have a ultimate capacity up to 200kN but clearly the concentration of loading in end bearing mode requires the investigation of its susceptibility to creep loss.

A system developed in the USA for the stabilisation of towers and holding down of pipelines involves the installation of an auger type arrangement with effectively 90% of the auger flight omitted. This allows auger type installation without removal of spoil and the insitu inclined plates provide end bearing resistance to pull out. The steel bar acts as at the tensile member of the anchor and ultimate loads up to 500kN are considered feasible. The anchor tendon is completely removable after temporary works.

The expander body anchorage system generates anchorage capacity by both expansion and soil compaction by in situ inflation of a stainless steel “balloon”. The thin walled folded body is inserted down the predrilled bore whilst mounted on the end of a disposable drill tube. Prestressing strand acts on the tendon tendon and is slid down into the closed body followed by grout injection through the tube. The expander body may be inflated to normal full size of 0.5m diameter over a 3m length using pressure in the 1.5 to 2.5 MPa range (220 to 350 psi) the system has been used for a number of years and can achieve
capacities of up to 1000kN in dense sands with one tonne of cement grout take. The number of strands which can pass into the body are however limited, and the tube and body component can be expensive, but the system is available and can also be used in a number of other ground conditions. Although stainless steel is used, it may degrade in the long term, (120 year life) particularly in the more aggressive environment. The body wall is thin and the effect of degradation on the bonded interface is unknown. It is possible to install removable strand tendons with this anchor system.

5.0 CORROSION PROTECTION REQUIREMENTS OF PERMANENT ANCHORS IN EUROPE

In many instances the consequences of failure of one or a number of anchors could have extremely severe effects on the stability of a structure. However, owing to the severity of ground anchor proof testing, complemented by load or creep monitoring prior to acceptance of each anchor, the risk of failure of a working anchor due to interfacial bonding is extremely small. But the need to take account of the risk of failure due to corrosion of steel components, must be tackled. An essential requirement in any system developed to eliminate this risk is that it must be compatible with the demands of load testing, either by proving the isolation of the tendon steel of each individual in-situ anchor, or by fully demonstrating the effectiveness of a robust protective system prior to use.

To date, it is believed that only a single world wide study of the corrosion of anchors has been carried out and the report published. A working group under the FIP (Fédération Internationale de la Précontrainte - 1986) collected 35 case histories of anchor failure by tendon corrosion protection requirement for a standard:-

“While the mechanisms of corrosion are understood, the aggressivity of the ground and general environment are seldom quantified at the site investigation stage. In the absence of aggressivity data it is unlikely the case histories involving tendon corrosion will provide reliable information for the prediction of corrosion rates in service.

Case histories of tendon corrosion indicate that failure can occur after service of only a few weeks or many years. Invariably corrosion is localised and in such circumstances no tendon type (bar, wire or strand) appears to have a special immunity.

Since there is no certain way of predicting localised corrosion rates, where aggressivity is recognised, albeit qualitatively, some degree of protection should be provided by the designer. In this regard, the anchor head is particularly susceptible to attack, and early protection of this component is recommended for both temporary and permanent anchorages.

Choice of degree of protection should be the responsibility of the designer (usually the Client’s Engineer) and such choice depends on such factors as consequences of failure, aggressivity of environment and cost of protection. In current practice the design solution normally ranges from double protection (implying two physical barriers) to simple grout cover.

Out of millions of prestressed ground anchorages which have been installed around the world, 35 case histories of failure by tendon corrosion have been recorded. With the passage of time, lessons have been learned and standards improved which augers well for the future. There is no room for complacency, however, and engineers must rigorously apply standards both in design and construction in order to ensure satisfactory performance during service.”

Although the record of failure is yet limited, it is probable that in the next decade the frequency of individual and group anchor failures will increase as anchors installed prior to the implementation of rigorous protective requirements suffer from corrosive attack and their reduced capacities no longer ensure the fulfilment of their intended role. Similar instances to those of the Thames wall failure (Barley, 1997B), and a tidal barrier wall failure (unpublished), where structural collapse or partial collapse, will occur. Many permanent anchors installed in the UK prior to the guidelines of DD81 (pre 1982), with the exception of anchors associated with the Thames Bank Raising Scheme, contained only limited protection against corrosion and are unlikely to withstand aggressive conditions, particularly those in a marine environment.

From the known vulnerability of anchors in aggressive environments, does it follow that all permanent anchors should carry the same degree of protection against corrosion despite the knowledge that corrosion in certain environments may be mildly based on the FIP report statements. European Standard EN1537, “The Execution of Ground Anchor Works” states:-
“There is no certain way of identifying corrosion circumstances with sufficient precision to predict corrosion rates of steel in the ground. All steel components which are stressed shall be protected against corrosion for their design life”.

and recommended that for permanent anchors:-

“The minimum corrosion protection surrounding the tendon(s) of the anchor shall be a single continuous layer of corrosion preventive material which does not degrade during the lifetime of the anchor.”

This recommendation was initially supported by two requirements:-

“The tendon of a permanent ground anchor shall be provided with either:

a) a single protective barrier to corrosion, the integrity of which shall be proven by testing each anchor insitu.”

b) two protective barriers to corrosion such that if one barrier is damaged during installation or anchor loading, the second barrier remains intact.”

In the “Eurocode”, the method of substantiating the former requirement involves the insitu testing of the total isolation of the steel tendon from the surrounding environment. This can be established by electrical resistance measurement, but clearly the efficiency of the test system itself must be established to ensure that a defective protective layer can be positively identified to satisfy the mandatory test requirements. Anchors not tested insitu must contain two protective barriers and must comply with further requirements that:-

“All corrosion protection systems shall have been subjected to at least one system test to verify the competence of the system. The results of all tests shall be documented.”

Methods of establishing the integrity of protective barriers during or after loading conditions, involve the use of a grouted gun barrel in which tendons are loaded. The gun barrel allows splitting and inspection of corrosion protection after test completion. The system of testing the integrity of the corrosion protection ducting in situ and used extensively in Australia, was put forward to the European Working Group but surprisingly not approved. This simple but effective method of checking the water filled duct for water loss when a differential head is applied has been successfully developed and used in large scale dam anchors where alternative methods would have been very difficult to implement. It does fulfil the basic requirement of verifying the performance of the protective barrier in situ prior to grouting.

Steel tendons proposed for permanent usage, not isolated by a proven impermeable membrane, but relying on the integrity of the grout cover or on steel coatings, are not proven for an adequate life span in all environments and therefore not approved.

For clarification the new European Standard details examples of approved corrosion protection systems in two tables, for temporary and permanent anchors, which allow the identification of the corrosion protection layers proven effective in each integrated system. Such tables provide Engineers and Clients with a better understanding of the systems which are available for selection, how appropriate they might be and the implicit level of risk associated with their use.

The elimination of risk of anchor failure due to steel corrosion demands proof beyond reasonable doubt that the protective system will work and last for the 60 to 120 year designed life span of the anchored structure.

6.0 THE FIXED ANCHOR

6.1 Design of the Fixed Length of an Anchor

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

Thus, in the vast majority of anchors, when applying the initial load, the bond stress is concentrated over the proximal length of the fixed anchor and at that time the distal component of the fixed anchor is unstressed and redundant. As load is increased in the anchor the ultimate bond stress at either or both the tendon to grout and grout to ground interface is exceeded and the residual bond stress at that location, after interfacial movement, is generally of a lower order. When ultimate bond stress is achieved at one interface, generally the grout to ground in soil anchors, then the bond stress at the other interface cannot increase further. That unit length of anchor has reached capacity limit and subsequently the capacity will decrease.

As load in the overall anchor is further increased the bond stress concentration zone progresses further along the fixed anchor. Prior to failure of the anchor the load concentration zone approaches the distal end of the anchor. Figure 2 typifies the distribution of bond stress along a normal fixed anchor during initial loading and when approaching failure, albeit the relationship between the ultimate bond stress and the residual bond stress will vary with ground conditions and grouting techniques.

In all anchors there is a minimum of two interfaces across which load must be transferred (tendon/grout and grout/ground). In corrosion protected anchors there may be as many as six interfaces when the inside and outside of the encapsulation duct(s) are considered. Thus, the load transfer is always a complex mechanism but the Industry is only beginning to interpret a simple approach acknowledging the phenomenon of “progressive debonding”. However in the vast majority of design recommendations, the occurrence of progressive bonding is not even considered in the design formulae:

For example, inspection of the British Standard for Ground Anchorages (BS8081) reveals the following design assumption:-

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.

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 to the rock/grout interface in the weaker medium (rough borehole).

d) Here are no discontinuities or inherent weakness planes along which failure can be induced.

e) There is no local debonding at the rock/grout interface.

They are however qualified:-

“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.”

The design assumptions of uniform load distribution are, however, not only limited to the British Standard but are generally adopted on a world-wide scale for the design of ground and rock anchors. In simple interpretation it means that the design of the fixed length of an anchor is based on:

\[ T_{ult} = \tau_{ult} A \]  \hspace{1cm} (1)

where \( T_{ult} \) = ultimate anchor load, \( \tau_{ult} \) = ultimate bond stress, \( A \) = cylindrical area of contact surface and generally

\[ A = \pi dL \]  \hspace{1cm} (2)

where \( d \) = diameter of bore, \( L \) = fixed length of anchor.

Hence

\[ T_{ult} = \tau_{ult} \pi dL \]  \hspace{1cm} (3)

or

\[ T_{ult} \propto L \]

Yet it is fully acknowledged in the Industry that the ultimate load is not proportional to the fixed length:-

“The increase in carrying capacity of an anchor tapers off steadily with length, so that in general a length of 6 to 7m is optimal from the economical point of view”. This simple but classic statement by Ostermayer was made 26 years ago (1974).

This “taper-off” and non-proportionately are demonstrated in Ostermayer’s curves Figure 3, and also in Figure 2 where the area under the bond stress distribution line is representative of the ultimate load in the anchor. The area under neither of the bond stress distribution lines increases uniformly with increasing fixed length. With due consideration of the extreme complexity of the load transfer across a multiple of interfaces, it would be extremely difficult to evolve mathematically a design approach for everyday use in anchor design.

There have been a number to attempts to introduce a non-linear factor into Equation 3; Casanovas (1989) recommend that:

Apparent fixed length over which the full ultimate bond stress can be mobilised

\[ L_{ve} = L \frac{1}{\log(0.3 \tau_{ult})} \]  \hspace{1cm} (4)

where \( L \) = fixed length, \( L_{ve} \) = apparent fixed length over which full \( \tau_{ult} \) operates and \( \tau_{ult} \) (kN/m\(^2\)) can be represented in sands by a relationship with \( N \), the standard penetration test value:

\[ \tau_{ult} = \begin{cases} 1.1 N & \text{in fine sand} \\ 1.5 N & \text{in medium sand} \\ 1.9 N & \text{in sands and gravel} \end{cases} \]  \hspace{1cm} (5)

Evaluation of this data provides an “apparent” fixed length in loose medium grained sand (\( N = 10 \)) of 7.1m for an actual fixed length of 10m or in very dense sand and gravel (\( N = 60 \)) an apparent fixed length of 3.06m. It could be said that the former was 71% efficient and the latter was 31% efficient in mobilising the ground strength.

Alternatively, inspection of the distribution of bond stress in Figure 2 does allow evaluation of a simple mathematical expression that acknowledges the occurrence of progressing debonding: (Barley 1995)

\[ T_{ult} \propto f_{eff} L \]  \hspace{1cm} (6)

where \( f_{eff} \) = efficiency factor which is itself a function of \( L \), and

\[ T_{ult} = f_{eff} \tau_{ult} \pi L \]  \hspace{1cm} (7)
In this case $\tau_{ult}$ is the ultimate bond stress of a relatively short fixed length where loss of efficiency due to the progressing debonding is small or negligible, or where

$$T_{ult} = f_{eff} LP_m$$  \hspace{1cm} (8)

where $P_m$ = capacity per metre length of a short length of bore (kN/m).

