STUDIES RELATING TO GROUND ANCHORAGE SYSTEMS

G.S. LITTLEJOHN

D.Sc.

UNIVERSITY OF EDINBURGH

1993

EX BIBL. UNIV. EDINBURGEN.

Contents

1. Abstract

This thesis comprises 28 papers which illustrate the nature and direction of development work and associated research undertaken between 1965 and 1993 on soil and rock anchorage systems. The research was performed in order to obtain a basic understanding of the behaviour of newly developed anchorage systems in a variety of ground types and conditions, in order to improve anchorage designs, construction methods and testing procedures, and thereby encourage the safe and economic application of ground anchorages worldwide.

Field development of anchorage construction methods in gravels, sand, clays, marls and chalk using cement grout injection techniques is described together with equations evolved to estimate the ultimate resistance to withdrawal for each ground type, based on systematic testing of full scale anchorages.

A new design method for single and multi tied stiff retaining walls installed in any soil is detailed and validated by large scale tests and closely monitored case histories. The interactions between wall, anchorage and soil are illustrated, coupled with the refinement of overall stability analyses in cohesionless soils using wedge and log spiral based mechanisms of failure.

For the rapid installation of anchorages in granular soils, vibratory driving is investigated in the laboratory and two distinct types of motion are found to exist. Theoretical equations of motion are developed to define the penetration processes and facilitate the design of vibrodrivers and vibrohammers.

World practice in relation to the design, construction, testing and behaviour of rock anchorages is appraised, and field studies permit an improved understanding of uplift capacity by general shear failure, load transfer mechanisms, bond at rock/grout and grout/tendon interfaces, debonding, service performance and post-failure behaviour.

Acceptance criteria related to service behaviour are created for load relaxation and creep displacement with time, which are independent of ground type and potentially of short duration.

The extent and nature of steel tendon corrosion is described based on an international study of the corrosion performance of post-tensioned anchorages. Guidance is provided on class of protection, design principles and acceptable protective systems.

For rock tunnelling by drill and blast methods of excavation, a fundamental understanding of rock bolt behaviour under static and dynamic loading is provided. Field, laboratory and finite element studies are combined to investigate the character of blast induced wave forms within a rock mass and the effect of these signatures on the rock bolt system. Attenuation relationships for peak particle velocity and peak dynamic bolt load are presented together with effect of bolt prestress, bolt length, and both single and two speed resin systems. Observations confirm that resin bonded rock bolts have a remarkable resilience to close proximity blasting, and the data provide a new understanding of stress transfer in tensioned bolts under static and dynamic conditions.

A simple device to control rock bolt tensioning is developed and applied as a result of observed variations in prestress during production bolting.

Ground anchorage technology is reviewed to highlight areas where further investigation and study would enhance understanding of anchorage behaviour and improve standards of practice.

2. Declaration

I certify that the publications contained in this thesis have not been previously submitted for any higher degree. The work contained in the publications in which I am first author was carried out by me or under my immediate supervision. In the case of publications in which I am not first author I declare that I have made a substantial and significant contribution to the work as described within my list of publications.

G.S. Littlejohn

3. Publications

3.1 Papers and Books

1. LITTLEJOHN, G.S. (1961) "Dense Tar Surfacing"
Road Tar, 15 (3), 14-17.

* 2. HSU, T.C., LITTLEJOHN, G.S. & MARCHBANK, B.M. (1965) "Elongation in the Tension Test as a Measure of Ductility"
Proc. Amer. Soc. Test. Mat., 65, 874-98.
(Hsu wrote the paper and supervised the research of Littlejohn and Marchbank who provided the data.)

* 3. LITTLEJOHN, G.S. (1968) "Recent Developments in Ground Anchor Construction"
Ground Engineering, 1(3), 32-36 & 46.

* 4. HANNA, T.H. & LITTLEJOHN, G.S. (1969) "Retaining Wall Tie-Backs"
Consulting Engineer, May, 50-53, June, 49-52.
(Hanna and Littlejohn contributed equally to this paper.)

5. LITTLEJOHN, G.S. (1970) "Anchorages in Soils - Some Empirical Design Rules"
"Ground Anchors - Safety Factors"
"General Specification for Ground Anchors"
The Consulting Engineer (Special Supplement - May) 9-12 & 37-38.

* 6. LITTLEJOHN, G.S. (1970) "Soil Anchors"
Proc. Ground Engineering Symposium, 33-44 plus reply to discussion 115-120, Institution of Civil Engineers, London.

7. LITTLEJOHN, G.S. (1971) "Design and Construction of Sand Anchors"
Proc. Conference on Foundations on Interbedded Sands, 95-101, C.S.I.R.O., Perth, Western Australia.

* 8. LITTLEJOHN, G.S., JACK, B.J. & SLIWINSKI, Z.J. (1971) "Anchored Diaphragm Walls - Some Design and Construction Considerations"
Journal of the Institution of Highway Engineers, 18 (4), 15-29.
(Littlejohn co-ordinated this paper and contributed half the data.)

9. LITTLEJOHN, G.S. (1973) "Monitoring Foundation Movements in Relation to Adjacent Ground"
Ground Engineering, 7 (4), 17-22.

10. LITTLEJOHN, G.S. (1974) "Observations of Brick Walls subject to Mining Subsidence"
Proc. Int. Conf. on Settlement of Structures, 384 393, discussion 764-766 & 792-795. British Geotechnical Society, University of Cambridge, England.

*Paper included in thesis.

11. LITTLEJOHN, G.S., RODGER, A.A. & SEAGER, D.L. (1974) "Current Vibrodriving Studies in Ground Engineering" Proc. 2nd Int. Conf. on Exploitation of Vibration, 1-36, National Engineering Laboratory/Society of Underwater Technology, East Kilbride, Scotland.
(Littlejohn wrote the paper. Rodger and Seager provided technical data.)

12. LITTLEJOHN, G.S. (1974) "Rock Anchors - Some Design Considerations" Proc. of Special Session on Prestressed Concrete and Foundations and Ground Anchors, 67-74, 7th Int. Fédération Internationale de la Précontrainte (F.I.P.) Conference, New York, U.S.A.

*13. LITTLEJOHN, G.S. & MACFARLANE, I.M. (1974) "A Case History Study of Multi-Tied Diaphragm Walls" Proc. Conf. on Diaphragm Walls and Anchorages, 113-121, Institution of Civil Engineers, London.
(Littlejohn wrote the paper. Macfarlane provided technical data.)

*14. LITTLEJOHN, G.S. & TRUMAN-DAVIES, C. (1974) "Ground Anchors at Devonport Nuclear Complex" Ground Engineering, 7 (6), 19-24.
(Littlejohn wrote the paper. Truman-Davies supervised the field work.)

15. LITTLEJOHN, G.S. (1975) "Acceptable Water Flows for Rock Anchor Grouting" Ground Engineering, 8 (2), 46-48.

*16. LITTLEJOHN, G.S. & BRUCE, D.A. (1975) "Rock Anchors - State-of-the-Art Part 1: Design" Ground Engineering, 8 (3), 25-32, 8 (4), 41-48.
(Littlejohn wrote the paper and supervised Bruce's Ph.D. programme.)

17. LITTLEJOHN, G.S. & BRUCE, D.A. (1975) "Rock Anchors - Design and Quality Control" Proc. 16th Symposium on Rock Mechanics, 53-64, University of Minnesota, Minneapolis, U.S.A.
(Littlejohn wrote the paper and supervised Bruce's Ph.D. programme.)

*18. LITTLEJOHN, G.S. & BRUCE, D.A. (1975) "Rock Anchors - State-of-the-Art Part 2; Construction" Ground Engineering, 8 (5), 34-45, 8 (6), 36-45.
(Littlejohn wrote the paper and supervised Bruce's Ph.D. programme.)

*19. LITTLEJOHN, G.S. & BRUCE, D.A. (1976) "Rock Anchors - State-of-the-Art Part 3: Stressing and Testing" Ground Engineering, 9 (2), 20-29, 9 (3), 55-60, 9 (4), 33-44.
(Littlejohn wrote the paper and supervised Bruce's Ph.D. programme.)

*20. LITTLEJOHN, G.S., NORTON, P.J. & TURNER, M.J. (1977) "A Study of Rock Slope Reinforcement at Westfield Open Pit and the Effect of Blasting on Prestressed Anchors" Proc. Conference on Rock Engineering, University of Newcastle-upon-Tyne, England.
(Littlejohn wrote the paper. Norton and Turner provided technical data.)

21. LITTLEJOHN, G.S. (1977) "Ground Anchors: Installation Techniques and Testing Procedures" Seminar on Diaphragm Walls and Anchorages, Institution of Civil Engineers, London.

*22. LITTLEJOHN, G.S., BRUCE, D.A. & DEPPNER, W. (1977) "Anchor Field Tests in Carboniferous Strata" Revue Francaise de Geotechnique No. 3, January 1978, 82-86.
(Littlejohn wrote the paper. Bruce and Deppner provided field data.)

23. LITTLEJOHN, G.S. (1978) "Mix Design for Underbase Grouting of the Ninian Central Platform" Conference on Ash Technology and Marketing, Central Electricity Generating Board, Sunbury House, Newgate Street, London E.C.1.

24. LITTLEJOHN, G.S. (1978) "Grouting the Ninian Central Platform's Underbase" Ground Engineering, 11 (8), 17-22.

25. LITTLEJOHN, G.S. (1979) "Drilling Practice in the Field of Grouting" Ground Engineering, 12 (1), 39-43, 52-53

26. LITTLEJOHN, G.S. (1979) "Surface Stability in Areas Underlain by Old Coal Workings" Ground Engineering, 12 (2), 22-30.

27. LITTLEJOHN, G.S. (1979) "Consolidation of Old Coal Workings" Ground Engineering, 12 (4), 15-21.

*28. LITTLEJOHN, G.S. & BRUCE, D.A. (1979) "Long Term Performance of High Capacity Rock Anchors at Devonport" Ground Engineering, 12 (7), 25-33.
(Littlejohn and Bruce jointly wrote the paper. Bruce provided the field data under the supervision of Littlejohn.)

*29. LITTLEJOHN, G.S. (1979) "Ground Anchors: State-of-the-Art" Amer. Soc. Civ. Engrs. National Convention, Boston, U.S.A. (April 1979). See also Concrete Beton No. 17, 5-19 (April 1980).

*30. LITTLEJOHN, G.S. (1979) "Design Estimation of the Ultimate Load Holding Capacity of Ground Anchors" Ground Engineering, 13 (8), 25-39.

31. LITTLEJOHN, G.S. (1980) "Wet Process Shotcrete" Proc. Symposium on Sprayed Concrete, Concrete International C180, 18-35, The Concrete Society, London.

*32. RODGER, A.A. & LITTLEJOHN, G.S. (1980) "A Study of Vibratory Driving in Granular Soils" Geotechnique, 30 (3), 269-93.
(Rodger and Littlejohn jointly wrote the paper. Littlejohn supervised Rodger's Ph.D. programme)

33. LITTLEJOHN G.S., CONNOR, R.M. & LEUENBERGER, H. (1981) "Support and Protection of Pipelines Offshore" Pipes and Pipelines International, 26 (2), 21-24 & 34.
(Littlejohn wrote the paper. Connor and Leuenberger provided technical data.)

*34. LITTLEJOHN, G.S. (1981) "Acceptance Criteria for the Service Behaviour of Ground Anchorages" Ground Engineering, 14 (3), 26-29 & 36.

35. LITTLEJOHN, G.S. (1981) "Grouting Platforms and Pipelines Offshore"
Proc. Int. Conf. on Geotechnical Aspects of Offshore and Nearshore Structures, Asian Institute of Technology, Bangkok.

*36. LITTLEJOHN, G.S. (1982) "Design of Cement Based Grouts"
Proc. Amer. Soc. Civ. Engrs. Conf. on Grouting in Geotechnical Engineering, New Orleans, 35-48.

37. LITTLEJOHN, G.S. (1982) "The Practical Use of Prestressed Ground Anchorages"
Proc. 9th Fédération Internationale de la Précontrainte (F.I.P.) Congress, Stockholm (6th to 10th June).

38. LITTLEJOHN, G.S., INGLE, J. & DADASBILGE, K. (1983) "Improvement in Base Resistance of Large Diameter Piles Founded in Silty Sand"
Proc. 8th European Conf. on Soil Mechanics and Foundation Engineering, Helsinki, 1, 153-156.
(Littlejohn wrote the paper. Ingle and Dadasbilge provided field data under Littlejohn's supervision.)

39. LITTLEJOHN, G.S. (1983) "Chemical Grouting"
Proc. South African Institution of Civil Engineers Grouting Course, University of Witwatersrand, Johannesburg (4th to 6th July).

40. LITTLEJOHN, G.S. (1983) "Plant and Equipment for Cement Based Grouts"
Proc. South African Institution of Civil Engineers Grouting Course, University of Witwatersrand, Johannesburg (4th to 6th July).

41. LITTLEJOHN, G.S. & CRAWLEY, J.D. (1983) "Preplaced Aggregate Concrete at Ko Ri Nuclear Power Plant, South Korea"
Concrete, 17 (19), 17-20.
(Littlejohn wrote the paper. Crawley provided field data under Littlejohn's supervision.)

42. LITTLEJOHN, G.S. & HEAD, J.M. (1984) "Specification for the Consolidation of Old Shallow Mine Workings"
Proc. Int. Conf. on Construction in Areas of Abandoned Mineworkings, 131-140, Engineering Technics Press, Edinburgh.
(Littlejohn and Head jointly wrote the paper and provided equal amounts of technical data.)

43. LITTLEJOHN, G.S. (1984) "Anchor Standards"
Proc. Conf. on Anchors and Diaphragm Walls, 74-82, May,
Kuala Lumpur, Malaysia.

44. LITTLEJOHN, G.S. (1984) "Grouted Preplaced Aggregate Concrete"
Proc. Conf. on Concrete in the Ground, 1-13, The Concrete Society, London.

45. LITTLEJOHN, G.S. (1985) "Chemical Grouting"
Ground Engineering, 18 (2), 13-16, (3), 23-28, (4), 29-34.

46. LARDNER, W.E. & LITTLEJOHN, G.S. (1985) "Suggested Method for Rock Anchorage Testing" International Society for Rock Mechanics, Commission on Testing Methods, Int. J. Rock Mech. Min. Sci. & Geomech. Abst. 22 (2), 71-83.

47. LITTLEJOHN, G.S. (1985) "Underpinning by Chemical Grouting" Chapter 8 of Underpinning, Editors: S. Thorburn and J.F. Hutchinson, Blackie & Son Ltd., Glasgow.

48. LITTLEJOHN, G.S. (1986) "Structures Liable to the Effects of Mining Subsidence" Chapter 9 of The Design and Construction of Engineering Foundations - 2nd Edition, Editor: F.D.C. Hendry, Chapman & Hall Ltd., London.

49. LITTLEJOHN, G.S. & STADLER, G. (1986) "Injektionsmischungen mit Zement" Felsbau, Verlag Gluckauf Jahrgang 4, S.188-190.
(Littlejohn wrote the paper. Stadler provided technical support.)

*50. LITTLEJOHN, G.S. (1987) "Ground Anchorages : Corrosion Performance" Proc. Instn. Civ. Engrs., Part 1, 82, 645-662.

*51. LITTLEJOHN, G.S., RODGER, A.A., MOTHERSILLE, D.K.V. & HOLLAND, D.C. (1987) "Monitoring the Influence of Blasting on the Performance of Rock Bolts at Penmaenbach Tunnel" Proc. Int. Conf. on Foundations and Tunnels, University of London, 1, 99-106.
(Littlejohn and Rodger jointly wrote the paper and supervised Mothersille's and Holland's research work.)

*52. LITTLEJOHN, G.S. (1988) "Sprayed Concrete for Underground Support" Proc. 3rd Int. Conf. on Underground Space and Earth Sheltered Buildings, Shanghai, China.

*53. LITTLEJOHN, G.S. (1988) "Rock Anchorages for Underground Support" Proc. 3rd Int. Conf. on Underground Space and Earth Sheltered Buildings, Shanghai, China.

54. LITTLEJOHN, G.S. & HUGHES, D.C. (1988) "Thermal Behaviour of Grouted Supports for Pipelines" Department of Energy - Offshore Technology Report `Grouts and Grouting for Construction and Repair of Offshore Structures', OTH 88289, 111-120.
(Littlejohn and Hughes jointly wrote the paper. Hughes provided the laboratory data under Littlejohn's supervision.)

55. HAIMONI, A., LITTLEJOHN, G.S. & WATERHOUSE, A. (1988) "Long Range Pumping of Grouts" Department of Energy - Offshore Technology Report `Grouts and Grouting for Construction and Repair of Offshore Structures', OTH 88289, 177-188.
(Haimoni wrote the paper and provided field data under the supervison of Littlejohn and Waterhouse.)

*56. RODGER, A.A., LITTLEJOHN, G.S., HOLLAND, D.C. & MOTHERSILLE, D.K.V. (1989) "Instrumentation Used to Monitor the Influence of Blasting on the Performance of Rock Bolts at Penmaenbach Tunnel" Proc. Conf. on Instrumentation in Geotechnical Engineering, University of Nottingham, 267-279.
(Rodger and Littlejohn jointly wrote the paper and supervised Mothersille's and Holland's research work.)

57. LITTLEJOHN, G.S., NEWMAN, R.L. & KETTLE, C.T. (1989) "Grouting to Control Ground Water During Basement Construction at St. Helier" Ground Engineering, 22 (1), 22-31.
(Littlejohn and Newman wrote the paper. Kettle provided field data under the supervision of Littlejohn.)

*58. LITTLEJOHN, G.S., RODGER, A.A., MOTHERSILLE, D.K.V. & HOLLAND, D.C. (1989) "Dynamic Response of Rock Bolt Systems" Proc. Int. Conf. on Foundations and Tunnels, University of London (Sept.), 2, 57-64.
(Littlejohn and Rodger wrote the paper and supervised Mothersille's and Holland's research work.)

59. LITTLEJOHN, G.S. & WATERHOUSE, A. (1990) "Field Data on Long Range Pumping of Cement Based Grouts" Proc. Instn. Civil Engrs., Part I, 88, 465-470.
(Littlejohn and Waterhouse jointly wrote the paper. Waterhouse provided field data under the supervision of Littlejohn.)

60. LITTLEJOHN, G.S. (1990) "Ground Anchorage Practice" Proc. Speciality ASCE Conference on Earth Retaining Structures, American Society of Civil Engineers, Cornell University, U.S.A., 1-40.

*61. LITTLEJOHN, G.S. (1990) "Corrosion Protection of Steel Tendons for Ground Anchorages" Ground Engineering, 23 (9), 33-40.

*62. LITTLEJOHN, G.S. (1991) "Routine On-Site Acceptance Tests for Ground Anchorages" Ground Engineering, 24 (2), 37-43.

63. LITTLEJOHN, G.S. (1991) "Inadequate Site Investigation" Ground Engineering, 24 (6), 28-31.

*64. LITTLEJOHN, G.S. (1992) "Ground Anchorage Technology - A Forward Look" Proc. Amer. Soc. Civ. Engrs. Conf. on Grouting, Soil Improvement & Geosynthetics, New Orleans, 39-62.

65. LITTLEJOHN, G.S. (1992) "Rock Anchorage Practice in Civil Engineering" Proc. Int. Symp. on Rock Support, Laurentian University, Sudbury, Ontario, May, 257-268.

66. LITTLEJOHN, G.S. (1992) "Advancing Anchorage Technology" Civil Engineering, J. Amer. Soc. Civ. Engrs., 62 (7) 61-64.

67. LITTLEJOHN, G.S. & HAJI-BAKAR, I. (1992) "Engineering Properties and Behaviour of Silicate Grouted Sand", Proc. Conf. Grouting in the Ground, Institution of Civil Engineers, London.
(Littlejohn wrote the paper and supervised Haji-Bakar's Ph.D. programme.)

68. LITTLEJOHN, G.S. & MOLLAMAHMUTOGLU, M. (1992) "Time-Dependent Behaviour of Silicate Grouted Sand" Proc. Conf. Grouting in the Ground, Institution of Civil Engineers, London.
(Littlejohn and Mollamahmutoglu jointly wrote the paper. Littlejohn supervised Mollamahmutoglu's Ph.D. programme.)

69. LITTLEJOHN, G.S. (1993) "Chemical Grouting" Chapter 5 of Ground Improvement, 1st Edition, Editor M. Moseley, Blackie Academic & Professional, Glasgow.

70. LITTLEJOHN, G.S. (1993) "Soil Anchorages" Chapter 10 of Underpinning and Retention, 2nd Edition, Editors S. Thorburn and G.S. Littlejohn. Blackie Academic & Professional, Glasgow.

71. LITTLEJOHN, G.S. (1993) "Underpinning by Chemical Grouting" Chapter 8 of Underpinning and Retention, 2nd Edition, Editors S. Thorburn and G.S. Littlejohn. Blackie Academic & Professional, Glasgow.

72. THORBURN, S. & LITTLEJOHN, G.S. (editors) (1993) "Underpinning and Retention" 2nd edition, Blackie Academic & Professional, Glasgow.
(Thorburn and Littlejohn shared the editorial function.)

73. LITTLEJOHN, G.S. (1993) "Overview of Rock Anchorages" Chapter 15 of Comprehensive Rock Engineering, Editors J.A. Hudson, E.T. Brown, C. Fairhurst & E. Hoek, Pergamon Press, Oxford.

*74. RODGER, A.A., LITTLEJOHN, G.S., HOLLAND, D.C. & XU, H. (1993) "Dynamic Response of Rock Bolts at Pen y Clip Tunnel in North Wales" Proc. Int. Cong. on Options for Tunnelling, Int. Tunnelling Assoc., Amsterdam.
(Rodger and Littlejohn jointly wrote the paper and supervised Holland's and Xu's research.)

*75. LITTLEJOHN, G.S. & CONWAY, J. (1993) "A Simple Device to Control Rock Bolt Tensioning" Tunnels & Tunnelling. (in press).
(Littlejohn and Conway jointly wrote the paper. Littlejohn planned and supervised the experimental work.)

76. LITTLEJOHN, G.S. (1993) "Consolidation Grouting" Report on Session 2, Proc.Conf. Grouting in the Ground, Institution of Civil Engineers, London. (in press)

3.2 Edited Reports

77. LITTLEJOHN, G.S. & MACLEOD, I.M. (editors) (1974) "Report on Structure-Soil Interaction in Relation to Buildings" Special Study Group, Institution of Structural Engineers, London.
(Littlejohn and MacLeod jointly wrote the report. Regional study groups of I.Struct.E. provided technical support.)

78. LITTLEJOHN, G.S. (chairman & editor) (1982) "Recommendations for Ground Anchorages" Draft for Development (DD81), British Standards Institution, London.

79. LITTLEJOHN, G.S. (chairman & editor) (1986) "Corrosion and Corrosion Protection of Prestressed Anchorages" FIP State-of-the-Art Report, Thomas Telford Ltd., London.

80. LITTLEJOHN, G.S. (chairman & editor) (1988) "Code of Practice for Ground Anchorages" BS.8081, British Standards Institution, London.

81. LITTLEJOHN, G.S. (chairman & editor) (1990) "Review of the SERC Civil Engineering Research Programme 1983-1988" Science and Engineering Research Council, Swindon, England.

82. LITTLEJOHN, G.S. (chairman & editor) (1991) "Inadequate Site Investigation" Ground Board Report, Institution of Civil Engineers. Thomas Telford Ltd., London.

83. DOBSON, R.S. & LITTLEJOHN, G.S. (1992) "The Institution of Civil Engineers Corporate Strategy : A vision into the next century". Special Publication, Thomas Telford Ltd., London.
(Dobson and Littlejohn jointly produced the publication. Littlejohn was Chairman of the ICE Working Group on Corporate Strategy.)

84. LITTLEJOHN, G.S. (chairman & editor) (1993) "Recommendations for the Design and Construction of Prestressed Ground Anchorages" FIP Commission on Practical Construction, Thomas Telford Ltd., London (in press).

85. LITTLEJOHN, G.S. (chairman & editor) (1993) "Site Investigation in Construction"
Part 1 : Without site investigation ground is a hazard
Part 2 : Planning, procurement and quality management of site investigation
Part 3 : Specification for ground investigation
Part 4 : Guidelines for the safe investigation of landfills and contaminated land by drilling.
Thomas Telford Ltd., London (in press).

4. Introduction

Over the past 25 years the author has researched a variety of subjects within the field of geotechnical engineering, with particular reference to ground anchorages, grouting for ground improvement and foundations, and structure-soil interaction in mining areas.

The selected papers in this thesis are concerned with ground anchorage systems ranging from low capacity rock bolts in ground reinforcement for tunnels, to medium to high capacity anchorages in soils and rocks for the stabilisation of civil engineering structures.

The exception is the first paper[2] on the ductility of metals where the research developed from a final year project tackled by the author as an undergraduate at the Department of Engineering, University of Edinburgh in 1961. This project stimulated the author towards a career in research and the final year dissertation led the external examiner, the late Professor Fisher Cassie, to invite the author to study for a Ph.D. at King's College, University of Durham in 1962.

Aside from the historical note, illustrating the value of individual final year projects, the purpose of this introduction is to highlight in chronological order the nature and direction of the author's research and development in ground anchorages over the period 1965 to 1993.

While in industry with the Cementation Company (1965-71) the author was responsible for the development of a range of ground anchorage construction systems based primarily on cement grout injection techniques[3]. As a result of these field studies in a variety of ground conditions, for example gravels, sands, clays and chalk, safe resistances to withdrawal were established for each ground type to enable the systems to be exploited commercially for the first time in the UK. Notable commercial "firsts" included anchorages in Thames Ballast (1967), London Clay (1968 - gravel placement; 1969 - multi underream), Keuper marl (1969 - straight shaft and multi-underream) and chalk (1969 - straight shaft).

During this early period of commercial application the use of post-tensioned anchorages as an earth support method for excavations was confirmed to be more cost-effective than the traditional application of internal bracing, but the new tie-back system involved more complex interactions between retaining wall, ground and individual anchorages. As a consequence, these interactions were studied and guidelines were published[4] on the important design principles and practical considerations necessary for the safe application of retaining wall tie-backs.

To facilitate soil anchorage design by civil and structural engineers, the author organised and supervised a programme of full scale anchorage trials on a large number of sites across the UK over a four year period. For given ground conditions and construction methods the tensile load-displacement behaviour of anchorages was analysed up to and including failure. From these studies a basic understanding of load transfer and failure mechanisms was evolved leading in turn to the publication[8] of empirical equations for the estimation of the ultimate resistance to withdrawal of anchorages installed in coarse sands and gravels (permeability > 100 μm/s), fine to medium sized sands (permeability = 100 to 1 μm/s), stiff clay (Cu > 90 kN.m^2), stiff to hard chalk (weathering grades I, II and III) and Keuper marl (weathering zones I and II). Of particular interest was a parallel development of construction methods to enhance the pull-out capacity of anchorages in stiff clay where the ultimate load of 150 kN for a straight shafted system was increased to 750 kN and 2400 kN for gravel placement and multi-underream systems, respectively. The latter system still provides the highest individual anchorage capacity in stiff clay to this day. In the anchorage design study, the work of Russian researchers Berezantzev (1961)* and Trofimenkov (1965)+ on slender piles proved invaluable in helping the author to create appropriate bearing capacity factors for sand and gravel anchorages, where the factors for uplift conditions were related to both angle of internal friction and slenderness ratio.

