ANCHOR FIELD TESTS IN CARBONIFEROUS STRATA

Essais en place d'ancrage dans un sédiment carbonifère

by

G.S. LITTLEJOHN

Geotechnics Research Group, University of Aberdeen, Scotland

D.A. BRUCE

Geotechnics Research Group, University of Aberdeen, Scotland

W. DEPPNER

Universal Anchorage Co, Bolton, England

SOMMAIRE

La communication présente les résultats préliminaires des essais en vraie grandeur des cinquante-sept ancrages dans des sédiments carbonifères. Le site géologique est brièvement décrit, ainsi que la construction de l'ancrage et les méthodes d'essais adoptées. Avec des scellements de différentes longueurs (0.75 à 6 m) à des profondeurs différentes (0.75 à 12 m), divers modes de rupture ont été obtenus. Des valeurs d'adhérence de l'interface roche-coulis sont données ainsi que l'influence de la pression d'injection et des écarts entre les brins sur le transfert des charges et le comportement à la rupture.

SUMMARY

The paper discusses the preliminary findings of full scale tests on 57 anchors installed in carboniferous sediments. The site geology is briefly described together with the anchor construction and testing methods adopted. To investigate the influence of anchor geometry on failure mode, anchors ranging in overall depth from 0.75 to 12 metres were tested with grouted fixed anchor lengths of 0.75 to 6 metres. The observed effects of depth of embedment, grout surcharge, tendon configuration, interstrand spacing and tendon density on anchor performance are discussed in relation to current practice. Measurements of interfacial bond and load transmission are presented.

INTRODUCTION

A world-wide survey of prestressed rock anchor practice by Littlejohn and Bruce (1975-1976) has highlighted a dearth of information concerning the fundamental behaviour of rock anchors with particular reference to the mechanism of load transfer and modes of failure.

In order to study phenomena such as rock mass failure, localised bond failure, critical embedment and debonding, full scale pull-out tests have been carried out on 57 instrumented rock anchors. The purpose of this paper is to highlight the preliminary findings.

SITE GEOLOGY

The anchors were installed in Upper Carboniferous sediments of the Middle Grit Group of the upper part of the Millstone Grit Series. The sequence fined downwards from gently dipping massive, coarse, gritty siliceous sandstones to finer grained flaggy and shaley sandstones. In addition, a total of eight soft, friable mudstone beds were exposed or inferred in the sequence. Different groups of anchors were installed from different stratigraphic levels due to the presence of various benches, but each intersected at least one argillaceous bed at the grouted fixed anchor level. The whole sequence was conspicuously vertically jointed, the major orientations being north, east-north-east, (most prominent), and south-east. Joint spacing varied greatly, being up to 1 metre in the coarser sandstones. The major geotechnical properties are provided in Table 1.

TABLE 1

Summary of geotechnical properties

	Range	Mean
Fracture Index	10 — 1	6
RQD	60 — 100	90
Unit weight (Mg/m^3)	2.45 — 2.60	2.50
Ultimate Pulse Velocity (km/sec)	2.00 — 4.50	3.50
Diametral Point Load Strength (N/mm^2)	0.50 — 6.00	3.80
Elastic Modulus - material (N/mm^2)	$(1.3 - 2.8) \times 10^{+4}$	$2.0 \times 10^{+4}$
Elastic Modulus - mass (N/mm^2)	$(0.5 - 1.6) \times 10^{+4}$	$1.0 \times 10^{+4}$

ANCHOR CONSTRUCTION

All holes were drilled vertically, by rotary percussion, to provide a nominal diameter of 114 m. The tendons, which consisted of 10 No. 7-wire Dyform 15.2 mm diameter strands of 300 kN individual capacity were assembled in a straight, parallel formation with a 10 mm clear spacing (fig. 1) unless otherwise specified. In order to dissociate the free (elastic) length from the surrounding grout or ground, grease impregnated tape was wrapped around each strand over the required length.

Prior to tendon homing each hole was water tested, and sealed with neat cement grout, if the water loss exceeded 3 litres/minute/atmosphere. On completion of sealing, neat Rapid Hardening Portland Cement grout ($w/c = 0.45$) was tremied into the hole and the tendon slowly homed.

The grout, prepared in a conventional paddle mixer, gave bleed readings of 1.3 to 2.1% and stressing only took place once the grout had achieved a crushing strength of 28 N/mm^2. At least two anchors of each type were installed.

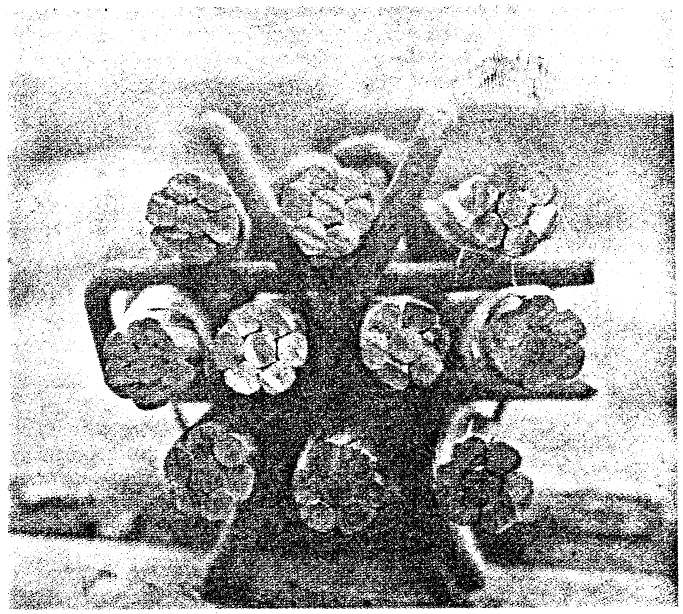


Fig. 1. — **Arrangement for standard ten strand tendon.**

ANCHOR TESTING

A hydraulic stressing system was evolved to enable the anchors to be incrementally, and cyclically loaded to failure, or to a maximum of 260 kN per strand. The system comprised remote loading through a simply supported beam and accommodated both multistrand and monostrand stressing modes. Dial gauges yielded anchor extensions and rock surface displacements to 0.01 mm accuracy, and annular load cells gave a direct reading of the applied load to 1% accuracy. A second, independent, direct measure of stress distribution was provided by strain gauges attached at strategic positions on a large proportion of the strands installed. At least one anchor of each type was instrumented in this way.

DISCUSSION OF RESULTS

Rock mass performance

Table 2 illustrates the overall performance of the shallow fully bonded anchors, where all the major modes of failure are reproduced.