Clearly, the only potential source for evaluation of the simply represented "efficiency factor", $f_{eff}$, is the back analysis of the results of failed test anchors installed into the same ground condition but utilising differing fixed anchor lengths.

Barley (1995) back analysed such data from 8 construction sites where anchors of different fixed lengths had been tested to failure in clays, silty clays, and sandy clays, boulder clay and glacial till.

Figure 4 illustrates the distribution of efficiency factors obtained, and the best fit curve is represented by;

$$f_{eff} = 1.6L^{-0.57}$$  \hspace{1cm} (9)

In the same paper he suggested an efficiency factor appropriate for use in sands of which relates efficiency to fixed length and soil friction angle.

$$f_{eff} = (0.91)^{L \tan \phi}$$  \hspace{1cm} (10)

This work and liaison with the University of Surrey lead to research work by Barkhordari in back analysing the extensive trial data presented by Ostermayer (1974) and Ostermayer and Scheele (1977).

Woods and Barkhordari (1997) proposed a different relationship which also related efficiency to fixed length and soil friction angle.

$$f_{eff} = \exp(-0.05L \tan \phi)$$  \hspace{1cm} (11)

but for incorporation in the design formula specifically for anchors in sand;

$$\tau_{ult} = f_{eff} Ln \tan \phi$$  \hspace{1cm} (12)

where $n$ = loading capacity per metre length for a short fixed length, $\phi$ = friction angle of soil.
Utilising $\phi$ and $n$ values of 36 and 75, 42 and 120, 43 and 175 and 47 and 200 representative of loose, medium dense, dense and very dense medium to coarse sand with gravel, their ultimate load versus fixed length curves fit very closely with the strength development lines proposed by Ostermayer and Scheele (1977).

In 1974 Ostermayer also chose to present his data in terms of skin friction (bond stress) against fixed length. The information covered ultimate bond stress values in medium and medium to high plasticity clays for post grouted and non-post grouted anchors, and for end of casing grouted anchors in fine to medium sand. All results reflected a fall in bond stress with increase in fixed length and Ostermayer presented curved boundary lines to illustrate this. The imposition of distribution lines, defined by the efficiency factor of $1.6L^{-0.57}$, on these graphs (Figure 5) illustrates that this simple "efficiency factor" concept and furthermore, the same mathematical expression, are quite consistent with Ostermayer's original boundaries. Thus from establishing an ultimate bond stress over a known fixed length the capacity of an anchor with any fixed length in that soil, using that construction technique, can be reasonably evaluated (Equation 7). As previously stated, the relationship between the stiffness of the fixed anchor (controlled by the steel tendon) and the stiffness of the ground governs the rate of progressive debonding as an anchor is loaded and hence affects fixed length efficiency. In this context the variation in soil stiffness for the range of the soils in which anchors are usually founded is not sufficiently large to result in significant variation. Hence, the best fit efficiency factor derived from anchors in silty clays can used as a guide factor in all soils using normal steel tendons.

In 1997 Barley extended his research work to back-analyse data from soil nail pull-out tests in London Clay which investigated tendons of considerably differing stiffness; 20mm and 50mm diameter deformed steel bars and 6mm and 20mm diameter glass reinforced plastic tendons with fixed lengths ranging from 1 to 20m. Figure 6 highlights the rapid fall-off in efficiency when using highly elastic members (GRP) and the smaller fall-off when using an unusually large stiff steel bar (50mm diameter).

Probably the most extensive attempts to model both the construction technique and performance of an anchor have been carried out by Mecsi (1995) with his analysis based on construction data and performance results from several hundred installed anchors. He has provided guidelines to "specific pull-out resistance" of anchors in various soils.

This term refers to the pull-out capacity of a 1m length of anchor which is similar in concept to that of Barley who also refers to short length capacity over which progressive debonding is small or negligible. Mecsi acknowledges that the actual value depends on the construction technique and other variables.

His recommendations for evaluation of the ultimate anchor capacity have other common ground with that proposed by Casanovas (1989) and Barley (1995) but in his evaluation incorporates values for tendon stiffness (rigidity index) and overburden pressure. He considers that full grout/ground bond stress is mobilised over a limited length of the anchor ($L_o$) and only a percentage of that value mobilised over the remaining length ($L - L_o$).
Figure 5: Ostermayer’s (1974) “boundary lines” may be reasonably represented by the distribution as per “efficiency factor” $1.6L^{-0.57}$. 
Hence:

\[ T_{ult} = t_{ult} \left[ L_o + \frac{1}{k} th[k(L - L_o)] \right] \]

when \( t_{ult} \) = specific pull-out resistance of a 1m length kN/m (=P_m); \( L_o \) = length over which is fully mobilised; \( k \) = "rigidity index" of the anchor tendon; \( h \) = height of overburden on mid level of fixed anchor.

This approach fully acknowledges progressive debonding and the "taper off" concept.

Figure 6: Comparison of efficiency factors associated with GRP and steel soil nails with that of the best fit ground anchor curve. (Barley 1997)

From the above, it is reasonable to propose that the design formulae for anchors in soils and weak rocks should always incorporate an efficiency factor or similar mathematical expression acknowledging non-linearity. With this understanding, test anchors of length 2.5 to 5 metres may easily be taken to failure to establish the ultimate bond stress of that length and then the fixed length of production anchors accurately, designed to provide the required factor of safety. In the trials it is important to control the grouted length tested.

Although anchors in strong rocks encounter the same progressive debonding phenomenon the availability of extremely high local bond capacity at the rock grout interface may transfer the progressive failure mechanism to the tendon grout interface and in such conditions the efficiency factor, if appropriate, would require the extensive review of other data.

6.2 The Influence of Tendon Bond Length on Anchor Capacity

In the above, the relationship between the tendon bond length and the grout/ground bond length (anchor fixed length) has been ignored and generally considered to be the same. However, inspection of the research and trial programmes from which the empirical values of efficiency have been evolved reveals that typically the tendon bond length has been between 60% and 80% of the fixed length which is in fact consistent with the practice carried out by many experienced specialist contractors. Why? Because the efficiency in load transfer (as defined in 5.1) is least when the load transfer mechanism over the entire fixed length is in shear and tension (Fig. 2) and, if other factors do not cause failure, greatest when the load transfer over the entire fixed length is in shear and compression (Figures 1 and 7). When the tendon bond length equals the fixed length the elastic properties of the tendon control the load transfer over the entire fixed length and consequently reduces the efficiency of mobilisation of grout/ground bond strength over the entire fixed length. Conversely, if successful use could be made of an end plate on a tendon itself, fully debonded through the entire fixed length, then there would be no contribution from the elastic properties of the tendon to the efficiency in load transfer in the fixed anchor. The only incompatibility that would exist would be that...
of the grout behaviour in compression with, and that of the ground. Basically, the anchor load transfer mechanism would be close to that of an unreinforced pile.

Unfortunately, although much tried in early anchoring days, (Figure 1) the total concentration of load on a single end plate or on a ultra short compression tube is not appropriate in the majority of weak rock and soil conditions.

All load transfer mechanisms from tendon to grout induce bursting forces in the grout of one degree or another. Generally, the greater the mechanical locking effect (end plate or major deformations) the greater the bursting forces. It follows that the shorter the tendon bond length the greater the mechanical locking to allow the potential transfer of full tendon load capacity. However, this can only be effected where the ground or strong rock will provide adequate confinement of the grout column to prevent bursting failure. In a moderately strong rock, a single 80mm diameter end plate allowed transfer of up to 1100kN load and mobilised a bond stress of 10Mpa over a 300mm bond length, when confinement was high. The mechanical interlocking worked well and enhanced bond by wedging. (Barley 1978). However, it is unlikely that the same mechanical interlocking load transfer mechanisms would achieve even 100kN in a soil where confinement is very limited.

So to reduce the inefficiency in load transfer (entire fixed length in shear and tension) it is appropriate to utilise tendon bond lengths long enough to eliminate the risk of bursting failure yet as short as possible to gain maximum efficiency from grout/ground bond (Fig. 7). This concept is supported by Briund, Powers and Weatherby (1998) "Should Grouted Anchors Have Short Tendon Bond Lengths?".

![Figure 7](image)

**Figure 7**: Probable Bond Stress Distribution in (a) Shear and Compression Combined Shear and Compression and Shear and Tension

The paper highlights the fact that the load capacity of a 9.2m fixed length utilising a 4.6m bond length in stiff clay was greater (961kN) than that of the same fixed length utilising a full 9.2m bond length. (783 kN). It was concluded "The ultimate soil resistance, Qu, was found to be 23% larger for the anchors with a short bonded length (4.6m) than for the anchors with a long bonded length (9.2m)".

Ludwig and Weatherby (1989) investigated load distribution over an 18m fixed length using a bond length of only 4.4m. Their results indicated that a similar amount of load was transferred to the ground above the bond length as within the bond length. It is possible that the 300mm diameter grout column may have contributed to the control of the bursting stresses over the low proportion bond length.

In choosing short bond lengths it should not be overlooked that natural weak soil and poor rock conditions do exist where the borehole confinement is so limited that the potential gains from the reduced
bond length may be lost. Here the prior application of pressure grouting methods to enhance the borehole constraint may achieve the greatest benefit.

And so, all anchor systems that utilise elastic tendon members to transfer load to the ground are inefficient, but some systems are less inefficient than others!

7.0 THE DEVELOPMENT OF AN EFFICIENT LOAD TRANSFER FOR THE SINGLE BORE MULTIPLE ANCHOR SYSTEM IN PERMANENT WORKS AND FOR TEMPORARY REMOVABLE ANCHOR WORKS

7.1 Introduction

Consider an anchor tendon load transfer system that would:

i) transfer the tendon capacity from the tendon to anchor grout over a short length (say 0.5m) without inducement of high bursting forces;

ii) repeat this load transfer at staggered lengths along the anchor borehole (say 1m spacing);

iii) carry the same load on each tendon simultaneously;

It would almost eliminate the occurrence of progressive debonding and hence grossly increase the efficiency in mobilisation of ground strength adjacent to an anchor borehole. Such was the intent of the development of resin encapsulated fixed strands ends illustrated in Photo 4.

A system (Single Bore Multiple Anchor), now achieving working loads of 800kN to 2000 kN in soils and weak rocks and achieving 3000 to 4000 kN has developed from that concept.

Research into the use of short corrosion protected load transfer lengths from strand to grout within short encapsulations continued through the early 1980s.

Initial site trials (1988) demonstrated that the capacity of a 10m fixed length in clay using a 6m tendon bond length (6m in shear and tension, 4m in shear and compression) was 370kN. In an identical bore with a 10m fixed length containing four strands each with a 2m bond length staggered at 2.5m centres and subjected to simultaneous strand loading achieved 640kN. Load distribution is illustrated in Figure 8, the areas under the distribution lines proportioning to 370 and 640kN.

The first full-scale commercial single bore multiple anchors, (SBMA), in which each "unit" anchor encapsulation was isolated from others in the overall anchor fixed length, and each unit individually loaded, was at Southampton in 1988 (Barley 1995). A total load of 1337kN was recorded on the annular anchor load cell during testing of five unit anchors founded in cohesive Bracklesham Beds. Recently, permanent anchors with working loads of 1300kN have been installed in this stratum which highlights the advancement of the system. Test anchors achieve loads circa 3600kN. This development and refinement of multiple anchor techniques has continued over a twelve year period and some 40,000 permanent unit anchors have been successful installed and tested (Photo 5).
In urban areas, the "contamination" of ground under adjacent properties with steel tendons left after use is not tolerated. Here a fully removable multiple anchor system is available. This "environmentally friendly" system allows removal of the tensile steel member from the grouted borehole after usage over both free and fixed length. Some 5000 unit anchors have been installed in major cities, (London, Edinburgh, Hong Kong and Gratz), with working capacities in soils as high as 2000kN. (Barley, Payne, McBarron 1999).