The programme of full scale tests also led to recommendations for site investigation to facilitate design and choice of anchorage construction method, load safety factors for the various components of a ground anchorage system, routine post-tensioning procedures to assess short term anchorage performance and an overload allowance to accommodate observed prestress losses during service. This publication remains a basic reference in current national and international codes of practice on ground anchorages.

*Berezantzev, V.G. et al (1961), Load Bearing Capacity and Deformation of Piled Foundations, Proc. 5th Int. Conf. Soil Mech., 2, 11-15.
+Trofimenkov, J.G. & Mariupolskii, L.G. (1965), Screw Piles used for Mast and Tower Foundations, Proc. 6th Int. Conf. Soil Mech., 2, 328-332.

For permanent anchorages and temporary anchorages formed in an aggressive environment, the author created a corrosion protection system[8,9], based on corrugated plastics sheathing and epoxy resin for the protection of prestressing steel tendon. The component design, choice of materials and fabrication details were based on the results of full scale testing in gun barrel apparatus, including long term creep observations[9]. The protective system was first used in 1968 for anchorages installed to resist hydrostatic uplift at Kilburn, London and these protected anchorages remain in operation today. Key elements of the protection system were adopted for 1000 anchorages associated with the ground works of the World Trade Centre in New York in 1969.

To exploit the potential of multi-tied continuous walls for deep excavations, particularly in urban areas, a novel repetitive single tied wall design method was evolved with BJ Jack of Cementation[10]. Prior to its application, the validity of the basic assumptions used in the method, e.g. triangular earth pressures for stiff walls, was confirmed by large scale physical testing in the laboratory, and economic viability was checked by comparisons with conventional strutted designs and experimental results of US, UK and Danish researchers, notably Terzaghi, Tschebotarioff, Rowe and Brinch Hansen. Thereafter, the results of monitored case histories[15] were published by the author to illustrate via prestress fluctuations, retaining wall displacements and bending moment profiles, that the unique design method which takes account of the continuous wall construction and excavation stages was amenable to various soil strata and provided safe and economic solutions. This empirical design method has been incorporated into a number of design software packages and is now used routinely for the design of anchored diaphragm and contiguous bored pile walls.

For the general problem of the overall stability of single line tied walls of any type in cohesionless soils, the author (working in conjunction with H. Locher of Losinger Ltd., Switzerland) evolved a simplified procedure of analysing the block mechanism of failure advanced by Professor Kranz (Germany) for waterfront structures in 1953 and further refined for anchorages in 1966 by Professors Jelinek and Ostermeyer (University of Munich). The simplified method[9,10] reduced the number of external forces acting on the block by incorporating the anchorage prestress as an internal force controlling the geometry of the block at failure. The overall analysis was also improved and made more realistic by the adoption of Brinch Hansen's concept of safety i.e. $F = \tan \varphi / \tan \varphi_n$.

For systems with several rows of anchorages in cohesionless soils, a logarithmic spiral shaped sliding surface was proposed by Locher and the author in 1970; the method was again simple because the anchorage forces were eliminated in the moment analysis.

Following a series of large scale laboratory tests at the University of Sheffield where anchored walls were overloaded to induce failure of the wall/ground/anchorage system, the researchers Anderson, Hanna and Abdel-Malek reviewed current methods of analysis and concluded in 1983 that the spiral method of analysis gave the best prediction of observed behaviour. The two simplified methods of assessing overall stability of anchored walls have been widely accepted in practice and are included in the current British Standard BS.8081 Ground Anchorages.

While at the University of Aberdeen (1971-76) the author focused more on rock anchorage technology to complement earlier research on soil anchorages. At this time there was a surprising dearth of data concerning the design, construction, stressing and testing of rock anchorages which had been used by the mining industry in the form of low capacity rock bolts for roof control since 1918, and by the civil engineering industry in the form of high capacity rock anchorages for dam raising since 1934. In essence, practice worldwide had developed in an ad hoc fashion, often on a regional basis, and there was an absence of field investigations on anchorage behaviour and well documented rock anchorage projects. As a consequence, within the construction industry where the success of soil anchorages was providing new technical challenges and markets for anchorages founded in rock, there were no comparable data on important issues such as design principles, load transfer mechanisms, failure conditions, construction tolerances, test criteria and service behaviour for rock anchorages.

Following a three year study of developments and practices in over twenty countries, the author published a world state-of-the-art on rock anchorages[18,20,21]. Aside from the presentation of a detailed analysis of this work, guidelines on good practice were proposed for all aspects of rock anchorage technology and topics worthy of further study and research were highlighted. Subsequent research at Aberdeen was based on these findings.

In regard to detailed field investigations, access to an abandoned quarry in Lancashire permitted study of rock mass failure, localised bond failure, critical depths of embedment, tendon debonding and post-failure performance using instrumented full scale vertical anchorages of different geometries installed in Upper Carboniferous sediments of the Middle Grit Group of the upper part of the Millstone Grit Series.

Conclusions and data from this work have been disseminated primarily through DD81 : 1982[78], BS.8081 : 1989[80] and international recommendations of the Fédération

Internationale de la Précontrainte (first published in 1982 and updated in 1993). In the summary paper[24] key findings included:

(i) uplift capacities for shallow anchorages in unweathered sandstone (slenderness ratio ≤8) were related to general failure of inverted cones within the rock mass, the shape being strongly controlled by the incipient rock mass structure,

(ii) above a critical slenderness ratio failure was localised permitting design recommendations on limiting depths of embedment,

(iii) bond values at the grout/tendon interface for straight parallel strands, locally noded strands and generally noded tendons, respectively, permitting design recommendations,

(iv) the influence of strand spacing on grout/strand bond leading to a minimum recommended clear spacing for straight parallel strand tendons,

(v) bond values at the rock/grout interface for a range of grout surcharges, showing that grout surcharge had no significant effect on bond capacity or debonding, but above a critical limit the surcharge inhibited explosive type failures,

(vi) the influence of tendon density (area of tendon/area of hole) on debonding leading to an upper limit in practice for multi-unit tendons,

(vii) a basic understanding of the load transfer mechanism by analysis of tendon load displacement behaviour, leading to recommendations on acceptance criteria for apparent debonding,

(viii) a high post failure capacity, as a proportion of the initial failure load recorded at the grout/tendon interface, permitting guidelines on post failure remedial measures and acceptance criteria.

The behaviour of vertical rock anchorages was also studied using the finite element method[Δ] to provide a predictive capacity, verified by field results, and to facilitate parametric and sensitivity analyses (Drs. Yap and Rodger were Ph.D. students of the author).

In regard to documented rock anchorage projects, the author created an important case history[16,30] by publishing his detailed design and construction procedures for 475 No. 2000 kN anchorages installed in hard slate as part of the construction of two dry docks for the Polaris submarine complex at Devonport. Short term load-displacement data were presented together with monitored service behaviour in the form of prestress fluctuations over a period up to 33,000 hours. Monitoring of the service behaviour of anchorages at this site indicated two distinct phases in terms of rate of prestress loss. Phase I was reflected by a stabilising but fairly rapid loss with time, occurring within a period of 3000 hours. Thereafter, a slower and more uniform rate of prestress loss was observed. Based on these results a minimum duration of monitoring was recommended for rock anchorages to cover Phase I and provide sufficient results to indicate a clear trend for Phase II and thereby permit an extrapolation of the results for the prediction of the service behaviour of the anchorages. The overall study also confirmed that the anchorages were performing satisfactorily. This proven performance has played a crucial role in convincing the Ministry of Defence and the Navy that post-tensioned anchorages can be used to strengthen existing docks at Devonport to withstand both overturning forces and earthquake loadings in the construction of the Trident Complex, recently approved by Parliament. In this regard, the proven resilience of high capacity anchorages to dynamic loading, when subjected to close proximity blasting, provided a further valuable and unique case history[22].

The subject of soil anchorages was not ignored at Aberdeen but experimental work was restricted to a basic investigation of vibratory driving in granular soils, since vibratory pile driving methods (originally developed in 1930 by Hertwig in Germany) were being studied for the rapid installation of anchorages in sands. To complement the field work by industry, the research at Aberdeen concentrated on laboratory experimentation and the development of theoretical equations of motion to predict the experimentally derived results. The research work[34] identified two distinct types of vibratory driving in granular soils, and for the case of "slow" vibrodriving defined the motion to be that of a rigid body

[Δ]Yap, L.P. & Rodger, A.A. (1984) "A Study of the Behaviour of Vertical Rock Anchors Using the Finite Element Method", Int. J. Rock Mech., Min. Sci. & Geomech. Abstr. 21(2), 47-61.

subject to viscous-Coulomb side and elasto-plastic end resistance under a combined sinusoidal excitation and static surcharge force. The development of the theory and an interactive computer simulation introduced a new understanding and provided sufficient information to facilitate the specification and design of field prototypes for vibrodriving and vibrohammering. Subsequently, the work has been extended into soil sampling, and at the present time vibratory impact moling in the field of trenchless technology.

On returning to industry with the Colcrete Group of Companies (1976-84) the author concentrated initially on providing updated recommendations on good practice[31] quality controls, construction tolerances, and classes of test for the new British Standard (DD81 : 1982[78]). The author also refined his work on corrosion protection by introducing the concepts of single and double protection (now recognised worldwide in practice) and started to develop new procedures for the short term testing of anchorages to ensure that they were fit for their intended purpose. Thereafter, new anchorage nomenclature was proposed to facilitate understanding and comparisons of national practices. This nomenclature has been adopted by the three international organisations concerned with anchorages, i.e. ISSMFE, ISRM and FIP. Anchorages were also classified by the author into four main types. For each type appropriate design procedures[32] were described and analysed, and appropriate load safety factors proposed for use by industry. These anchorage types have been adopted by standards bodies addressing design such as BSI, FIP and the US Post-Tensioning Institute (PTI).

In relation to short term testing, new acceptance criteria for service behaviour were proposed by the author in 1981[36] after a two year period of field appraisal on contract anchorages installed in soils and rocks.. These criteria were novel in that they accommodated testing by either residual load-time or creep displacement-time behaviour, were independent of ground type and potentially of short duration up to 50 minutes. Essentially time intervals on a logarithmic scale were established and for each time interval a single load relaxation or creep criterion was specified to ensure a stabilising trend. The approach which is now well established was first adopted by BSI in 1982[78], ISRM in 1985[48], and FIP in 1993[84]. The criteria have also been included in the current draft US code (PTI) which should be published in 1994.

In 1982 the author published design guidance on cement based grouts[38] which are used for ground anchorage applications, and elsewhere, based on experimental work and studies carried out at both Cementation and Colcrete.

At this time the author created an FIP Working Group, comprising anchorage specialists from 14 countries, to study over a three year period the corrosion and corrosion protection of prestressing steel tendons in ground anchorages. A comprehensive report written by the author in liaison with the working group was published by FIP in 1986[79]. This report covered types and mechanisms of corrosion, aggressivity of the ground in relation to steel and cement based grouts, and a detailed analysis of corrosion failures collected by the working group. The analysis of failures also permitted the inclusion of rigorous recommendations concerning principles of protection and properties of a protective system, together with typical examples of acceptable protective systems. The author published a summary[52] of the findings from the corrosion failures which illustrated for example that corrosion was invariably localised and in such circumstances no tendon type (bar, wire or strand) had a special immunity, and failure could occur after only a few weeks of service or many years. Quenched and tempered plain carbon steels and high alloy steels were also found to be more susceptible to hydrogen embrittlement than other varieties, and are no longer recommended for tensioned anchorages.

Since joining the University of Bradford (1985 to date) the author has focused on (a) the behaviour of rock bolt supports in tunnels, when subjected to close proximity blasting, (b) the design and performance of anchorages installed in weak mudstones (UCS < 5 N/mm^2) with particular reference to load transfer and failure mechanisms and (c) the service behaviour of anchorages in a variety of ground types, e.g. sandstone, mudstone, slate, granite, microdiorite and fossil scree. The results of studies under (b) and (c) will be published during 1994 and 1995 and are therefore outside the scope of this thesis.

For rock tunnelling by drill and blast methods of excavation, there was no predictive capacity in 1985 for optimising the distance from the tunnel face to a safe location for the installation of permanent rock bolts, and the resulting designs were considered by the author to be conservative and costly. To provide a basic understanding of rock bolt behaviour under static and dynamic loading and to enable the development of new improved design procedures, a three part programme was undertaken.

(i) Full scale tests on active tunnel construction sites at Penmaenbach (1987) and Pen y Clip (1991) Tunnels in North Wales, where the initial tunnel support comprised rock bolts in conjunction with sprayed concrete[54,55]. Axial load and acceleration were measured on rock bolts positioned at various distances from the blast face. The influence of prestress load on rock bolt performance was studied as was the difference in bolt response resulting from single speed and two speed resin bonding.

In addition, stress distribution along the bolt length was studied at Pen y Clip using a specially designed transducer system.

(ii) Large scale laboratory model tests to examine stress distribution within resin bonded bolts under static and dynamic loading.

(iii) Finite element computer simulation of the response of rock bolts to transient loading to assist with the interpretation and generalisation of field and laboratory experimental results. This work was a development of the previous finite element studies of the static behaviour of rock anchorage systems published by Yap and Rodger.

Aside from the establishment of unique attenuation relationships for peak particle velocity and peak dynamic load the results of the field monitoring at Penmaenbach Tunnel[53,58,60] constructed through very strong rhyolite (fracture spacing typically 0.2 m to > 0.5 m), showed that all deformations were elastic and no significant load loss or resin-bolt debonding was recorded even for bolts located only 0.7 m from the blast face (accelerations up to 640 g). As a consequence, a large number of bolts scheduled for replacement were in fact undamaged, resulting in substantial savings. The field monitoring also provided corroboration of an important result found from laboratory and finite element studies that prestressing the rock bolts decreases the vibrational loading. No appreciable difference was observed between the dynamic load responses of fully bonded 3.5 m and 6 m bolts, but single speed bolts experienced twice the dynamic loading of equivalent two speed bolts due to their shorter decoupled length. In relation to blasting practice, where typically each blast generated 35 events (delays) in a blast duration of 6 seconds, the effect of varying the charge weight per delay on the vibration induced in nearby rock bolts was found to increase with improvement in rock mass quality. This work is referenced in BS.8081 and has provided fundamental information and a new understanding of the character of blast induced wave forms within a rock mass and how these signatures affect the overall rock bolt system.

Based on the above findings the safe distance for permanent rock bolts at Pen y Clip Tunnel (longest hard rock tunnel in UK) was reduced to 3 m effecting a 38% reduction in the cost of rock bolt support. The monitoring at Pen y Clip provided further unique attenuation relationships (strong microdiorite with a discontinuity spacing of 0.1 to 0.2 m) and confirmation that safe distances to the blast face could be reduced to 1 m without significant damage to the bolt, in spite of the presence of a weaker rock mass.

Basic data have also been obtained on the nature of the stress distribution and bond development along the resin-bolt interface resulting from both static and dynamic loading, and on the influence of prestress loading on bolt response[74]. The static load distribution of rock bolts was exponential throughout the length of bolts tensioned after the slow setting resin had cured. For standard two speed rock bolts, the loads were evenly distributed over the slow setting resin element, and exponentially distributed over the fast setting element. Two speed rock bolts were more effective than single speed rock bolts in resisting tensile loads since these bolts could transfer the tensile load further into the stable rock mass. In practice, the two speed rock bolts did not always transfer the load as required because load distribution is critically dependent upon the choice of slow and fast setting times for the resins in relation to the time of tensioning. This aspect needs to be more tightly specified and controlled in future production bolting.

Prestress loading was found to be highly variable in production bolting at Pen y Clip, and as a result a parallel field trial was undertaken (with J. Conway) to develop a field control for rock bolt tensioning to ensure that rock bolts are installed with the specified load, and to facilitate monitoring of critical load fluctuations in service[75].

An additional major finding from both tunnel projects, was that the frequency response spectra of rock bolts responding to different blasts and at different distances from the blast, were found to be of a similar form. The response spectra are assumed to depend on the form of rock bolt construction and the characteristics of the co-vibrating rock mass. Looking ahead, this finding has indicated the possibility of developing a new non-destructive testing system for ground anchorage integrity based on examination of the signature of the response spectra. It is now intended to investigate the feasiblity of such a system using the database of over 9000 records (on FM magnetic tape) obtained from Penmaenbach and Pen y Clip projects.

Development of such a system has also been facilitated by a field investigation carried out by the author and A.A. Rodger at a large excavation in the centre of Edinburgh (Port Hamilton), where support was provided by over 200 anchorages installed in mudstone. The anchorages had been installed two years previously and the fixed anchors were positioned as close as 5 m to a proposed tunnel to be excavated by blasting. Geophones were installed in boreholes at the level of the distal end of the fixed anchors and at various distances from the tunnel blast face. Vibration measurements were also taken on the anchor heads at waling level and anchorage loads were monitored by check lifting before and after blasting. On this site, the vibrations did lead to load losses on occasions, and thereby permitted the research team to obtain vibrational waveforms corresponding to permanent load loss. A further important finding was corroboration that measurements of anchor head vibration can be related to those at the distal end of the fixed anchor for, in this case, a buried dynamic source. The detailed results for this project (1991-93) have been reported in full to Lothian Regional Council who should permit publication during 1994.

Two publications[63,64] illustrate the outcome of the author's investigations into corrosion protection of steel tendons and on-site acceptance tests for ground anchorages which were first disseminated by the author through BSI and FIP standards. These and other related publications[67,68] arose as a result of a programme of invited lectures to several leading US civil engineering consulting firms in Boston and New York (all involved in Boston's Tunnel and Central Artery Project, potentially utilising over 10,000 anchorages), where it became apparent that US practice and standards were lagging behind national and international codes evolved in Europe. Following these lectures, the author has been appointed as foreign adviser to the PTI ground anchorage committee, and is retained to advise the PB/MK Team (Parsons Brickerhoff and Morrison Knudson) on behalf of the Universities Research Association for the development of anchorages in the Taylor Marl for the Superconducting Super Collider being built at Waxahachie, Texas. Straight-shafted anchorages, with and without postgrouting, and underreamed anchorages are being investigated in relation to short term tensile load-displacement behaviour, creep and ultimate capacity.

In 1991 the author reviewed world practice and prepared a forward look[66] to highlight those areas where further investigation and development coupled with improved standards would enhance understanding of anchorage behaviour and maintain ground anchorage technology at the forefront in the field of geotechnical processes.

5. Selection of Papers

This selection comprises 29 papers,and with the exception of the first paper on the ductility of metals, the remaining papers cover the author's published work on ground anchorage systems over the period 1965 to 1993.

ELONGATION IN THE TENSION TEST AS A MEASURE OF DUCTILITY

By T. C. Hsu,[1] G. S. Littlejohn,[2] and B. M. Marchbank[2]

KEY WORDS: ductility, tension test, elongation, necking, strain distribution, aluminum alloys, steel, strain rate.

ABSTRACT: Four quantities, independent of each other, are derived to represent the size and shape of the strain distribution curve. These are: the plastic strain at maximum load, the nondimensional length of the neck, the maximum strain due to necking, and a factor of uniformity related to the increase in elongation owing to necking. The strain distribution curves of seven nonferrous metals and several steels were plotted. Strains were determined by measuring distances between lines of a grid pattern previously photographed on the specimen. The effects of strain rate and specimen size and shape were investigated. When the specimen width-to-thickness ratio was changed from 4 to 8, the mode of fracture changed from shear to tear. The effect of both size and shape was more pronounced for aluminum alloy than for steel specimens, but the per cent elongation due to necking was identical for all specimens less than $\frac{1}{16}$ in. thick. Strain rate had opposite effects on aluminum and steel as far as the strain due to necking and factor of uniformity are concerned. Although additional tests at various strain rates are needed to conclusively demonstrate the latter result, it does indicate that ductility cannot be accurately represented by a single index.

REFERENCE: T. C. Hsu, G. S. Littlejohn, and B. M. Marchbank, "Elongation in the Tension Test as a Measure of Ductility," *Proceedings*, Am. Soc. Testing Mats., Vol. 65, 1965, p. 874.

Ever since the tension test was first used by engineers over a hundred years ago, it has been considered to be a test for ductility as well as for strength. It was natural for the early test engineers to measure the total elongation of the broken specimen, first between the shoulders and then, more sophisticatedly, between two gage marks, and to take the elongation as a measure of ductility. The broken tension specimen has, however, a rather complicated shape, so that on it, several measurements other than the elongation can be made which may also be considered to represent the ductility of the material. In the history of the tension test, one can trace (1) the proposals of one measurement and another as the ductility index, and (2) the attempts to correlate such measurements taken from specimens of different size and shape. The following review is written along these two separate lines:

(1) The proposals for the ductility indexes are summarized in Table 1. By no means exhaustive, this table sufficiently shows that there has always been an element of doubt in the choice of the ductility index.

(2) The analysis of the strain distribution along the tension specimen arose out of the need to correlate results obtained from specimens of different size, shape, and gage length. The experimental and theoretical studies of the strain distribution are summarized in Table 2.

[1] Senior lecturer, Mechanical Engineering Dept., University of Edinburgh, Edinburgh, Scotland.

[2] Formerly of Civil and Mechanical Engineering Dept., University of Edinburgh, Edinburgh, Scotland.

TABLE 1—CHRONOLOGICAL SUMMARY OF THE CHOICE OF DUCTILITY INDEX.

Author	Year	Measurements Made	Proposed Ductility Index	Remarks	Reference
Fairbairn.......	1850	Total elongation and contraction at fracture	...	...	*1*
Mallet.........	1858	Elongation and reduction in area of 1-in. square bar, 3½ ft parallel length	...	Maximum strain at fracture noticed, p. 325, *2*	*2*
Kirkaldy.......	1863	Total elongation, diameter outside the neck and minimum section at fracture	...	...	*3*
Whitworth.....	1873	...	Percentage elongation said to be of equal importance as tensile ultimate strength	...	Discussion by J. Whitworth, pp. 106–107, *4*
Barba..........	1880	Percentage elongation and reduction in area	...	...	*5*
Webster........	1883	...	Maximum uniform strain and reduction in area	...	Correspondence by J. J. Webster, pp. 147–150, *6*
Hackney.......	1883	...	Percentage elongation and percentage reduction in area used	...	*6*
Tetmajer.......	1890	...	Maximum uniform strain	...	*7*
Unwin.........	1903	...	Uniform deformation at maximum load	...	p. 179, *8*
Sachs..........	1925	...	Percentage reduction in area emphasized	...	*9*
MacGregor.....	1944	...	Maximum uniform strain and the maximum strain due to neck formation	...	*10*
Zaitsev.........	1957	...	Maximum uniform strain and strain at fracture were used	...	*11*

As can be seen in Table 2, crude attempts to determine the strain distribution along the tension specimen are almost as old as the tension test. In particular, Martens' strain distribution curves (instantaneous longitudinal strain plotted against the distance along the undeformed specimen) seem to be remarkably accurate (Fig. 36, Ref. *14*, cf. Unwin's strain distribution curves in Fig. 3, Ref. *8*, and Fig. 50, Ref. *16*). The theoretical analysis of the strain distribution, being hitherto stimulated by the need to correlate results from

TABLE 2—CHRONOLOGICAL SUMMARY OF THE STUDY OF STRAIN DISTRIBUTION IN TENSION TEST.

Author	Year	Experimental Work	Theoretical Work	Remarks	Reference
Fairbairn........	1850	...	Extension per unit length = $0.18 + 25/l_g$ (l_g = gage length)	...	*1*
Kirkaldy........	1863	Punch marks ½-in. apart put on specimen	...	...	*3*
Committee of Civil Engrs....	1868	Variation of strain along specimen measured	...	...	p. 74, *6*
Barba...........	1880	...	Percentage elongation proportional to $\sqrt{A_o}/l_g$ (A_o = original area, l_g = gage length)	...	*5*
Adamson........	1886	...	Separate use for uniform elongation and the elongation between maximum load and fracture	...	*12*
Bauschinger.....	Before 1887	...	Elongation in neck proportional to $\sqrt{A_o}$	...	*13*
Tetmajer........	1890	...	Separated local and general elongation	...	7
Martens.........	1894	Strain distribution curves plotted	...	Also plotted by Pralon, 1894, p. 269n, *15*	*14*
Martens.........	1899	Strain distribution curves during neck formation	...	...	Fig. 220, *15*
Martens.........	1899	...	Percentage elongation = 100 $(a+b\sqrt{A_o}/l_g)$ (a, b are constants)	...	pp. 123–124, *15*
Unwin..........	1903	Elongation measured in ½-in. gage lengths along specimen	Percentage elongation = 100 $(a+b\sqrt{A_o}/l_o)$	...	*8, 16*
Bartella.........	1922	...	Percentage elongation = $\mu(\sqrt{A_o}/l_g)^\alpha$ (μ, α, are constants)	...	Quoted in *18*

TABLE 2—*Continued.*

Author	Year	Experimental Work	Theoretical Work	Remarks	Reference
Tiedemann......	1927	...	Separated uniform and local elongation	...	*17*
Oliver...........	1928	...	Percentage elongation = $\mu(\sqrt{A_o}/l_o)^a$	...	*18*
Simizu and Ide...	1957	Percentage elongation in terms of maximum uniform strain and maximum strain due to neck formation	...	...	*19*

specimens of different size and shape, was confined to formulas relating the elongation to the cross-sectional area and gage length. Of such formulas, called "elongation equations", the one by Martens and Unwin and that by Bartella and Oliver are well known, and their implications will now be discussed.

Unwin's elongation formula may also be written as

$$\text{Elongation} = \epsilon_u l_g + \beta\sqrt{A_o}$$

where:

ϵ_u = longitudinal strain outside the neck,
l_g = gage length,
A_o = original cross-sectional area, and
β = a constant.

In this form the formula merely states that the elongation can be divided into two parts: one, which is due to a uniform strain along the specimen and is proportional to the gage length, and the other, due to local elongation in the neck, is proportional to the square root of the cross-sectional area. The term $(\beta\sqrt{A_o})$ concerns the so-called Barba's Law that geometrically similar specimens develop geometrically similar necks. Barba's Law is considered to be outside the scope of this investigation.