TABLE 2

Average interfacial bond values at maximum test loads for shallow fully bonded anchors

Anchor No	Embedment (m)	Maximum Test Load (kN)	Rock-grout bond (N/mm^2)	Grout-tendon bond (N/mm^2)	Grout Crushing strength (N/mm^2)	Mode of failure
1	0.75	440	1.64	1.23	45	Rock mass
2	0.75	500	1.86	1.40	45	Rock mass
3	0.75	450	1.68	1.26	46	Rock mass
4	1.50	1 495	2.72	2.09	60	Rock mass
5	1.50	1 355	2.52	1.89 F	62	Grout-tendon
6	1.50	1 206	2.24	1.68	50	Rock mass
23	1.50 (u)	1 834	3.41 *	2.56	50	Rock mass
24	1.50 (u)	1 594	2.97 *	2.23	51	Rock mass
51	2.25	2 411	2.99	2.24	35	None-rock mass imminent
52	2.25	1 978	2.45 F	1.84	36	Rock-grout
53	2.25	1 891	2.35 F	1.76	37	Rock-grout
7	3.00	2 353	2.19	1.64	49	Strand fracture
8	3.00	2 469	2.30	1.72 F	43	Grout-tendon
9	3.00	2 122	1.97	1.48 F	44	Grout-tendon

(*) Calculated as a straight shaft. F - failure value at interface.

Up to embedment depths of 1.5 m, failure occurred mainly in the rock mass. For greater depths failure tended to be localised at one of the grout interfaces. For rock mass failure, the shape of the rock volume mobilised in each case was strongly controlled by the incipient rock mass structure (fig. 2).

For example, the major radial fractures developed along the trends of the major joint directions, whilst the projected shape of the rock volume mobilised below the surface was strongly influenced by the laminar nature of the mass. Under similar conditions, underreamed anchors sustained higher loads than their straight shaft counterparts. Underreaming was carried out with a UAC patented tool to form pairs of «bells» of 230 mm diameter and whilst a greater extent of rock was mobilised, the pattern of surface fracturing was not radically different.

For the shallow anchors installed in unweathered rock in this project, the ultimate resistance to rock mass failure is reasonably estimated from the empirical rule P (kN) = 600 d^2, where d is the depth of embedment (m). For the traditional and conservative design concept pertaining to the weight of an inverted 90° cone, and using a unit weight of 2.5 Mg/m³ for the rock, factors of safety ranging from 14 to 45 are indicated, assuming the apex at the base of the anchor. Employing the observed extent of the fissuring to speculate on the size of cones mobilised, included angles of (117° - 144°) and (90° - 114°) can be calculated for the apex positioned at the mid point and base of the anchor, respectively. Assessing the weights of these cones the factors of safety against pull-out are (14 - 56) and (8 - 29) for the apex at mid point and base, respectively. These figures highlight that other «rock strength» parameters constitute the major component of resistance to pull-out, and assuming that the actual failure volumes were more akin to cones with apices at the mid point of the anchor, then average «rock strength» values mobilised over the surface area varied from 0.076 - 0,185 N/mm². These values may be compared with the design recommendations of 0.034 N/mm² by Saliman & Schaefer (1968), and 0.024 N/mm² by Hilf (1973). Based on the rock surface displacements the tests show considerable surface disturbance for maximum loads of 900 kN for slenderness ratios (distance from rock surface to the proximal end of the grouted fixed anchor divided by the borehole diameter) up to 8, but at a value of 13 for loads of 1360 kN no surface movement was observed. Above a value of 13, failure was localised, invariably at the grout-tendon interface, and it is considered that this type of observation is invaluable when assessing the relevance of stressing through a bearing plate of a simply supported beam.

Grout-tendon interface

Bearing in mind the high grout strengths measured prior to the stressing of each anchor (> 35 N/mm²) no correspondence between ultimate average grout-tendon bond values and grout strength was detected. The presence of surcharge grout (up to 9 m) did not markedly affect grout-tendon bond values (table 3) or the phenomenon of debonding. A grout surcharge in excess of 3 m did however lead to a steady and quieter pull-out of strands compared with the sudden explosive type of failure which may be observed without surcharge.

Fig. 2. — **Failure volume induced at anchor 4, showing the control of the dominant NW-SE joints.**

TABLE 3

Average bond values at maximum loads for a range of grout surcharges

Anchor	Surcharge (m)	Max. rock-grout (N/mm²)	Max. grout-steel (N/mm²)	Grout strength (N/mm²)
16	0.00	2.39	1.31 F — 1.78	58
17	0.00	1.86	1.01 F — 1.40 F	58
18	1.44	1.98	1.39 F — 1.48	44
19	1.88	1.95	1.46 F	46
22	1.70	2.00	1.29 F — 1.50 F	48
20	3.00	1.79	1.34 F	48
21	3.00	1.95	1.46 F	50
56	3.00	2.24	1.68	47
57	3.00	2.22	1.66	48
43	6.00	2.16	1.64 F	44
44	6.00	2.11	1.58 F	45
45	9.00	2.08	1.56 F	46
46	9.00	2.24	1.68	4[illegible]

It is also noteworthy that of the five anchors with less than 2 m surcharge, four had an initial failure followed by a higher maximum, or a maximum. followed by failure at a lower load on the subsequent cycle. Average ultimate bond values of 1.01 - 1.68 N/mm^2 were recorded compared with a design range of 0.25 - 1.35 N/mm^2 which are commonly observed in practice according to Littlejohn & Bruce (1975). In this respect it should be noted that for a single strand anchor PC I (1974) indicates a bond of about 3.1 N/mm^2, and the Australian Standard CA 35 (1973) suggests a working bond of up to 2.1 N/mm^2 in design for single or multi-strand tendons.

With regard to load resisting characteristics in relation to tendon configuration, individually noded strand tendons were more effective than generally noded tendons (table 4) but both showed distinct advantages over parallel, straight tendons. To effect general tendon noding the strands were bound intermediate to the spacers in the fixed length. Individual strand nodes were produced by unravelling each strand and introducing a small metal collar onto the straight central wire at the appropriate point: the peripheral wires were then returned to their original lay around it, with a proturbance thereby created at that point.

Strand spacing was also varied but no reduction in bond was observed down to a clear spacing of 5 mm. Thereafter, only when strands were actually in contact was any significant reduction in bond observed (table 5). Nevertheless, the use of centraliser/spacer units in the grouted fixed anchor zone is strongly advised, and spacings lower than 5 mm are only recommended where noding is employed to increase mechanical interlock.

TABLE 5

Average bond values at grout-tendon interface for different interstrand spacings (6 strand tendon)

Anchor No	Spacing between strands (mm)	Max. Test Load (kN)	Grout Crushing Strength (N/mm^2)	Max. Bond (N/mm^2)
35	10	1 535	60	1.79 F
36	10	1 555	62	1.81
41	5	1 555	42	1.81
42	5	1 555	44	1.81
37	0	1 351	38	1.57 F
38	0	1 455	40	1.69 F

TABLE 4

Average bond values at maximum test loads for different tendon configurations (10 strand tendon)

Anchor No	Tendon Configuration	Max. (kN)	Test Load (% fpu)	Grout Crushing Strength (N/mm^2)	(Max. Bond) Rock-grout	(N/mm^2) Grout-tendon	Remarks
20	straight, parallel strands	1 920	64	48	1.79	1.34 F	Grout-tendon failure
21		2 093	70	50	1.95	1.46 F	Grout-tendon failure
31	general noding of tendon	2 481	83	48	2.31	1.73	load held - large extensions
32		2 248	75	49	2.09	1.38 F	Initial yield at 1 978 kN
33	local noding of strands	2 411	80	50	2.24	1.68	Load held
34		2 411	80	48	2.24	1.68	Load held