7.2 Practical Constraints

7.2.1 General
Theoretically, the multiple anchor system would work to its maximum efficiency when utilising a large number of low load capacity unit anchors, each with relatively short unit fixed lengths over which no progressive debonding exists. However, the following constraints control the actual number of unit anchors and the unit anchor capacities:

i) The bond length or bond mechanism used at the tendon/grout interface of each unit to allow safe use of the full tendon capacity.

ii) The diameter and type of the corrosion protection of the fixed anchor (encapsulation).

iii) The influence of the passage of the “free” length tendons from the deep unit anchors (distal) on the bond capacity of the shallower unit anchors (proximal) and the resulting congestion in the borehole.

iv) The arrangement at the anchor head of the multiple of individual jacks in the hydraulically synchronised stressing system. (All unit anchors have different free lengths and hence require different amounts of extension and ram travel).

7.2.2 The unit anchor tendon
The difficulties in handling and coupling rigid bars, and the extremely low capacity of a single wire tendon, immediately exclude both types of tendon for consideration.

Strand is readily available in three sizes, 12mm, 15mm and 18mm, with a type variety in each group (normal, superstabilised, and dyform or compact). Extensive research information from strand and encapsulation pull-out tests has allowed a number of options of bond mechanism to be considered. These range from non-deformed strand to deformed tendon, or deformed tendon, or to mechanical locking devices. For permanent works requiring the encapsulation of the strand within a double plastic corrugated duct system (developed to comply with the corrosion protection
requirement of BS8081), the deformed strand is the preferred system, whilst for temporary works either deformed strand or a mechanical device for removable anchors is available (Barley and McBarron 1997).

Photo 5 : Use of Single Bore Multiple Anchors in Natchez, Mississippi where high load anchors founded in loess provided overall stability of soil nailed slope (Civil Engineering ASCE December 1997)

Although research has established that the full capacity of the entire range of strands could be achieved within encapsulation lengths of 1 to 1.5m, in practice, the unit encapsulation lengths have been standardised in the 2 to 3m range as a general safeguard.

Further research has determined the encapsulation size, complete with a double plastic layer, could be as little as 22mm, but the common diameter now in use is 50mm for ease of fabrication.

Initially, unit anchors contained only single strand, but the demand for higher unit tendon capacity to ensure failure at the ground/grout interface in preliminary trial anchors, necessitated incorporation of two strands. Subsequent development has confirmed that a multiple of strands may be incorporated satisfactorily into the double protected encapsulations of individual unit anchors to allow mobilisation of even higher unit anchor loads.

7.2.3 Multiple stressing jack and load measurement arrangement

The initial choice of the unit anchor tendon system determined that the range of test loads required in production unit anchors was between 200 to 300kN (75% characteristic strength of strands). In order to demonstrate factors of safety in the range of 2 to 3 in the test loading of preliminary trial anchors, or to achieve failure at the grout/ground interface, unit anchor test loads up to 600kN have accordingly been required.

In utilising a multiple of hydraulically synchronised jacks, the arrangement which maintains the unit anchor tendons on the minimum pitch circle diameter has been found to be most appropriate. This allows use of normal 150mm to 200mm diameter ducts at the head of the anchor, with only nominal deviation of the strand alignment. Safe loading to test loads of 4200kN can now be carried out utilising a specially designed seven-unit jack arrangement. (Photo 6).

Each of the jacks is coupled via a central manifold to a single hydraulic powerpack. Thus, during load application, the load in each anchor is always the same. The hydraulic pressure is measured by a pair of matching calibrated gauges and, based on the ram area of the identical jacks, the applied load is known. Any error in measurement of pressure is identified immediately by observation of discrepancy between the two gauge readings, and by checking the gauge pressure on the powerpack itself. Any friction within the
“system” can be established by carrying out loading and unloading cycles. Owing to continual difficulties over a 15 year period in achieving compatibility between loads established from pressure gauge readings with those recorded by load cells (strain gauged, vibrating wire or hydraulic), more emphasis has now been placed on determining loads by accurate reading of hydraulic pressure gauges alone.

Photo 6: Seven Hydraulically Synchronised Jacks Allow Simultaneous Loading of Seven Unit Anchors in a Bore Hole to 4200 kN in soil.

In the case of preliminary trial anchors, each individual jack also has its own pressure gauge and lock off valve. If, from the load/extension data, the failure or onset of failure of a unit anchor is suspected then its valve is closed and the load in that unit can be observed independently while further testing of other unit anchors is continued.

7.2.4 Effect of adjacent tendons on proximal unit fixed anchor lengths

All mechanisms which transfer load from tendon to grout, or encapsulation grout, subject the grout to bursting stresses. Owing to the very limited tensile strength of cementitious grout it is, in the majority of cases, the surrounding soil or rock which effectively confines the grout and prevents the grout column bursting at low loads. The presence of a number of strands in close proximity and within a compressible sheath, adjacent to the bond system of proximal anchors, provides a considerable weakness in the grout column and reduces the effective confinement. Research has been carried out to investigate the influence of the presence of the adjacent strands on the bond capacity of both encapsulations and mechanical devices. In soil conditions where confining stresses are limited, a system of surrounding the adjacent strands in non-compressible sleeves, and reinforcing the grout, has been developed to ensure these problems do not result in low capacity pull-out failure.

From the testing of the numerous anchors, it has been established that friction within the free length of the strands of distal anchors can, due to their passage of upper encapsulations, be greater than that in proximal anchors. For this reason, it is recommended that the lower limit of the apparent tendon free length acceptance criterion is 80% (or strand extensions are not less than 80% theoretical). This limit is consistent with that specified in the new European Standard and nominally less than that adopted by the British Standard, BS8081. It should be borne in mind, however, that via the nominal friction the load is still transferred into the overall fixed anchor length.
7.2.5  Effect of load change in a production single bore multiple anchor

It has been normal practice in the U.K. for over a twenty year period to apply a preload of 110% of working load to production anchors.

This generally provides more than a reasonable overload to ensure that, within the life of the anchor, load loss due to soil creep or tendon relaxation does not cause the load to fall below the designed working load. This procedure complies with BS8081 and as such is applied to more than 95% of installed anchors. However, there are occasions in which the full working load is not applied to an anchor, and subsequent load change results entirely from the amount of movement of the anchorhead in the axial direction.

When SBM anchors are installed for use in the normal applications, where full working load is applied, then no special considerations are necessary. However, where the anchors are intended to be partially or fully loaded by structural movement of the anchor head, then consideration must be given to the designed variations in the unit anchor free lengths. When the anchor head moves, the load increase in the proximal unit anchor will be greater than that in the distal unit anchor due to its shorter elastic length; thus the load locked into each unit anchor at a datum, or an intermediate level, must be varied such that when the calculated amount of movement necessary to load the anchor occurs, then after this movement the unit loads will be equal, and no individual unit anchor overloaded.

7.3  Test Anchor Programmes

One of the major benefits accruing from the installation and testing of preliminary trial anchors using the multiple anchor system, is that each unit anchor provides a full and comprehensive set of data with regard to its own elastic and non-elastic behaviour and bond capacity; i.e. a six unit anchor provides six times as much data as a normal anchor. Attempts have not been made to fully isolate the grout column associated with each unit anchor, and it is accepted that some upward transfer of load may exist between unit anchors during normal loading. However, in the trials carried out to date, the determination of failure capacity of some middle or lower unit anchors has not been prevented by this phenomenon. Furthermore, after reaching a general stage of failure, subsequent tests have been carried out to substantiate the information from individual unit anchors. The proximal anchor is loaded to failure first, and the associated grout column pulled away remote from the one below. This is repeated, working progressively towards the distal anchor.

In addition to the trials carried out to establish ultimate capacities, in the majority of cases, load holding tests have been carried out at locked off loads of 1.1 x working load, to ensure load loss does not exceed 1% load per unit time over 8 time periods (5,15,50,150 mins; 8,24 hours; 3, 10 day) in order to comply with the requirements of BS8081. Apart from two sites, one where tendon contamination was experienced, and the other where erroneous ground information was provided, no SBM unit anchors tested to date have failed this criterion, and generally load losses have been well within these limits.

7.4  Multiple Anchor Design

Given a load per metre run of wall to be retained by the ground anchors, it requires an experienced judgement to optimise on the anchor loads and anchor spacing, albeit it can be assumed that in the majority of soils the number of multiple anchors required will be in the order of half or 2/3 of the number of normal anchors. Having selected the working load per anchor, the number of unit anchors per anchor and the capacity per unit anchor requires an astute scrutiny of the ground conditions. Unit anchors can be founded in differing ground conditions. For example, recently on a river quay wall the upper unit anchors of high capacity were founded in sands and gravels, whilst the lower units of lower capacity were founded in chalks. Total anchor working load was 900kN and trial anchors achieved 2300kN.

Having established the working load requirement of each unit anchor, which is either identical or double that of the others, then use is made of Equation 7 (cohesive stratum) or Equation 12 (non-cohesive stratum) to establish the required fixed length of each anchor unit. Each of these equations, accommodating the efficiency factor acknowledges that efficiency reduces with increased unit length.

Use of Equation 6 demands the input of a value of ultimate bond stress associated with a short fixed length; From piling and anchoring

\[ \tau_{ult} = \alpha c_u \]  

(14)
where $\alpha =$ adhesion factor, $c_u =$ average undrained shear strength over fixed anchor (kN/m²).

Recommended values of $\alpha$ established from piling are in the 0.2 to 0.5 range, whilst the range of 0.3 to 0.6 has been achieved in anchoring over 6 to 8m lengths. Trials on SBMA utilising 2.5 to 3m unit length, have frequently achieved adhesion value of unity, albeit such value is influenced by drilling and grouting techniques. Thus, bearing in mind the efficiency factor, $f_{eff}$ accommodates the loss of efficiency due to long fixed length adhesion values associated with the short fixed length concept are likely to be in the 0.8 to 1.0 range.

Unfortunately, it is not always economic for the site investigation to provide full and comprehensive data on the clay shear strength over the full depth range. In an increasing number of situations, particularly in boulder clays and glacial tills, only standard penetration test data are available. Such data can be used in two ways to design the fixed length of anchor:

i) make use of the relationship and factors recommended by Stroud (1984) to allow clay shear strength to be estimated.

$$c_u = f_i N$$  \hspace{1cm} (15)

where $f_i =$ factor ranging from 4.4 to 6.0, $N =$ standard penetration test value. Thus make use of the derived clay shear strength value in the previous Equation 2).

ii) On the basis of failure loads exhibited in the trial anchor, determine a direct relationship between bond stress and $N$ for anchors in clays and;

$$\tau_{ult} = f_{10} N$$  \hspace{1cm} (16)

where $\tau_{ult} =$ ultimate bond stress and $f_{10}$ is one of a number of proposed new factors relating ultimate bond stress to standard penetration test values.

Such relationships have previously been proposed by Littlejohn (1970) for anchors in chalk , Barley (1988) for anchors in chalk, mudstone and sandstone and Barley (1995) for anchors in clay $f_{10}$ values have ranged from as low as 3 in stiff clay at shallow depth to 10 in boulder clays or 20 to 30 in chalks. Consistent with design approach above incorporating the efficiency factor, $f_{eff}$ relating efficiency to the choice of fixed length, the ultimate anchor load in may be represented by:

$$T_{ult} = \pi DL f_{eff} f_{10} N$$  \hspace{1cm} (17)

Each Unit anchor is designed individually based on ground strength at that depth.

8.0 SUMMARY

Forty years of development of anchors, associated load transfer mechanisms and corrosion protection systems complimented by the enormous refinement of anchoring plant and equipment have taken the industry into the 21st Century with a climate of healthy confidence.

However, without the implementation of the essential demands for construction expertise and general compliance with the available Codes and Practice, ground anchors are still a high risk consideration. Anchors which do not comply with Codes of Practice should not be eliminated from usage but where the consequences of anchor failure are severe, their performance in the short and long term (up to 120 years) must be fully researched and documented with particular attention to the items which currently prevent Code compliance.