The separation of the local from the general elongation is possible only if the gage length is longer than the neck. Oliver, when deriving his formula, used gage lengths shorter than the neck and thus mixed together what Martens and Unwin had separated. It can be shown that Oliver's formula implies that the strain at the fractured section is infinite, that there is no uniform elongation outside the neck, and that the form of the strain distribution curve is a hyperbola of a certain type (Appendix I).

The historical survey of the tension test suggests, therefore, that the choice of the ductility index is still open to consideration, and that the strain distribution along the specimen may warrant further study. Our investigation was undertaken with these two points in view.

Experimental Technique:

Our experimental technique consists of printing, by photographic means, a fine grid pattern on the surface of tension specimens and measuring the strain along the specimens. Grid patterns have been photographed before on metal surfaces for the study of plastic flow in metal forming as well as in tension test [*20–23*].[3] The refinement in this

[3] The italic numbers in brackets refer to the list of references appended to this paper.

technique consists of using a checkerboard pattern instead of a line grid (like that of the graph paper), the advantages of the former being its greater accuracy owing to the use of the edge of a square rather than a line as the gage mark, the greater ease of photographic work, and the less likelihood for the pattern to disappear after deformation. Two master plates of checkerboard pattern were used, one of 211 and one of 88 lines per inch, the choice depending on the size of the specimen.

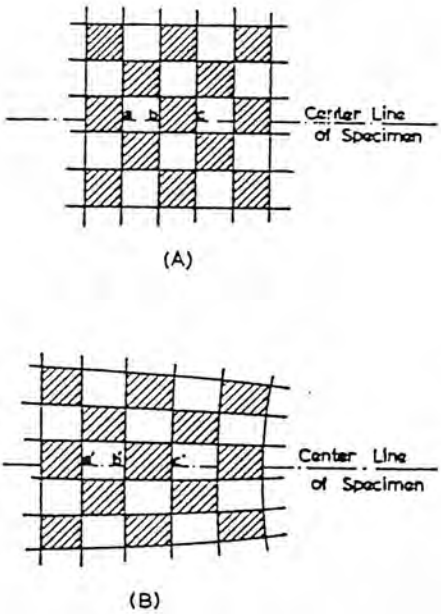


FIG. 1—Measurement of longitudinal strain in a tension specimen.

To measure the strain at various points along the specimen, the elongated specimen was first aligned on the microscope stage so that the centerline coincided with a line on the microscope screen, as in Fig. 1*b*. The engineering strain at midpoint between *a* and *b* is taken to be (Fig. 1) $\epsilon = (a'b' - ab)/ab$.

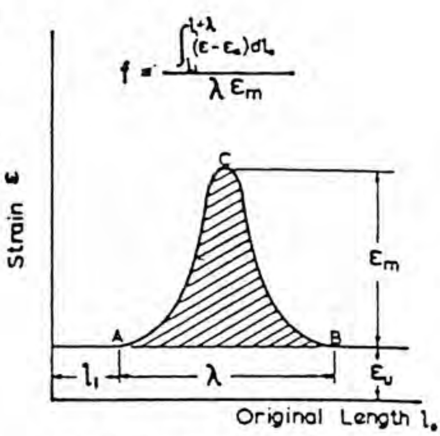


FIG. 2—The strain distribution curve.

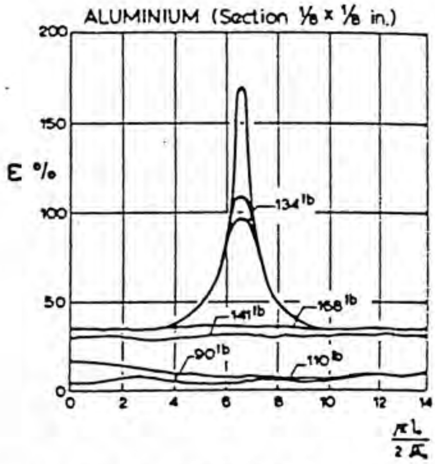


FIG. 3—Strain distributions during the formation of the neck.

The Strain Distribution Curve:

To show the variation of the strain ϵ may be plotted against the distance along the *undeformed* specimen (l_0) measured from a convenient point as in Fig. 2; or against a nondimensional distance as in Fig. 3, where A_0 is the original cross-sectional area and the abscissa represents the number of equivalent diameters. The experimental points are omitted from Fig. 3 for the sake of clarity. The specimen represented in Fig. 3 was stretched to fracture in seven steps, four before and three after the neck had started to form. In the first three steps (90, 110, and 141-lb load), the strains increased irregularly but were generally uniform along the specimen. When the load reached its maximum (168 lb), the uniform strain also reached its maximum (36 per cent), and thereafter the load dropped and the strain became localized in the neck. The highest curve in Fig. 3 corresponds to the fractured specimen, and it will henceforth be called the strain distribution curve.

For analyzing the elongation in a fractured tension specimen, the strain distribution curve is to be represented by four quantities as follows. In order to clarify the relation between the four quantities and the dimensions of the fractured specimen, the strain ϵ is plotted in Fig. 2 against the distance l_o.

(1) The maximum uniform strain ϵ_u, which is the plastic strain at maximum load, as well as the permanent strain outside the neck in the fractured specimen.

(2) The nondimensional length of the neck

$$\rho = \frac{\sqrt{\pi}}{2} \frac{\lambda}{\sqrt{A_o}}$$

where λ is the length of the neck measured along the undeformed specimen and $2\sqrt{A_o}/\sqrt{\pi}$ is the equivalent diameter of the original cross section.

(3) The maximum strain due to necking, ϵ_m, which is equal to the strain at the minimum section in the fractured specimen less the maximum uniform strain.

(4) The factor of uniformity, f, which is defined as

$$f = \frac{\int_{l_1}^{l_1+\lambda} (\epsilon - \epsilon_u)\,dl_o}{\lambda \epsilon_m}$$

and is equal to

$$\frac{\text{Average height of the cross-hatched area, Fig. 2}}{\text{Maximum height}}$$

An area in Fig. 2 represents an elongation, in inches, hence the cross-hatched area represents the increase in elongation due to necking. The value of f

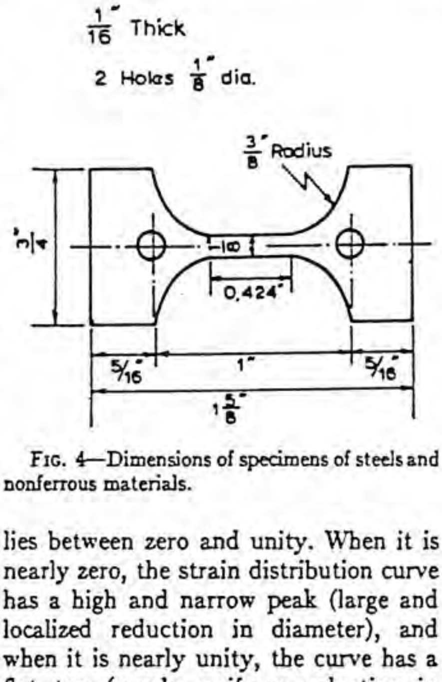


FIG. 4—Dimensions of specimens of steels and nonferrous materials.

lies between zero and unity. When it is nearly zero, the strain distribution curve has a high and narrow peak (large and localized reduction in diameter), and when it is nearly unity, the curve has a flat top (nearly uniform reduction in diameter).

For practical purposes, the four quantities defined above represent completely the strain distribution curve, because when their values are known, the strain distribution curve can be reconstructed. Thus, A and B in Fig. 2 can be located by ϵ_u and λ $(= 2\rho\sqrt{A_o}/\sqrt{\pi})$, C is midway between A and B (for a symmetrical neck) and at ϵ_m above them,

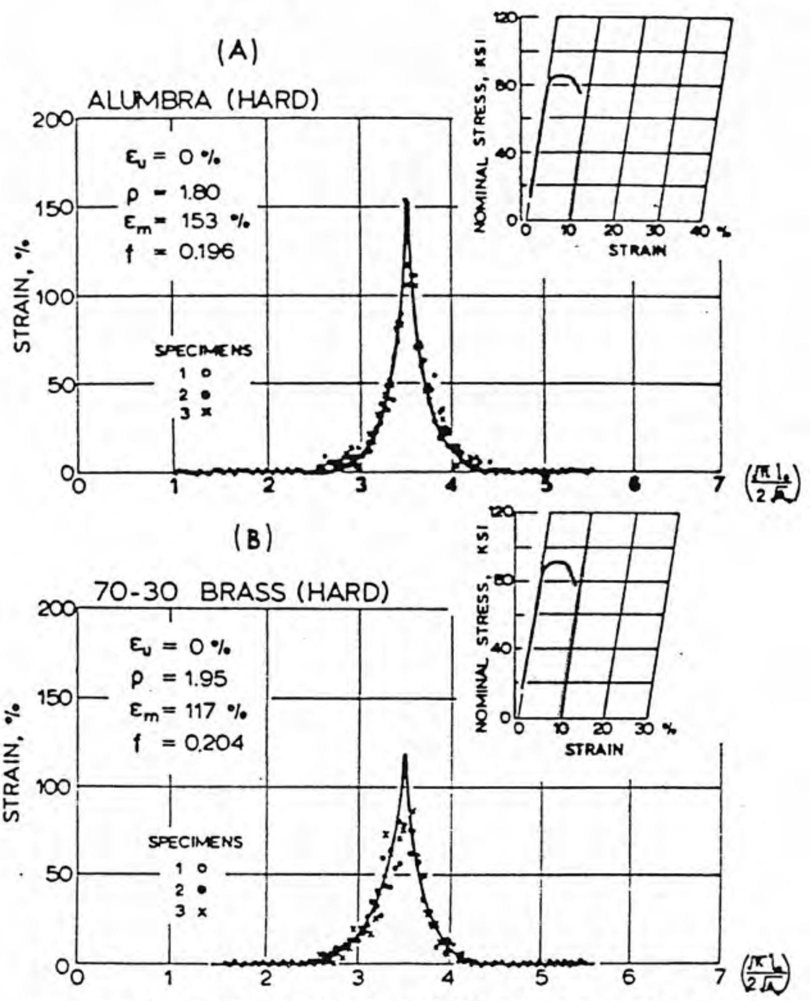
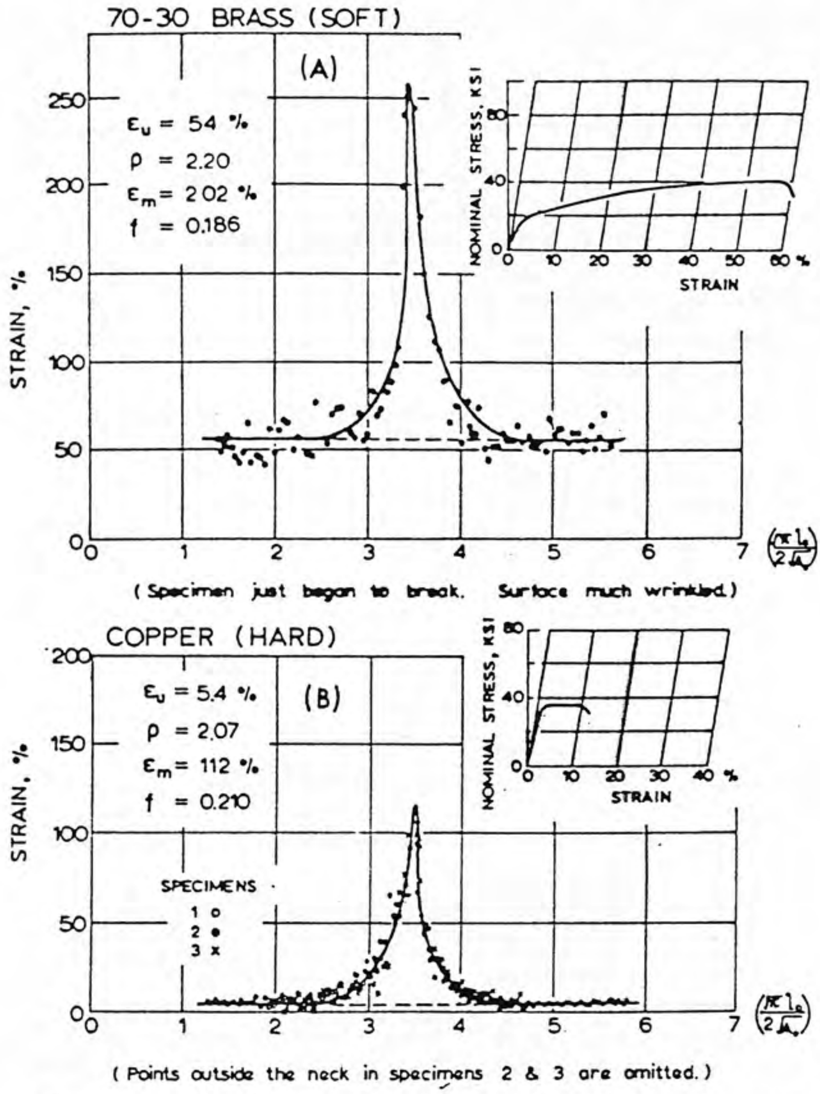


FIG. 5—Strain distribution curves for alumbra (hard) and 70-30 brass (hard).

and the curve ACB can be constructed with zero slope at A, B, and C and enclosing an area equal to $(f\lambda\epsilon_m)$. (For precise representation of the strain distribution curve, see Appendix II.)

Given appropriate values, the four quantities ϵ_u, ρ, ϵ_m, and f can represent the most extreme types of elongation in engineering materials. Thus, for some manganese steels which stretch uniformly until fracture occurs [*18*], the value of ρ is infinity, ϵ_m is equal to zero, and f is unity because the whole parallel length may be considered to be in the neck. For nylon, which develops a well-defined uniform neck of great length [*29*], ρ is large but finite and f is unity. Some aluminum alloys neck until the smallest section is practically a needle point, and for them ϵ_m is almost infinity



Hard—The materials were originally about ⅛ in. thick in the annealed state; then cold-rolled to $\frac{1}{16}$ in. thick.

Soft—From the cold-rolled state ($\frac{1}{16}$ in. thick), copper was annealed at 400 C for 1 hr; 70-30 brass annealed at 525 C for 1 hr.

FIG. 6—Strain distribution curves for 70-30 brass (soft) and copper (hard).

and f is zero. Tension specimens of brittle materials break without a neck, so that ρ and ϵ_m are equal to zero, ϵ_u is very small, and f is unity or zero, depending on mathematical interpretation.

Experimental Results:

Strain distribution curves for nonferrous materials—Seven nonferrous materials, of composition and heat treatment given

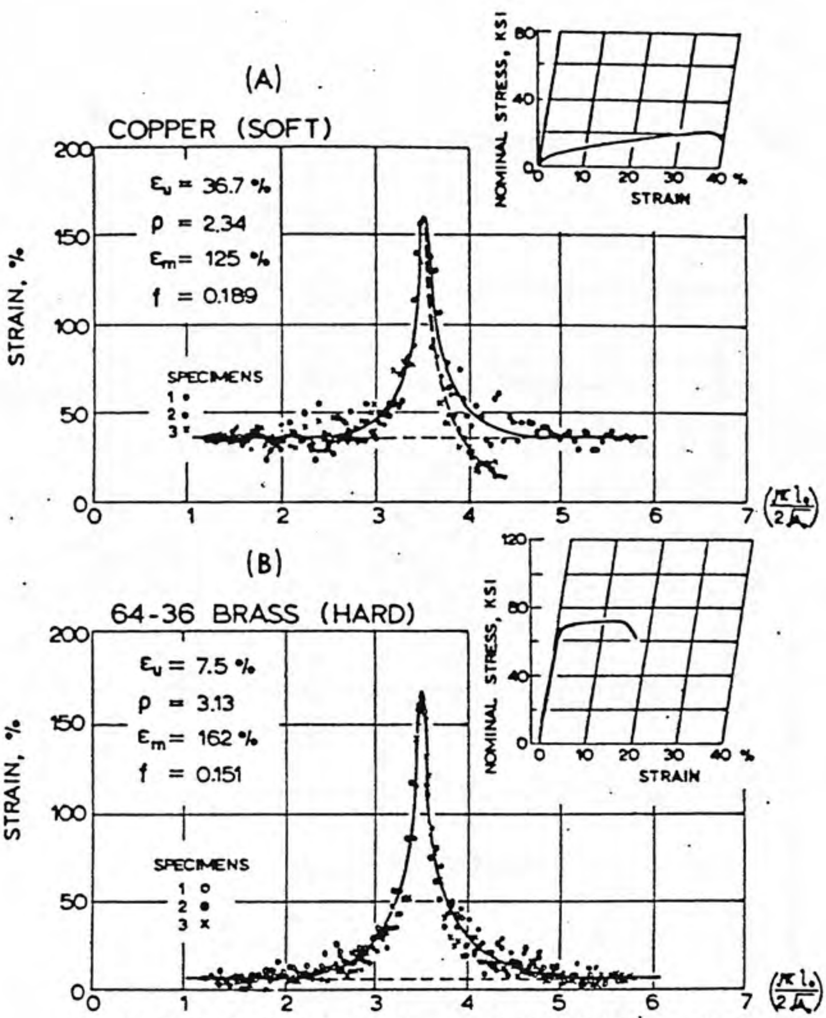


FIG. 7—Strain distribution curves for copper (soft) and 64-36 brass (hard).

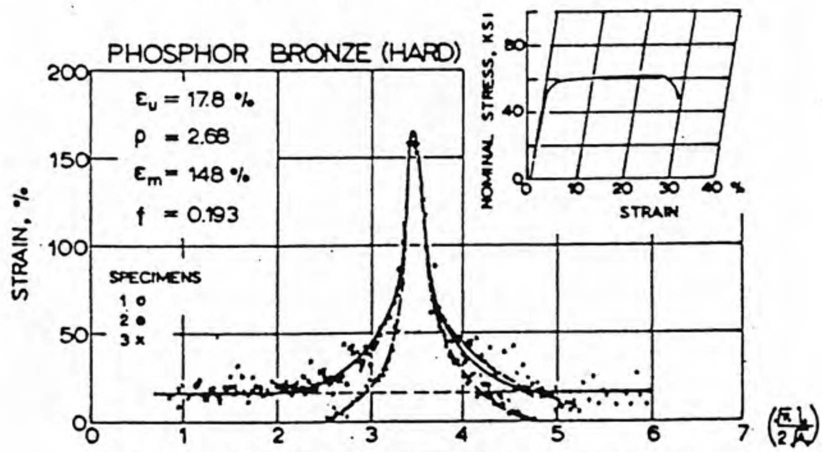


FIG. 8—Strain distribution curves for phosphor bronze (hard).

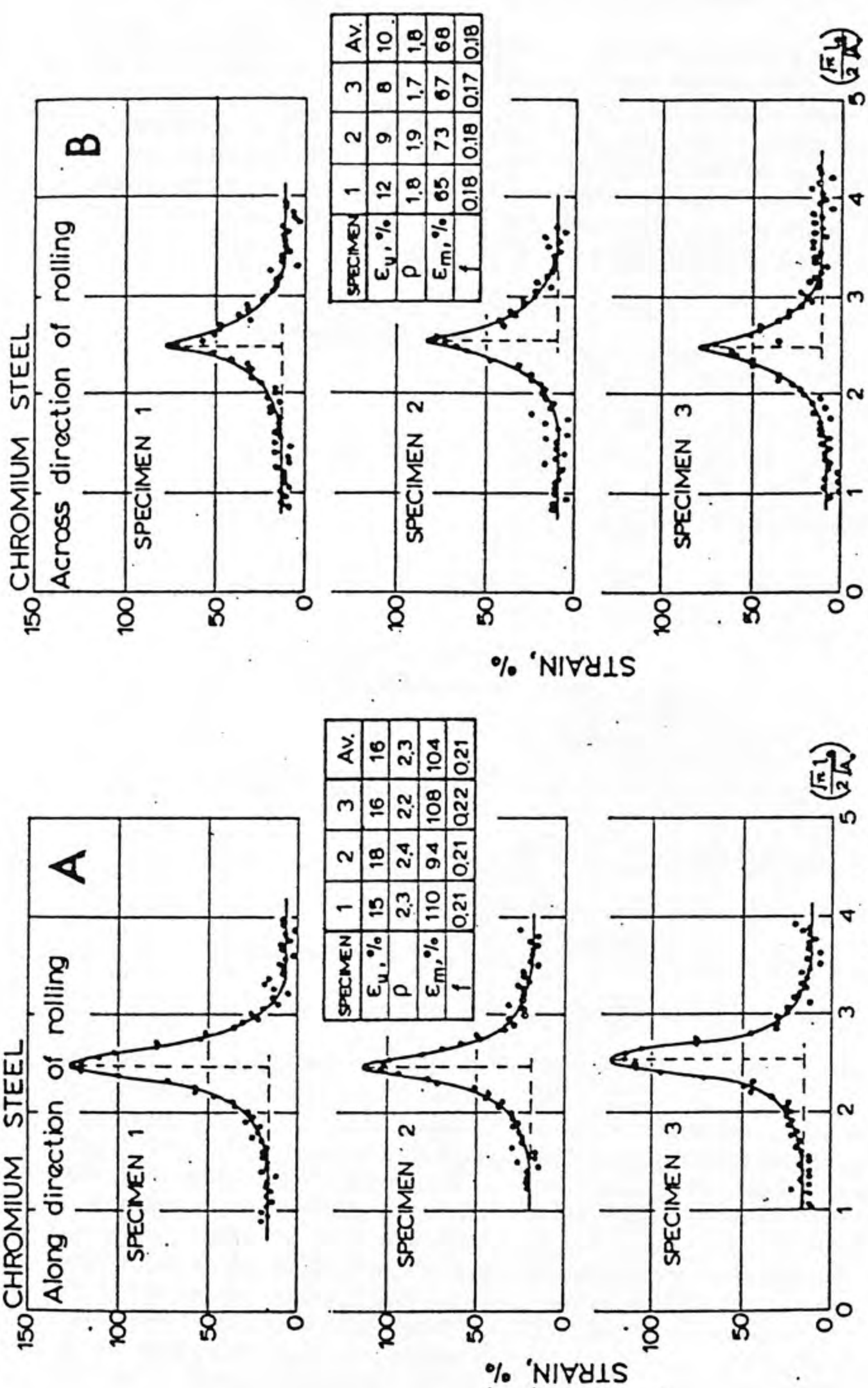


FIG. 9—Strain distribution curves for chromium steel.

in Appendix III, were tested. The specimen size, conforming to British Standard 18:1956 (gage length equals $4\sqrt{A_o}$), is shown in Fig. 4. Their strain distribution curves are shown in Figs. 5–8. The stress-strain curves for the materials, determined in a tensometer and shown in the insets in Figs. 5–8, are based on the nominal stress calculated with the original cross-sectional area and the strain calculated with the parallel length in Fig. 4. The inclined ordinates are due to the characteristics of the tensometer.

For most of the materials, three specimens were measured and the results are superimposed one on another. In cases of unsymmetrical necks (Figs. 7*a*, 8), the higher leg is chosen to represent the strain distribution curve—a practice in line with British Standards 18:1956 (Clause 22, Note).

As shown in Figs. 5–8, the strain distribution curves of the nonferrous materials vary from each other as much as their stress-strain curves. Thus, in Fig. 5*a*,*b* the elongation is entirely due to necking, whereas in Fig. 6*a* it is largely due to the uniform strain. Wide variation is noticeable in the value of ϵ_m (Figs. 5*b*, 6*a*) as well as in p (Figs. 5*a*, 7*b*).

Strain distribution curves for steel—Careful examinations of the experimental points on Figs. 5*a* and 7*a* will show that specimens of the same material sometimes develop necks of slightly different shape. For this reason it was decided to present the typical results for the steels in three separate curves for two cases (Fig. 9*a*, *b*). The chemical composition and heat treatment of the materials are given in Appendix IV and

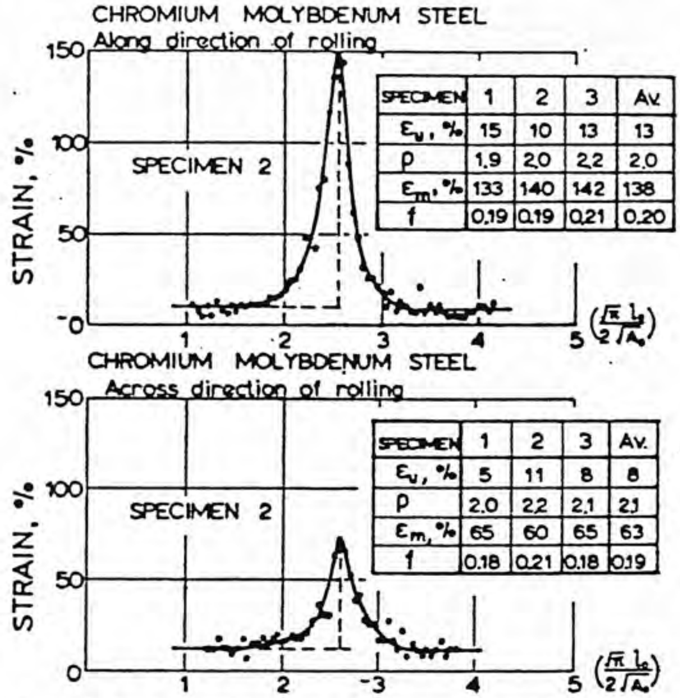
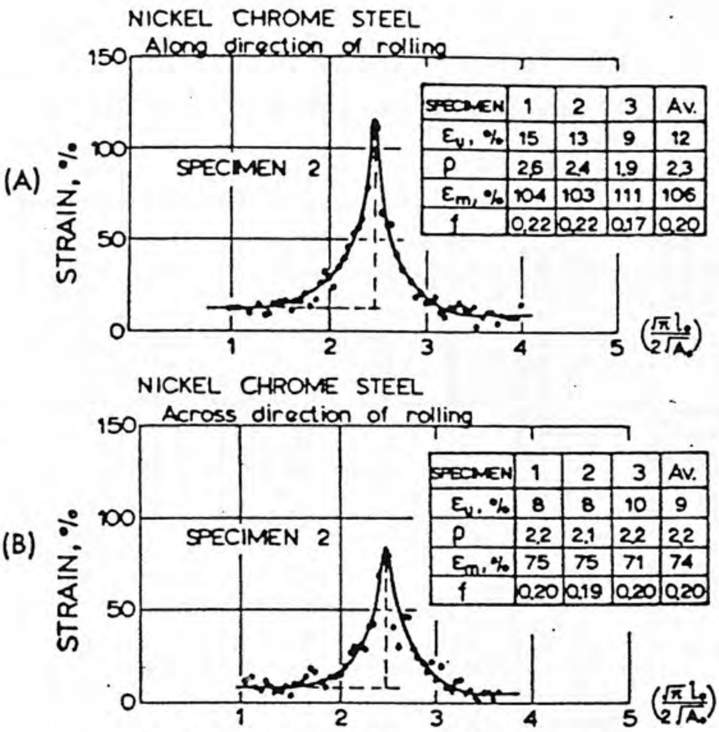


FIG. 10—Strain distribution curves for chromium molybdenum steel.