In relation to the extent of debonding table 6 illustrates the basic characteristics for a strand tendon, where the steel represents 10.7% of the hole area, and

TABLE 6

Extent of effective debonding for 6 and 10 strand tendons, at various tendon stress levels

Anchor No	No of Strands	Effective debonded length (m) at tendon stress of (% fpu)					
		40%	50%	62.5%	75%	80%	Failure
20	10	1.12	1.52	2.04	—	—	2.48 (1 920 kN)
21	10	1.12	1.52	1.94	—	—	2.63 (2 093 kN)
35	6	0.81	0.95	1.10	1.29	1.52	1.75 (1 535 kN)
36	6	0.91	1.03	1.05	1.15	1.20	None

f.p.u. : characteristic strength of the tendon (0.1% proof stress = 83.5% fpu) a 10 strand tendon (17.8% hole area). The inference is clear for the less congested tendons, namely that the rate of effective debonding is slower and the failure load per strand is greater. Whilst it is appreciated that 10 strand anchors would normally have a fixed anchor greater than 3 m, the homing of a high density tendon (17-18% hole area) can give problems due to damage or contamination of the strands, and fixing of the centraliser/spacer units can be difficult and time consuming. Therefore whilst the former installation is feasible, it is recommended that the tendon density be limited to 15% of the hole area wherever possible.

Debonding is a little understood phenomenon and although the analysis of strain gauge and load/extension data are by no means complete the major conclusions to date are:

1) effective debonding occurs at low loads (15 kN/strand) and progresses distally with increasing load. The effective debonded length comprises wholly debonded, and partially debonded sections. The latter, pertaining to adhesive bond failure, may be determined from strain gauge records, and the extent of this adhesion zone appears to be proportional to the applied load.

2) For grout/tendon failure the limit of effective debonding was 0.5 to 1.0 m from the distal end. Under working conditions (50-60 % fpu), where fpu is the characteristic strength of the tendon, an effective debonded length of 1 to 2 m should be anticipated. The partially debonded zone extends some distance distally of the point of effective debonding, possibly about 0.8 m at 62.5% fpu, Based on this preliminary information it is clear that there should be no reduction in the current minimum fixed anchor length of 3 m, often recommended in practice.

In general, where localised failure of the complete tendon at the grout/tendon interface was observed subsequent restressing mobilised on average a total tendon load of about 85% of that recorded at first failure. When tested with a monojack individual strands commonly yielded pull-out resistances in excess of the initial average multijack value. «Failed» anchors may therefore have a useful role to play at a lower capacity for temporary works. In these circumstances it is strongly recommended that post failure cyclic loading tests be carried out in order to assess maximum safe working loads for anchors which might otherwise be discarded.

REFERENCES

HILF (J.W.). — Reply to Aberdeen Questionnaire (unpublished) Geotechnics Research Group, University of Aberdeen (1973).

LITTLEJOHN (G.S.) and BRUCE (D.A.). — «Rock Anchors - State of the Art», *Ground Engineering*, Vol. 8, Nos 3, 4, 5 & 6, Vol. 9, Nos 2, 3 & 4 (1975-1976).

PC I. — «Tentative Recommandations for Prestressed Rock and Soil Anchors», PC I Post-Tensioning Committee, Chicago, USA (1974).

SALIMAN (R.) and SCHAEFER (R.). — «Anchored Footings for Transmission Towers», Preprint No 753, ASCE Annual Meeting, Pittsburg, USA (1968).

Standards Association of Australia. — Prestressed Concrete Code CA 35, Section 5: Ground Anchorages (1973).

World-Wide Interest in Ground Anchors

The international delegates who attended the Soft Ground Tunnelling Session in Tokyo were impressed by reports on the versatility of the New Austrian Tunnelling Method (NATM). Particularly interesting were the accounts of successful tunnel installations in weakly cemented granular soils and in clay formations. Even though the short term stability of these materials was skillfully utilized, specific support measures were required almost immediately in order to minimise the tunnel contractor's risk.

The NATM experience has shown that unreliable and weak formations could be permanently strengthened through the installation of anchor systems. The latter are the most effective elements of modern tunnel lining installations which may otherwise consist of steel arches, wire mesh and layers of gunite concrete. All these various details of the NATM are conveniently adaptable to the prevailing circumstances.

Protection against the onset of instability is the first objective of support installations. The timing of the work is important. The final achievement is the provision of adequate strength against collapse by means of skillfully installed anchors. NATM experts believe that stabilization is the result of composite action of the support system with the surrounding formation. There is conclusive evidence that ultimately the whole anchor penetration zone contributes more to stability than the visible elements of the tunnel lining.

While the technological advancement reports of tunnelling experts provided a remarkable first day success in Tokyo, the major event on anchorages was the Speciality Session No. 4, organized by Prof. P. Habib of France. It was a formidable gathering of civil engineers concerned with anchored retaining walls and slope stabilization schemes, utilizing ground anchors and rock bolts. There were 17 conference papers on anchors by comparison with 7 on soft ground tunnelling, inclusive of the one paper on the NATM with specific references to anchor installations.

Only three contributors of Prof. Habib's session reported on model tests, whereas the authors of 14 papers had obtained their scientific data for the development of performance theories and empirical calculation methods by directly monitoring the actual anchor behaviour with sophisticated built-in instrumentation. The prevailing emphasis on in situ measurements and full scale tests was confirmed by Prof. Habib's choice of 6 conference papers for presentation and discussion in Tokyo, whereas the French expert's own written contribution, admittedly, was on model tests.

Relevant as supporting evidence was the publication (in volume 2 of the Tokyo Proceedings) of the 1972 research paper by Sills, Burland and Czechovski on the behaviour of the Neasden anchored diaphragm wall in stiff clay. The contribution of the scientists of the British Building Research Establishment, incidentally, provided the first monitored data on "sinking" caused by the downwards pull which four rows of inclined anchors had exerted on the wall. In 1974 similar settlement measurements were reported by Littlejohn and Macfarlane. The latter information from the Vauxhall site was perhaps even more remarkable than the first report, as the occurrence of the 9 m thick gravel layer on top of the London clay did not prevent "sinking".

The range of in situ observations varied considerably in the 14 papers mentioned before. The Italian researchers, Evangelista and Sapia for example, used data from two anchors, whereas the French expert, Y. Fenoux, based his findings on the observation of 2732 anchors. Most noticeable were the merits of academic endeavours. Within the large anchor research programme of Munich's University Institute which had yielded in 1974 the prize-winning paper of the London Conference, there was impressive scientific material made available for Tokyo. The German contributors, Ostermayer and Scheele, presented new research results which would have been unobtainable without the continuity of University backing. For brevity's sake only two of the interesting findings can be mentioned to stimulate similar research. (1) The distribution of skin friction along the fixed anchor length in non-cohesive soils was deduced from measured tensile forces. Referring to various tests of the same soil density, there was a limit value of skin friction observed which is almost identical for fixed anchors, irrespective of their different lengths. With increasing soil density this limit value increases expotentially.