The design of ground anchors utilising formulae which do not accommodate the acknowledged "taper off of capacity with length" should be superseded by use of formulae which incorporate "efficiency factors" or similar mathematical expressions that correct for the progressive debonding phenomenon.

The multiple anchor system, which is developed specifically to increase the efficiency in anchor utilisation of available or developed ground strength can safely provide working loads of 800 to 2000kN in the majority of competent soils and weak rocks.

For temporary works in urban areas "environmentally friendly", high or low capacity anchors are now available and after usage allow either free length, or free length and fixed length steel tendons to be removed.

Work to modern Codes, use modern design approaches, work with modern plant, but utilise age and experience : then let quality anchors take the strain.
9.0 ROCK BOLTS AND CABLE BOLTS

The structural elements used as ground anchors, cable bolts and rock bolts are similar in many ways but may be neatly characterised into three groups on the basis of scale and the length – capacity relationship. It is suggested that the three groups of devices have evolved in response to stability problems that find expression over three scales: large scale instability, intermediate scale instability and small scale instability. It appears that the length and capacity of each group of devices is related to the geometry and mass of the potential instability. Ground anchors tend to have the highest capacity and are usually longer than 10m, cable bolts tend to be between 3m and 10m long and rock bolts tend have the lowest capacity and are usually shorter than 3m.

Rock bolts and cable bolts appear to be most successful in maintaining local rather than regional stability around excavations in jointed rock or rock expected to fracture during stress redistribution. The deformation and collapse mechanisms in these regimes subjects the groups of reinforcement to a variety of loading modes over and above uniform axial loading. For example, the reinforcements are often required to respond to discrete displacements and rotations that may occur at joints and fractures that intersect along their length. In these circumstances the rock bolts or cable bolts are indeed acting as rock reinforcement.

The design of rock reinforcement is a complex problem that would require any formal design procedure to take into account both the geometry of the jointing system and the forces and displacements that may occur at the joints. The inherent difficulties in such an approach has led to the development of relatively informal design procedures involving rules and charts based on precedent (Lang, 1961) and rock mass classification schemes (Barton et al., 1974, Bieniawski, 1976). These procedures are simple, rapid, very popular and appear to have been highly successful. However, it is believed that attempting to understand the complexity of the design problem and the mechanical behaviour of individual and groups of reinforcements are central to raising the level of rock bolting and cable bolting to the standards set in ground anchor technology.

The balance of this discussion will attempt to explore the developing technology of rock reinforcement from a purely mechanical perspective. This results in design and selection procedures that are very different from the popular precedent and rock mass classification approaches.

10.0 THE DEVELOPMENT OF ROCK BOLT AND CABLE BOLT DEVICES

A review of the historical development of excavation stability will show that in the distant past many ingenious devices have been used to stabilise rock and soil materials for both civil and mining purposes, for example, see Brown (1999). The initial reinforcement devices took the form of natural bamboo and timber, manufactured wooden dowels and steel rods. Since then, a large range of devices has subsequently been developed in order to address the need for installing large numbers of reinforcements quickly and cheaply. Consequently, the development of rock reinforcement devices is linked in many ways to manufacturing technology and to drilling, blasting and excavation technology. However, the historical development of rock reinforcement will be viewed here from a purely mechanical perspective.

The application of rock bolts began in the early 1900’s and become a systematic practice in the 1950’s, some 100 years after the invention of reinforced concrete technology. Similarly, the application of cable bolts began in the early 1960’s, some 70 years after the patent for prestressed concrete was awarded. It is important to note that the first modern rock bolt elements were initially identical to the elements used in reinforced concrete and the first cable bolt elements were initially identical to those used in prestressed concrete. Furthermore, it appears that the development of an understanding of the mechanics of rock reinforcement has also followed in the footsteps of reinforced and prestressed concrete.

10.1 Reinforcement Action

Most ground reinforcement devices seek to provide additional stabilising forces and control the displacements within soil or rock materials. Structural engineers will immediately recognise these as the aims in reinforced and prestressed concrete technology. These aims are generic in all modern and indeed, ancient reinforcement technologies (eg, from fibre reinforced composites to straw reinforced clay bricks). In each case the intrinsic strength of these particulate or aggregated materials are characteristically weak in tension, shear, bending and torsion. Excess stress in these modes leads to excess strain that may manifest on both microscopic and macroscopic scale as initiation and propagation of dislocations or discontinuities.
Cracked materials are said to behave ‘discontinuously’ or as a dislocated system of parts. Reinforcement seeks to arrest or limit discontinuous behaviour by providing additional reaction forces and controlling the displacement at discontinuities. To achieve this, the reinforcement must transfer load from one side of a discontinuity to the other. This requires the reinforcement to be connected to, or embedded within the material either side of the discontinuity. The amount of reaction that can be mobilised and the amount of deformation sustained depend on the stress-strain characteristic of the reinforcement and the stiffness of the connection either side of the discontinuity. In general, the reinforcement must possess three attributes:

1. A sufficient force capacity to satisfy the force demand of the unstable mechanism.
2. A sufficient displacement capacity to satisfy the displacement demand of the unstable mechanism.
3. An overall response stiffness that can negotiate the force-displacement path to equilibrium.

The development of rock bolts and cable bolts has basically consisted of progressively modifying reinforcement stiffness in response to different geomechanical environments. Consider Figure 9, which shows the development of cable bolts. The initial cable bolts like the initial rock bolts were smooth steel rods or wires cement grouted into boreholes. Under axial load, Poisson contraction at the point of load application very quickly sets up a debonding front at the element/grout interface, which, with increasing load propagates along the reinforcement length producing a relatively soft axial response. Since then, cable bolt and rock bolt variants have evolved based on simply changing the ‘degree’ and the ‘extent’ of coupling between the rock and the bolt. The degree and the extent of coupling are very important because they control the stiffness of the device. The result of these developments is essentially a toolbox of reinforcement devices that may be used to address the different stiffness demands of different instabilities (eg. rock bursts to gravitational collapse). The same developments can be seen in conventional and fibre reinforced concrete.

Although the technique of prestressing will not be discussed here it is interesting to note that for many earthen materials, it is unlikely that unstressed reinforcement can prevent (as opposed to control) the onset of discontinuous behaviour. This is due to the low tensile or flexural ‘cracking’ strain limits of these materials (in the order of about 100 microstrain) compared to the stiffness of commonly used reinforcement materials (in the order of about 200Gpa). In other words, the structure has to deform or sometimes even crack in order to receive a reactive response from the reinforcement.

11.0 REINFORCEMENT SYSTEM MECHANICS

11.1 Reinforcement Systems

A reinforcement system comprises a system of four principal components: the rock, the element, the internal fixture and the external fixture. Each component has intrinsic load-displacement behaviour and is involved in two load transfer interactions. While the rock is not generally thought of as part of a reinforcement device, it has a marked influence on system behaviour through its interaction with other components and must be considered an integral part of the system.

11.2 Reinforcement System Response

The reinforcement system response is determined by the combined behaviour of the principal components of the system and their multiple interactions. Clearly, the response of one or more principal components or the interaction between any two components may dictate the overall response of the system. A mechanical systems approach allows the overall mechanical behaviour of the system to be predicted which in turn enables the design of the individual components to be optimised. Six different styles of reinforcement system response are shown in Figure 10a in relation to the intrinsic material behaviour of an isolated element. The style of system response will influence the suitability of a given reinforcement system to a given set of in situ requirements.

11.3 Fundamental Classes of Reinforcement System

The ‘degree’ to which the components are coupled together (ie. mechanically or frictionally) and the ‘extent’ of how they are coupled together (ie. continuously or discretely) will determine the response of the system and may be used to classify reinforcement devices into three fundamental types of system:

2. Continuously Frictionally Coupled (CFC) Systems.
3. Discretely Mechanically or Frictionally Coupled (DMFC) Systems.
<table>
<thead>
<tr>
<th>TYPE</th>
<th>LONGITUDINAL SECTION</th>
<th>CROSS SECTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Multiwire Tendon</td>
<td><img src="multiwire_tendon.png" alt="Diagram" /></td>
<td><img src="cross_section_multiwire_tendon.png" alt="Diagram" /></td>
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<tr>
<td>(Clifford, 1974)</td>
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</tr>
<tr>
<td>Birdcaged Multiwire Tendon</td>
<td><img src="birdcaged_multiwire_tendon.png" alt="Diagram" /></td>
<td><img src="cross_section_birdcaged_multiwire_tendon.png" alt="Diagram" /></td>
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<tr>
<td>(Jirovec, 1978)</td>
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<tr>
<td>Single Strand</td>
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<td><img src="cross_section_single_strand.png" alt="Diagram" /></td>
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<tr>
<td>(Hunt &amp; Askew, 1977)</td>
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</tr>
<tr>
<td>Coated Single Strand</td>
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<td><img src="cross_section_coated_single_strand.png" alt="Diagram" /></td>
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<td>(VSL Systems, 1982)</td>
<td></td>
<td></td>
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<tr>
<td>(Dorsten et al., 1984)</td>
<td></td>
<td></td>
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<tr>
<td>Barrel and Wedge Anchor On Strand</td>
<td><img src="barrel_and_wedge_anchor_on_strand.png" alt="Diagram" /></td>
<td><img src="cross_section_barrel_and_wedge_anchor_on_strand.png" alt="Diagram" /></td>
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<tr>
<td>(Matthews et al., 1983)</td>
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<tr>
<td>Swaged Anchor On Strand</td>
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<tr>
<td>(Schmuck, 1979)</td>
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<td>High Capacity Shear Dowel</td>
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<td><img src="cross_section_high_capacity/shear_dowel.png" alt="Diagram" /></td>
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<tr>
<td>(Matthews et al., 1986)</td>
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<td></td>
</tr>
<tr>
<td>Birdcaged Strand</td>
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<td><img src="cross_section_birdcaged_strand.png" alt="Diagram" /></td>
</tr>
<tr>
<td>(Hutchins et al., 1990)</td>
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<td></td>
</tr>
<tr>
<td>Bulbed Strand</td>
<td><img src="bulbed_strand.png" alt="Diagram" /></td>
<td><img src="cross_section_bulbed_strand.png" alt="Diagram" /></td>
</tr>
<tr>
<td>(Garford, 1990)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ferruled Strand</td>
<td><img src="ferruled_strand.png" alt="Diagram" /></td>
<td><img src="cross_section_ferruled_strand.png" alt="Diagram" /></td>
</tr>
<tr>
<td>(Windsor, 1990)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 9: A summary of the development of cable bolt configurations.
Table 1: A selection of devices classified according to their mechanical configuration.

<table>
<thead>
<tr>
<th>Reinforcement Device</th>
<th>CMC</th>
<th>CFC</th>
<th>DMFC</th>
<th>Rock Bolts</th>
<th>Cable Bolts</th>
<th>Ground Anchors</th>
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<tbody>
<tr>
<td>Deformed bar</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Hollow deformed bar</td>
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<tr>
<td>Ribbed bar</td>
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<tr>
<td>Rolled Thread bar</td>
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<td></td>
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<tr>
<td>Tubular Bolt</td>
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<td></td>
<td></td>
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<tr>
<td>Yielding bolt</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Square twisted bolt</td>
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<tr>
<td>Self drilling bolt</td>
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<tr>
<td>Cone Bolt</td>
<td></td>
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<td></td>
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<tr>
<td>Split Set</td>
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<td></td>
<td></td>
<td></td>
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<tr>
<td>Swellex bolt</td>
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<tr>
<td>GD Rock Nail</td>
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<td></td>
</tr>
<tr>
<td>Plain bar</td>
<td></td>
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<td></td>
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<tr>
<td>Pigtail or Wriggle bolt</td>
<td></td>
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<tr>
<td>Fibreglass bolt</td>
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<td></td>
<td></td>
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<tr>
<td>Slot and wedge bolt</td>
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<tr>
<td>Expansion shell bolt</td>
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<tr>
<td>Pre-stressing wire</td>
<td></td>
<td></td>
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<tr>
<td>Pre-stressing bar</td>
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</tr>
<tr>
<td>Pre-stressing strand</td>
<td></td>
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<tr>
<td>Polymer strand</td>
<td></td>
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<tr>
<td>Birdcaged strand</td>
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<tr>
<td>Bulbed Strand</td>
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<td>Ferruled Strand</td>
<td></td>
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</tr>
</tbody>
</table>

Windsor and Thompson (1993) have given a more detailed description of these three classes previously. This classification is thought to be valid for all reinforcement devices despite the fact that they are available in so many different configurations. A selection of devices from a large classification database is given in Table 1 and typical response ‘shapes’ for the three different classes are given in Figure 10b.