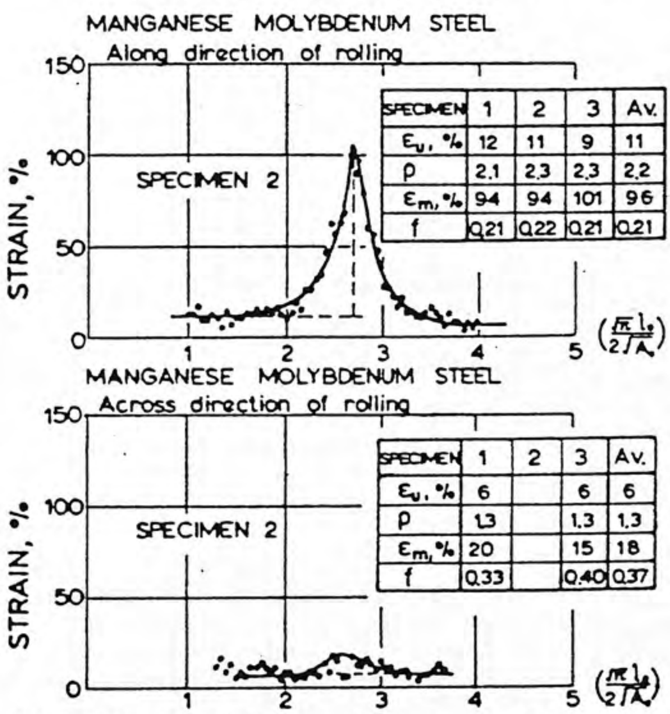
(A) Along direction of rolling

SPECIMEN	1	2	3	Av.
ε_u, %	15	13	9	12
ρ	2.6	2.4	1.9	2.3
ε_m, %	104	103	111	106
f	0.22	0.22	0.17	0.20

(B) Across direction of rolling

SPECIMEN	1	2	3	Av.
ε_u, %	8	8	10	9
ρ	2.2	2.1	2.2	2.2
ε_m, %	75	75	71	74
f	0.20	0.19	0.20	0.20

FIG. 11—Strain distribution curves for nickel chrome steel.



Along direction of rolling

SPECIMEN	1	2	3	Av.
ε_u, %	12	11	9	11
ρ	2.1	2.3	2.3	2.2
ε_m, %	94	94	101	96
f	0.21	0.22	0.21	0.21

Across direction of rolling

SPECIMEN	1	2	3	Av.
ε_u, %	6		6	6
ρ	1.3		1.3	1.3
ε_m, %	20		15	18
f	0.33		0.40	0.37

FIG. 12—Strain distribution curves for manganese molybdenum steel.

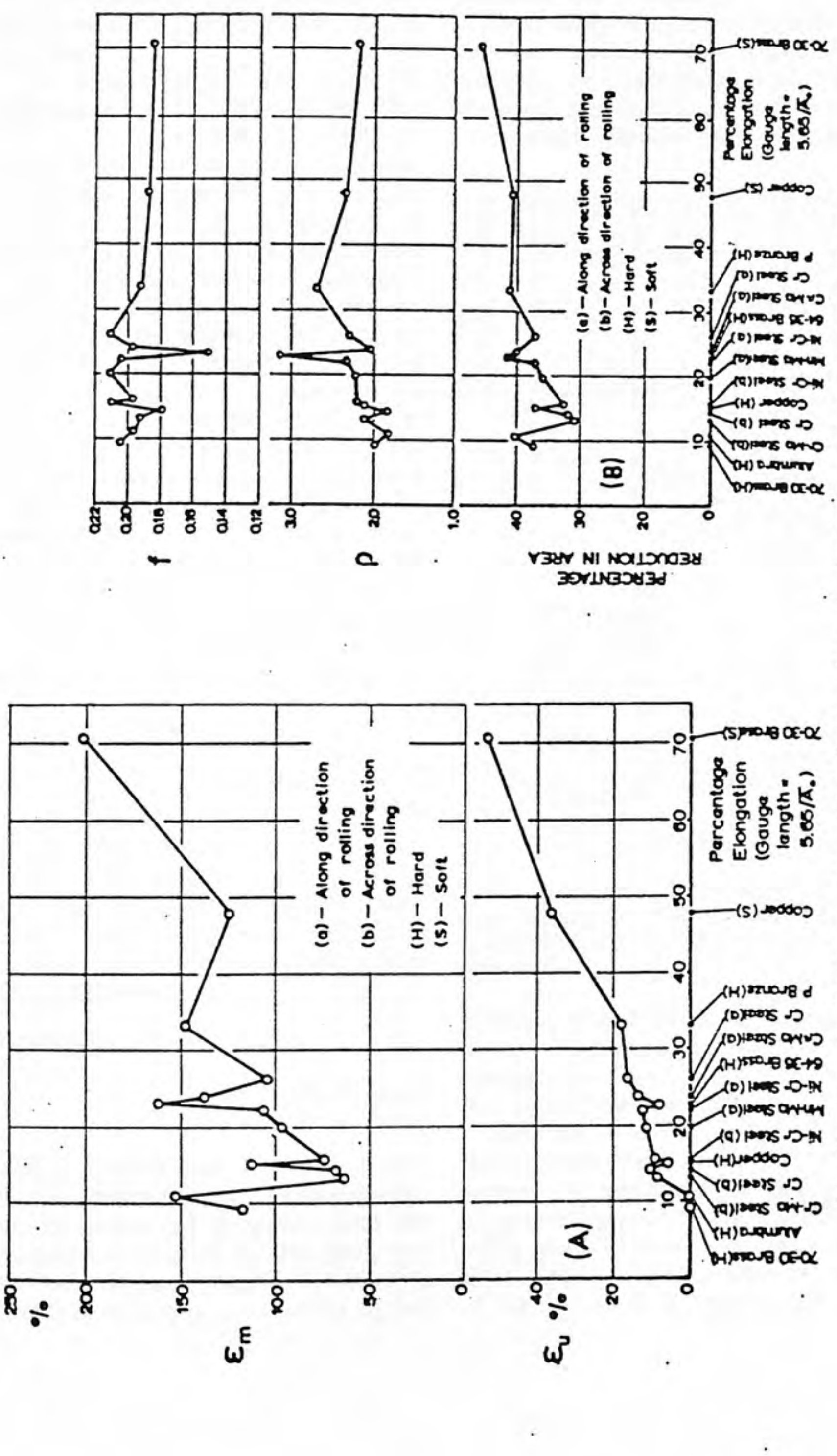


FIG. 13—Correlation between ε_u, ρ, ε_m, f and the percentage reduction in area with the percentage elongation.

the specimens are also as shown in Fig. 4. Each material was tested in two directions, along and across the direction of rolling.

The same variety of the strain distribution curve is noticeable in Figs. 9–12 as in Figs. 5–8. In particular, tested across the direction of rolling, manganese-molybdenum steel hardly developed a neck at all, although all the other results show well-formed necks.

The apparent success of Oliver's elongation formula can now be explained by the partial similarity between the actual curves in Figs. 5–8 and 9–12, and those implied by the Oliver's formula in Fig. 22 (see Appendix I), where each hyperbola and its mirror image together constitute a strain distribution curve (to suitable scales on the axes).

Correlation between the percentage elongation and ϵ_u, ρ, ϵ_m and f—It can be easily shown that, for a gage length of $5.65\ \sqrt{A_o}$ (five equivalent diameters), the percentage elongation is given by

Percentage elongation

$$= 100\left(\epsilon_u + \frac{f\rho\epsilon_m}{5}\right)$$

It is possible, therefore, to compare each of the four quantities ϵ_u, ρ, ϵ_m, and f with the percentage elongation determined by this formula, as in Fig. 13*a*, *b*. In Fig. 13*b*, the percentage reduction in area is calculated

Percentage reduction in area

$$= 100\left(\frac{\epsilon_m + \epsilon_u}{1 + \epsilon_m + \epsilon_u}\right)$$

It is obvious from Fig. 13*a*, *b* that ϵ_m, for example, bears no more relation to the percentage elongation than the percentage reduction in area does, and similarly for ϵ_u, ρ, and f. It is hardly necessary to plot ϵ_u, ρ, ϵ_m, and f against each other to show that they are not functions of each other.

It was shown in a previous section that the four quantities were sufficient to represent the strain distribution curve; it has just been shown that each of them is necessary, because they vary inde-

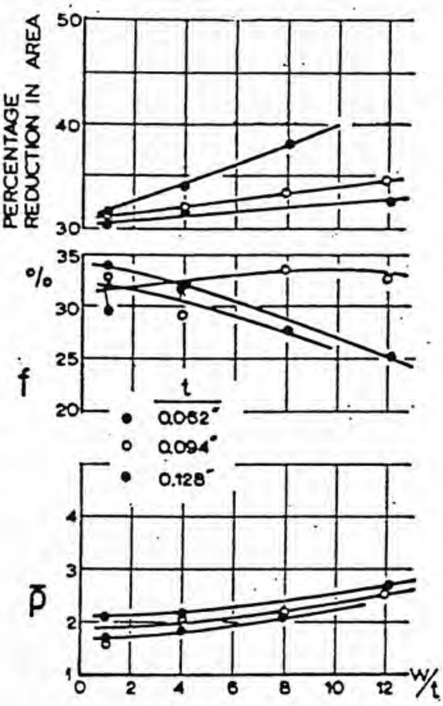


FIG. 14—Variation in the length of neck, factor of uniformity, and percentage reduction in area with respect to width-to-thickness ratio (mild steel).

pendently of each other. In other words, it was shown that each of them was effectual, and now it has been shown that none of them is redundant.

These results show the limitations of the empirical relations sometimes found between the percentage elongation and the percentage reduction in area [19,24].

The effects of shape and size of specimens—Specimens of the following range of size and shape of cross section were

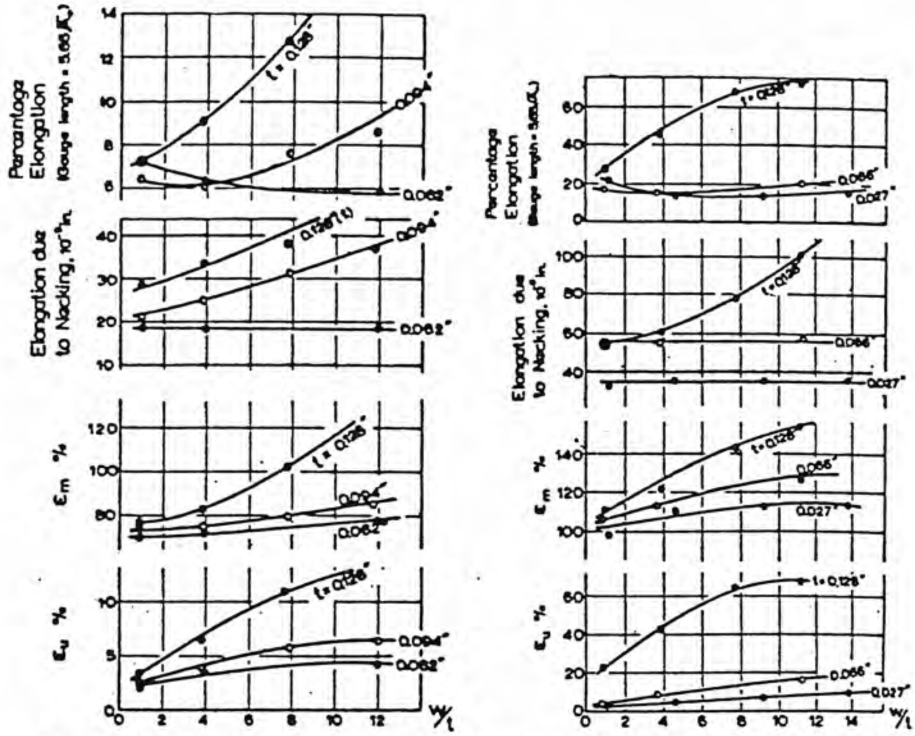


FIG. 15—Variation in ϵ_u, ϵ_m, percentage elongation and the elongation due to necking with respect to width-to-thickness ratio, (*left*) mild steel and (*right*) aluminum.

made of a mild steel and an aluminum alloy according to British Standards 18:1956.

	Thickness, in.	Cross Section $\frac{\text{width}}{\text{thickness}}$
Mild steel.......	$\frac{1}{16}$ to $\frac{1}{8}$	1 to 12
Aluminum alloy..	0.027 to $\frac{1}{8}$	1 to 12

(The thickness of 0.027 in. was due to the slight reduction from $\frac{1}{32}$ in. when the surface was polished for the photographic work.) In most cases, two specimens of the same size and shape were tested and the average measurements were taken. The results for the mild steel and the aluminum alloy are shown in Figs. 14–16. It should be added that rolled materials of different thicknesses were used, hence the curves in Figs. 14–16 represent, not only the effects of size, but those of metallurgical variation as well.

When the width-to-thickness ratio changed from 4 to 8, the fractured surfaces in both mild steel and aluminum specimens changed from shear (Fig. 17*a*) to tear (Fig. 17*c*). In all cases, however, the strain was measured along the centerline on the broader side of the specimen. (The strain varies considerably across the width of the specimen as shown in Fig. 18, which represents the axial strain at various points along a line *AB* (*inset*) on the undeformed specimen, near but not at the fracture.) In most of the specimens in this series of tests, the neck consisted of a contraction in thickness rather

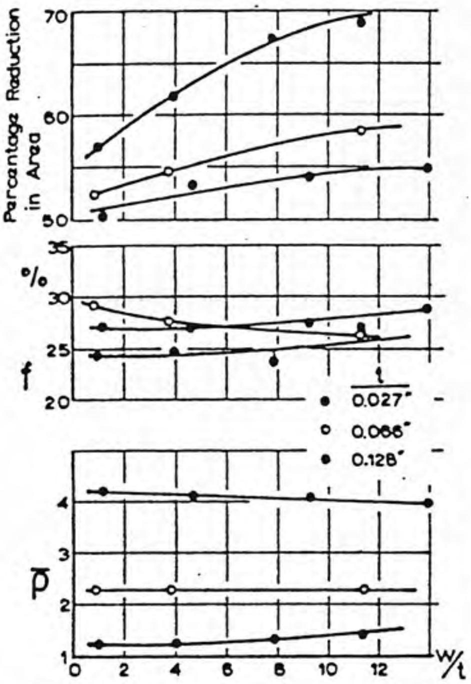


FIG. 16—Variation in the length of neck, factor of uniformity, and percentage reduction in area with respect to width-to-thickness ratio (aluminum).

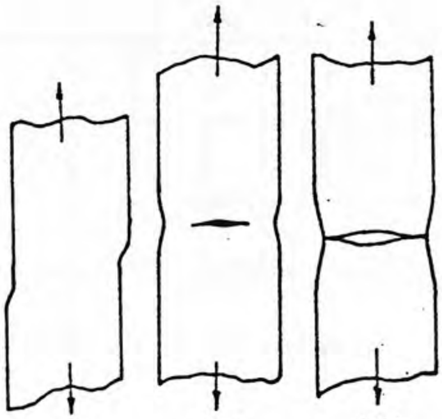


FIG. 17—Two types of failure in the tension specimen.

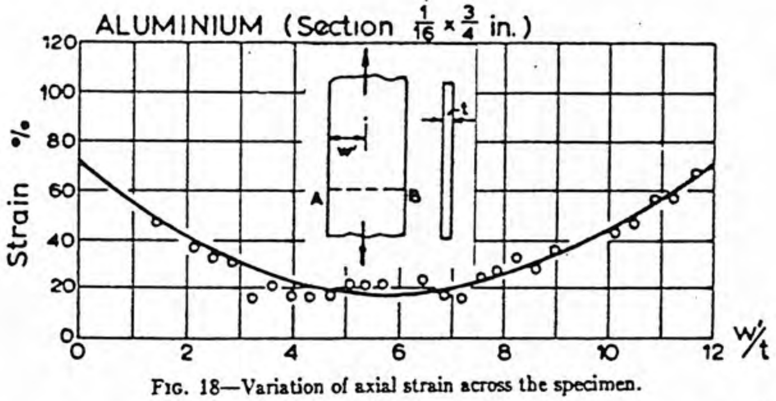


FIG. 18—Variation of axial strain across the specimen.

than in width, hence, the nondimensional length of the neck used in Figs. 14 and 16 ($\bar{p}$, equal to λ/t) is based on the thickness rather than on the equivalent diameter.

As can be seen in Figs. 14–16, the effects of both the size and the shape are, in general, more pronounced in aluminum. For some unknown reason, the elongation due to necking, repre-

MILD STEEL

	Strain Rates, in/in/sec (A)	(B)	(C)
	11.0×10^{-4}	35.0	44.7
ε_u %	2.0	2.75	3.5
ρ	1.57	1.49	1.37
ε_m %	48	44	40
f %	29	37	37

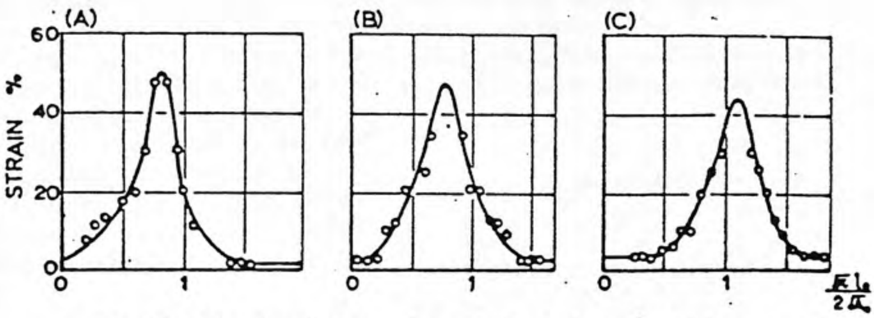


FIG. 19—Effect of strain rate on the strain distribution curve (mild steel).

ALUMINIUM

	Strain Rates, in/in/sec (A)	(B)	(C)
	11.0×10^{-4}	35.0	44.7
ε_u %	12.5	16.0	20.0
ρ	1.39	1.26	1.05
ε_m %	80	128	133
f %	31	27	22

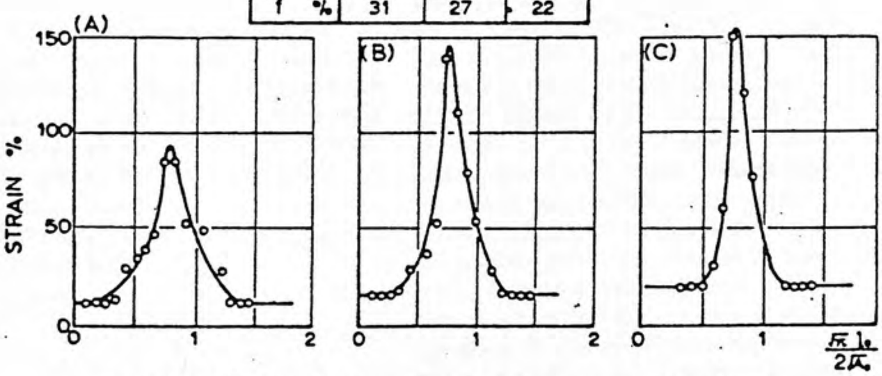


FIG. 20—Effect of strain rate on the strain distribution curve (aluminum).

sented by the cross-hatched area in Fig. 2, remains constant with respect to width-to-thickness ratio for all specimens not thicker than $\frac{1}{16}$ in. (Fig. 15).

The effects of speed of testing—The same steel and aluminum were used for investigating the effects of strain rate. The specimens were of $\frac{1}{4}$ by $\frac{1}{4}$ in. in cross section. For the lowest strain rate (*a*, Figs. 19 and 20), the universal testing machine was used and the average strain rate was determined from the interval between yield point and fracture. For higher strain rates (*b* and *c*, Figs. 19 and 20), the specimens were mounted on special guides and tested in an Izod testing machine, the average strain rate being calculated from the total elongation, the parallel length, and the velocities of the hammer before and after the impact.

By Figs. 19 and 20, it appears that increased strain rate has opposite effects on the mild steel and aluminum in so far as the trends of ϵ_m and f are concerned. However, for conclusive results, further investigation with proper equipment for varying strain rates is required. The results are shown in Figs. 19 and 20 to demonstrate the application of the techniques rather than to present the properties of the materials.

Significance of ϵ_u, ρ, ϵ_m *and* f:

(1) The significance of ϵ_u can be discussed conveniently by plotting the nondimensional stress-strain curve of the material as in Fig. 21, where $\bar{\epsilon}$ is the natural strain at the smallest section, σ is the true stress (force per unit minimum section) and σ_o is the yield stress. Let F be the current load and A the current area, then

$$F = \sigma A$$

or

$$\ln F = \ln(\sigma/\sigma_o) + \ln A + \ln \sigma_o$$

However, $A/A_o = e^{-\bar{\epsilon}}$ by the incompressibility of the material, hence

$$\ln F = \ln(\sigma/\sigma_o) - \bar{\epsilon} + \ln(A_o\sigma_o)$$

which shows that, with $\ln(\sigma/\sigma_o)$ as ordinate and $\bar{\epsilon}$ as abscissa, the lines for constant loads (F) are straight lines inclined at 45 deg with the horizontal axis (Fig. 21). The origin corresponds to the yield point ($\sigma = \sigma_o$), F_o is the load producing the yield stress, and the stress-strain curve is OG.

As can be seen in Fig. 21, from O to G the load increases with increasing strain,

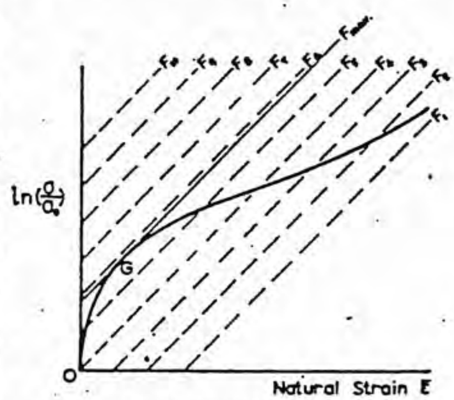


FIG. 21—Nondimensional stress-strain curve.

at G the load reaches its maximum (F_{max}) and beyond G the load decreases. Between O and G, the neck cannot develop, because the material at any local contraction, being stronger than the rest of the specimen, cannot contract further. Beyond G, the neck is bound to develop, because the material at any local contraction, being weaker than the rest of the specimen, must be further stretched. Thus, at maximum load (at G), the uniform strain also reaches its maximum. Therefore, the maximum uniform natural strain ($\bar{\epsilon}_0$) is the strain at which the slope of the nondimensional stress-strain curve (Fig. 21) is unity; in other words, when

$$\frac{d}{d\bar{\epsilon}}\left(\ln\frac{\sigma}{\sigma_o}\right) = 1, \quad \text{or} \quad \frac{d\sigma}{d\bar{\epsilon}} = \sigma$$

To convert natural strain into engineering strain, one may write

$$\epsilon_G = e^{\bar{\epsilon}_G} - 1$$

One may also say that the value of ϵ_u depends on the curvature of the nondimensional stress-strain curve (Fig. 21). If the curvature of curve OG is small and increases slowly towards G, then ϵ_u is large; otherwise, it is small.

(2) The quantity $(\epsilon_m + \epsilon_n)$, as it is measured in this investigation, corresponds to the surface strain when the surface of the material at the centerline begins to fracture (Fig. 17). Therefore, this quanity may be said to be the strain at fracture.

It should be noticed that the percentage reduction in area calculated from ϵ_m and ϵ_u, namely, $100 \times (\epsilon_m + \epsilon_u)/(1 + \epsilon_m + \epsilon_u)$, is not the same as the percentage reduction in area measured in the standard way, that is, by the minimum diameter. In most ductile materials, fracture starts at the center of the specimen and when the specimen breaks, there is a cavity in the material. Therefore, the percentage reduction in area as determined by the minimum diameter represents the strain at fracture only if the cavity is counted as solid material.

(3) Once the neck is formed, the material in it undergoes nonuniform deformation under complicated triaxial stresses. It can be seen from Fig. 2 that the length of the neck ρ depends on the extent of the plastic zone in the early stages of the neck formation. The factor of uniformity f depends partly on how fast the plastic zone contracts during the neck formation, and partly on the value of ϵ_m. The product $(f\rho\epsilon_m)$ represents the elongation due to necking (in equivalent diameters), and it constitutes a measure of the "accident insurance" in design based on ultimate strength, especially in structural members in which deformation causes stress relaxation.

Ductility Indexes Based on the Tension Test:

Perfect ductility, like perfect roundness, is easy to define; but degrees of ductility, like degrees of roundness, cannot be represented adequately by a single index. A perfectly ductile material is one which can be deformed infinitely without breaking. All materials are, of course, only partially ductile and partial ductility can take many different forms. It may be thought that the strain at fracture can be used as a simple ductility index, but it cannot be, because both the strain and the stress at fracture can be any of the many different types of triaxial stress and strain. Besides, in many engineering applications, the strain at fracture may not be important; rather, the worsening nonuniform deformation may determine the performance of the materials. As shown here, deformation can become nonuniform in many different ways.

Ductility indexes are used by engineers in the hope that they indicate the performance of engineering materials in load-bearing structures and manufacturing processes. Those who are familiar with the literature connected with testing materials are accustomed to the complaints that test results are not reliable guides in some particular applications [25]. These complaints are not surprising because ductility indexes, as Gillet [26] pointed out, are used for a great variety of purposes, and there are usually only two indexes to choose from—the percentage elongation and the percentage reduction in area. Strictly speaking, tension test results can predict only the performance of the materials in processes similar to the tension test, and the stresses and strains in most manufacturing processes are quite unlike those involved in the determination of percentage elongation and percentage reduction in area. The correspondence between tension test results and the performance in manufacturing processes is partly empirical, and partly due to the predominance of some aspect of ductility in the particular processes.

Ductility indexes being necessarily imperfect, too much faith in them is as dangerous as too little. Engineers cannot afford either to ignore or to trust them completely and their workable applications begin somewhere between credulity and skepticism. Such being the case, it may be desirable to supply the production engineers with several ductility indexes—each representing an aspect of imperfect ductility. In other words, though we cannot find "truer" ductility indexes, we can define some which are better differentiated than those in current use. If the different aspects of partial ductility are mixed together in a single index—as the uniform and local strains are mixed in different proportions in the percentage elongation—the index may become less useful because the significance of its constituent parts may be masked by each other. On the other hand, if several ductility indexes of purer significance are available, different engineers may choose what suits their particular purposes, or derive new indexes by combining them. A case in point is the apparently contradictory variations in ductility with respect to strain rate. In Austin and Steidel's results for titanium, the percentage reduction in area increases with strain rate but the percentage elongation decreases (Fig. 16, Ref. 27). And in Jones and Moore's results for hard-drawn copper, the former decreases and the latter increases with strain rate (Figs. 6 and 7, Ref. 28). In this investigation, opposite trends can also be seen in the curves for the percentage elongation and percentage reduction in area in Figs. 14–16. Such apparent contradictions, due to the mixture of several quantities in the two ductility indexes, pose the awkward question as to which of them represents "true" ductility.