In loose soils the limit value of skin frictions is more or less constant along the fixed anchor length. In dense soils it is obtained only in a short section of the length. This zone of maximum skin friction is shifting towards the anchor end with increasing load. A similar phenomenon was observed when a constant load was applied for an extended length of time.

(2) A chart was compiled showing the apparent relationship between the failure load and the number of blows in accordance with the Standard Penetration Test procedure, using a 50 kg hammer for soil testing. The researchers found that if the information on soil type and density was adequate, one could deduce from the table the load carrying capacity of the anchor.

The most convincing compromise between a university research report and a collation of reliable back-up information on ground engineering equipment performances was the Tokyo

Testing of ground anchors in Britain

conference paper by Littlejohn, Bruce and Deppner. Dr Stewart Littlejohn of the Geotechnics Research Group of Aberdeen University* provided academic guidance and industrial experience, D.A. Bruce the motivated Ph.D, research depth, and Bill Deppner the construction industry orientation and the technical resources of the firm Universal Anchorage Company Limited. The industrial academic liaison will yield a series of technical reports on UAC sponsored full scale anchor field tests. The Tokyo paper, for instance, describes the preliminary findings on the performance characteristics of 57 rock anchors. The field work, however, still continues in the dis-used Whitnell Quarry near Bolton, where UAC's Chief design engineer, A.D. Barley is in charge of a further programme of 27 full scale test anchors. Ultimately the experiments will comprise a total of 84 instrumented test anchors which otherwise embody the characteristic merits of UAC anchor installations. The total cost of this "value for money" research programme amounts to about £40,000 of which £10,000 was provided by a Science Research Council grant to the University body for planning, instrumentation and the analysis of tests. The all-in price works out to less than £500 per anchor – for science sake – plus the research enthusiasm and ingenuity provided by UAC's senior engineers in addition to their working day commitments. It is also noteworthy that Dr. Littlejohn and Bill Deppner had agreed that all the results of tests should be published without reservations so that the construction industry would benefit immediately from the research findings.

There are 8 principal research objectives to be pursued within the framework of this scientific test programme: (a) Load transfer mechanism. (b) Bond-stress distribution and debonding. (c) Comparison of straight shafted and under-reamed anchors. (d) Merits of multi and monostrand stressing. (e) Friction losses in tendon free length in various media. (f) The effects of interstrand or tendon "noding". (g) The influence of end plates on bar tendons. (h) Minimum slenderness ratio required to ensure localised shear failure within the rock mass.

The Tokyo paper by Littlejohn, Bruce and Deppner was chosen for inclusion in this journal without prejudice against the other conference contributions on anchors – solely because it seemed to be the most praxis-near example of co-operation between science and industry. It is respectfully acknowledged that the fully illustrated official version of this and other papers on anchors may be printed in due course by the Conference Organizers.

SITE GEOLOGY

The anchors were installed in Upper Carboniferous sediments of the Middle Grit Group of the upper part of the Millstone Grit Series. The sequence fixed downwards from gently dipping massive, coarse, gritty siliceous sandstones to finer grained flaggy and shaley sandstones. In addition, a total of eight soft, friable mudstone beds were exposed or inferred in the sequence. Different groups of anchors were installed from different stratigraphic levels due to the presence of various benches, but each intersected at least one argillaceous bed at the grouted fixed anchor level. The whole sequence was conspicously vertically jointed, the major orientations being north, east-north-east, (most prominent), and south-east. Joint spacing varied greatly, being up to 1 metre in the coarser sandstones. The major geotechnical properties are provided in Table 1.

ANCHOR CONSTRUCTION

All holes were drilled vertically, by rotary percussion, to provide a nominal diameter of 114mm. The tendons, which consisted of 10 no. 7-wire Dyform strands of 300 kN individual capacity were assembled in a straight, parallel formation with a 10mm clear spacing (Fig. 1) unless otherwise specified. In order to dissociate the free (elastic) length from the surrounding grout or ground, grease impregnated tape was wrapped arou nd each strand over the required length.

* Now Technical Director of Colcrete

	Range	Mean
Fracture Index	10 – 1	6
R.Q.D.	60 – 100	90
Unit weight (Mg/m^3)	2.45 – 2.60	2.50
Ultimate Pulse Velocity (Km/sec)	2.0 – 4.5	3.50
Diametral Point Load Strength (N/mm^2)	0.5 – 6.0	3.80
Elastic Modulus – material (N/mm^2)	(1.3–2.8)x10^{+4}	2.0x10^{+4}
Elastic Modulus – mass (N/mm^2)	(0.5–1.6)x10^{+4}	1.0x10^{+4}

Table 1. Summary of geotechnical properties

Prior to tendon homing each hole was water tested, and sealed with neat cement grout, if the water loss exceeded 3 litres/minute/atmosphere. On completion of sealing neat Rapid Hardening Portland Cement grout ($^{W}/c$ = 0.45) was tremied into the hole and the tendon slowly homed.

The grout, prepared in a conventional paddle mixer, gave bleed readings of 1.3 to 2.1% and stressing only took place once the grout had achieved a crushing strength of 28 N/mm^2. At least two anchors of each type were installed.

ANCHOR TESTING

A hydraulic stressing system was evolved to enable the anchors to be incrementally, and cyclically loaded to failure, or to a maximum of 260 kN per strand. The system comprised remote loading through a simply supported beam and accommodated both multistrand and monostrand stressing modes. Dial gauges yielded anchor extensions and rock surface displacements of 0.01 mm accuracy, and annular load cells gave a direct reading of the applied load to 1% accuracy. A second, independent, direct measure of stress distribution was provided by strain gauges attached at strategic positions on a large proportion of the strands installed. At least one anchor of each type was instrumented in this way.

DISCUSSION OF RESULTS

Rock Mass Performance

Table 2 illustrates the overall performance of the shallow fully bonded anchors, where all the major modes of failure are reproduced.

Up to embedment depths of 1.5m failure occurred mainly in the rock mass. For greater depths failure tended to be localised at one of the grout interfaces. For rock mass failure, the shape of the rock volume mobilised in each case was strongly controlled by the incipient rock mass structure. For example, the major radial fractures developed along the trends of the major joint directions, whilst the projected shape of the rock volume mobilised below the surface was strongly influenced by the laminar nature of the mass. Under similar conditions, underreamed anchors sustained higher loads than their straight shaft counterparts. Under-reaming

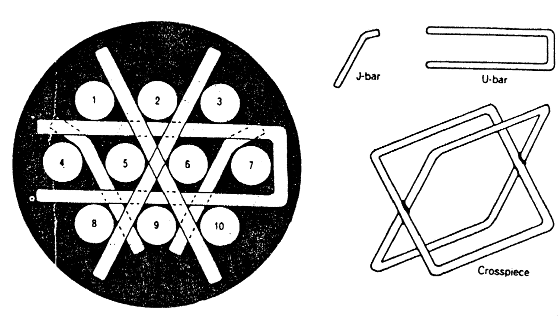


Fig. 1 arrangement for standard ten strand tendon

Anchor No.	Embedment (m)	Maximum Test Load (kN)	Rock-grout bond (N/mm^2)	Grout-tendon bond (N/mm^2)	Grout Crushing strength (N/mm^2)	Mode of Failure
1	0.75	440	1.64	1.23	45	Rock mass
2	0.75	500	1.86	1.40	45	Rock mass
3	0.75	450	1.68	1.26	46	Rock mass
4	1.50	1495	2.72	2.09	60	Rock mass
5	1.50	1355	2.52	1.89F	62	Grout-tendon
6	1.50	1206	2.24	1.68	50	Rock mass
23	1.50(u)	1834	3.41*	2.56	50	Rock mass
24	1.50(u)	1594	2.97*	2.23	51	Rock mass
51	2.25	2411	2.99	2.24	35	None-rock mass imminent
52	2.25	1978	2.45F	1.84	36	Rock-grout
53	2.25	1891	2.35F	1.76	37	Rock-grout
7	3.00	2353	2.19	1.64	49	Strand fracture
8	3.00	2469	2.30	1.72F	43	Grout-tendon
9	3.00	2122	1.97	1.48F	44	Grout-tendon