Figure 10: a) Types of force-displacement response. b) Force-displacement responses for the three classes.
12.0 REINFORCEMENT SCHEME MECHANICS

12.1 Reinforcement Schemes
A reinforcement scheme is really just a larger system made of individual reinforcement systems. The scheme may comprise rock bolt systems and cable bolt systems and possibly even ground anchor systems. The individual reinforcements may be arranged in a variety of geometrical configurations and installed as pre-reinforcement, post reinforcement, which may be pre-tensioned or post-tensioned and pre-grouted or post-grouted. The behaviour of the collapse mechanism depends on the combined results of the behaviour of natural components of the rock mass (e.g., rock blocks and discontinuities) and the artificial components (e.g., reinforcement systems) and their interactions in response to an excitation in the force-displacement field. Unfortunately, determining reinforcement scheme behaviour is particularly complicated. In some cases, it is possible to estimate reinforcement scheme response by applying analytical and numerical procedures to simple rock mass collapse mechanisms (e.g., deformation of a continuum, simple flexure of stratified beams, the agitation of simple systems of polyhedral blocks, etc.). However, in many cases a formal solution is rendered intractable by the numerous complexities of rock mechanics (Bray, 1967). These complexities often mean that a proper definition of the problem is often impossible. Consequently, the design of a reinforcement scheme is best conducted on the basis of a simple mechanism and the response of the scheme is best assessed by analytical simulation of that mechanism.

12.2 Rock Mass Mechanism Characteristics
The behaviour of many simple rock mass mechanisms can be described within a ‘characteristic force-displacement’ field. For example, consider for two very common cases. Firstly, a circular tunnel in homogeneous rock under hydrostatic stress. The characteristic force-displacement field is oriented radially and characterised by \((\sigma_r, \delta_r)\). Secondly, a polyhedral block undergoing translation (i.e., free-falling, sliding on a plane or sliding on a multiplicity of planes). The characteristic force-displacement field is oriented parallel to the direction of translation and is characterised by \((F_t, \delta_t)\).

The force-displacement relationship of a mechanism may be explored using the Mechanism Characteristic Diagram. This diagram displays the various characteristics and relations associated with a particular mechanism in force-displacement space. A simple Mechanism Characteristic Diagram for the problem of a reinforced block resting on a plane given in Figure 11. The block may slide due to gravity or be driven down the plane by stored energy in the rock mass. In both cases, the characteristic force-displacement field is oriented parallel to block translation. The characteristics shown on the diagram are now defined.

![Mechanism Characteristic Diagram for a simple block sliding mechanism.](image)

Figure 11: Mechanism Characteristic Diagram for a simple block sliding mechanism.
The ‘Excitation Characteristic’ is a relation that describes the excitation force-displacement field driving the rock mass mechanism. For the case of a block on a plane under the action of gravity plus an additional driving force \((E)\), the excitation characteristic is displacement dependent until the block detaches from the mass upon which it reduces to a displacement independent, constant characteristic. Most excitation characteristics will comprise some initial, non-linear, energy/displacement dependent function that eventually decays to some displacement independent gravitational characteristic.

The ‘Rock System Response’ describes the combined force-displacement response of the system of natural components comprising the rock mass. For the case of a block on a plane, the rock system response is the response of the shearing surface defined here by peak and residual strengths. In the absence of reinforcement, the rock mass system response is insufficient to arrest the mechanism and an additional contribution is required from a system of artificial components.

The ‘Reinforcement System Response’ is simply the load displacement behaviour of the reinforcement system. The ‘Reinforced Rock System Response’ describes the combined response of the rock and the scheme of reinforcement systems. If the Rock-Reinforcement System Response intersects the Excitation Characteristic, then stability is achieved in terms of both force equilibrium and displacement compatibility.

The ‘Mechanism Characteristic’ is a relation that describes the force-displacement path of the mechanism and is determined by the interaction of the excitation characteristic and the response of all components acting in the mechanism. The diagram defines the relative importance of the natural and artificial systems.

Although the resultant forces and displacements may act in one plane different reinforcement response modes (e.g., shear, and bending) as shown in Figure 12 could be generated depending on the relative orientation of the reinforcement and the displacement vector. Furthermore, these response modes might modify the behaviour of other components in the scheme.

![Diagram](https://example.com/diagram.png)

*Figure 12: A block with reinforcing elements oriented to reinforce different types of release surfaces.*

### 12.3 An Example of Reinforced Rock Mass System Characteristics

An example will be considered involving the reinforcement of a cubic block resting on a joint plane inclined at 45º as shown in Figure 13. The response of the rock reinforcement is explored using realistic joint and reinforcement responses and an assumed excitation characteristic. The unit weight of the rock is set to 30 kN/m³ and the shear strength of the joint is characterised by a peak friction angle of 35º. The reinforcement scheme involves a radial array of 6 metre long cable bolts that modify the normal stress conditions and thus the natural response of the rock component.
The stepped nature of the Rock System Response reflects the additional normal confinement provided by Cable 2 and Cable 3. Also note that Cable 4 is very inefficient compared with the other three cables. The Reinforced Rock System Response intersects the Excitation Characteristic at approximately 38 mm. Prior to equilibrium being reached, Cable 1 installed parallel to the plane has ruptured in tension. This is reflected in a spike in the Mechanism Characteristic at about 22 mm.

In summary, the Mechanism Characteristic Diagram is simply a graphical way of showing how a system of components undergoing a particular mechanism might simultaneously achieve force equilibrium and displacement compatibility. The fact that equilibrium is achieved does not necessarily mean that the design is satisfactory. For example, a particular demand may result from the need to arrest the mechanism at a given displacement and this will dictate the required reinforcement stiffness and the initial conditions (ie. time of installation, prestressing, pregrouting). The need to consider force equilibrium and displacement compatibility results in many complexities, some of which will be discussed in the following section. Other complexities associated with defining the demand of the mechanism (eg. the shape, size and mass of blocks) will be explored later.

13.0 COMMON INVALID ASSUMPTIONS IN REINFORCEMENT MECHANICS

Two simplifying assumptions are normally made in a stability analysis involving reinforcement. Firstly, the response of a reinforcement system is taken as its fully mobilised capacity and secondly, a one-dimensional analysis for equilibrium of forces is conducted. These assumptions are implicit in methods that calculate the factor of safety as simply the quotient of the resisting forces divided by the driving forces. For example, in the case of a block that would free fall from the roof of an underground excavation, the driving force is the weight of the block. The resisting forces are assumed to be the fully mobilised capacity of the elements in the reinforcement system with the resultant acting through the block centroid. The so-called factor of safety is then taken to equal the number of elements multiplied by their capacity divided by the weight of the block.

13.1 Reinforcement System Capacity and Response

The most obvious deficiency in the above approach is that the capacity of a reinforcement device is not necessarily its response. Furthermore, in the stability analysis of a reinforced block, the analysis must change from a ‘ubiquitous’ in nature to ‘specific’ in nature. In other words, it is no longer acceptable to assume that the block and the reinforcement systems may occur everywhere and anywhere, they are positioned in a specific configuration.
The ubiquitous assumption allowed the sum of the forces from all the reinforcement systems to be applied through the block centroid as a resultant force. The assumption that the reinforcement response was independent of displacement allowed this resultant to be set equal to the sum of the fully mobilised force capacities of all the reinforcing systems. However, the response of reinforcement is dependent on its deformation, which is dependent on its position in relation to the block.

13.2 Reinforcement Array Geometry

Many of the deficiencies in the approach are a direct result of ignoring the relative position of the block in relation to the reinforcements. The spatial relationship of the block and the reinforcement array raises a number of geometric issues concerning reinforcement length, orientation and position. These issues affect the capacity of the reinforcement system, the scheme, the stability of the reinforced block and the mode of block instability. Two issues will be explored here:

1. The effect of reinforcement position on reinforcement density and length.
2. The effect of length and orientation on load transfer and thus, system response.

Figure 14 shows how the number of active elements for the block may vary with the position of the block face relative to the reinforcement pattern. For the block shape given, the number of reinforcing systems sampled is shown to vary from 4 to 2. This problem will be addressed later by analysing discrete locations of the block face relative to the reinforcement pattern.

The specific position of the block relative to the reinforcement pattern will also define the effective point of reinforcement action. This point divides the reinforcement length into a block length and an anchor length as shown in Figure 15. The variation in these embedment lengths will affect the response of the reinforcement, especially for CMC and CFC devices.

13.3 Deformation Modes

In reality, the capacity of the reinforcement depends on the actual geometry of the reinforcement relative to the block and the discontinuities forming the block faces. That is, the resulting action of the reinforcement is at the discontinuity and not at the collar. Further, the reinforcement action is related to its orientation and involves both vertical and horizontal forces. These forces may cause the block to translate horizontally and to rotate as shown in Figure 16a. The resulting block displacements can be resolved into the axial, shear and rotational displacements of the reinforcement as shown in Figure 16b. These reinforcement displacements result in tension, shear and bending in the reinforcement element.
The assumption that blocks can only translate is usually applicable to surface excavations but is not generally applicable to underground excavations. It has also been found that the disposition of reinforcement in relation to the block centroid is less important for sliding blocks formed in surface excavations compared with falling and rotating blocks from overhanging surfaces. In the latter cases, reinforcement will be generally non-uniformly loaded and a simple force equilibrium approach is not valid. Reinforcing elements are loaded approximately equal only when the reinforcement is evenly distributed about the block's centre of mass. For proper analysis of a reinforced block, it is necessary to consider all 6 modes of possible displacement for a block (ie. 3 translations and 3 rotations) and the equilibrium associated with each of these modes of displacement. Furthermore, the analysis needs to satisfy compatibility of block displacements with the displacements occurring in the various reinforcement systems comprising the reinforcement scheme. An analysis procedure that satisfies these requirements is encoded in the SAFEX computer program package (Windsor and Thompson, 1992) and is described in more detail in Thompson (1989).

Figure 15: A tetrahedral block and the associated 'specific' reinforcement dimensions.

Figure 16: Reinforcement deformation mode caused by block displacement.
14.0 DEMAND, CAPACITY AND FAILURE

For each rock reinforcement scheme there are a number of selection requirements over and above achieving stability. For example:
- Serviceability
- Durability
- Procurement Logistics
- Installation Logistics
- Economic considerations

These selection requirements often limit, or indeed, dictate the reinforcement systems chosen and the arrangement of the scheme. However, in most circumstances it is also desirable to define what constitutes a ‘good’ design or the level of stability, safety or confidence required of the reinforcement scheme. This immediately raises the necessity to first define the demand, capacity and failure of the scheme.

14.1 Demand and Capacity

Demand may be defined as the force-displacement requirements to bring a rock mass collapse mechanism to equilibrium. Capacity may be defined as the force-displacement response that can be delivered by the reinforcement. Clearly, there is a demand for the overall mechanism that needs to be satisfied by the capacity of the overall reinforcement scheme and there is a demand for each reinforcement system in the scheme. The concepts of demand and capacity are illustrated in terms of both force and displacement in the mechanism characteristic diagrams given in Figure 17a and Figure 17b. In Figure 17a the demand characteristic is independent of displacement (as in gravitational collapse mechanisms) and in Figure 17b it is initially displacement dependent (common in stress or energy driven collapse mechanisms). In the first case, if force capacity exceeds demand then the mechanism can be arrested and equilibrium is achieved at a displacement depending on the reinforcement stiffness. In the second case, the force capacity, displacement capacity and stiffness of the reinforcement collectively determine if the mechanism can be arrested.