Of course, engineers are more interested in a workable than in the "true" ductility index, and several indexes, each of a different significance, are more likely to be workable than one or two mixtures of them. In the four quantities defined in this paper, the maximum uniform strain (ϵ_u) marks the limit of plastic stability, the maximum strain due to neck-formation (ϵ_m) is the strain range between the threshold of instability and fracture, and the length of the neck (ρ) and the factor of uniformity (f) represent, respectively, the extent and the evenness of the nonuniform deformation. All four are dimensionless numbers, independent of the different units of length in different countries and none of them is encumbered by the arbitrary gage length on the effect of which so much has been written.

Needless to say, all the valid ductility indexes proposed in the past can be derived from the four quantities discussed above. The calculation of the four quantities from the dimensions of a fractured round tension specimen is shown in Appendix V.

Acknowledgment:

This investigation was carried out in the Department of Civil and Mechanical Engineering, University of Edinburgh. We gratefully acknowledge the encouragement of the late Professor R. N. Arnold, then Regius Professor of Engineering, and the help of the Imperial Chemical Industries, Ltd. and Colvilles, Ltd. in supplying the test materials.

APPENDIX I

IMPLICATIONS OF OLIVER'S ELONGATION FORMULA

Let

ϵ = engineering strain at any point along the specimen

l_o = distance from the fractured section (measured on the undeformed specimen)

l_g = gage length (= $2l_o$)

$$\text{Elongation} = 2\int_0^{l_g/2} \epsilon \, dl_o$$

Percentage elongation

$$= 100\,\frac{\text{elongation}}{\text{gage length}} = \frac{100}{l_o}\int_0^{l_g/2} \epsilon \, dl_o$$

According to Oliver,

$$\text{Percentage elongation} = \mu\left(\frac{\sqrt{A_o}}{l_g}\right)^{\alpha}$$

Hence

$$\frac{100}{l_o}\int_0^{l_g/2} \epsilon \, dl_o = \mu(\sqrt{A_o}/2l_o)^{\alpha}$$

Differentiating the last equation, we get

$$\epsilon = \frac{(1-\alpha)\mu(\sqrt{A_o})^{\alpha}}{100 \times 2^{\alpha} \times l_o^{\alpha}} = (\text{a constant})\,\frac{1}{l_o^{\alpha}}$$

For the various metals investigated by Oliver, α varied from 0.14 to 0.74 [18], hence the strain distribution curve (ϵ against l_o) is a hyperbola like one of those in Fig. 22.

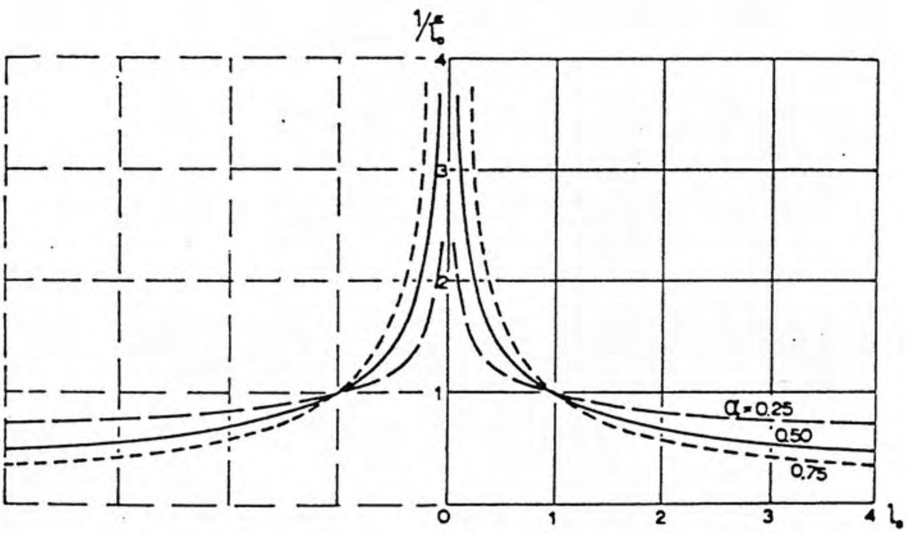


FIG. 22—Nature of the strain distribution curve according to Oliver.

APPENDIX II

REPRESENTATION OF THE STRAIN DISTRIBUTION CURVE

Strictly speaking, the four quantities ϵ_u, ρ, ϵ_m, and f, are not sufficient to represent the exact shape of the strain distribution curve. Thus, in Fig. 23, the areas under the dotted and the solid curves are the same, hence both curves satisfy the same set of values for ϵ_u, ρ, ϵ_m, and f. For precise representation, therefore, a fifth quantity, the maximum slope of the curve, is required.

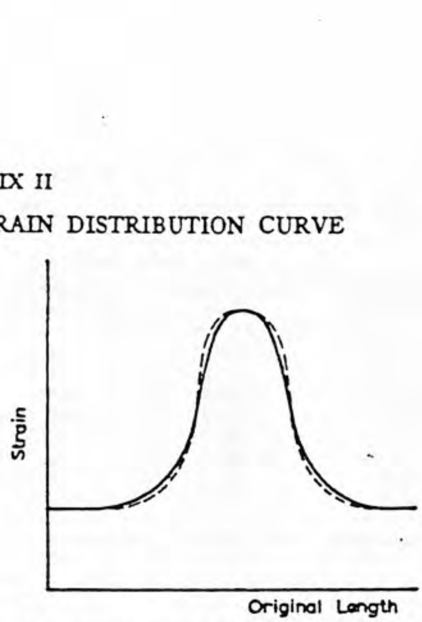


FIG. 23—Two strain distribution curves of the same factor of uniformity.

APPENDIX III

CHEMICAL COMPOSITION AND HEAT TREATMENT OF THE NON-FERROUS MATERIALS

Element	Composition, weight per cent				
	Copper	70-30 Brass	64-36 Brass	Alumbra	Phosphor Bronze
Copper........	balance	70.05[a]	64.6[a]	76.3[a]	balance
Tin...........	nd[b] <0.001	<0.001	<0.001	<0.001	5.5[a]
Lead..........	nd <0.002	nd <0.002	0.002/0.003	nd <0.002	0.005/0.007
Iron..........	nd <0.01	0.01/0.02	0.02/0.03	0.01/0.02	nd <0.01
Nickel........	<0.01	nd <0.01	nd <0.01	0.01/0.02	<0.01
Manganese....	<0.01	<0.01	<0.01	<0.01	<0.01
Aluminum.....	<0.01	<0.01	<0.01	1.8[a]	<0.01
Magnesium....	...	...	...	...	...
Silver.........	0.005/0.007	0.005/0.007	0.005/0.007	0.005/0.007	0.005/0.007
Antimony.....	nd <0.002	nd <0.002	nd <0.002	nd <0.002	nd <0.002
Bismuth.......	nd <0.001	nd <0.001	nd <0.001	nd <0.001	nd <0.001
Arsenic........	nd <0.006	nd <0.006	<0.006	<0.006	nd <0.006
Silicon........	nd <0.01	nd <0.01	nd <0.01	0.01/0.02	nd <0.01
Phosphorus....	<0.03	<0.03	<0.03	<0.03	0.03/0.05
Chromium.....	...	...	...	...	...
Titanium......	...	...	...	...	...
Oxygen.......	0.035[a]	...	...	...	...
Zinc..........	0.05/0.07	balance	balance	balance	0.1/0.2

[a] Chemical value.
[b] Not detected.

APPENDIX IV

CHEMICAL COMPOSITION AND HEAT TREATMENT OF THE STEELS

Element	Composition, weight per cent			
	Chromium Steel	Chromium Molybdenum Steel	Nickel Chrome Steel	Manganese Molybdenum Steel
Carbon.............	0.38	0.40	0.35	0.365
Silicon.............	0.29	0.27	0.28	0.21
Sulfur..............	0.043	0.016	0.014	0.028
Phosphorus.........	0.035	0.026	0.035	0.026
Manganese.........,	0.68	0.57	0.58	1.23
Nickel.............	0.47	0.21	1.49	0.09
Chromium..........	12.6	1.34	1.40	0.12
Molybdenum.......	...	0.65	0.20	0.45
Process.............	electrically melted	electrically melted	electrically melted	basic open hearth
Heat treatment.....	heated to 950 C; cooled in oil. heated to 750 C; cooled in air.	heated to 860 C; cooled in oil. heated to 640 C; cooled in air.	heated to 840 C; cooled in oil. heated to 610 C; cooled in air.	heated to 860 C; cooled in oil. heated to 610 C; cooled in air.

APPENDIX V

CALCULATION OF ϵ_u, ρ, ϵ_m AND f FROM DIMENSIONS OF A FRACTURED ROUND TENSION SPECIMEN

Let

d_o = original diameter,

d_u = diameter of tested specimen outside the neck,

d_m = diameter of the minimum section of the broken specimen,

l_g = gage length,

l_a = distance between the gage marks in the broken specimen joined together at the fracture, and

λ_a = length of the neck in the broken specimen joined together at the fracture (which can be measured by placing straight edges against the broken specimen).

Assuming that the gage length includes the whole neck ($l_a > \lambda_a$), we get

$$\epsilon_u = \frac{d_o^2 - d_u^2}{d_u^2}$$

$$\lambda = l_g - \frac{l_a - \lambda_a}{\epsilon_u} = l_g - \frac{d_u^2}{d_o^2 - d_u^2}(l_a - \lambda_a)$$

$$\rho = \frac{\sqrt{\pi}}{2\sqrt{A_o}}\left(l_g - \frac{l_a - \lambda_a}{\epsilon_u}\right)$$

$$= \frac{1}{d_o}\left[l_g - \frac{d_u^2}{d_o^2 - d_u^2}(l_a - \lambda_a)\right]$$

$$\epsilon_m = \frac{d_o^2}{d_m^2} - 1 - \epsilon_u$$

$$= \frac{d_o^2}{d_m^2} - 1 - \frac{d_o^2 - d_u^2}{d_u^2} = \left(\frac{1}{d_m^2} - \frac{1}{d_u^2}\right)d_o^2$$

$$f = \frac{l_a - \epsilon_u l_g}{\lambda \epsilon_m}$$

$$= \frac{d_m^2}{d_o^2}\cdot\frac{d_o^2 - d_u^2}{d_u^2 - d_m^2}\cdot\frac{l_a d_u^2 - l_g(d_o^2 - d_u^2)}{l_g(d_o^2 - d_u^2) - (l_a - \lambda_a)d_u^2}$$

Note that the value of ϵ_m so calculated is not the same as that measured by the technique used in this investigation, unless the hole at the fractured section is negligibly small, as in some mild steel specimens.

REFERENCES

[1] W. Fairbairn, "An Experimental Inquiry into the Strength of Wrought Iron Plates and their Riveted Joints as Applied to Ship Building and Vessels Exposed to Severe Strains," *Philosophical Transactions*, Part I, 1850, pp. 677–725.

[2] R. Mallet, "On the Coefficients T_e and T_r of Elasticity and of Rupture in Wrought Iron in Relation to the Volume of the Metallic Mass, its Metallurgic Treatment and the Axial Direction of its Constituent Crystals," *Proceedings*, Institute Civil Engrs., Vol. 18, 1858, pp. 296–348.

[3] D. Kirkaldy, *Results of an Experimental Enquiry into the Tensile Strength and Other Properties of Various Kinds of Wrought-Iron and Steel*, second edition, printed for and sold by the author, Glasgow, 1864.

[4] W. Hackney, "The Manufacture of Steel," *Proceedings*, Institute Civil Engrs., Vol. 42, 1875, pp. 2–68; Discussion, pp. 76–128.

[5] M. J. Barba, "Résistance des matériaux," *Mémoires de la Société des Ingénieurs Civils*, Part I, 1880, p. 682.

[6] W. Hackney, "The Adoption of Standard Form of Test-Pieces for Bars and Plates," *Proceedings*, Institute Civil Engrs., Vol. 76, 1883, pp. 70–158.

[7] L. von Tetmajer, "Methoden und Resultate der Prüfung der Festigkeitsverhältnisse des Eisens und anderer Metalle," *Mittheilungen der Anstalt zur Prüfung von Baumaterialien am eidgen*, Polytechnikum, Zürich, Part IV, 1890.

[8] W. C. Unwin, "Tensile Tests of Mild Steel and the Relation of Elongation to the Size of the Test Bar," *Proceedings*, Institute Civil Engrs., Vol. 155, 1903, pp. 170–292.

[9] G. Sachs, (V.D.I., Aug. 28, 1925) quoted in *The Metallurgist*, Vol. 2, Sept., 1926, pp. 138–139.

[10] C. W. MacGregor, "The True Stress-Strain Tension Test—Its Role in Modern Materials Testing," *Journal of Franklin Institute*, Vol. 238 (2 and 3), Aug. and Sept., 1944, pp. 111–135, 159–176.

[11] G. P. Zaitsev, "The Problem of Finding the Plasticity and Strength Constants of Metals," *The Physics of Metals and Metallography*, Vol. 5 (1), 1957, pp. 89–96.

[12] E. Maitland, "The Treatment of Gun Steel," *Proceedings*, Institute Civil Engrs., Vol. 89, 1886–1887, pp. 114–240. Discussion by D. Adamson, pp. 167–170.

[13] *Mittheilungen des mechanisch-techn.* Laboratoriums der Königlichen technischen Hochschule, München, Part VI, quoted on p. 123, Ref *15*.

[14] A. Martens, "Ergebnisse von Versuchen über die Festigkeitseigenschaften von Kupfer," *Mittheilungen aus den königlichen technischen Versuchsanstalten*, Berlin, Part III, 1894, Fig. 36.

[15] A. Martens, *Handbook for Testing Materials*, Translated by G. C. Henning, first edition, John Wiley & Sons, Inc., New York, 1899.

[16] W. C. Unwin, *The Testing of Materials of Construction*, third edition, Longmans, Green & Co., London, 1910. Chapter IV.

[17] O. Tiedemann, "Zeitschrift für Metallkunde," p. 299, June 1927, quoted in *The Metallurgist*, Vol. 3, Oct., 1927, pp. 149–150.

[18] D. A. Oliver, "Proposed New Criteria of Ductility from a New Law connecting the Percentage Elongation with Size of Test Piece," *Proceedings*, Institute Mechanical Engrs., Vol. 2, 1928, pp. 827–864.

[19] T. Simizu, M. Ide, "On the Relation between the Percentage of Elongation and Area Contraction in Tensile Test," *Transactions*, Society Mechanical Engrs., Japan, Vol. 23 (131), July, 1957, pp. 518–522.

[20] W. H. Brown and M. H. Jones, "Strain Analysis by Photogrid Method," *The Iron Age*, Vol. 158, Sept., 1946, pp. 50–55.

[21] F. Hewlett, "Photogrids Measuring Flow and Stretch in Metal Specimens and Parts," *Aircraft Production*, Vol. 7 (9), Sept., 1945, pp. 425–427.

[22] G. A. Brewer and R. B. Glassco, "Determination of Strain Distribution by the Photo-grid Process," *Journal of the Aeronautical Sciences*, Vol. 9 (1), Nov., 1941, pp. 1–7.

[23] G. A. Brewer, "Photogrid Process applied to Problems in Machine Design," *Machine Design*, Vol. 19, Feb., 1947, pp. 120–122.

[24] A. C. Elliott, "The Relation of the Constants of the Elongation Equation to Contraction of Area," *Proceedings*, Institute Civil Engrs., Vol. 158, 1904, pp. 390–396.

[25] J. B. Caine, "Case against Tension Test," *Foundry*, Vol. 85 (11), Nov., 1957, pp. 86–92; Vol. 86 (9), September, 1958, pp. 78–85.

[26] H. W. Gillet, "The Limited Significance of

the Ductility Features of the Tension Test," *Proceedings*, Am. Soc. Testing Mats., Vol. 40, 1940, pp. 551–578.

[27] A. L. Austin and R. F. Steidel, "The Tensile Properties of Some Engineering Materials at High Rates of Strain," *Proceedings*, Am. Soc. Testing Mats., Vol. 59, 1959, pp. 1292–1319.

[28] P. G. Jones and H. F. Moore, "An Investigation of the Effect of Rate of Strain on the Results of Tension Test of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 40, 1940, pp. 610–624.

[29] A. Nadai, *Theory of Flow and Fracture of Solids*, Vol. I, 2nd ed., McGraw-Hill, London. 1950.

by Dr. G. S. Littlejohn, Ground Engineering Division, Cementation Co. Ltd.

Ground anchors in civil engineering: 2

Recent developments in Ground Anchor Construction

During recent years there has been an increasing demand for a means of anchoring both temporary and permanent structures, due primarily to the increasing tendency to design buildings with a number of basement floors. This makes it necessary to carry out very deep excavations in both soil and rock, the floor of the excavation being often at considerable depth below the foundations of the neighbouring properties (see Fig. 1.). In such cases, shoring of the piling in the traditional way by means of interior strutting is unattractive since the working space available is often severely limited.

It is the existence of this type of problem in connection with the shoring of sheet piling and support walls, together with the anchoring of foundations, masts and towers that has brought about the development of simple flexible methods of making anchorages in gravels, sands, clays and more recently chalk.

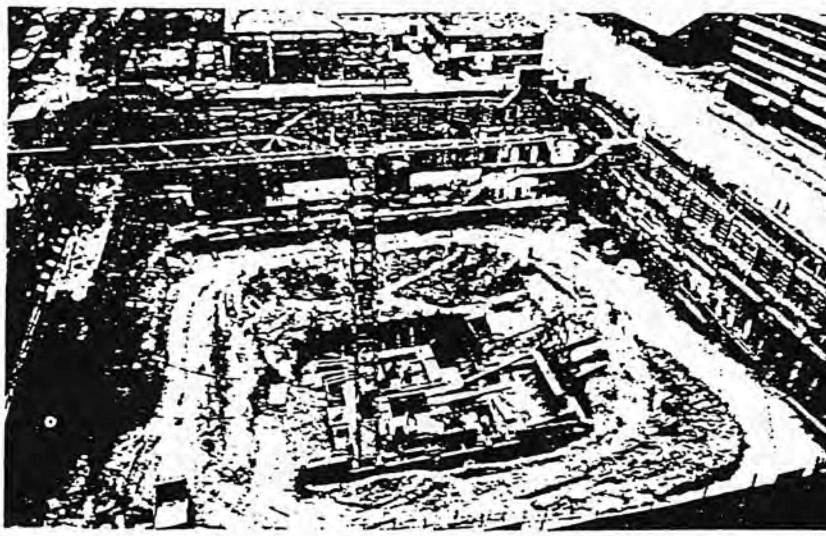


Fig. 1: Basement excavation for Rand Daily Mail Building with piles anchored by 300 steel cables, each of 40 ton load.

1. *ANCHORS IN GRAVEL*

Alluvium anchors can be formed in any load bearing ground down to and including clay but the highest resistances to withdrawal are obtained in gravels and coarse sands where the permeability is not less than 10^{-2} cm/sec. In homogenous ground of this type, anchors are designed to resist safe working loads of up to 80 tons.

Construction

The method which is employed for anchorages in gravel entails a number of working operations as follows:—

(a) Driving a lining tube, 2 in—4 in (5—10 cm) nominal diameter, through the ground to the desired depth (see Stages 1—3, Fig. 2).

(b) Homing of the cable which consists of high tensile steel strands or wires (see Stage 4).

(c) Pressure injection (grouting of hole with neat cement and water) whilst withdrawal of the lining tube takes place (see Stages 5—9).

Grout W/C ratios of 0.5 and 0.65 are recommended for gravels and coarse sands respectively, and the injection pressure may vary from 5 to 150 lb/in² (0.35 to 10.5 kg/cm²) depending on the permeability of the ground. In the case of temporary works, where only a minimum time is required between anchoring and tensioning a high alumina cement is used which enables the cable to be stressed 24 hours after construction.

(d) Withdrawal of the lining tube completely (see Stage 10).

(e) Following hardening of the grout the cable is stressed to the desired load (see Stage 11).

Thus the anchorage is based on grout injection and consists basically of a cable which is bonded into a grouted zone of alluvium and is known as the fixed anchorage. The rest of the cable is encased in a protective sheath to prevent the cable from coming into contact with the surrounding ground and also to provide a safeguard against corrosion.

A special VSL movable anchorage is

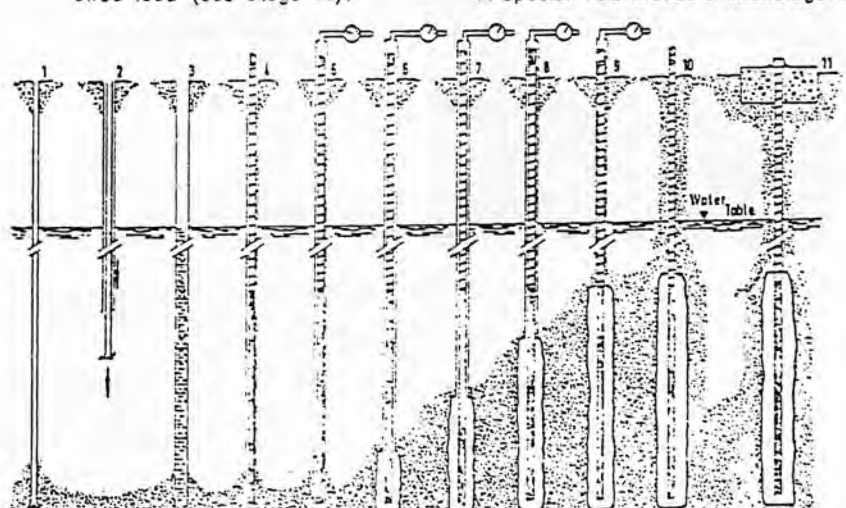


Fig. 2: Stages in the construction of an alluvium anchorage.

ısed for stressing, which allows the ıost-tensioning to be carried out in any equired number of stages and at any ime after construction. This post ten- ioning pre tests the anchor, thus en- uring its safety.

afety.

The importance of this feature in pre- tressed ground anchors cannot be ver-emphasised and the following otes are included to define the term Safety" in more detail.

The notation below is used by the ementation Co. Ltd. for Ground An- hors.

Tb=minimum breaking load of the steel cable.

Tf=failure load of the grouted fixed anchor.

Tt=maximum allowable test load to which an anchor can be temporarily subjected in order to check its capacity.

Tw=the working load of the anchor.

Sb=Factor of Safety against breaking of cable.

Sf=Factor of Safety against bond failure between grouted fixed anchor and adjacent ground.

The measured Factor of Safety gainst cable failure (Sb=1.5 (or great-). The careful checks carried out on l the tensile steel and anchorage com- ɔnents employed guarantees this safe- for each ground anchor.

The measured Factor of Safety jainst withdrawal of the complete ɛed anchor (Sf=1.5 to 3.0) is evalu- ed on site by carrying out a tempor- y test loading. The allowable test load 't) however is limited by the elastic nit of the steel cable and consequent- Sf=Tt/Tw (or greater). This method testing takes into account the fact at the local ground conditions in the ed anchor zone, which are of the most importance, often vary consider- ly. It normally establishes, however, ıly very small minimum values for the fety against fixed anchorage with- awal.

In homogeneous ground therefore, other form of check is used where a ıt anchor with an overdesigned cable pulled to failure, to establish the imate resistance to withdrawal of fixed anchor. In this way the opti- ım fixed anchor length for the remain- anchors can be determined. If how- er, the jacking capacity is insufficient fail a typical working anchor, then a ıt anchor is constructed with a re- ced fixed anchor length whose fail- load, Tf is expected to be less than test load Tt.

t should be noted that the Factors Safety referred to in this section are asured values and consequently ould not be compared with the larger ety Factors often employed by ndation engineers to take account situations which defy calculation.

istance to Withdrawal

s already indicated, working loads up to 80 tons can be attained in vels, and test anchors constructed depths of 50 ft. (15.2 m) below und surface have mobilised maxi- m resistances to withdrawal of 200 tons, when pulled to failure.

Typical ground anchor details to produce these high resistances are as follows:—

Total depth of anchorage=50 ft (15.2 m)

Length of fixed anchor=12 ft (3.6 m)

Effective diameter of fixed anchor =16 in (40.6 cm)

Quantity of cement injected=6 cwt (305 kg)

Angle of internal friction (ϕ)=40 deg.

As a result of testing anchorages with different fixed anchor lengths it may be concluded that the tensile force is mainly transferred to the ground by skin friction, and the following empirical rule has been established for the calculation of ultimate resistance to withdrawal (Tf) of anchors constructed in coarse sands or gravels.

$$Tf = L.N \tan \phi$$

where

L=Length of fixed anchor (ft) and N=12—16 tons per foot.

A recent example of a successful contract, carried out by Losinger & Co., Berne, is shown in Fig 3, where a total of 111 temporary anchors of 65 ton capacity (Sf=3) were installed in semi-coarse gravelly ground.

A reinforced concrete retaining wall had been constructed by the E.L.S.E. (slurry trench) method along one side of the site of an underground car park at the Berne Town Hall. The wall is 380 ft (115.8 m) long, 50 ft (15.2 m) high and 32 in (80 cm) thick and is very close to an existing line of buildings. The anchors were formed in the alluvium beneath these buildings.

2. *ANCHORAGES IN SAND.*

(a) Alluvium Anchors using Cement Grout

Construction.

When the standard procedure already described for gravels is adopted in fine to medium sized sands, the fixed anchor formed consists of a relatively smooth grout cylinder (see Figs 4 and 5) since the sand does not allow permeation of the dilute cement grout. The diameter of the fixed anchor depends on the size of casing and the injection pressure employed, and in compact medium sand with an injection pressure of 75 lb/in², (5.2 kg/cm²) the diameter of the fixed anchor will vary from 4 in (10.1 cm) to 8 in (20.3 cm) for 2 in (5.08 cm) and 4 in (10.1 cm) casing, respectively.

Resistance to Withdrawal

Typical working loads (Sf=1.5) for anchors of this type are illustrated in Table 1.

From this table it can be observed that relatively low capacity anchors are formed in fine cohesionless material using cement grout, and since underreaming is not practical, especially under the water table, the loading capa-

Above—Fig. 3: Diaphragm wall for underground car park in Berne with 111 temporary alluvium anchors. Below—Fig. 4: Grout cylinder formed in sand.