Table 2 Average interfacial bond values at maximum test loads for shallow fully bonded anchors

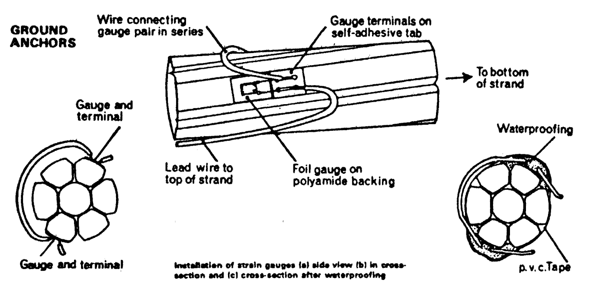
F – failure value at interface * calculated as a straight shaft

was carried out with a UAC patented tool to form pairs of "bells" of 230mm diameter and whilst a greater extent of rock was mobilised the pattern of surface fracturing was not radically different.

For the shallow anchors installed in unweathered rock in this project, the ultimate resistance to rock mass failure is reasonably estimated from the empirical rule P (kN) = 600 d^2, where d is the depth of embedment (m). For the traditional and conservative design concept pertaining to the weight of an inverted 90° cone, and using a unit weight of 2.5 Mg/m^3 for the rock, factors of safety ranging from 14 to 45 are indicated assuming the apex at the base of the anchor. Employing the observed extent of the fissuring to speculate on the size of cones mobilised, included angles of (117° – 144°) and (90° – 114°) can be calculated for the apex positioned at the mid point and base of the anchor, respectively. Assessing the weights of these cones the factors of safety against pull-out are (14 – 56) and (8 – 29) for the apex at mid point and base, respectively. These figures highlight that other "rock strength" parameters constitute the major component of resistance to pull-out, and assuming that the actual failure volumes were more akin to cones with apices at the mid point of the anchor, then average "rock strength" values mobilised over the surface area varied from 0.076 – 0.185 N/mm^2. These values may be compared with the design recommendations of 0.034 n/mm^2 by Saliman & Schaefer (1968), and 0.024 N/mm^2 by Hilf (1973). Based on the rock surface displacements the tests show considerable surface disturbance for maximum loads of 900 kN for slenderness ratios (distance from rock surface to the proximal end of the grouted fixed anchor divided by the borehole diameter) up to 8, but at a value of 13 for loads of 1360 kN no surface movement was observed. Above a value of 13, failure was localised, invariably at the grout-tendon interface, and it is considered that this type of observation is invaluable when assessing the relevance of stressing through a bearing plate of a simply supported beam.

Grout-Tendon Interface

Bearing in mind the high grout strengths measured prior to the stressing of each anchor (> 35 N/mm^2) no correspondence between ultimate average grout-tendon bond values and grout strength was detected. The presence of surcharge grout (up to 9m) did not markedly affect grout-tendon bond values (table 3) or the phenomenon of debonding. A grout surcharge in excess of 3m did however lead to a steady and quieter pull-out of strands compared with the sudden explosive type of failure which may be observed without surcharge.



Waterproofing of strain gauges

Anchor No.	Tendon Configuration	Max. Test Load (kN)	Max. Test Load (% fpu)	Grout Crushing Strength (N/mm^2)	(Max. Bond) (N/mm^2) Rock-grout	(Max. Bond) (N/mm^2) Grout-tendon	Remarks
20)	straight,	1920	64	40	1.79	1.34F	Grout-tendon failure
)	parallel						
21)	strands	2093	70	50	1.95	1.46F	Grout-tendon failure
31)	general	2481	83	48	2.31	1.73	load held – large extensions
)	noding of						
32)	tendon	2248	75	49	2.09	1.38F	Initial yield at 1978 kN
33)	local noding	2411	80	50	2.24	1.68	Load held
34)	of strands	2411	80	48	2.24	1.68	Load held

Table 4 Average bond values at maximum test loads for different tendon configurations (10 strand tendon)

Anchor	Surcharge (m)	Max. rock-grout (N/mm^2)	Max. grout-steel (N/mm^2)	Grout strength (N/mm^2)
16	0.00	2.39	1.31F–1.78	58
17	0.00	1.86	1.01F–1.40F	58
18	1.44	1.98	1.39F–1.48	44
19	1.88	1.95	1.46F	46
22	1.70	2.00	1.29F–1.50F	48
20	3.00	1.79	1.34F	48
21	3.00	1.95	1.46F	50
56	3.00	2.24	1.68	47
57	3.00	2.22	1.66	48
43	6.00	2.16	1.64F	44
44	6.00	2.11	1.58F	45
45	9.00	2.08	1.56F	46
46	9.00	2.24	1.68	47

Table 3 Average bond values at maximum loads for a range of grout surcharges

It is also noteworthy that of the five anchors with less than 2m surcharge four had an initial failure followed by a higher maximum, or a maximum followed by failure at a lower load on the subsequent cycle. Average ultimate bond values of 1.01 – 1.68 N/mm^2 were recorded compared with a design range of 0.25 – 1.35 N/mm^2 which are commonly observed in practice according to Littlejohn & Bruce (1975). In this respect it should be noted that for a single strand anchor PC I (1974) indicates a bond of about 3.1 N/mm^2, and the Australian Standard CA35 (1973) suggests a working bond of up to 2.1 N/mm^2 in design for single or multi-strand tendons.

With regard to load resisting characteristics in relation to tendon configuration, individually noded strand tendons were more effective than generally noded tendons (Table 4) but both showed distinct advantages over parallel, straight tendons. To effect general tendon noding the strands were bound intermediate to the spacers in the fixed length. Individual strand nodes were produced by unravelling each strand and introducing a small metal collar onto the straight central wire at the appropriate point: the peripheral wires were then returned to their original lay around it, with a proturbance thereby created at that point.

Strand spacing was also varied but no reduction in bond was observed down to a clear spacing of 5mm. Thereafter, only when strands were actually in contact was any significant reduction in bond observed (Table 5). Nevertheless, the use of centraliser/spacer units in the grouted fixed anchor zone is strongly advised, and spacings lower than 5mm are only recommended where noding is employed to increase mechanical interlock.