![Figure 17: The concepts of demand and capacity shown in force-displacement terms.](image)

14.2 Failure of a Reinforcement Scheme

Clearly, all reinforcement schemes seek to prevent failure. Failure is often thought of as physical collapse. In design terms, this leads to the notion that failure occurs when the so-called factor of safety drops below unity or when the forces driving the mechanism exceed the restraining forces.

However, consider the case given in Figure 18. It involves an unstable, 146.1 tonne, hexahedral block situated in the roof of a rectangular drive. Two alternative schemes of cable bolt reinforcement are assessed, one using standard strand the other, birdcaged strand. Both schemes involve the same pattern and density of reinforcement holes such that the block is penetrated by 6 vertical holes. Both schemes achieve equilibrium as shown in the stability assessment results given in Figure 18.
<table>
<thead>
<tr>
<th>STANDARD STRAND</th>
<th>BLOCK NUMBER 1: BLOCK WEIGHT 146.1 TONNES</th>
</tr>
</thead>
<tbody>
<tr>
<td>ELEMENT MEMBER</td>
<td>1  2  3  4  5  6</td>
</tr>
<tr>
<td>BEARING</td>
<td>90 90 90 90 90 90 deg</td>
</tr>
<tr>
<td>ELEVATION</td>
<td>90 90 90 90 90 90 deg</td>
</tr>
<tr>
<td>TOTAL LENGTH</td>
<td>5.0 5.0 5.0 5.0 5.0 5.0 m</td>
</tr>
<tr>
<td>BLOCK LENGTH</td>
<td>2.5 2.5 2.5 2.5 2.5 2.5 m</td>
</tr>
<tr>
<td>ANCHOR LENGTH</td>
<td>2.5 2.5 2.5 2.5 2.5 2.5 m</td>
</tr>
<tr>
<td>AVAILABLE AXIAL LOAD</td>
<td>27.0 27.0 27.0 27.0 27.0 27.0 tonne</td>
</tr>
<tr>
<td>AXIAL LOAD</td>
<td>24.3 24.3 24.3 24.3 24.3 24.3 tonne</td>
</tr>
<tr>
<td>AXIAL DESIGN DISP.</td>
<td>40.0 40.0 40.0 40.0 40.0 40.0 mm</td>
</tr>
<tr>
<td>AXIAL DISPLACEMENT</td>
<td>26.8 26.8 26.8 26.8 26.8 26.8 mm</td>
</tr>
<tr>
<td>AVAILABLE SHEAR LOAD</td>
<td>13.5 13.5 13.5 13.5 13.5 13.5 tonne</td>
</tr>
<tr>
<td>SHEAR LOAD</td>
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<tr>
<td>SHEAR DISPLACEMENT</td>
<td>0.0 0.0 0.0 0.0 0.0 0.0 mm</td>
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<table>
<thead>
<tr>
<th>NORTH</th>
<th>WEST</th>
<th>R.L.</th>
<th>TOTAL</th>
<th>DIP</th>
<th>AZIMUTH</th>
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<tbody>
<tr>
<td>-0.00</td>
<td>0.00</td>
<td>-26.81</td>
<td>26.8</td>
<td>90</td>
<td>176</td>
</tr>
</tbody>
</table>

| CENTROID ROTATIONS (d:m:s) | 0: 0: 0 0: 0: 0 0: 0: 0 |

| BIRDCAGE OR BULBED STRAND BLOCK NUMBER 1 : BLOCK WEIGHT 146.1 TONNES |
|--------------------------|------------------------------------------|
| ELEMENT MEMBER           | 1  2  3  4  5  6                         |
| BEARING                  | 90 90 90 90 90 90 deg                    |
| ELEVATION                | 90 90 90 90 90 90 deg                    |
| TOTAL LENGTH             | 4.0 4.0 4.0 4.0 4.0 4.0 m                |
| BLOCK LENGTH             | 2.5 2.5 2.5 2.5 2.5 2.5 m                |
| ANCHOR LENGTH            | 1.5 1.5 1.5 1.5 1.5 1.5 m                |
| AVAILABLE AXIAL LOAD     | 25.5 25.5 25.5 25.5 25.5 25.5 tonne      |
| AXIAL LOAD               | 24.3 24.3 24.3 24.3 24.3 24.3 tonne      |
| AXIAL DESIGN DISP.       | 7.0 7.0 7.0 7.0 7.0 7.0 mm               |
| AXIAL DISPLACEMENT       | 5.9 5.9 5.9 5.9 5.9 5.9 mm               |
| AVAILABLE SHEAR LOAD     | 12.8 12.8 12.8 12.8 12.8 12.8 tonne      |
| SHEAR LOAD               | 0.0 0.0 0.0 0.0 0.0 0.0 tonne            |
| SHEAR DESIGN DISP.       | 50.9 50.9 50.9 50.9 50.9 50.9 mm         |
| SHEAR DISPLACEMENT       | 0.0 0.0 0.0 0.0 0.0 0.0 mm               |

<table>
<thead>
<tr>
<th>NORTH</th>
<th>WEST</th>
<th>R.L.</th>
<th>TOTAL</th>
<th>DIP</th>
<th>AZIMUTH</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.00</td>
<td>0.00</td>
<td>-5.90</td>
<td>5.9</td>
<td>90</td>
<td>184</td>
</tr>
</tbody>
</table>

| CENTROID ROTATIONS (d:m:s) | 0: 0: 0 0: 0: 0 0: 0: 0 |

Figure 18: The results from stability analyses for two different types of reinforcement scheme.
Each standard strand has a system force capacity of 27.0 tonnes requiring an anchorage length above the block of 2.5 m. This scheme requires 30m of reinforcement, hole drilling and grouting. The total axial force capacity of the scheme is 152 tonnes; thus the conventional factor of safety is about 1.11. Axial force utilisation is about 90% and axial displacement utilisation is about 67%. This scheme achieves equilibrium at about 27mm of vertical displacement of the block.

Each birdcaged strand has a lower system force capacity of 24.3 tonnes but is axially stiffer than standard strand and requires only a 1.5m anchorage length above the block. This scheme requires 24m of reinforcement, hole drilling and grouting. The total axial force capacity of the scheme is 153 tonnes; thus the conventional factor of safety is about 1.05. Axial force utilisation is about 95% and axial displacement utilisation is about 84%. The scheme achieves equilibrium at 6mm of vertical displacement of the block.

Given that both schemes achieve equilibrium, which is the better design? One is apparently ‘safer’ than the other, has lower utilisation in terms of both force and displacement but is more expensive and allows greater displacement of the block. The answer of course depends on the project requirements and what constitutes failure. For example, would the larger block displacement lead to loosening of the rock around the block?

The implicit assumption made in most rock reinforcement design procedures is that failure is associated with forces alone. However, in some circumstances, excessive deformation may lead to a detrimental sequence of unwanted effects peripheral to the rock reinforcement scheme. In others a certain amount of deformation may be sought in order to redistribute loads elsewhere. So what constitutes failure for the project at hand?

The condition of failure, or more to the point the conditions of failure, may be defined in terms of both force and displacement for the individual components of the reinforcement scheme or the for whole reinforced rock system. Clearly, a number of conditions of failure could be specified, for example:

- Exceeding the force capacity in a number of the reinforcement systems.
- Exceeding the displacement capacity in a number of the reinforcement systems.
- Exceeding a given force utilisation in a number of the reinforcement systems.
- Exceeding a given displacement utilisation in a number of the reinforcement systems.
- Exceeding a given tolerable displacement of the unstable mass.
- Exceeding a given utilisation of the natural resisting forces.

The proceeding discussion implies that some consideration should be given to the mechanical requirements of the reinforcement scheme prior to commencing the reinforcement design process.

15.0 DESIGN AND SELECTION OF REINFORCEMENT SYSTEMS

There are two aspects to the design of reinforcement. Firstly, there is the design of the reinforcement system (ie. selection of the principal components followed by physical assessment of the force-displacement response of the assembled system). The main requirements here are to achieve an economically and mechanically efficient system, with compatibility of geometry, capacity and utilisation between the principal components. Secondly, there is the design of the rock reinforcement scheme (ie. selection of an arrangement of systems followed by analytical or numerical assessment of the scheme response). The main requirements here are to select an economically and mechanically efficient reinforcement scheme with reasonably uniform utilisation of all systems that will bring about force equilibrium of the mechanism at an appropriate displacement. In some circumstances it may also be appropriate to ensure that there is a sufficient level of unalleviated force and displacement capacity in reserve to negotiate subsequent perturbations. Consequently, the design process may be thought of as comprising two parts: selection of an appropriate reinforcement system and selection of an appropriate reinforcement scheme.

There are certain considerations that constrain the choice of an appropriate system. Table 2 sets out these considerations together with their applicability to each class of reinforcement. The first five constrain and simplify system selection considerably. In practice, the table would need to be more specific for there are exceptions to these loose generalisations. However, even this crude comparison allows the rational selection of an appropriate type of system.

The usage, entry and serviceability requirements of the excavation determine the level of durability required. The extraction and construction processes determine the adjustability required. Installation logistics also concern the excavation processes, the availability of equipment and the management of the quality control and assurance program.
Table 2: Reinforcement system selection considerations and generalised attributes.

<table>
<thead>
<tr>
<th>Selection Consideration</th>
<th>CMC</th>
<th>CFC</th>
<th>DMFC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Durability</td>
<td>High</td>
<td>Low</td>
<td>Medium</td>
</tr>
<tr>
<td>Adjustability</td>
<td>Low</td>
<td>Medium</td>
<td>High</td>
</tr>
<tr>
<td>Installation Logistics</td>
<td>High</td>
<td>Low</td>
<td>Medium</td>
</tr>
<tr>
<td>Economy</td>
<td>Low</td>
<td>High</td>
<td>Medium</td>
</tr>
<tr>
<td>Force Capacity</td>
<td>High</td>
<td>Low</td>
<td>High</td>
</tr>
<tr>
<td>Displacement Capacity</td>
<td>Low</td>
<td>High</td>
<td>Medium</td>
</tr>
<tr>
<td>Stiffness</td>
<td>High</td>
<td>Low</td>
<td>Medium</td>
</tr>
<tr>
<td>Load Relaxation</td>
<td>Low</td>
<td>High</td>
<td>High</td>
</tr>
<tr>
<td>Creep</td>
<td>Low</td>
<td>High</td>
<td>High</td>
</tr>
</tbody>
</table>

There are also economic issues and with these it is important to compare the total cost per installed reinforcement unit as opposed to simply cost per unit. Once a number of candidate systems have been selected, the problem becomes one of arranging a scheme of reinforcement systems to satisfy the mechanical demands of the collapse mechanism.

16.0 DESIGN OF REINFORCEMENT SCHEMES

The design of reinforcement for rock excavations requires consideration of many interrelated issues. If the process is simplified and thought of purely in mechanical terms then it could be considered to comprise six basic steps:

1. Formulation of a rock mass model.
2. Assessment of rock mass demand.
3. Dimensioning of trial reinforcement schemes.
4. Analysis of candidate reinforcement schemes.
5. Selection of an appropriate reinforcement scheme.
6. Performance assessment of the selected reinforcement scheme.

The design of reinforcement for an excavation in structured rock is particularly difficult due to a number of problems associated with completing steps 1 and 2. These steps are essentially concerned with defining the ‘demand’ of the rock mass or as shown previously the ‘excitation characteristic’. To date, the discussion has sidestepped that issue and the mechanics of reinforcement have been explored on the basis of a given shape and volume of material that needs to be stabilised. In order to simplify future discussion, only gravitational demand will be considered. Even with this simplification, defining demand is still often very difficult and is the key area that requires future research and development.