Table 1 Anchor Detail	Diameter of Casing 4 in (10.1 cm)	2 in (5.08 cm)
Depth of anchorage	30 ft (9.1 m)	30 ft (9.1 m)
Length of fixed anchor	12 ft (3.6 m)	12 ft (3.6 m)
Effective diameter of fixed anchor	8 in (20.3 cm)	4 in (10.1 cm)
Quantity of cement injected	4.5 cwt (228 kg)	2 cwt (101.6 kg)
Angle of internal friction (ϕ)	35 deg	35 deg
Working Load (Sf=1.5)	25 tons	10 tons

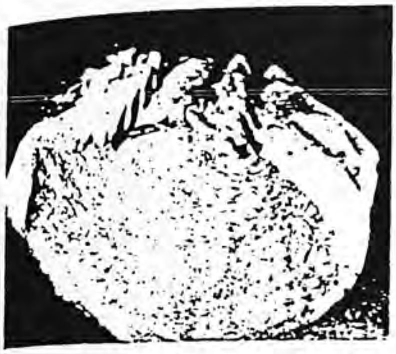


Fig. 5: Grout sample taken from fixed anchorage formed in compact sand.

city can only be improved by increasing the overall depth of the anchor.

(b) Alluvium Anchors using Chemical Grout.

In compact fine sands which do not allow permeation of dilute cement grout and which cannot be underreamed, high capacity anchors can be formed at relatively shallow depths by the use of highly penetrating epoxy resin grout. These grouts, only recently developed, have very low viscosities (20 cp at 20 deg C) and are formulated for use in formations of low permeability (10^{-2} to 10^{-4} cm/sec) under both saturated and dry conditions.

The grout does not fill the voids of the soil with a gel but deposits from a solution a resin which drains to the contact points between particles and sticks them together. Thus the grout imparts high strength by adhesion to the ground to yield fixed anchor zones having unconfined compressive strengths of the order of 1000—5000 lb/in² (70.3 kg—351.5 kg/cm²)

Construction

The construction stages for epoxy resin anchorages are identical to those already described in Section 1, except for Stage (C) where the grouting technique required is more sophisticated.

Grouting of Fixed Anchor

Prior to the grout injection, a water test is carried out in the hole to evaluate the ground permeability. Following

Fig. 6: Spherical chemical grout anchor in homogenous soil.

this operation, a flushing fluid is injected at low pressure — 15 lb/in² (1.05 kg/cm²)—to displace the void water in the ground surrounding the injection cell, whilst at the same time it provides a suitable medium for deposition to occur. This flushing fluid may not always be necessary but it provides the best known conditions for the formation of a consolidated system. Without interruption to the flow, the resin grout (basically a diluted resin-hardener system) is then switched into the circuit, and sufficient quantity injected — at 25 lb/in² (1.75 kg/cm²) pressure approx.—to produce the required geometry of fixed anchor.

The shape of the fixed anchor depends on the homogeniety and permeability of the ground but in a reasonably homogeneous soil the shape would approximate to a sphere (see Fig 6)

Resistance to Withdrawal

At Stevenston in Scotland, anchorages of this type were formed in compact fine to medium sized sands (ϕ=35—39 deg), at very shallow depths, and high resistances to withdrawal were produced, as shown in Figs 7 and 8.

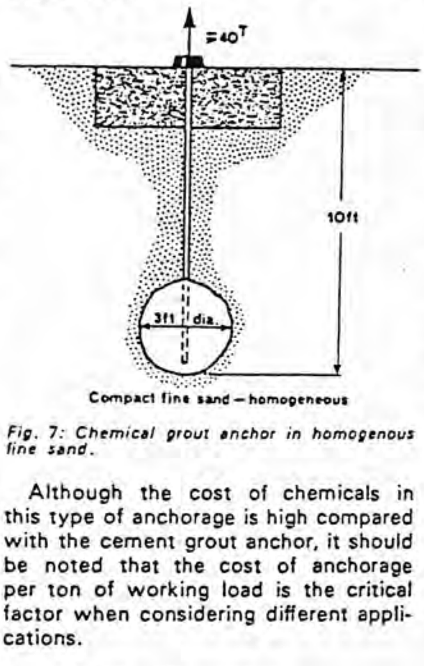
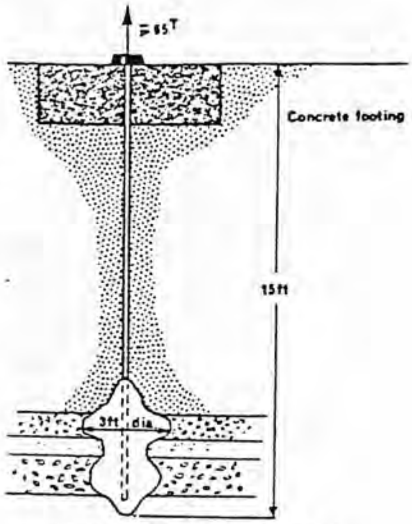


Fig. 7: Chemical grout anchor in homogenous fine sand.

Although the cost of chemicals in this type of anchorage is high compared with the cement grout anchor, it should be noted that the cost of anchorage per ton of working load is the critical factor when considering different applications.

3. *ANCHORAGES IN CLAY*

These anchorages are normally de-



Compact fine sand – slightly stratified

Fig. 8: Chemical grout anchor in stratified fine sand.

signed to carry safe working loads of up to 40 tons, and are constructed in stiff to very stiff clays, cohesion= 2000 lb/ft² (97648 kg/m²) (or greater). Originally the technique of anchoring was simply to auger a 4 in (10 cm) hole to the required depth in the clay, and then grout the cable into the fixed anchor zone using a tremie pipe. Anchorages of this type however, are of low capacity since an adhesion of only (0.3—0.35) C may be mobilised at the grout-clay interface of the fixed anchor. Thus a fixed anchor 30 ft (9.1 m) long and 4 in (10 cm) diameter, constructed in London Clay (C=3500 lb/ft²) (17088 kg/m²) may only develop 15 tons at failure.

In view of this situation, the following construction methods have been employed to increase the fixed anchor dimensions whilst maintaining a nominal 4 in (10 cm) borehole for the remainder of the anchorage.

a) Underground craters using explosives
b) Under-reaming using an expanding bit; and
c) Driving irregular gravel into the clay adjacent to the fixed anchor.

a) Construction of clay anchors using explosives

Extremely interesting results have been obtained using this technique and high loading capacities have been de-

Table 2	Anchorage No.	
Anchor Details	1	2
Depth of anchorage	30 ft (9.1 m)	30 ft (9.1 m)
Wt. of Gelignite	5 lb (2.2 kg)	2.5 lb (1.1 kg)
Volume of chamber blown	35 ft³ (.99 m³)	17 ft³ (.48 m³)
Length of fixed anchor zone	4.25 ft (1.3 m)	4 ft (1.2 m)
Effective diameter of fixed anchor.	3.25 ft (.96 m)	2.33 ft (.71 m)
Cohesion of clay	3200 lb/ft² (15624 kg/m²)	3200 lb/ft² (15624 kg/m²)
Maximum Test Load (Tt)	65 tons	55 tons

:loped at nominal depths. The con- ruction procedure is as follows:

1. Auger 4 in (10 cm) dia. hole to required depth.
2. Place explosive charge at bottom of borehole.
3. Fill borehole with compacted sand.
4. Detonate charge (Wt of gelignite=1—5 lb (.45—2.3 kg) depending on size of fixed anchor required.
5. Home cable.
6. Grout fixed anchor chamber using tremie pipe (W/C of grout=0.45)

:sistance to Withdrawal

Vertical anchorages of this type ve been successfully constructed to rry working loads of 50 tons on an perimental site at Herne Bay, Kent, d in all cases these loads were sus- ined for two to three months and the tal upward movement of the fixed ichor was less than ¼ in (6.3 mm). ible 2 illustrates the anchor dimen- ons obtained using different explosive arges.

Although high loading capacities may achieved using the technique des- ibed, the blasting operation and asso- ated vibrations may well restrict the plication to open sites. This is ex- emely important in the case of clay ice the amplitude of the seismic dis- rbance depends on the resistance of e ground to distortion. Clay has a wer resistance to stress than rock and vibrates, though with a low frequen- , at a higher amplitude for a given ergy input.

) Construction of clay anchors using an under-reamer.

The basic idea in this method is to ll the hole to the depth at which is intended to start under-reaming, d then instal the under-reaming tool. is is rotated whilst air is blown down rods and through the tool, and it ts its own hole to a larger diameter the fixed anchor zone, than the lled hole. The under-reamer devel- ed by the Cementation Co. Ltd, re- ires a 3½ in (8.8 cm) hole and can pand out to 9 in (22.8 cm) depend- on the size of cutters that are made. r larger sizes, it has been found that amount of air which can be passed wn the rods is insufficient to clear cuttings from the hole.

sistance to Withdrawal

At Westfield Properties in Durban, uth Africa, it was required that a ill, 64 ft (19.5 m) x 8 ft (2.4 m) high tied back to prevent disturbance to adjacent building whilst the wall was epened by about 12 ft (3.6 m) and derpinned. The ground consisted of zone of very fine sand underlain by ff saturated clay and a series of sand chors and clay anchors were installed indicated in Figure 9. Sand anchors re constructed above a fluctuating ter. table, and clay anchors below. Working loads of up to 34 tons f=2 to 2.5) were applied to fixed

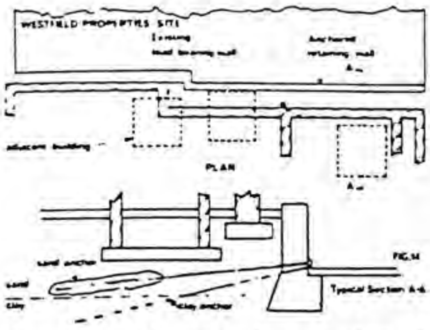


Fig. 9: Use of sand and clay anchors at Westfield Properties site in Durban.

anchors [length=12 ft (3.6 m) diameter=9 in (22.8 cm)] in the saturated clay, but with approximately 16 ft (4.9 m) of cover.

c) Construction of clay anchors using irregular gravel

This simple and flexible method may be employed in a wide range of clays, down to consistencies where under-reaming may not be practical.

After the hole has been drilled and cased to the required depth, irregular fine to medium sized gravel is injected into the hole as the casing is withdrawn the fixed anchor length. Following this stage a smaller casing, fitted with a non-recoverable point, is driven by percussion through the gravel, thus forcing it to penetrate the surrounding clay. The cable is then homed, the point is displaced and the gravel injected with neat cement grout, as the smaller casing is withdrawn the fixed anchor length. When the injection is complete both casings are removed completely from the hole.

Resistance to Withdrawal

In stiff clays (C=2000—3000 lb/ft²) (9764—14647 kg/m²) anchors have been constructed, using this technique, to carry safe working loads of 50 tons (Sf=1.5) at depths of 30 ft (9.1 m). Fixed anchor lengths are normally 12 ft (3.6 m) and the effective diameter of the grouted gravel varies from 5 in (12.7 cm) to 8 in (20.3 cm).

As already indicated, the local ground conditions in the fixed anchor zone are all important, and it should be noted that stiff clays often contain weak zones due to the presence of fissures or sand lenses, which may significantly reduce the anchorage capacity. For this reason site pull-out tests are recommended to determine the actual Factor of Safety of the anchorage design.

4. ANCHORAGES IN CHALK.

Although anchor trials have been carried out as far back as 1955 to study the resistance to withdrawal and creep of cables grouted into stiff chalk, it is only recently that the opportunity to construct chalk anchors for retaining walls has presented itself.

Construction.

The construction stages now employed are as follows:

a) Drive a lining tube, 2—4 in (5—10.1 cm) nominal diameter, through the overburden and at least 2 ft into the chalk. Drill beyond this point to a depth where the fixed anchor can be formed in a stable zone of chalk, outside the possible influence of the excavation.
b) Water test borehole to determine severity of fissuring and stabilise hole if necessary, using weak cement grout placed by tremie.
c) Redrill borehole, 12 hours after stabilisation, and repeat water tests.
d) Following a successful water test, home cable.
e) Inject cement grout (W/C=0.5) into borehole using a tremie pipe, and subsequently remove this pipe and pump in additional grout at low pressures (30 lb/in² approx.) (2.10 kg/cm²)
f) On completion of the grout stage i.e. when further grout cannot be injected at 30 lb/in² (2.10 kg/cm²) withdraw casing from borehole. (Normally, ⅓—½ cwt (17—25 kg) of cement is injected per foot run of anchor, but experiments in stiff chalk at Ramsgate have indicated that the cement consumption may rise to 2 cwt (102 kg) per foot run).

Resistance to Withdrawal

At the Reading Inner Distribution Road, chalk anchors were installed to tie back a temporary sheet-piled retaining wall nearly 30 ft (9.1 m) high (see Figure 10). Site boreholes show in general that below 10 ft (3.04 m) of made ground, a clayey sandy gravel approximately 6 ft (1.8 m) thick overlies 13 ft (3.9 m) of dense sandy gravel. This material is underlain by a redeposited stiff rubbly chalk which changes with depth to a stiff/very stiff chalk.

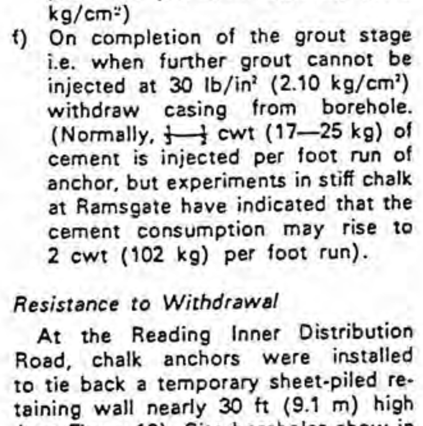


Fig. 10: Reading contract — the anchorage illustrated is capable of resisting a working load of 65 tons (Sf=2) when formed in sound "rock" chalk.

Approximately 80 anchors were successfully constructed in the upper zone of the chalk to resist working loads of 50—80 tons (Sf=1.5 to 2) and this work is particularly significant because the upper chalk layers were heavily fractured with softening along the fissures.

Since this upper chalk provides most of the problems of bearing capacity

continued at foot of page 46

GROUND ANCHORS . . . *continued from page 36*

and settlement of foundations on chalk, the opportunity was taken to study its engineering behaviour. Test anchors were pulled to failure and it was established that the ultimate resistance to withdrawal, due primarily to skin friction, varied from 2.8 to 7.5 tons/ft^2 (28 to 82 tons/m^2) of grouted fixed anchor (cf 8 to 10 tons/ft^2 (87 to 109 tons/m^2) of fixed anchor in Ramsgate chalk). In this range the consistency of the chalk in the fixed anchor zones at Reading changed from rubbly chalk with soft zones (S.P.T.=30 blows/ft) to unfissured 'rock' chalk (S.P.T.=80 blows/ft).

Creep tests are also being carried out on the working anchors over a period of six months to determine the loss of prestress, if any, due to fixed anchor movement under continuous load. After three months the results indicate that the working loads have been sustained, and the main proportion of the relaxation is probably due to cable extension. At Reading, allowance was made for loss of prestress by post-tensioning each anchor to working load plus 15 per cent.

In conclusion, it is considered that this type of field data may help to optimise the factors affecting anchorage design and construction and certainly the type of experience gained at Reading will be of value in cliff stabilisation work on the South Coast of England.

Retaining wall tie-backs

T H Hanna University of Sheffield and **G S Littlejohn** Cementation Ground Engineering Ltd

Recent developments in earth support methods mean that much better efficiency can now be achieved by using prestressing techniques for supporting braced cuts. The second half of this article will appear next month

In construction of retaining walls a balance between safety, functional performance and minimum cost must always be a prime consideration for the engineer. Responsibility is heightened in today's congested sites on which buildings often include several basement floors. In the light of factors like these, it is essential to have a simple and flexible system which can accommodate a wide range of structural and ground conditions.

Over the years many methods of temporary and permanent earth support have been developed. New techniques are constantly being applied to keep pace with the increased efficiency of modern construction. It is now common to have several site activities in progress simultaneously—deep excavation, foundation construction and steelwork erection—but the optimised construction programme is complicated and compromise usually results.

In many cases it is possible to achieve appreciably increased efficiency by using prestressing techniques for supporting braced cuts, **1**. This method of wall support eliminates interior struts, which in turn brings quite large economic and constructional advantages. This is especially so in cramped excavations, in wide cuts or on sites where the contract programme calls for the use of efficient construction machinery.

During the late 1950s a number of engineers in several countries started to use a system of wall support whereby the earth loads against the wall are resisted by inclined preloaded anchors fixed in the retained soil mass, **1**. The system comprises three components—the wall, the anchors and the beams which distribute anchor forces to the wall member.

While the design of conventionally strutted excavations is relatively straightforward, the design of walls supported by prestressed and inclined ground anchors entails a fundamental difference because construction methods play a more significant part in determining the wall forces and displacements which result. The many construction variables possible, however, make a full discussion very complex. In the following study the problem has been simplified and only general trends are mentioned.

In assessing an engineering problem it is useful to establish the practical limits within which the problem lies. General details of walls which have been constructed and tied back using prestressed ground anchors are listed in **3**. In the present context, prestressed anchors are individually loaded to a predetermined design load value before the structure load is resisted by them. In many of the walls quoted in **3**, complete information is not given in the cited reference; data is also scarce on stresses in the wall and anchors and on resulting wall and ground displacements. An examination of **3** reveals that many types of wall have been successfully built in a wide range of soil conditions. The variation in anchor dimensions, loads, positions and inclinations reveals the differing design approaches currently being considered.

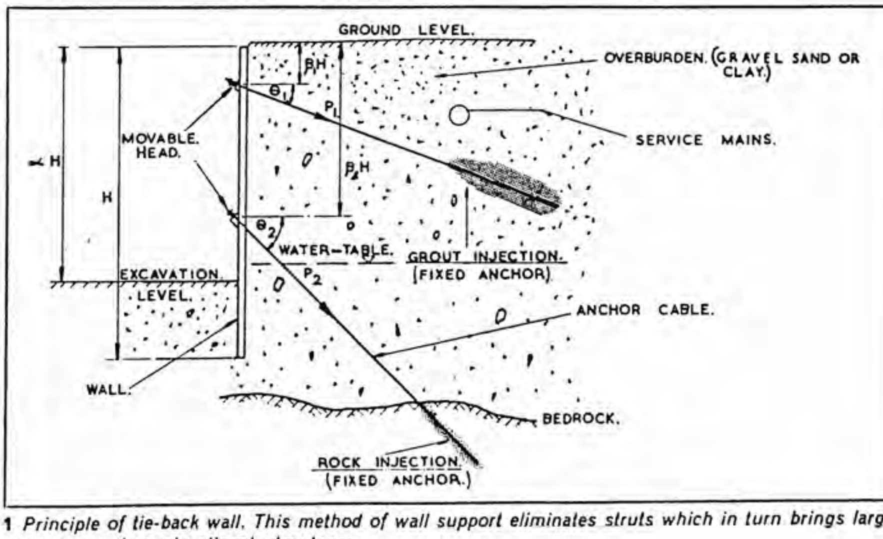
Several interrelated steps are required during a wall design. Of paramount importance is the site investigation which should provide the following basic information:

1 Soil succession.
2 Quantitative strength, compressibility and classification data.
3 Ground water conditions.
4 Expected changes in the above conditions due to climate, construction activity etc.

Because retaining walls can extend for thousands of yards it is imperative that general variations in soil properties across the site be determined, and that local variations within a few feet be known. Having investigated the site, procured representative samples and obtained realistic soil parameters, the design may proceed. Two problems predominate in design; the design of the wall system and the design of the anchor system. The problems are interrelated and from a design point of view should not be treated separately. Anchor spacing is dictated primarily by wall height, flexibility, anchor load and allowable wall stresses. From reported designs, β_1, the depth to the first anchor level varies widely and has been as large as 35% of the wall height H, see **3**.

Anchor inclination

Anchor inclination is kept small and ideally should be less than 20° to the horizontal. In many cases, however, this is not possible due to the proximity of adjacent foundations and to the advantages of anchor formation in rock strata. Inclinations as high as 65° have been used although values of 20°-45° are more usual. At present anchor design is semi-empirical and Littlejohn[22] discusses some of the approaches used in the design of alluvium anchors. Hanna[23] suggests that an extension of friction pile theory, suitably modified to allow for suction on the base and end bearing at the top of the fixed anchor, may be employed. The anchor geometry, the overall length and the position of the



1 *Principle of tie-back wall. This method of wall support eliminates struts which in turn brings large economic and constructional advantages* anchorage zone within the retained soil mass are the important variables. The size of the anchor zone is determined from anchor design load and soil conditions, but the present design approaches are based on experience. Much laboratory and field work is desirable to predict anchor capacity using soil mechanics principles.

Overall anchorage length is more difficult to design with precision. The most widely used method is that based on the work of Kranz[24] which is detailed in the German Recommendations for Waterfront Structures[25]. This method assumes that failure of the wall would take place along a 'failure plane' as shown in **2**. By considering the static forces acting on the wall-soil-anchor zone bounded by such an assumed surface of failure, the overall factor of safety of the wall-anchor system may be estimated. An approximate value of the factor of safety of a particular wall may also be obtained. Broms[26] has extended this method to allow for the vertical axial force in the wall member. It should be recognised, however, that several uncertainties exist, both in method of analysis and determination of the anchor forces. It is thus considered prudent to provide an overall factor of safety, based on this method of analysis, of at least 1.5 for temporary works and at least 2 for permanent works. For individual ground anchors, proven safety factors of at least 1.5 or greater should be provided.

Such factors of safety may be inferred from the prestressing records obtained during anchor stressing. Usually these records are valid for short loading periods of up to about an hour and therefore do not provide directly the time factor of safety of the anchor under service loading. Experience has shown that the decrease in anchor carrying capacity under long term loading is relatively modest for most soil types and, provided the factor of safety during stressing is greater than 1.5, the anchor can be considered sound. Special ground conditions such as stiff fissured clay subjected to permanent anchor loading deserve additional consideration and at this stage a somewhat higher factor of safety on short term loading is desirable.

In addition to the above stability calculation, a check should also be provided to ensure that there is a safety against a slip failure beneath the toe of the wall, and beyond the anchor zone either during or after wall construction.

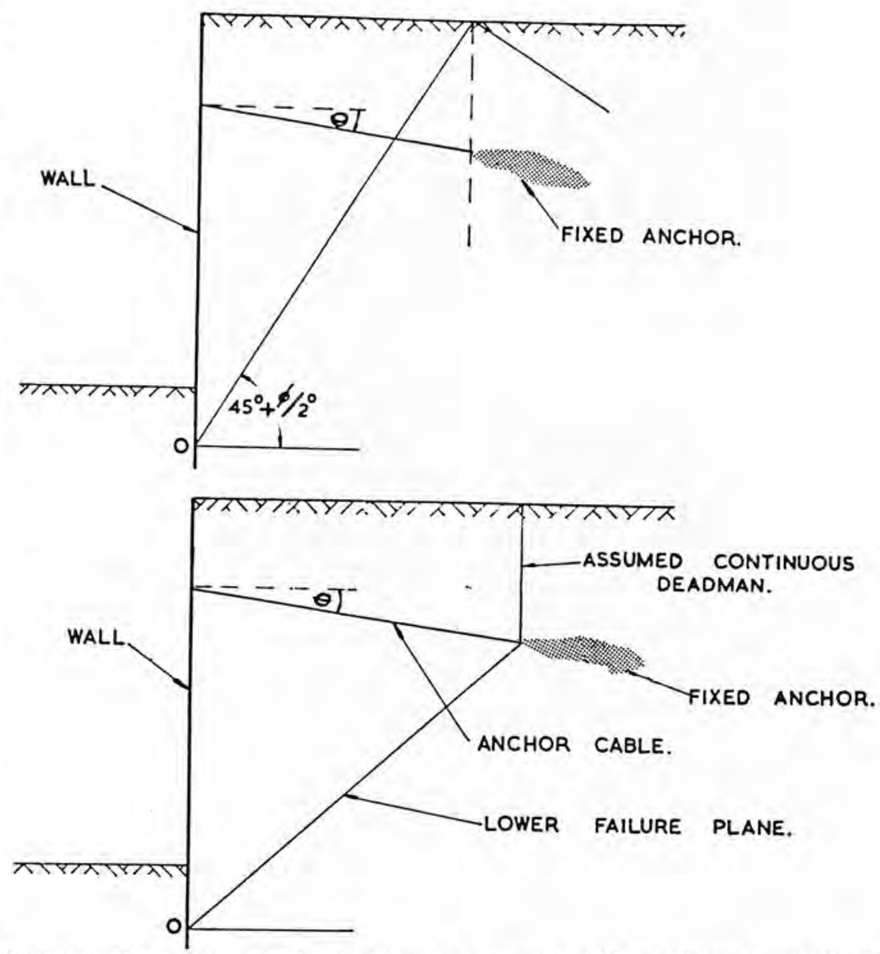
Wall design

Design of the wall member includes proportioning of the wall, selection of anchorage levels and an assessment of the overall stability of the wall-anchor system. Many wall systems are currently in use, each of which possesses its own design features, ground conditions and location determining the type of wall chosen. The conventional types of wall associated with braced excavations are as follows: vertical wood sheeting; interlocking steel sheet piles; solider piles with timber beam lagging; diaphragm wall (slurry trench type); contiguous bored piles.

As already indicated the construction procedure plays an important part in anchored wall design. Consequently the range of forces acting on the wall during construction and while in use must be considered. On large contracts the wall construction process is an assembly line operation as follows:

Installation of wall members.
Excavation within wall limits to first level of anchors.
Installation of tie-backs.
Tensioning of tie-backs to required load.
Repeat stages 2 to 4 with respect to second and subsequent levels of anchors.