In relation to the extent of debonding Table 6 illustrates the basic characteristics for a 6 strand tendon, where the steel represents 10.7% of the hole area, and a 10 strand tendon (17.8% hole area). The inference is clear for the less congested tendons, namely that the rate of effective debonding is slower and the failure load per strand is greater. Whilst it is appreciated that 10 strand anchors would normally have a fixed anchor greater than 3m, the homing of a high density tendon (17–18% hole area) can give problems due to damage or contamination of the strands, and fixing of the centraliser/spacer units can be difficult and time consuming. Therefore whilst the former installation is feasible it is recommended that the tendon density be limited to 15% of the hole area wherever possible.

Anchor No.	Spacing between strands (mm)	Max. Test Load (kN)	Grout Crushing Strength (N/mm^2)	Max. Bond (N/mm^2)
35	10	1535	60	1.79F
36	10	1555	62	1.81
41	5	1555	42	1.81
42	5	1555	44	1.81
37	0	1351	38	1.57F
38	0	1455	40	1.69F

Table 5 Average bond values at grout-tendon interface for different interstrand spacings (6 strand tendon)

Debonding is a little understood phenomenon and although the analysis of strain gauge and load/extension data are by no means complete the major conclusions to date are:

1) effective debonding occurs at low loads (15 kN/strand) and progresses distally with increasing load. The effective debonded length comprises wholly debonded, and partially debonded sections. The latter, pertaining to adhesive bond failure, may be determined from strain gauge records, and the extent of this adhesion zone appears to be proportional to the applied load.
2) At failure the limit of effective debonding was 0.5 to 1.0m from the distal end. Under working conditions (50–60% fpu), where fpu is the characteristic strength of the tendon, an effective debonded length of 1 to 2m should be anticipated. The partially debonded zone extends some distance distally of the point of effective debonding, possibly about 0.8m at 62.5% fpu. Based on this preliminary information it is clear that there should be no reduction in the current minimum fixed anchor length of 3m, often recommended in practice.

In general, where localised failure of the complete tendon at the grout/tendon interface was observed subsequent restressing mobilised on average a total tendon load of about 85% of that recorded at first failure. When tested with a monojack individual strands commonly yielded pull-out resistances in excess of the initial average multijack value. "Failed" anchors may therefore have a useful role to play at a lower capacity for temporary works. In these circumstances it is strongly recommended that post failure cyclic loading tests be carried out in order to assess maximum safe working loads for anchors which might otherwise be discarded.

REFERENCES

Hilf, J.W. (1973)
Reply to Aberdeen Questionnaire (unpublished) Geotechnics Research Group, University of Aberdeen.

Littlejohn, G.S. and Bruce, D.A. (1975-76)
"Rock Anchors – State of the Art", Ground Engineering, Vol. 8, Nos. 3, 4, 5 & 6, Vol. 9, Nos. 2, 3 & 4.

PC I, (1974), "Tentative Recommendations for Pre-Stressed Rock and Soil Anchors", PC I Post-Tensioning Committee, Chicago, U.S.A.

Saliman, R. and Schaefer, R. (1968) "Anchored Footings for Transmission Towers", Preprint No. 753, ASCE Annual Meeting – Pittsburg, U.S.A.

Standards Association of Australia (1973)
Prestressed Concrete Code CA35 Section 5 : Ground Anchorages.

Anchor No.	No. of Strands	Effective debonded length (m) at tendon stress of (% fpu)					
		40%	50%	62.5%	75%	80%	Failure
20	10	1.12	1.52	2.04	–	–	2.48 (1920 kN)
21	10	1.12	1.52	1.94	–	–	2.63 (2093 kN)
35	6	0.81	0.95	1.10	1.29	1.52	1.75 (1535 kN)
36	6	0.91	1.03	1.05	1.15	1.20	None

f.p.u. – characteristic strength of the tendon (0.1% proof stress = 83.5% fpu)

Table 6 Extent of effective debonding for 6 and 10 strand tendons, at various tendon stress levels.

ANCHOR FIELD TESTS IN CARBONIFEROUS STRATA

Essais en place d'ancrage dans un sédiment carbonifère

by

G.S. LITTLEJOHN
Geotechnics Research Group, University of Aberdeen, Scotland

D.A. BRUCE
Geotechnics Research Group, University of Aberdeen, Scotland

W. DEPPNER
Universal Anchorage Co, Bolton, England

SOMMAIRE

La communication présente les résultats préliminaires des essais en vraie grandeur des cinquante-sept ancrages dans des sédiments carbonifères. Le site géologique est brièvement décrit, ainsi que la construction de l'ancrage et les méthodes d'essais adoptées. Avec des scellements de différentes longueurs (0.75 à 6 m) à des profondeurs différentes (0.75 à 12 m), divers modes de rupture ont été obtenus. Des valeurs d'adhérence de l'interface roche-coulis sont données ainsi que l'influence de la pression d'injection et des écarts entre les brins sur le transfert des charges et le comportement à la rupture.

SUMMARY

The paper discusses the preliminary findings of full scale tests on 57 anchors installed in carboniferous sediments. The site geology is briefly described together with the anchor construction and testing methods adopted. To investigate the influence of anchor geometry on failure mode, anchors ranging in overall depth from 0.75 to 12 metres were tested with grouted fixed anchor lengths of 0.75 to 6 metres. The observed effects of depth of embedment, grout surcharge, tendon configuration, interstrand spacing and tendon density on anchor performance are discussed in relation to current practice. Measurements of interfacial bond and load transmission are presented.

INTRODUCTION

A world-wide survey of prestressed rock anchor practice by Littlejohn and Bruce (1975-1976) has highlighted a dearth of information concerning the fundamental behaviour of rock anchors with particular reference to the mechanism of load transfer and modes of failure.

In order to study phenomena such as rock mass failure, localised bond failure, critical embedment and debonding, full scale pull-out tests have been carried out on 57 instrumented rock anchors. The purpose of this paper is to highlight the preliminary findings.

SITE GEOLOGY

The anchors were installed in Upper Carboniferous sediments of the Middle Grit Group of the upper part of the Millstone Grit Series. The sequence fined downwards from gently dipping massive, coarse, gritty siliceous sandstones to finer grained flaggy and shaley sandstones. In addition, a total of eight soft, friable mudstone beds were exposed or inferred in the sequence. Different groups of anchors were installed from different stratigraphic levels due to the presence of various benches, but each intersected at least one argillaceous bed at the grouted fixed anchor level. The whole sequence was conspicuously vertically jointed, the major orientations being north, east-north-east, (most prominent), and south-east. Joint spacing varied greatly, being up to 1 metre in the coarser sandstones. The major geotechnical properties are provided in Table 1.

TABLE 1
Summary of geotechnical properties

	Range	Mean
Fracture Index	10 — 1	6
RQD	60 — 100	90
Unit weight (Mg/m^3)	2.45 — 2.60	2.50
Ultimate Pulse Velocity (km/sec)	2.00 — 4.50	3.50
Diametral Point Load Strength (N/mm^2)	0.50 — 6.00	3.80
Elastic Modulus - material (N/mm^2)	$(1.3 — 2.8) \times 10^{+4}$	$2.0 \times 10^{+4}$
Elastic Modulus - mass (N/mm^2)	$(0.5 — 1.6) \times 10^{+4}$	$1.0 \times 10^{+4}$

ANCHOR CONSTRUCTION

All holes were drilled vertically, by rotary percussion, to provide a nominal diameter of 114 m. The tendons, which consisted of 10 No. 7-wire Dyform 15.2 mm diameter strands of 300 kN individual capacity were assembled in a straight, parallel formation with a 10 mm clear spacing (fig. 1) unless otherwise specified. In order to dissociate the free (elastic) length from the surrounding grout or ground, grease impregnated tape was wrapped around each strand over the required length.