In structured rock the mutual intersection of discontinuities divides the rock into fully and partially formed blocks of rock. If an excavation plane cuts through this assembly of blocks a new set of blocks are formed at the excavation surface. Some of these ‘exposed’ or ‘surface’ blocks will have a shape that will allow them to fall, slide or rotate into the excavation should the block driving forces exceed the block stabilising forces. In order to understand how such a rock mass may best be reinforced or supported the assembly of blocks must be investigated. The ideal outcome from an investigation would be to predict the exact shape, size, stability and spatial position of each block that forms around the excavation. These block characteristics define the rock mass demand and provide the information need to dimension trial reinforcement schemes.

16.1 Defining Demand Deterministically

The ability to properly define the block characteristics is dependant on the quality and quantity of data that describes the rock mass surrounding the proposed excavation and the current predictive capabilities of the current block analysis techniques. If the exact position of each discontinuity is known in advance, together with the parameters that describe its geometry and strength characteristics then a ‘specific’ procedure such as that described by Warburton (1983) or Cundall (1991) could be used to conduct an assessment and make an accurate prediction of block shapes and sizes and assess their stabilities. Unfortunately, this data is usually not available until after the excavation has been formed therefore the
‘ubiquitous’ procedures such as those defined by Priest (1985) and Goodman and Shi (1985) need to be used to determine all the possible block shapes and sizes that could form. In this approach sets of discontinuities and an excavation surface are assumed to be able to occur everywhere and anywhere in space. This assumption means that all possible combinations of discontinuities and excavation surfaces are considered which produces a list of all the possible block shapes that form around the excavation boundary. The Warburton and the Goodman and Shi methods are equally important in that both cope with arbitrary polyhedra. However, the differences between ‘specific’ and ‘ubiquitous’ approaches have important consequences. In general, the specific approach will not, without an extreme number of analyses, provide the complete range of possible block shapes. However, it does provide the sizes of the specific blocks generated within each specific analysis. The ubiquitous approach can provide a complete range of possible block shapes but provides little information on the size of the possible block shapes that form. The recognition of these differences allows a third approach to be formulated that is somewhat of a hybrid of the specific and ubiquitous approaches. Windsor (1997) has described a hybrid approach, which comprised four basic phases of block analysis:

1. Shape analysis.
2. Size analysis.
4. Shape-size-stability characterisation

In this approach all phases are conducted using vector analyses. This brings the mathematical efficiency needed to analyse many blocks very quickly but lacks the visual description which aids comprehension that is inherent in the Goodman and Shi and Priest graphical methods. In the block shape analysis the discontinuities and excavation surfaces are assumed to be ubiquitous. In the block size and stability analysis each block shape may assume any size and occur anywhere on the excavation boundary within minimum and maximum constraints set by the dimensions of the discontinuities and the excavation. The final phase is used to produce single graphs that collectively characterise the block assembly in terms of the relative size and stability of the block shapes that could possibly form at the excavation surface. Although this approach has been received reasonably well, it made many unrealistic simplifying assumptions. One of the most important was that it conveniently ignored the stochastic nature of the rock jointing. This common characteristic dictates that the shapes, sizes and stability of blocks and thus the demands placed on a rock reinforcement scheme are also stochastic. Each block shape possible will occur over a range of possible sizes. Each block shape will be characterised by a shape-scale-stability relationship, indicating the stability of the shape over its possible size range. The implication is that in a deterministic design the complete size range of all possible block shapes must be addressed because by definition, each is equally likely to occur. This is never the case and a deterministic definition of demand and subsequent reinforcement design like the approach proposed by Windsor (1997) is unacceptable. In order to design reinforcement and support properly a probabilistic solution is required that takes into account the natural variation in the discontinuity characteristics and provides the probability of occurrence of all possible block shapes, their sizes and their stability.

16.2 Defining Demand Probabilistically

Probabilistic block analyses have been investigated in the past by many workers along both specific and ubiquitous lines (eg. Shapiro and Delport, 1991; Kuszmaul, 1993). Some of the more recent investigations, in particular, the approaches identified and being explored and developed by Kuszmaul (1994), Stone (1994) and Mauldon (1995) show great promise and suggest that a formal solution to this important practical problem may one day be possible. In the past, the author has unsuccessfully attempted to obtain a partial solution to this problem using two simulation techniques. Both techniques were found to be inefficient and too time consuming to be acceptable for use in standard practice. However, two important discoveries were made which suggested that with some modifications, the hybrid approach, described earlier, might be useful in providing at least a partial solution to the stochastic problem of defining block shape and block size.

16.3 Simulation Based Probabilistic Assessment of Demand

The hybrid block analysis has been modified to account for the variation in the rock structure. The block shape, size and shape/size/stability analysis phases described earlier are retained as ‘deterministic engines’. In other words, they conduct a set of fundamental calculations for shape, size stability etc. The engines may
be driven once with a unique set of input parameters to provide a unique deterministic answer or a 
multiplicity of times with variation of the input parameters to produce a stochastic result. The way that the 
proposed simulation is conducted distinguishes it from other probabilistic analyses. The main difference is 
the introduction of what for want of a better word will be termed a ‘possibilistic’ analysis. A possibilistic 
analysis is conducted to determine the extreme conditions within which the probabilistic analysis is then 
conducted. Consequently, considerably smaller rock volumes are involved in each realisation and 
considerably less computational time and effort are invested in dealing with discontinuities that cannot be 
involved in block formation. The simulation procedure for defining demand has been described by Windsor 
(1999). The input requirements for the simulation comprise:

- The excavation face orientations and dimensions.
- The rock unit weight distribution and associated statistical parameters.
- The number of discontinuity sets and, for each set:
  - The mean orientation, Fisher’s constant and the chosen confidence level.
  - The trace length distribution and associated statistical parameters.
  - The spacing distribution and associated statistical parameters.
  - The joint friction distribution and associated statistical parameters.
  - The joint cohesion distribution and associated statistical parameters.

The last 4 variables may be simulated using the exponential, normal, lognormal or beta distributions. The 
simulation procedure results in a prediction of:

- The relative probability of each block size of each possible block shape forming.
- The relative probability of each block size of each block shape forming and being unstable.

The results are given as relative frequency distributions and cumulative frequency distributions of the 
characteristics that define demand. These are termed demand characteristics. For example, block height, 
face area, volume, weight, stability index (Factor of Safety), out of balance force, out of balance pressure and 
the displacement vector of the block.

Some example outputs from a simulation on an underground excavation roof intersected by 5 sets of 
discontinuities are given in Figures 19 to 24. The simulation comprised 1 million realisations on each of the 
10 possible block shapes. The 10,000,000 realisations produced a total of 258,197 removable blocks (ie. 
2.58%) of which, 91,982 were unstable blocks (ie. 0.92%). Figures 19 and 20 show the relative frequency 
distribution and the cumulative frequency distributions of apex height and stability index respectively for 
block number 1. The relatively frequency distribution in Figure 19 has been normalised to the maximum 
frequency to better show the shape of the distribution. Figures 21 and 22 show the cumulative frequency 
distributions for apex height and stability index for all removable blocks. Figure 22 clearly shows that some 
but not all of the removable blocks are unstable depending on their size. Figure 23 and 24 show the 
distribution of dip and azimuth of the block displacement vectors.

The 10 million realisations took a total of 847 minutes (14.1 hours) to complete on a standard PC. 
However, it was noted in this exercise and from work on other problems that the distributions seem to 
become relatively smooth and precise after about 250,000 realisations. This implies that for ‘design work’ 
on a problem involving 5 sets of discontinuities a simulation of 1250,000 realisations followed by storage 
and plotting of results could be comfortably completed within 1 working day.

17.0 DIMENSIONING TRIAL ROCK REINFORCEMENT SCHEMES

The previous sections have discussed the second step in the reinforcement design process for the case of 
blocky rock masses; namely assessment of the rock mass demand. The demand was determined in terms of 
the relative frequency distributions for the block characteristics of the different shaped unstable blocks over 
their anticipated range of occurrence

17.1 Demand Characteristics

Consider Figure 25 which shows the frequency distributions and cumulative frequency distributions for 
two particular blocks i and j and the distributions for both blocks collectively for a certain block 
characteristic. A distribution can be drawn for any given block demand characteristic (e.g. mass). These 
characteristics define particular aspects of demand which enable particular aspects of a trial rock 
improvement scheme to be selected and dimensioned.
Figure 19: Apex height - relative frequency and cumulative frequency distributions for block no.1.

Figure 20: Stability index - relative frequency and cumulative frequency distributions for block no.1.

Figure 21: Cumulative frequency of apex height for all removable blocks.

Figure 22: Cumulative frequency of stability index for all removable blocks.

Figure 23: Distribution of translation dips (Note there is a high percentage of dips at 90°).

Figure 24: Distribution of translation azimuths (Note there is a high percentage of azimuths at 0°).
The block characteristics found to be particularly useful in reinforcement and support design include:

- Altitude (H) - perpendicular distance to the apex.
- Face area (A_f) - block area in the excavation face.
- Out-of-balance force (OBF) - difference between driving and resisting forces.
- Displacement vector (α / β) - movement direction.
- Face perimeter (L_p) - perimeter of the face area.
- Shear surface area (A_s) - sum of shearing areas.
- Force demand $F_{αβ}$ - Force in a given direction required to maintain equilibrium.
- Pressure demand $P_{αβ} = F_{αβ} / A_f$.
- Internal shear demand ($S_{in}$) = OBF / A_s
- External shear demand ($S_{ex}$) = OBF / L_p x 1mm

These block demand characteristics may be used for dimensioning of:

- Reinforcement systems and schemes (eg. rock bolts, cable bolts and ground anchors) in terms of axial and shear capacity, length, orientation, density, pattern type and pattern dimensions.
- Support systems and schemes (eg. shotcrete, polymer membranes, sets, liners, etc.) in terms of shear, tensile, flexural and bond capacity and cover area, cover dimensions and cover sequence.
- Restraint systems and schemes (eg. mesh, straps, laces, etc.) in terms of shear, tensile, and flexural capacity, grid dimensions, grid orientation, cover area and cover sequence.
17.2 Use of the Demand Characteristics in Trial Reinforcement Scheme Dimensioning

The use of the relative frequency distributions in rock improvement scheme design requires careful consideration of a number of issues:

1. It is extremely important to recognise that a ‘trial’ rock improvement scheme must be dimensioned before it can be assessed for reliability. The distributions given here are for use in dimensioning trial solutions before the design process enters the infinitely more complex reliability assessment step (ie. a simulation based probabilistic force-displacement analysis). The reliability assessment procedure is discussed later.

2. The distributions provide a measure of the relative probability of the block characteristics occurring for each removable block. Consider the case of a rock mass containing one unstable removable block. The actual portion of the shape envelope that tends to produce say, blocks with large apex heights does not necessarily coincide with the portion of the envelope that tends to produce say, blocks with large pressure demand. The altitude distribution may be used to dimension the length of reinforcement and the pressure demand distribution may be used to select reinforcement capacity and array dimensions etc. The blocks with large altitude will be supplied with appropriate length reinforcement and the blocks with large pressure demand will be supplied with an appropriate reinforcement capacity at a suitable density. The resulting trial design is somewhat inconsistent and may be more appropriate for some blocks than others. This is why the trial design must be analysed for reliability after it has been selected and dimensioned.

3. The relative frequency distributions of $\Omega$ tend to a negative exponential form or comprise a peak frequency at a low value of $\Omega$ and decay to very low frequencies at higher values of $\Omega$. This means that the higher values of $\Omega$ are all poorly defined. Unfortunately, the designer must choose an upper design limit ($\Omega = \Omega_{\text{max}}$) in the relatively ill defined asymptotic region. It is more helpful to consider this problem in terms of cumulative frequency. An $X\%$ cumulative frequency may be chosen to account for $X\%$ of blocks. This means that the design must be dimensioned to account for all blocks in the range $\Omega_{\text{min}} < \Omega < \Omega_{X\%}$. The author uses 95% level (i.e $\Omega_{95\%}$) to achieve reliability for the larger blocks.