With regard to Stage 4, it is common practice



2 *Method of determining anchor length. This method assumes that failure of the wall would take place along a failure plane*

SOIL	Wall	H (ft)	αH (ft)	N	θ'	a (ft)	g (ft)	P (ton)	1	2	3	4	5	Tendon	Spacing (ft)	Reference
—	a	35–38	31	2	20–30	35	—	35–50	.29	.58	—	—	—	A	8	1
Rock at 45 ft	b	45	45	3	45	12 ft in rock	—	35	.18	.46	.67	—	—	B	4–5	2
—	b	—	75	—	30	40–60	—	19–56	—	—	—	—	—	C	—	3
Rock at 47 ft	b	47	47	4	40	90	17	55–100	.17	.36	.55	.75	—	D	14	4
Moraine Rock at 40 ft	a	36	27	2	20–45	50	16	20–25	.12	.55	—	—	—	B	—	5
Rock at 22 ft	b	22	22	2	45	35	15	135	.36	.81	—	—	—	D	—	6
Rock at base of wall	b	40	40	3	45	60	10–15	—	.2	.5	.8	—	—	B, D	7	7
Rock at 65 ft	c	65	65	—	45	40–115	—	300	—	—	—	—	—	—	—	8
Sand, gravel	a	60	48	3	15	—	13	20	.3	.46	.6	—	—	B	—	9
Sand, gravel	a	59	50	3	15–17	65	—	6–11	.29	.45	.62	—	—	B	8	9
—	a	50	35	1	30	49	15	27	.2	—	—	—	—	B	—	10
Till	b	56	50	3	35	32–45	15–20	—	.14	.34	.54	—	—	D	7	11
Clay, rock at 52 ft	b	52	52	5	30–35	31–48	12	26–45	.12	.31	.5	.7	.85	B, E	5–10	12
Silt, clay, sand	b	45	35	3	25–35	24–55	3.5	30–60	.18	.4	.63	—	—	F	8	13
Clay	b	51	45	2	30–45	27–39	2	—	.2	.29	—	—	—	B	8	14
Sand, gravel, clay	c	56	46	1	35	73	23	—	0.17	—	—	—	—	D	—	15
Limestone	c	38	33	2	30	—	—	35	.05	.22	—	—	—	D	—	15
Sandy gravel	a	46	36	2	22-27	33–46	7.5	36	0.25	0.46	—	—	—	B	5.5	16
Sandy gravel	c	40	22	1	10	33–36	6.5	54	0.14	—	—	—	—	B	8.2	16
Rock at 72 ft	b	75	72	5	45	100–150	30–35	90–268	0.15	0.39	0.51	0.57	0.71	D	—	17, 18
Sand and gravel	c	37	26	1	11–18	40	—	25	0.23	—	—	—	—	E	2.5–5.5	19
Loose sand	c	49	41	2	30	72	—	65-70	0.34	0.67	—	—	—	D	6–12	20
Gravel	c	50	20	2	30–45	50–90	13	65	0.07	0.13	—	—	—	D	4–8	21
Shale	c	—	40	3	—	—	14–18	75–100	—	—	—	—	—	D	—	30
Till	b	—	15–30	2	—	—	24	80	—	—	—	—	—	D	8	30
Limestone	—	—	25	2	—	—	16	87	—	—	—	—	—	D	8	30

Legend: a—sheet pile wall; b—solider pile and lagged wall; c—diaphragm wall; N—number of anchor levels; 1a—length of tendon; 1g—grouted length; H—see 1; A—12 in hole 42 in underream; B—1¼ in diameter rod; C—16 in hole 32 in underream; D—multi-strand; E—1 in dia rod; F—36 in dia underream with HT rod

3 *General details of walls constructed and tied back using prestressing. Note the wide range of both walls and soil conditions* to load representative anchors to around 150% of the design load and to record the anchor top displacement as this load is incrementally applied. Thus a minimum value of the safety factor of the anchor is obtained for this period of individual loading. Subsequently the overload is removed until the theoretical design load or the design jacking force value for the anchor is reached.

The design load acting on the wall is calculated using simplified assumptions based on site experience and an approximate pressure envelope. The trapezoidal earth pressure diagram used for strutted walls in clays and sands has been shown to be appropriate for walls in clay till soils (Hanna and Seaton[4]). A similar type of pressure envelope has recently been suggested by Broms[26]. The decision to use a triangular or trapezoidal earth pressure load is controlled by the mechanics of wall movement. Where the wall can kick out into the excavation a trapezoidal pressure envelope is considered valid.

Since assumed design pressure envelope is resisted by the horizontal components of the anchor loads, it is important that when the wall member is watertight, ground water pressures be added to the pressure envelope. In special cases pressures due to freezing (McRostie and Schriever[6]) swelling and adjacent construction activity must also be considered. Anchor positions are invariably arrived at in an approximate manner, and consequently the basic wall design problem is related to the earth pressure distribution. Typical forces, moments and pressures acting on an anchored wall are illustrated in **4**. Similar diagrams may also be produced to simulate the various stages of excavation and anchor construction.

Boundary conditions

Changes in boundary conditions related to a typical construction sequence are illustrated in **5**. Initially the wall member or part of it is formed in the ground, **5a**. Excavation then proceeds to a depth $\propto H$ and the first level of anchors is installed, test loaded and finally tensioned to a predetermined design value. The tensioning load may be somewhat less than the theoretical value and is considered in detail later. A net earth pressure on the wall, a shear stress on the embedded part of the wall, an axial thrust and a displacement of the wall, are shown in **5b**, **5c** and **5d**. They illustrate the changes in the load, displacement and boundary conditions which result on installation of the second anchor level and on completion of the final excavation respectively. It must be emphasised that these sketches are idealised and do not represent the wall-soil behaviour in detail. The decision to allow for wall flexibility is somewhat tentative at present due to the absence of any field or laboratory data on the actual pressure distributions on a tie-back wall.

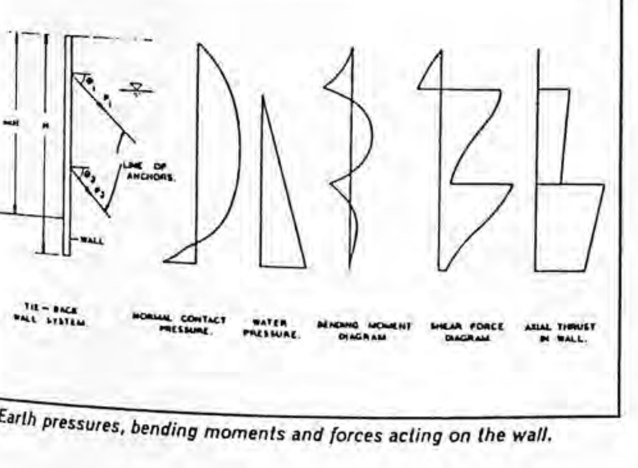
An important factor illustrated by **5** is the large axial force in the wall member caused by the vertical load component of the inclined anchor. This axial load is resisted by the side friction on the wall and the end resistance at wall toe level. With relatively wide walls, or where the wall bears on sound rock, little trouble is expected. However, with sheet piling and walls of H-Section solider piles large forces will be taken in end bearing. For example, a wall with three anchor levels, incorporating 45 ton anchors inclined at 45° and spaced at 6 ft centres, will be subjected to an axial force of 16 tons per ft run of wall. In such cases a simple bearing capacity check must be applied, and allowance made for deterioration of ground conditions at wall toe level. In view of the many unknowns present it might be argued that the design loading for the anchors should be calculated, assuming large earth pressures. From the discussion above, however, it will be realised that this approach can be dangerous because of the additional vertical loads imposed on the wall members.

In cases where solider beams with timber lagging are used, bending moments in the pile can be based on active earth pressure triangular distribution. Consideration should also be given to anchor forces and additional loads such as surcharges, construction forces etc. The lagging boards, which are flexible compared with the steel H piles, carry a small load and resulting bending stresses are very small. Field measurement by Hanna and Seeton[4] suggest that the earth pressures are less than the active values. Consequently for design purposes an active pressure load is considered safe.

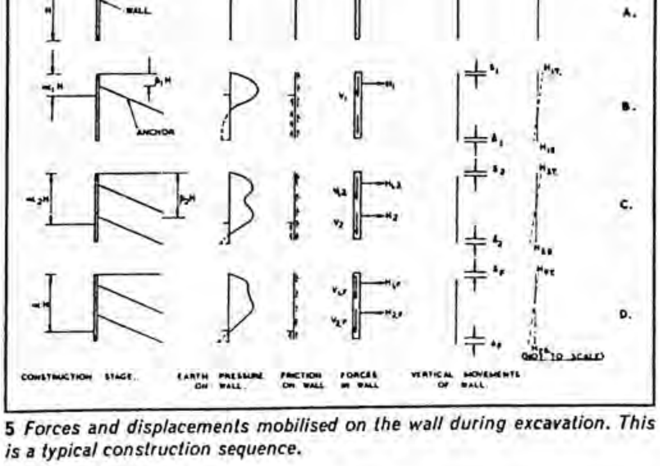
Having arrived at the design load for the wall the engineer must decide whether to use widely-spaced high capacity anchors or lower capacity anchors more closely spaced. In clay soils the force which an anchor will carry is significantly influenced by the soil strength, and in such cases the safe anchor load possible dictates spacing. With anchors in rock, sands and gravels a wide range of working loads may be realised. In sands and gravels for example, loads between 20 and 100 tons are easily obtained. In each case load and the spacing dictate the wall system used, the wall thickness and hence the overall cost.

References

1. *Lon buried tie-backs leave cofferdam unobstructed:* Construction Methods and Equipment, May 1960, 130-133.
2. *Grouted-in drilling steel serves as tie-back:* Construction Methods and Equipment, October 1966, 82-85.
3. Etheridge D. C.: *Tie-backs unclutter deep foundations,* Construction Methods & Equipment, Sep '66.
4. Hanna T. H. and J. E. Seeton: *Observations on a tied-back solider-pile and timber lagging wall,* Ontario Hydro Research, Vol 19 No. 2 1967, 22-28.
5. Nordin P. O.: *Insitu anchoring,* Rock mechanics and engineering geology, Vol 4, 1966, 25-37.
6. McRostie G. C. and W. R. Schriever: *Frost pressures in the tie-back* system at the National Arts Centre Excavation, Engineering Journal, Engineering Institute of Canada, March 1967, 17-21.
7. *Tie-backs remove clutter in excavation:* Engineering News Record, June 8 1961, 34-36.
8. *Tie-back system gives elbow room:* Engineering News Record, May 9 1968.
9. Jessberger J. L.: *Theorie und praxis eines neuzeitlichen verankerungsverfaherns,* Bautechnik, 7.7.63.
10. Proctor P. W. and C. A. Pequignot: *Injection anchors remove clutter from critical path,* Civil Engineering and Public Works Review, Dec 1967.
11. *Soil-anchored tie-backs aid deep excavation:* Engineering News Record, Aug 10 1968, 34-35.
12. *Northern approach—New Mersey Tunnel:* Construction News, January 25, 1968, 19.
13. Booth W. S.: *Tie-backs in soil for unobstructed deep excavation,* Civil Engineering, Sept 1966.
14. *Tie-back wall braces building excavation:* Construction Methods and Equipment, Nov 1962, 116-119.
15. *Car parks under Paris:* Engineering, June 14 1968.
16. Weber E.: *Symposium on earth and rock anchors* Swiss Society for Soil Mechanics and Foundation Engineering Bulletin No. 62, 1965.
17. *Foundation for tallest tower; water out, trains in:* Engineering News Record, Oct 31 1968, 30-32.
18. *Tie-backs eliminate internal bracing:* Construction Methods and Equipment, July 1968, 104-109.
19. Mayer A. and Rosset F.: *Anchorage of an in-situ wall on the UNESCO site in Paris,* Technical Bulletin of French-Switzerland, Dec 11 1965.
20. Sondage-Injections-Forages (1967), *Soil Anchors:* Technical Bulletin, 11 Avendue du Colonel Bonnet, Paris 16.
21. Losinger and Co.: Private correspondence 1965.
22. Littlejohn G. S.: *Recent developments in ground anchor construction,* Ground Engineering, May 1968.
23. Hanna T. H.: *Factors affecting the loading behaviour of inclined anchors used for the support of tie-back walls,* Ground Engineering, Vol 1 No. 4, Sep 1968.
24. Kranz E.: *Ueber die Verankerung von Spundwaenden,* Wm. Ernst & Sohn, Berlin, 1966.
25. *Recommendations of the Committee for Waterfront Structures,* Wm Ernst & Sohn, Berlin, 1966.
26. Broms B. B.: *Swedish tie-back systems for sheet pile walls,* Proc. 3rd Budapest Conference on Soil Mechanics and Found. Eng. October, 1968.
27. Broms B. B. and H. Bennermark: *Stability of clay at vertical openings,* Proc. ASCE Journal of Soil Mech. and Found. Eng. Division, Vol 93, 1967.
28. Broms B. B. and H. Bennermark: *Stability of cohesive soils behind vertical openings in sheet pile walls,* Proc. 3rd Budapest Conference on Soil Mechanics and Foundations Engineering, October 1968, 404-409.
29. Slater W. M.: *Prestressed anchors and tie-backs in greater use,* Daily Commercial News and Building Record, May 24 1967, 16-17.
30. Coneco International: Private correspondence with T. H. Hanna, 1969. ■



4 *Earth pressures, bending moments and forces acting on the wall.*



5 *Forces and displacements mobilised on the wall during excavation. This is a typical construction sequence.*

Retaining wall tie-backs

T H Hanna University of Sheffield and **G S Littlejohn** Cementation Ground Engineering Ltd

This is the second and concluding part of an article on earth support methods. Last month's piece ended with a section on boundary conditions around retaining walls. We open here with some construction considerations

During construction three distinct operations are involved: construction of wall members, construction and prestressing of anchors and soil excavation.

The type of wall and the decision to employ anchors are dictated by a number of engineering considerations of which ground conditions, site use, time available and site mechanisation are important. The presence of boulders rules out the use of sheet piles and favour a concrete wall or a soldier pile and timber lagging system. The presence of continuous water bearing and pervious lenses within the soil mass may necessitate the sinking of 'bleeder' wells or holes at 15-100ft centres to prevent bottom heave as excavation proceeds. Drainage of the soil behind a soldier pile wall is possible by spacing the horizontal lagging boards and providing filters behind them. With sheet piling and diaphragm walls these structures are normally considered to be watertight, although they can be perforated.

In sensitive soils and in cases where the wall does not make good contact with bedrock or is not toed into a hard stratum, loss of ground due to soil flow beneath the wall toe is likely. This subject is well covered by Broms and Bennermark [27, 28]. These authors outline analytical methods of predicting this condition. Where the wale beams are not continuous, contact lagging boards should be considered. This method of timbering is simpler than the conventional one and its chief merit is speed of erection and dismantling. Details are illustrated in 6.

The ends of the anchors bear on wale beams which are fixed to the wall members by struts. In some designs this has been costly and time consuming because of the necessary welding and site fitting. A survey of walls built recently reveals a trend away from the continuous wale system to very short lengths of wale, a move that seems to have been dictated by cost considerations alone. The wale beam must satisfy several requirements. First, the wale support struts and beams must be correctly aligned to receive the anchor loads. This is easily achieved by proper supervision of the anchor hole drilling and good site fabrication and welding. Secondly, all the structural units must be designed against local buckling both during test loading and while in service.

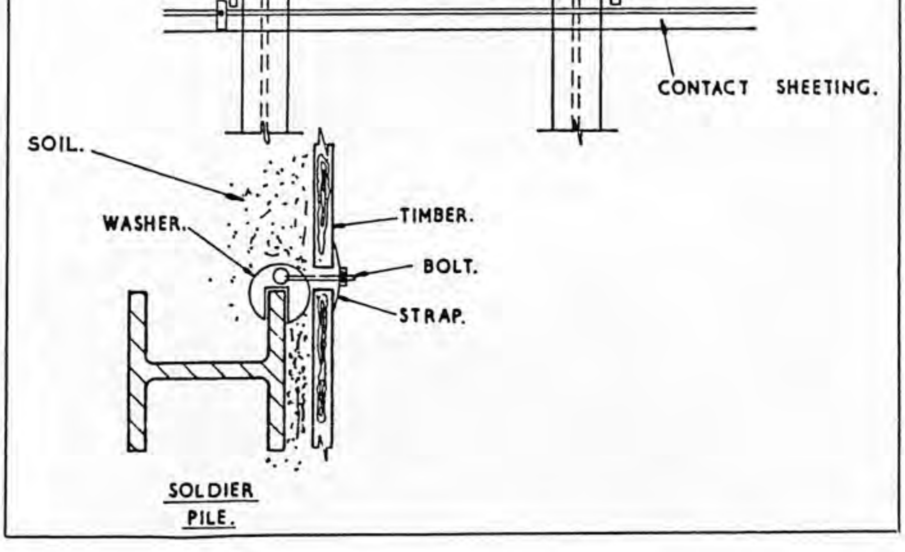
A reinforced concrete diaphragm wall 2ft in thickness supporting a 40ft excavation in London clay is shown in 7. The wall is 50ft deep and is supported by two rows of prestressed anchors. The top row, at 15ft depth, is inclined 15°. It is 46ft long and each anchor has a 55-60 ton working load in the Thames ballast. The bottom row at 31ft depth is inclined at 20° and carries a working load of 50 ton in the London clay. These anchors are 45-55ft long. On this site, where 370 anchors were used, each anchor was tested to 25% above its working load, and one in ten was tested to 140% of the working load value.

Excavation
In certain ground conditions an excavation, with the associated release in ground pressures, may result in a deterioration of ground strength. Water usually aggravates the situation and can cause swelling pressures to develop, producing a loss in bearing capacity near the base of the wall. It is essential, therefore, to assess the deterioration potential of a site before starting construction.

Construction errors are ever present and their importance should be realised. In the authors' experience serious wall distress may be caused by excavating too near the base of the wall and accidentally penetrating beneath the wall to cause a reduction in wall base support. This is associated with surface settlement and wall translation. Several cases are on record (Slater[29]) where observed wall movements of a foot or more took place due to neglect during excavation operations. In such cases the calculated penetration depth of the wall below excavation level should be increased by 20% to allow for over-excavation. Two cases merit close attention. First, where the anchors are steeply inclined, very large axial forces are transmitted to the wall. Secondly, where the excavation extends below the wall toe level, as occurs with excavations penetrating bedrock, an adequate bench in the rock must be provided to support the wall base.

Anchor construction
It has already been stated that local variations in ground conditions within a few feet are important and can alter anchor performance. This is especially significant in clay. Whilst this degree of coverage cannot be obtained from a conventional site investigation, much guidance is possible through experience from other sites and through observations during anchor hole drilling. Fluctuations in drilling

6 Detail of steel clip assembly for contact timber attachment. This method is simpler than the conventional one, and is also quicker to erect and dismantle



rate, degree of blocking of the drill bit, change in content of the air or water flushings; all these contribute to an understanding of anchor performance and soil variability. Because of variations in clay consistency from point to point, it is not surprising to find that the design anchor load cannot be mobilised at a particular point. In such cases an additional anchor should be installed or the length of the anchorage zone increased.

Anchor hole formation is aided by various flushing techniques. In sands and gravels, for example, water flushing widens and cleans the hole and ensures a better bond at the grout-soil inter-face. However, in clays, marls, chalks and in any soil or rock stratum liable to deterioration from water action, water flushing should be minimised and where possible air flushing employed.

After drilling and casing the hole to the required depth, the anchor member consisting of high tensile steel rods, wires or strands is 'homed' into position. The length of the anchor hole is such that the anchor load is mobilised in the soil mass beyond the zone bounded by the critical wedge as shown last month. The anchor shaft is decoupled from the anchorage zone by some form of sheath and is referred to later. The cable is then grouted to the soil over the fixed anchor zone. Normally, grout is injected under pressure. Care should be taken not to exceed the theoretical overburden pressure since this could cause fissuring in the ground and possibly lead to ground heave at the surface as well as possible damage to existing anchors. During the grouting stage therefore, a careful note of injection pressure is required together with grout consumption. In order to cater for the wide range of soils encountered the choice of grout is also important.

In sands and gravels which allow permeation of ordinary cement grouts, the water/cement ratio normally varies from 0.5 to 0.65 but for high capacity anchors at shallow depths in fine sand it may be necessary to employ extra fine cement or synthetic resin grout. In clays, a stiff grout (water/cement ratio 0.4) is recommended. Where the ground is heterogeneous, high alumina cement is often employed since it enables the anchor to be tensioned within 24 hours. Consequently, if the ground conditions have deteriorated locally without being observed, the tensioning stage will indicate a reduced capacity, and remedial measures can be taken immediately.

Grout admixtures to give an expanding or non-shrink grout have been employed on many sites but it is considered that their usefulness is restricted to impermeable ground. In rock, for example, the use of an expanding agent increases the 'confined' compressive strength thus improving the bonding ability. In clays, grout fluidifiers are sometimes employed to give a pumpable low water/cement ratio grouts.

When the anchorage zone is formed by pressure grouting, care must be taken to ensure that the anchor borehole is not filled with grout to the ground surface. It is common practice partly to fill the space surrounding the anchor rod or wires with a weak grout which is placed in position but not pressurised. A mud slurry or a sand fill may also be used; they are perhaps preferable except for the additional time involved in their placing. If the anchor rods are grouted up to ground surface, during anchor stressing the column of grout is stressed and a false measure of the load developed by the anchorage zone is obtained. Experience with test anchors in fine sand suggests that the crushing load on a 5in diameter grout column 14ft long may vary from 10 to 25 ton dependent on the lateral restraint provided by the surrounding soil.

Anchors may be constructed before or after wale beams are placed. To ensure that the wales fit over the anchors, it is essential that the anchors are in line. Considerable distress can otherwise result. This is not so critical with flexible cables but may be more serious with bar anchors. It is noteworthy that economy is possible when tying back re-entrant corners. This is accomplished by bolting the walls together through the retain soil mass, thus eliminating some of the anchors and reducing the possibility of anchors fouling. To date, few engineers appear to have taken advantage of this simple technique.

Post tensioning

During stressing it is usual to record the cable movement at the top end of the anchor as the load is incrementally applied. The load is measured by means of a load cell or jack pressure gauge. However, unless a reference datum is established which is independent of the wall system it is impossible to interpret the load/deformation curve. For this reason optical techniques of recording anchor movements must be used. Thus the real elongation and displacement of the anchorage can be obtained. Since the apparent stretch of the movable anchorage comprises several interrelated movements—including wall displacement, elastic elongation of cable and fixed anchor displacement (see Hanna [13])—the interpretation of an anchor load test requires skill and experience with the particular anchor system being used. A typical loading curve for a test anchor formed in Thames Ballast is shown in **8**. Note how the fixed anchor movement is negligible although friction between the protective sheath and the steel does distort the curves when loading and unloading takes place quickly. In fine sands however, fixed anchor displacement during the initial tensioning is fairly common and this should not be associated with failure. If the fixed anchor consists of a relatively smooth grout cylinder then some relative displacement at the grout/sand interface may be necessary to mobilise load. In these circumstances the load carrying capacity of the anchor is established from a second tensioning cycle. The fixed anchor movement should be negligible provided the initial test load is not exceeded.

As mentioned in an earlier section, each anchor may be tested to its theoretical design load plus a margin of safety. This margin of safety is usually 50%, although a range between 25% and 100% is on record. The possible test load is limited by the elastic limit of the steel cable. After proof loading of an anchor to the specified factor of safety, the anchor force is released to the required design jacking value. The earth pressure load that the wall is designed to resist is normally based on an active pressure, or at rest pressure assumption. Neglecting errors and limitations of predicting the wall load, the purpose of prestressing the individual anchors is to ensure that the sum of the horizontal components of the anchor load equals the earth pressure design load.

The load that is jacked into the anchor during prestressing will change as wall construction progresses. This results from the yield of the wall due to vertical and horizontal movements. In an attempt to allow for these movements, it is considered valid to stress the anchor initially to a load somewhat less than the theoretical load. A value of 80% is often used. This procedure is valid for rigid walls such as diaphragm and contiguous bored piles. Detailed studies are required to establish the practical validity of this prestressing approach. Work by Hanna and Seeton on a soldier pile and timber lagging wall in which the anchors were stressed to 85% of their 'theoretical' load confirmed that anchor loads about 20% greater than the jacking load were subsequently developed. Many other factors condition loading. Wall movements are the most uncertain but little is known about their magnitude. Most reports suggest that the movements are of negligible magnitude. In a few special cases relatively large movements are reported and it is towards such cases that detailed field study must be directed.

Because the anchor loads change during excavation and construction the decision to reload anchors, say, after the installation of a lower row or after a period of several days or weeks is somewhat doubtful and should not be followed without good reasons.

Corrosion protection

The majority of tie-back walls are of a temporary nature with a working life not in excess of

7 *A 40ft excavation in London clay. 370 anchors were used on the site, and all were tested to well above the working load*

2 years. In these cases, where the ground conditions are not hostile, a decoupling sheath consisting of a greased tape will suffice for protection. However, for permanent anchors or when anchoring in marine conditions proper corrosion protection of the cable is essential. Cathodic protection has been employed in North America[1*] where a full length electrical conductor consisting of a steel wire core covered with zinc is placed with each cable.

In Europe the trend has been to apply protective coating to the cables. Where a cable consists of several high tensile steel strands, the individual strands can be coated with grease, after which plastic is extruded over the strands under factory controlled conditions. This protected cable is then delivered to site and when the cable is fabricated the fixed anchor length is stripped, degreased and cast into a corrugated plastic tube using high strength synthetic resin. This quick-setting resin is more than capable of transmitting the working load over the bond length while the remaining length of the cable is perfectly decoupled.

Wall-anchor behaviour

When extending the tie-back principle of wall support to new ground conditions it is prudent to monitor overall wall behaviour. Many methods of studying wall behaviour are available. The choice of field instrumentation is dependent on the quantities to be measured and the accuracy desired. A starting point, therefore, is to examine the parameters which are of interest and of engineering use. A list of field instrumentation techniques considered appropriate for wall and anchor study is contained in 9. Many instrumentation techniques are available, and the list contains methods with which the authors are familiar. Some data on an instrumented tie-back wall is reported by Hanna and Seeton. Perhaps the greatest use of field instrumentation lies in its ability to simply supply data on performance. Only when such data is available covering a wide range of design conditions can the performance of a tie-back wall be fully understood. Very useful guidance can be obtained from an anchor test programme, similar to a pile test programme. Unfortunately, cost limits such work to relatively large projects.