Prior to tendon homing each hole was water tested, and sealed with neat cement grout, if the water loss exceeded 3 litres/minute/atmosphere. On completion of sealing, neat Rapid Hardening Portland Cement grout (w/c = 0.45) was tremied into the hole and the tendon slowly homed.

The grout, prepared in a conventional paddle mixer, gave bleed readings of 1.3 to 2.1% and stressing only took place once the grout had achieved a crushing strength of 28 N/mm². At least two anchors of each type were installed.

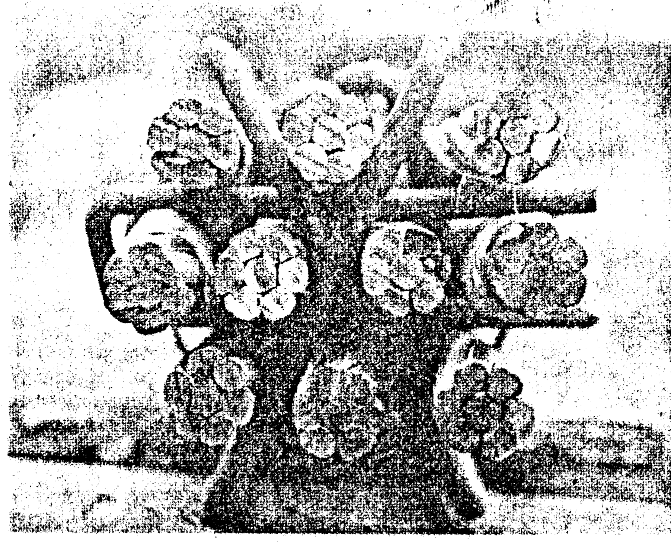


Fig. 1. — **Arrangement for standard ten strand tèndon.**

ANCHOR TESTING

A hydraulic stressing system was evolved to enable the anchors to be incrementally, and cyclically loaded to failure, or to a maximum of 260 kN per strand. The system comprised remote loading through a simply supported beam and accommodated both multistrand and monostrand stressing modes. Dial gauges yielded anchor extensions and rock surface displacements to 0.01 mm accuracy, and annular load cells gave a direct reading of the applied load to 1% accuracy. A second, independent, direct measure of stress distribution was provided by strain gauges attached at strategic positions on a large proportion of the strands installed. At least one anchor of each type was instrumented in this way.

DISCUSSION OF RESULTS

Rock mass performance

Table 2 illustrates the overall performance of the shallow fully bonded anchors, where all the major modes of failure are reproduced.

TABLE 2

Average interfacial bond values at maximum test loads for shallow fully bonded anchors

Anchor No	Embedment (m)	Maximum Test Load (kN)	Rock-grout bond (N/mm²)	Grout-tendon bond (N/mm²)	Grout Crushing strength (N/mm²)	Mode of failure
1	0.75	440	1.64	1.23	45	Rock mass
2	0.75	500	1.86	1.40	45	Rock mass
3	0.75	450	1.68	1.26	46	Rock mass
4	1.50	1 495	2.72	2.09	60	Rock mass
5	1.50	1 355	2.52	1.89 F	62	Grout-tendon
6	1.50	1 206	2.24	1.68	50	Rock mass
23	1.50 (u)	1 834	3.41 *	2.56	50	Rock mass
24	1.50 (u)	1 594	2.97 *	2.23	51	Rock mass
51	2.25	2 411	2.99	2.24	35	None-rock mass imminent
52	2.25	1 978	2.45 F	1.84	36	Rock-grout
53	2.25	1 891	2.35 F	1.76	37	Rock-grout
7	3.00	2 353	2.19	1.64	49	Strand fracture
8	3.00	2 469	2.30	1.72 F	43	Grout-tendon
9	3.00	2 122	1.97	1.48 F	44	Grout-tendon

(*) Calculated as a straight shaft. F - failure value at interface.

Up to embedment depths of 1.5 m, failure occurred mainly in the rock mass. For greater depths failure tended to be localised at one of the grout interfaces. For rock mass failure, the shape of the rock volume mobilised in each case was strongly controlled by the incipient rock mass structure (fig. 2).

For example, the major radial fractures developed along the trends of the major joint directions, whilst the projected shape of the rock volume mobilised below the surface was strongly influenced by the laminar nature of the mass. Under similar conditions, underreamed anchors sustained higher loads than their straight shaft counterparts. Underreaming was carried out with a UAC patented tool to form pairs of «bells» of 230 mm diameter and whilst a greater extent of rock was mobilised, the pattern of surface fracturing was not radically different.

For the shallow anchors installed in unweathered rock in this project, the ultimate resistance to rock mass failure is reasonably estimated from the empirical rule P (kN) = 600 d^2, where d is the depth of embedment (m). For the traditional and conservative design concept pertaining to the weight of an inverted 90° cone, and using a unit weight of 2.5 Mg/m^3 for the rock, factors of safety ranging from 14 to 45 are indicated, assuming the apex at the base of the anchor. Employing the observed extent of the fissuring to speculate on the size of cones mobilised, included angles of (117° - 144°) and (90° - 114°) can be calculated for the apex positioned at the mid point and base of the anchor, respectively. Assessing the weights of these cones the factors of safety against pull-out are (14 - 56) and (8 - 29) for the apex at mid point and base, respectively. These figures highlight that other «rock strength» parameters constitute the major component of resistance to pull-out, and assuming that the actual failure volumes were more akin to cones with apices at the mid point of the anchor, then average «rock strength» values mobilised over the surface area varied from 0.076 - 0,185 N/mm^2. These values may be compared with the design recommendations of 0.034 N/mm^2 by Saliman & Schaefer (1968), and 0.124 N/mm^2 by Hilf (1973). Based on the rock surface displacements the tests show considerable surface disturbance for maximum loads of 900 kN for slenderness ratios (distance from rock surface to the proximal end of the grouted fixed anchor divided by the borehole diameter) up to 8, but at a value of 13 for loads of 1360 kN no surface movement was observed. Above a value of 13, failure was localised, invariably at the grout-tendon interface, and it is considered that this type of observation is invaluable when assessing the relevance of stressing through a bearing plate of a simply supported beam.

Grout-tendon interface

Bearing in mind the high grout strengths measured prior to the stressing of each anchor (> 35 N/mm^2) no correspondence between ultimate average grout-tendon bond values and grout strength was detected. The presence of surcharge grout (up to 9 m) did not markedly affect grout-tendon bond values (table 3) or the phenomenon of debonding. A grout surcharge in excess of 3 m did however lead to a steady and quieter pull-out of strands compared with the sudden explosive type of failure which may be observed without surcharge.