4. The distributions of $\Omega$ may require a rock improvement scheme design that comprises a combination of reinforcement systems, support systems and restraint systems. For example, Figure 26 is a schematic representation of the relationship between the size of a particular shaped block and the approximate force and size capacity of primary and secondary reinforcement (e.g. cable bolts and rock bolts), shotcrete, mesh and straps. The diagram also shows a number of limiting block sizes that are constrained by the capacity of the rock improvement scheme design. For example, the primary reinforcement pattern limited block is the largest size block that can possibly move through the primary reinforcement pattern. These block limits are a subset of ‘capacity limits’ defined by practical issues associated with improvement schemes. The capacity limits divide the block size range into sections which together with the distribution of $\Omega$ in the range $\Omega_{\text{min}}$ to $\Omega_{X\%}$ indicate what type of rock improvement systems are required. Clearly, a number of alternative designs involving different types of rock improvement may be selected. However, the selection is again somewhat simplified by considering other issues such as project requirements in terms of serviceability, longevity, installation equipment and compatibility within the excavation and construction sequence, etc. However, the choice of dimensions will invariably result in a design that is more appropriate and more reliable for some block shapes than for others. This supports the suggestion that trial designs must be analysed for reliability.

5. The distributions for all removable, unstable block shapes may be drawn on one diagram together with an aggregated distribution for all the removable, unstable block shapes. Issues 2, 3 and 4 were discussed with respect to one unstable block but can be equally applied to the case of multiple unstable block shapes. Basically, the design must account for the demand for all the blocks in the range $\Omega_{\text{min}}$ to $\Omega_{X\%}$. However, some of the demand characteristics are affected by the displacement vector $(\alpha / \beta)$. For example, force demand $F_{\alpha\beta}$ and pressure demand $P_{\alpha\beta}$ are a function of the direction in which a block moves $\alpha / \beta$ and the direction in which the capacity is supplied $\alpha / \beta$. Reconsider, Figures 23 and 24 which show the variation in $(\alpha / \beta)$ for each block and for all blocks. Force, pressure and shear capacities must be dimensioned and oriented to satisfy all blocks in the range $\Omega_{\text{min}}$ to $\Omega_{X\%}$. Some of the dimensioning procedures needed to account for this are given in Windsor 1997. A restricted choice of orientation for the reinforcements will invariably result in a design that is more appropriate for some block shapes than for others. Again, this requires that the trial designs must be analysed for reliability.
Figure 26: Schematic representation of block scale and resulting demands on the rock improvement scheme.

ROCK IMPROVEMENT SCHEME CAPACITY

- PRIMARY REINFORCEMENT DESIGN
- PRIMARY & SECONDARY REINFORCEMENT DESIGN
- MESH DESIGN
- PLAIN & REINFORCED SHOTCRETE DESIGN
- STRAP DESIGN

BLOCK SIZE (e.g. HEIGHT, AREA, VOLUME, MASS)

- MESH PATTERN LIMITED
- STRAP PATTERN LIMITED
- P. and S. REINFORCEMENT PATTERN LIMITED
- PRIMARY REINFORCEMENT PATTERN LIMITED
- EXCAVATION FACE LIMITED
18.0 RELIABILITY ASSESSMENT OF TRIAL REINFORCEMENT SCHEMES

Once a trial rock improvement scheme design has been selected and dimensioned it must be assessed to ensure that it satisfies a certain reliability level of stability in terms of force equilibrium and displacement compatibility. In most but not all circumstances, the stability of each block shape is markedly affected by its position in relation to the rock improvement scheme as shown earlier in Figure 14. Thus, the reliability assessment is a complex force-displacement analysis in which the position of each block shape (over its complete shape and size range) is allowed to vary in relation to the rock improvement scheme.

Fortunately, the work conducted to obtain the demand characteristics can also be used to simplify the reliability analysis. The demand distributions for the different block characteristics may be approximated by standard statistical distributions in which the random variable $\Omega$ is continuous over a finite interval $(a, b)$. These distributions may then be used to simulate a distribution of block characteristics for each unstable block shape. The blocks may occur anywhere within a region of the rock improvement scheme constrained by the ‘block existence zone’ (Windsor 1999) associated with that block shape. This may be achieved by selecting a block vertex in the excavation face and positioning it within the block existence zone. The vertex position defined by the three coordinates $(x, y, z)$ is then generated to occur within this region.

Consider Figure 27 which shows the results from force-displacement analyses on a block (shape and size invariant) which has simply been moved in relation to the reinforcement scheme. In this Case A - the block is reinforced with 5 bolts and is unstable. In Case B - the block is reinforced with 5 bolts and is unstable. Now consider the amount of information given in the results. In a reliability assessment the variants of shape, size and position must all be simulated in accordance with the demand characteristics of the block. The outcome is an array of results for each block in terms of block displacement and the forces and displacements generated in the reinforcement scheme. The large amount of valuable information produced must be used to determine the suitability of the scheme in terms of the ‘conditions of failure’ discussed earlier. Here one must consider the end aim of this exercise and the manipulation of results from numerous stability analyses. The end aims are to define the reliability of the design in terms of block stability and rock reinforcement scheme performance. Furthermore, recall that there could be many different shaped blocks involved that give rise to many different forms of demand, all of which must be satisfied. Consequently, the results must be presented in a format that enables the designer to modify inappropriate designs to achieve both reliability and efficiency across the whole scheme. A reliable and efficient design is one in which:

1. Rock mass stability criteria are satisfied (eg. maximum block displacement).
2. Rock reinforcement scheme performance criteria are satisfied (eg. force capacity utilisation)

Because large amounts of information are produced it is convenient to considered this visually. Thus, a grid simulation of block position is suggested where the chosen block vertex is sequentially placed throughout a grid of specific positions in relation to the reinforcement scheme. This enables three-dimensional surfaces to be drawn with the grid axes (or block position) as abscissae and stability or performance characteristics (eg. block displacement, the force, moment, displacement utilisation of the scheme etc.) as the ordinate. The stability and performance criteria chosen are constant and may be represented by a flat plane on this diagram that intersects the 3D surface to indicate the probability of whether or not the trial reinforcement scheme satisfies the reliability and performance criteria. This approach also provides the designer with clues on how to improve unsatisfactory designs.

19.0 DESIGN OF COMBINED REINFORCEMENT – SUPPORT SCHEMES

Reinforcing systems are often used in ‘combined reinforcement-support schemes’. Support systems are usually constructed on or fixed to the boundary of an excavation where they provide an areal, reactive force in response to deformation of the boundary. Modern support systems used with reinforcement in combined improvement schemes include:

- Straps or beams (usually steel or reinforced concrete).
- Mesh (usually rigid or articulated steel grids).
- Membranes (usually polymer sprays).
- Shotcrete (sprayed concrete either plain or reinforced).
- Concrete (poured and may be plain, reinforced or prestressed).
Figure 27: Stability assessment results for two positions of the same shape and sized block. Note that:

a) As different positions of the block are simulated its stability is affected by the number and relative geometry of the reinforcements that are intersected.
b) The block intersected by 5 bolts is unstable, the block intersected by 4 bolts is stable.
c) Such outcomes are implicitly ignored in the conventional design procedures.
In comparison, reinforcement can provide either active or reactive forces linearly within the mass and at a point on the boundary. The distinction between the two in terms in how they supply artificial forces to the rock (ie. active or reactive, point or areal) is important when considering the response of the combined improvement scheme. In a combined scheme, reinforcement and support must interact together as a system of components to satisfy the force and displacement demands and negotiating the collapse mechanism to achieve both force equilibrium and displacement compatibility.

The notion of combined reinforcement and support is similar to the idea of ‘belt and braces’ but with the important distinction that in this case, one is not a back-up for the other and it is not necessary for one component to fail in order to activate a response from the second. The circumferential tension in the belt does not necessarily need to exceed the belt capacity in order for axial tension to be developed in the braces. It may be that by providing a small tension in the belt and a small tension in the braces a more efficient and safer design can be achieved. Basically, if both are required then they should be designed to work together.

The partnership, may be simple or complex. For example, the reinforcement may be relatively rigid and the support relatively flexible and vice-versa. The reinforcement may be supplementary to the support and vice-versa, the reinforcement may not respond until called upon by the support and vice-versa. Here, only two ways that reinforcement and support may be combined will be discussed. In the first, the reinforcement simply provides points of fixture and reaction for an areal support system. In this application the reinforcing system is a structural anchor. Ground anchors are commonly used in this application in conjunction with a relatively rigid structural diaphragm. The ground anchors may be prestressed to supply a large active collar force and depending on construction sequence and the flexural rigidity of the diaphragm the system may be configured to impart an active pressure at the boundary. In the second, the reinforcement provides both points of fixture and reaction for an areal support system and reinforcing actions within the mass. Rock bolts and cable bolts are commonly used in this application with relatively flexible support schemes (commonly straps, mesh, shotcrete or combinations). In either case, the key is to recognise the importance of each component of the system, its role and responsibility in arresting the instability.

The rationale for simulation based design and reliability assessment for support schemes is the same as that for reinforcement schemes. The demand characteristic is taken as that not accounted for by the reinforcement scheme (ie the smaller blocks that may exist between the reinforcement systems). Again block position, shape and size must be simulated to with respect to the support system. For example, Figure 28 shows the position of a block in relation to the fixture points for a sheet of mesh and a surface coated in shotcrete. The block shape, size and position all affect the loading and displacement of the mesh and the likely geometry of a yield line crack pattern in the shotcrete and thus its moment capacity and deflection.

Figure 28: Block shape and position relative to mesh fixture points and shotcrete yield line crack patterns.
20.0 SUMMARY GROUND REINFORCEMENT ISSUES

The second half of this paper has reviewed the development of rock bolt and cable bolt devices from a purely mechanical perspective and their behaviour as a system of components. The mechanics of a scheme of reinforcement systems was then explored in terms of its force and displacement behaviour in response to simple collapse mechanisms. The paper then described procedures for the deterministic and probabilistic design of trial rock reinforcement schemes followed by a brief description of a simulation based reliability assessment of these trial designs. This systems approach to understanding mechanical behaviour and to design is novel in that reinforcement design is most commonly conducted by alternative approaches based on precedent rules or rock mass classification. These latter approaches are extremely simple, popular and apparently very successful whereas the proposed systems approach is complex, cumbersome and not so popular. However, it has been explored here in order to highlight some of the important mechanical concepts associated with reinforcement and its design. It is believed that mechanical concepts embodied in the very complexities of reinforcement design hold the keys to future development of this discipline.

A number of issues have been discussed which probably deserve examination and assessment of their validity by the civil and mining engineering community. Some of these are listed below:
1. Ground reinforcement is related to reinforced and prestressed concrete technology.
2. Device variation results in variation in the degree and extent of coupling to the rock.
3. Reinforcement devices are systems of principal components.
4. Reinforcement schemes are systems comprising reinforcement systems.
5. Reinforcement behaviour must be considered in terms of both force and displacement.
6. Stability analysis must be specific and consider force equilibrium - displacement compatibility.
7. Design of reinforcement schemes for structured rock is better conducted probabilistically.
8. The design of rock reinforcement schemes should involve:
a) Probabilistic assessment of rock mass demand.
b) A definition of what constitutes the stability and performance criteria.
c) Dimensioning of trial schemes.
d) Reliability assessment of these trial candidate schemes.
9. The design of combined reinforcement and support schemes follow the logic given above in 8.
10. Research and development is required in the areas 8a to 8d given above.

It is hoped that this discussion will foster interest and critical debate such that the discipline of ground reinforcement can progress to the level of formality now demanded in ground anchor technology.

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22.0 REFERENCES


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