Fig. 2. — Failure volume induced at anchor 4, showing the control of the dominant NW-SE joints.

TABLE 3

Average bond values at maximum loads for a range of grout surcharges

Anchor	Surcharge (m)	Max. rock-grout (N/mm^2)	Max. grout-steel (N/mm^2)	Grout strength (N/mm^2)
16	0.00	2.39	1.31 F — 1.78	58
17	0.00	1.86	1.01 F — 1.40 F	58
18	1.44	1.98	1.39 F — 1.48	44
19	1.88	1.95	1.46 F	46
22	1.70	2.00	1.29 F — 1.50 F	48
20	3.00	1.79	1.34 F	48
21	3.00	1.95	1.46 F	50
56	3.00	2.24	1.68	47
57	3.00	2.22	1.66	48
43	6.00	2.16	1.64 F	44
44	6.00	2.11	1.58 F	45
45	9.00	2.08	1.56 F	46
46	9.00	2.24	1.68	4[illegible]

It is also noteworthy that of the five anchors with less than 2 m surcharge, four had an initial failure followed by a higher maximum, or a maximum, followed by failure at a lower load on the subsequent cycle. Average ultimate bond values of 1.01 - 1.68 N/mm² were recorded compared with a design range of 0.25 - 1.35 N/mm² which are commonly observed in practice according to Littlejohn & Bruce (1975). In this respect it should be noted that for a single strand anchor PC I (1974) indicates a bond of about 3.1 N/mm², and the Australian Standard CA 35 (1973) suggests a working bond of up to 2.1 N/mm² in design for single or multi-strand tendons.

With regard to load resisting characteristics in relation to tendon configuration, individually noded strand tendons were more effective than generally noded tendons (table 4) but both showed distinct advantages over parallel, straight tendons. To effect general tendon noding the strands were bound intermediate to the spacers in the fixed length. Individual strand nodes were produced by unravelling each strand and introducing a small metal collar onto the straight central wire at the appropriate point: the peripheral wires were then returned to their original lay around it, with a proturbance thereby created at that point.

Strand spacing was also varied but no reduction in bond was observed down to a clear spacing of 5 mm. Thereafter, only when strands were actually in contact was any significant reduction in bond observed (table 5). Nevertheless, the use of centraliser/spacer units in the grouted fixed anchor zone is strongly advised, and spacings lower than 5 mm are only recommended where noding is employed to increase mechanical interlock.

TABLE 5

Average bond values at grout-tendon interface for different interstrand spacings (6 strand tendon)

Anchor No	Spacing between strands (mm)	Max. Test Load (kN)	Grout Crushing Strength (N/mm²)	Max. Bond (N/mm²)
35	10	1 535	60	1.79 F
36	10	1 555	62	1.81
41	5	1 555	42	1.81
42	5	1 555	44	1.81
37	0	1 351	38	1.57 F
38	0	1 455	40	1.69 F

TABLE 4

Average bond values at maximum test loads for different tendon configurations (10 strand tendon)

Anchor No	Tendon Configuration	Max. (kN)	Test Load (% fpu)	Grout Crushing Strength (N/mm²)	(Max. Bond) Rock-grout	(N/mm²) Grout-tendon	Remarks
20	straight, parallel strands	1 920	64	48	1.79	1.34 F	Grout-tendon failure
21		2 093	70	50	1.95	1.46 F	Grout-tendon failure
31	general noding of tendon	2 481	83	48	2.31	1.73	load held - large extensions
32		2 248	75	49	2.09	1.38 F	Initial yield at 1 978 kN
33	local noding of strands	2 411	80	50	2.24	1.68	Load held
34		2 411	80	48	2.24	1.68	Load held

In relation to the extent of debonding table 6 illustrates the basic characteristics for a strand tendon, where the steel represents 10.7% of the hole area, and

TABLE 6

Extent of effective debonding for 6 and 10 strand tendons, at various tendon stress levels

Anchor No	No of Strands	Effective debonded length (m) at tendon stress of (% fpu)					
		40%	50%	62.5%	75%	80%	Failure
20	10	1.12	1.52	2.04	—	—	2.48 (1 920 kN)
21	10	1.12	1.52	1.94	—	—	2.63 (2 093 kN)
35	6	0.81	0.95	1.10	1.29	1.52	1.75 (1 535 kN)
36	6	0.91	1.03	1.05	1.15	1.20	None

f.p.u. : characteristic strength of the tendon (0.1% proof stress = 83.5% fpu) a 10 strand tendon (17.8% hole area). The inference is clear for the less congested tendons, namely that the rate of effective debonding is slower and the failure load per strand is greater. Whilst it is appreciated that 10 strand anchors would normally have a fixed anchor greater than 3 m, the homing of a high density tendon (17-18% hole area) can give problems due to damage or contamination of the strands, and fixing of the centraliser/spacer units can be difficult and time consuming. Therefore whilst the former installation is feasible, it is recommended that the tendon density be limited to 15% of the hole area wherever possible.

Debonding is a little understood phenomenon and although the analysis of strain gauge and load/extension data are by no means complete the major conclusions to date are:

1) effective debonding occurs at low loads (15 kN/strand) and progresses distally with increasing load. The effective debonded length comprises wholly debonded, and partially debonded sections. The latter, pertaining to adhesive bond failure, may be determined from strain gauge records, and the extent of this adhesion zone appears to be proportional to the applied load.

2) For grout/tendon failure the limit of effective debonding was 0.5 to 1.0 m from the distal end. Under working conditions (50-60 % fpu), where fpu is the characteristic strength of the tendon, an effective debonded length of 1 to 2 m should be anticipated. The partially debonded zone extends some distance distally of the point of effective debonding, possibly about 0.8 m at 62.5% fpu. Based on this preliminary information it is clear that there should be no reduction in the current minimum fixed anchor length of 3 m, often recommended in practice.

In general, where localised failure of the complete tendon at the grout/tendon interface was observed subsequent restressing mobilised on average a total tendon load of about 85% of that recorded at first failure. When tested with a monojack individual strands commonly yielded pull-out resistances in excess of the initial average multijack value. «Failed» anchors may therefore have a useful role to play at a lower capacity for temporary works. In these circumstances it is strongly recommended that post failure cyclic loading tests be carried out in order to assess maximum safe working loads for anchors which might otherwise be discarded.

REFERENCES

HILF (J.W.). — Reply to Aberdeen Questionnaire (unpublished) Geotechnics Research Group, University of Aberdeen (1973).

LITTLEJOHN (G.S.) and BRUCE (D.A.). — «Rock Anchors - State of the Art», *Ground Engineering*, Vol. 8, Nos 3, 4, 5 & 6, Vol. 9, Nos 2, 3 & 4 (1975-1976).

PC I. — «Tentative Recommandations for Prestressed Rock and Soil Anchors», PC I Post-Tensioning Committee, Chicago, USA (1974).

SALIMAN (R.) and SCHAEFER (R.). — «Anchored Footings for Transmission Towers», Preprint No 753, ASCE Annual Meeting, Pittsburg, USA (1968).

Standards Association of Australia. — Prestressed Concrete Code CA 35, Section 5: Ground Anchorages (1973).