Conclusions

The tie-back wall is an attractive method of supporting deep excavations in sands, gravels, clays and rock. Due to the many construction advantages the use of tie-backs is increasing throughout the world. It has been mentioned that a range of wall types, anchor systems and design approaches is in use. Due to the lack of experience and reported behaviour, caution is necessary in extending the tie-back principle to new ground conditions. For this reason opportunity should be taken of field instrumentation by which the performance of walls can be related to design assumptions. Factors which require detailed field assessment include:

Methods of theoretically predicting anchor load capacity

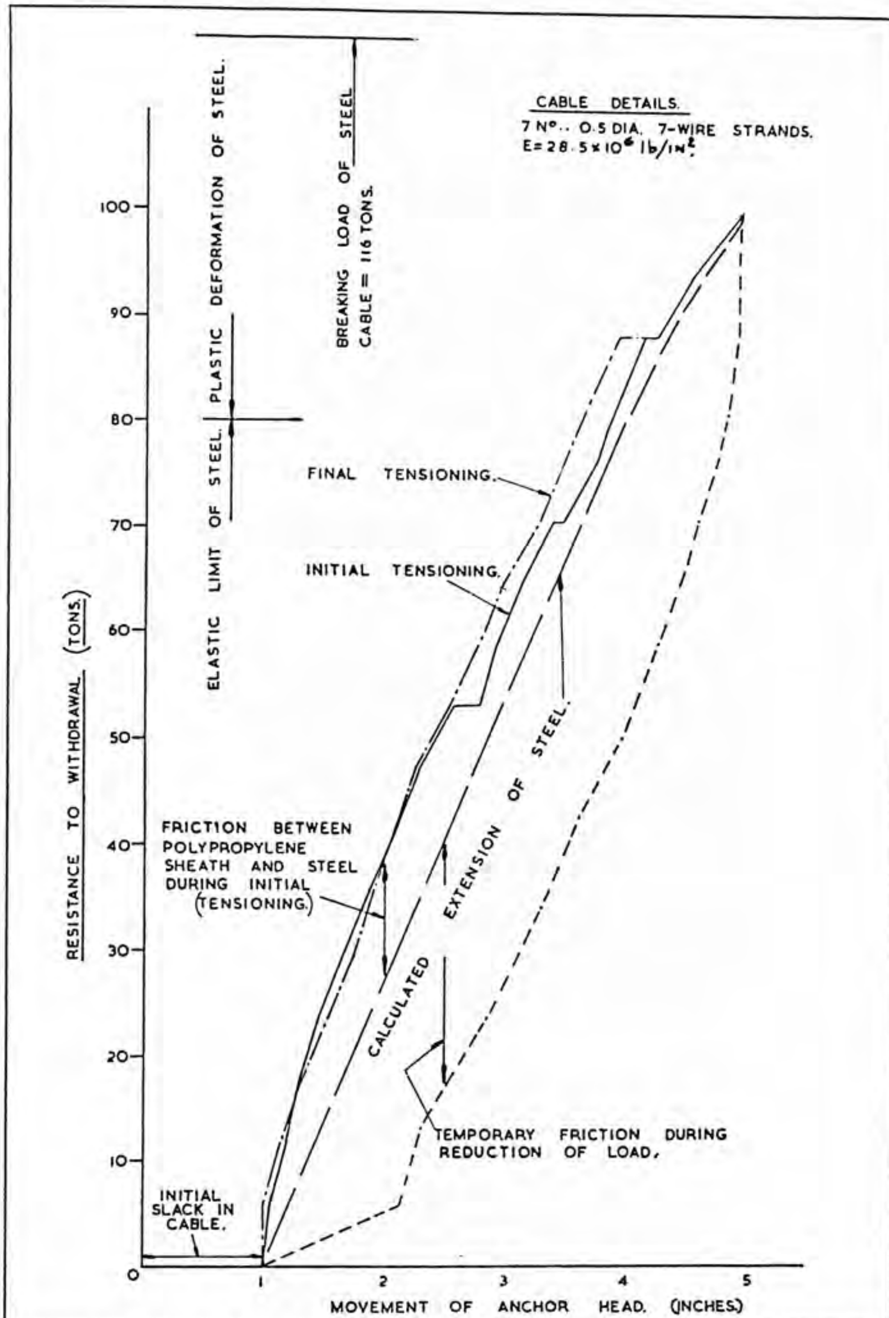
Methods of carrying out anchor load tests

The influence of adjacent anchors in a line or group on load capacity

Variables which affect wall design, construction and behaviour

The long term performance of tie-back walls and anchors

All of these factors are being simulated in a large scale model study into the behaviour of anchorage systems at Sheffield University. Where possible, the work is being extended into the field and actual structures are observed. It is believed that the data obtained from these studies will result in more economic and safe designs and perhaps may lead to new uses of anchorages in civil engineering. ■



8 *Load extension curve for test anchor in London ballast. Note how fixed anchor movement is negligible, although friction is possible due to quick loading and unloading*

Quantity to be measured	Methods available
Anchor load	Load cell—several types available. Accuracy to 1% of maximum load. Mechanical methods (4).
Wall forces	Vertical force in the wall member may be measured by load cells cast in the wall or indirectly by recording wall compressions. Contact pressure can be measured by earth pressure cells cast in the wall. Also necessary to measure pore water pressures on the wall with piezometers.
Surface movement of soil behind wall	Use survey pins located 5 to 20ft apart and extending to 3 times excavation depth away from the wall. Vertical movements can be measured to 0.0005 inch by precise level. Horizontal movements by invar tape measurements between pins. Care required to prevent disturbing pins.
Wall movements	Top settlement by precise levelling. Horizontal top yield by invar tape measurement. For precise work an inclinometer duct may be cast in or fixed to the wall. Note: This gives a measure of the position of the wall with respect to either the top or bottom of the wall. Accurate to about 1in in 150ft.
Soil movements	At depth they may be monitored by an inclinometer duct. Care needed during installations. Expensive.

9 *Field instrumentation techniques appropriate for wall and anchor study*

4. SOIL ANCHORS

G. S. LITTLEJOHN, BSc, PhD, MICE, FGS, Cementation Ground Engineering Ltd

Following a note on the background to recent injection anchor developments in the UK, the main applications associated with prestressed soil anchors are described. The type of site investigation and the soil properties required to facilitate anchorage design and choice of construction technique are then discussed. As a result of testing soil anchors to failure empirical design rules are presented, which relate ultimate load holding capacity to local soil properties and anchor dimensions, for coarse sands and gravels, fine to medium sized sands, stiff clay, stiff to hard chalk and Keuper marl. Safety factors which are applied to these rules are included together with recommendations for the post-tensioning and testing of individual anchors. Data on the long-term behaviour of soil anchors is limited but prestress losses due primarily to fixed anchor displacement are listed for guide purposes when estimating realistic overloads. Corrosion protection is discussed in relation to fully restressable cables for temporary and permanent works. Finally, the importance of pull-out tests is emphasized together with field observations of anchorage performance, where these field data are related to the original design criteria.

History

Although prestressed rock injection anchors have been installed regularly since 1934, when the late Andre Coyne employed anchor stressing at Cheurfas Dam in Algeria, it is only in recent years that the field technique has been extended to use soils. Initial development occurred during the late 1950s when anchorage construction techniques for cohesionless soils were introduced in Europe, the majority of the applications being associated with coarse sands or gravels. In many of these cases the employment of soil anchors using grout injection techniques brought about an increase in site construction efficiency and the resulting savings quickly encouraged engineers throughout the world to adopt the techniques. By 1966 the behaviour of injected anchor systems was being studied in the UK, and several systems from continental Europe had been introduced by specialist contractors.

2. Since these procedures and expertise were largely based on experience of anchoring in heavy alluvium, the period 1966–69 in the UK was marked by great developments in anchor construction to extend field applications to soils such as stiff clay, marl, fine to medium sized sand and chalk. As a result of this work, involving field anchor tests taken to failure and observations on the long-term behaviour of prestressed anchors, certain empirical design rules with realistic safety factors have been produced relating ultimate resistance to withdrawal of individual anchors to soil properties and anchor dimensions.

3. Thus in 1970 it is only the soft compressible soils which do not readily lend themselves to anchorage systems, i.e. where the natural soil compressibility severely restricts the amount of ground restraint which can be safely mobilized without high prestress losses. At present these restrictions on anchor loading can increase the cost per ton of resistance to such an extent that anchors are not attractive when compared with alternative solutions, except for special applications.

Applications

4. Although many of the injection anchor systems are relatively new it would appear from the literature now accumulating that the market for these techniques is developing rapidly. In view of this trend some of the main applications where injection anchors have already been successfully employed are described.

Retaining wall tie-backs

5. This method of wall support eliminates internal bracing in cramped excavations or in wide cuts, thus

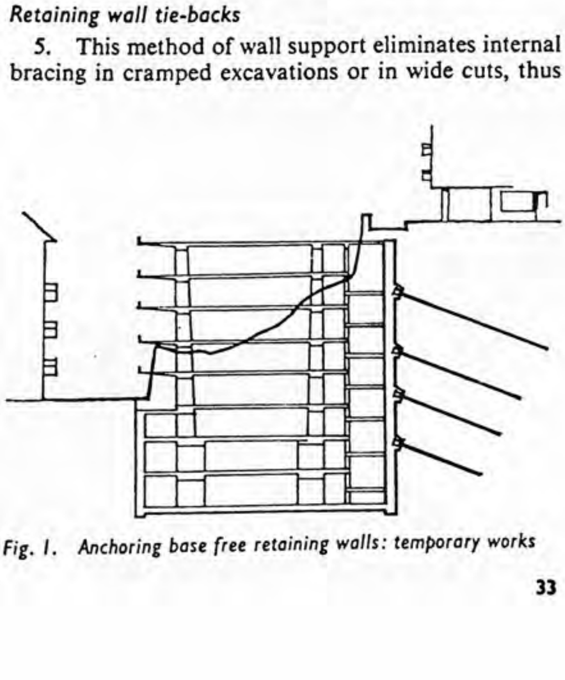


Fig. 1. Anchoring base free retaining walls: temporary works

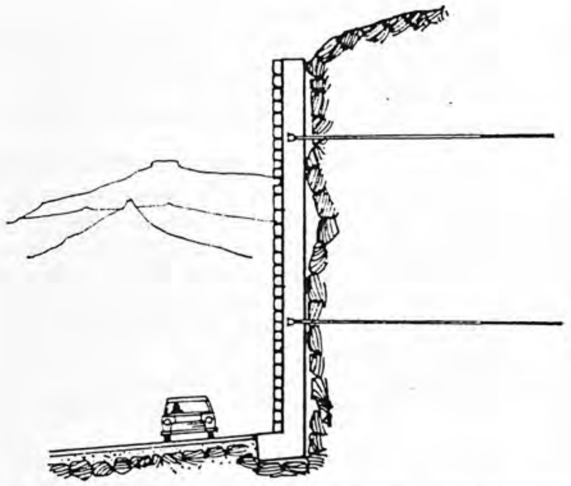


Fig. 2. Revetment of rock with anchored retaining walls: permanent works

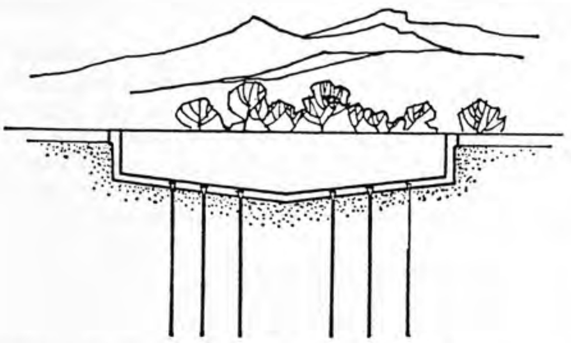


Fig. 3. Resistance to buoyancy: stormwater tanks

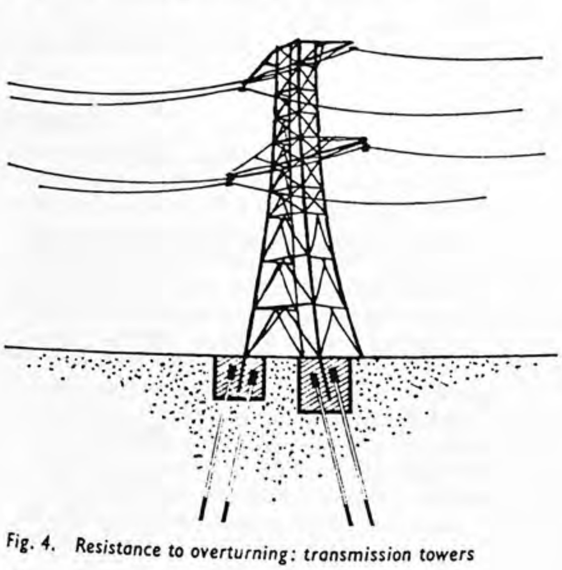


Fig. 4. Resistance to overturning: transmission towers

permitting the use of modern construction methods for excavation and subsequent works (see Fig. 1). In addition the anchoring of base-free retaining walls adjacent to new highways, railways and canals can reduce the amount of excavation necessary to a minimum (see Fig. 2). Anchored gunite or concrete retaining walls for the revetment of rock which is gradually being broken up by frost and weathering is another application since the permanently applied wall pressure counteracts ice formation and prevents the occurrence of disintegrating pressures.

Resistance to buoyancy

6. When basin-shaped structures subjected to a rising groundwater level are in an unloaded state the danger of their floating exists. Prestressed anchors founded in the ground beneath the structure can be used to resist upward water pressure (see Fig. 3). This application is usually associated with cofferdams, dry docks and effluent tanks but, due to the increasing tendency to build downwards in city areas, anchorage of basement car parks can now be included. A possible alternative to anchors is mass concrete but the additional cost of the extra excavation normally renders this method more expensive.

Resistance to overturning

7. Tall buildings and masts subjected to wind loading, and transmission towers with high surge loads, must be capable of resisting large overturning movements.

8. Ground anchors tied to the foundations of such structures, where the prestress is calculated on the worst design case, enable the foundations to resist the applied forces without upward movement (see Fig. 4).

Resistance to sliding

9. In pipe jacking or thrust boring where a bridge, underpass, culvert or pipe section is being pushed through an embankment, it is necessary to employ some form of thrust block to provide a reaction for the jacks. Where the passive resistance of the soil is not

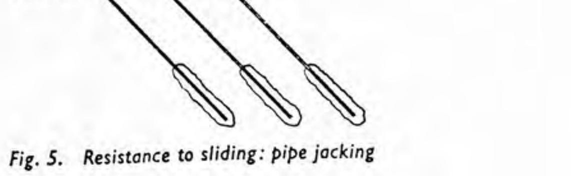


Fig. 5. Resistance to sliding: pipe jacking

available or sufficient to resist the jacking forces safely, prestressed inclined anchors provide a horizontal force component and a vertical contact pressure at the horizontal interface between the block and subgrade, thus increasing the resistance to sliding (see Fig. 5).

Preloading to minimize structural settlements

10. Where settlement of a new structure in compressible ground is severely limited, prestressed anchors can be used to preload the ground where the anchorage forces are of the same order and act over the same area as the subsequent structural loads. In this way ground settlements can be induced before construction (see Fig. 6). In low permeability soils the borehole above the fixed anchor can be filled with a drainage material such as sand to accelerate consolidation.

11. The reverse problem, i.e. heave of an excavation floor, can be tackled in the same way, where the anchorage forces are equivalent to the overburden pressure which has been removed. This aspect is particularly relevant at present around London since the proposed inner ringway involves some deep permanent cuts in the London clay.

Pile and plate loading tests

12. Where sites are remote or where access to and space available at a site location are restricted, e.g. between existing railway tracks or highway lanes, the use of ground anchors for loading tests can be more attractive than kentledge. A typical pile test is shown in Fig. 7 and this system is commonly used to mobilize test loads up to 1000 tons.

13. Piles can also be prestressed using anchors founded in the ground beneath the pile base. In this way the pile can resist both compressive and tensile forces with the minimum of movement.

14. Other applications which are almost wholly restricted to rock injection anchors include the prestressing of dams for increased strength or before raising, rock bolting for roof strata control and cliff stabilization.

Site investigation

15. When a potential application for anchors is being studied, the value of a site investigation, orientated towards obtaining the soil properties which facilitate anchor design and choice of anchor construction technique, cannot be overemphasized. Lack of the relevant data will lead to a request by the designer for additional information, especially if the soil conditions are highly variable, or perhaps test anchors if the ground is fairly homogeneous. On small contracts, however, the cost of obtaining additional test data may not be acceptable and the resulting designs may be very conservative. In this way the obvious advantages of the anchor system can be diminished to such an extent that an alternative system, which does not depend to the same degree on soil properties, may become more attractive to the designer.

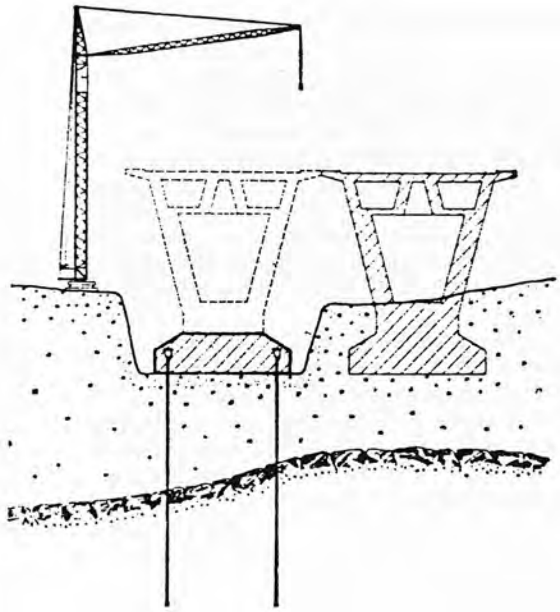


Fig. 6. Preloading to minimize structural settlements

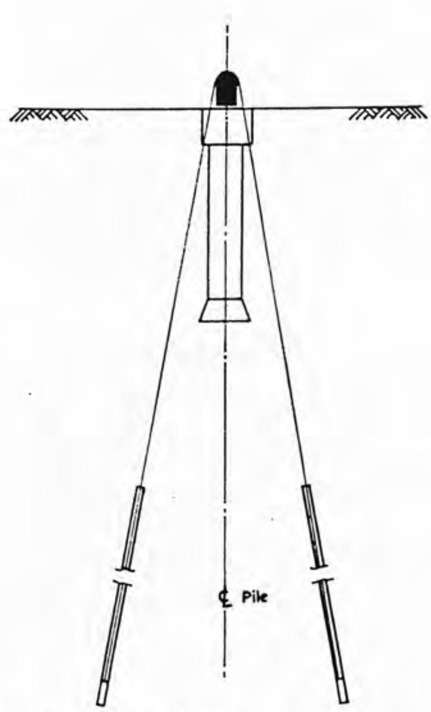


Fig. 7. Pile and plate loading tests

16. The ultimate resistance to withdrawal of an injection anchor depends on the ground restraint which can be mobilized adjacent to the grout injection zone (or fixed anchor). Anchors may be spaced at intervals of only one or two metres and therefore a knowledge of the local variations in soil properties is valuable. A conventional site investigation cannot supply this order of detailed information, but sufficient boreholes should be installed to enable a fairly accurate soil profile to be drawn which will indicate the changes of strata across the site together with groundwater level. In soft heterogeneous ground for example the variation in level and the thickness of a gravel layer (the potential anchorage medium) have a direct bearing on the length and inclination of the anchor.

17. Having assessed the soil succession from a few preliminary logging boreholes, the relevant engineering characteristics and classification data required for particular horizons in the soil mass can be determined for the remaining boreholes from undisturbed and disturbed samples.

18. The tests for which undisturbed samples are required are shear strength (unconfined and triaxial tests), consolidation and density. In cohesionless soils the angle of internal friction ϕ combined with the effective overburden pressure, which is dependent on the location of the groundwater table and the unit weight of the soil, enables the pull-out capacity of a given anchor to be calculated. In purely cohesive soils the undrained cohesion C_u is required for the design of temporary anchors, i.e. less than two years, but for a longer working life ϕ' and C' may be the relevant shear strength parameters. Consolidation and compressibility indices assist the design engineer in his assessment of the long-term behaviour of prestressed anchors and, as more case histories become available where the relevant soil properties have been documented, the accuracy of predicting variations of prestress with time will improve and lead to less conservative anchor designs.

19. The tests for which disturbed samples are required include mechanical and chemical analyses. Particle size distributions of frictional soils are invaluable since they enable the permeability and therefore the groutability of the soil to be assessed.

20. If the ground permeability k_w is greater than 100 µm/s cement grout may be used to permeate the soil adjacent to the anchor cable during the injection stage. In finer soils, however, the natural pores in the injection zone are not large enough to accept cement particles. A knowledge of this limit is therefore extremely useful when predicting the effective diameter of the grouted fixed anchor.

21. Grading samples can also be used in conjunction with standard penetration tests to estimate the relative density and then ϕ, if values of ϕ are not already available. Chemical analyses of the soil and groundwater, to determine sulphate content and pH value say, are important since these results will determine the type of cement grout and degree of corrosion protection required.

22. Finally, changes in the site conditions due to climate or adjacent construction activity should be assessed. Allowance for water level fluctuations due to climatic changes may now be regarded as a routine procedure, but little attention is normally paid to adjacent activities such as freezing, piling or blasting operations. In frost sensitive soil, such as silty clay, freezing can cause severe expansion and is capable of imposing pressure of 14–29 kN/m² on the anchored wall.[1]

23. Little data is available on the effect of piling or blasting on clay anchors, but in cohesionless soils it has been shown[2] that the compaction radius of a driven pile can extend to 6 d, where d is the pile diameter, and for blasting operations the type of empirical rule shown by equation (1) may be used to relate weight of charge W kg to radius of sphere of influence r m.

$$W = cr^3 \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where c is the coefficient ($15{\cdot}6 \times 10^{-3}$ for 60% dynamite).[3]

24. These relationships can indicate a potential hazard on the assumption that anchor performance is only affected when the relative density of the soil in the anchorage zone is altered.

Load carrying capacity

25. When the soil conditions have been obtained for a given anchorage application the design of the injection anchor may proceed. As a result of testing anchorages in a wide range of soils where the length of the injection zone (fixed anchor) has been varied, the following guide rules have been established for Cementation ground anchors relating ultimate load holding capacity T_f to local soil parameters and fixed anchor dimensions. Having estimated the ultimate load holding capacity, a factor of safety against pull-out S_f is applied to give the working load of the anchor T_w.

Coarse sands and gravels ($k_w > 100$ µm/s)

26. In coarse sands and gravels where the ground permeability k_w is greater than 100 µm/s, cement grout can be used to permeate the soil in the fixed anchor zone. In homogeneous ground of this type, anchors are designed to resist safe working loads of 80 t ($S_f = 1{\cdot}5$–$2{\cdot}5$).

27. The anchorage construction technique is to drill and drive a casing (102 mm nominal diameter) to the required depth, home a prepared steel cable or bar and then inject grout under a nominal pressure (30–1000 kN/m²) as the casing is gradually withdrawn over the fixed anchor length. For further information on anchor construction techniques see ref. 4.

28. When this technique is used in coarse alluvium

the ultimate load carrying capacity T_f may be estimated from the following empirical rule

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

where L is the fixed anchor length in metres, $n = 40$–60 t/m and ϕ is the angle of internal friction.

29. In equation (2) the factor n automatically takes into account the depth of overburden above the fixed anchor ($h = 12{\cdot}2$–$15{\cdot}1$ m), fixed anchor diameter (400–610 mm) and the range of fixed anchor lengths (0·9–3·7 m) over which the rule has been tested.

30. Equation (1) indicates that the resistance to withdrawal is transferred to the ground by skin friction and the absence of an end resistance component is considered to be due primarily to the limited fixed anchor displacement which took place during the loading procedure, thus preventing the mobilization of any significant end restraint. It can be readily appreciated that equations similar to equation (2) can be used only by specialist contractors familiar with their own particular anchorage system. However, equation (2) shows the type of simple rule currently being used by anchorage contractors.

31. For more general use it is necessary to relate anchor pull-out capacity with anchor geometry and soil parameters, and equation (3) is recommended for consideration.

$$T_f = A\bar{\gamma}\left(h+\frac{L}{2}\right)\pi DL \tan\phi + B\bar{\gamma}h\frac{\pi}{4}(D^2-d^2) \qquad (3)$$

(side resistance) + (end resistance)

where A ratio of the contact pressure at the fixed anchor/soil interface to the effective pressure of the overburden
B bearing capacity factor
$\bar{\gamma}$ unit weight of soil overburden (submerged unit weight beneath the water table)
h depth of overburden to top of fixed anchor
L length of fixed anchor
D effective diameter of fixed anchor
d effective diameter of grout shaft or column above fixed anchor.

32. The value of A depends on the installation procedure and, for the technique described where the casing is driven using rotary percussive techniques, A lies within the range 1–2. If the soil is not compacted or displaced during the casing installation and no residual grout pressure is left at the fixed anchor grout/soil interface on completion of the injection stage, A might reduce to a value approximating to the coefficient of earth pressure at rest K_0.

33. The value of the bearing capacity factor B depends on the angle of shearing resistance of the soil adjacent to the top of the fixed anchor and the ratio h/D. Consequently B resembles N_q, although from field experience the values published by Terzaghi or Meyerhof are too high to be applied directly to ground anchors. The high value of the ratio h/D in anchor work and the fact that the anchor develops end restraint in a zone of soil disturbed during construction appear to be mainly responsible for the reduced bearing value. In this connexion bearing capacity factors N_q have been published[5] for piled foundations (see Fig. 8) where $h/D = 25$. In addition test results have been published on the ultimate tensile and compressive loads on screwed piles,[6] from which it may be deduced that the ultimate bearing capacity is equal to 1·3–1·4 times the ultimate resistance to withdrawal for equivalent bearing areas. This combined information should be considered when assessing the value of B since the estimated values agree fairly closely with field observations in cohesionless soils to date.

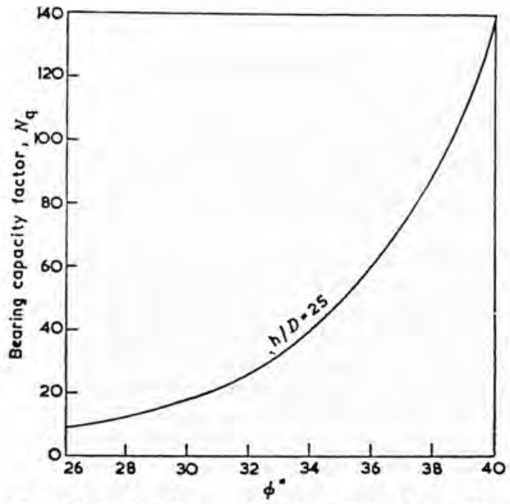


Fig. 8. Relationship between bearing capacity factor N_q and angle of internal friction ϕ (after Berezantzev et al.[5])

34. In compact Thames ballast ($\phi = 40°$) pull-out tests on injection anchors have indicated values of 1·7 and 101 for A and B respectively. From Fig. 8 the estimated value of B is 99–106, based on a ratio $N_q/B = 1{\cdot}3$–$1{\cdot}4$.

Fine to medium sized sands ($k_w = 100$–1 μm/s)

35. In this type of soil the fixed anchor formed consists of a smooth grout cylinder since the sand does not allow permeation of the dilute cement grout. As a result working loads of only 40 t ($S_f = 1{\cdot}5$) are normally mobilized when cement is used and the empirical rule equivalent to equation (2) is

$$T_f = Ln' \tan\phi \quad . \quad . \quad . \quad . \quad (4)$$

where $n' = 13$–$16{\cdot}5$ t/m. The range of application is $L = 0{\cdot}9$–$3{\cdot}7$ m, $D = 180$–200 mm and $h = 6{\cdot}1$–$9{\cdot}2$ m.

36. In compact fine to medium sized sand ($\phi = 35°$) pull-out test results on field anchors indicate values of 1·4 and 31 for A and B respectively in equation (3), where the anchor construction procedure is similar to the technique described for coarse sands and gravels (estimated $B = 35$–38). In fine soil the value of A depends primarily on the residual grout pressure at the fixed anchor/soil interface, since during the injection stage the cement forms a filter cake at the interface through which only water travels. Thus the injection

pressure on the filter cake is transmitted to the soil causing local compaction and a resulting increase in the fixed anchor diameter. When the injection stage is complete there is sufficient shear strength in the grout to enable a residual contact pressure to be locked into the system. In this connexion equation (5) has been used by some contractors for the estimation of the ultimate load holding capacity.

$$T_f \simeq p_i \pi DL \tan \phi \quad . \quad . \quad . \quad . \quad (5)$$

where p_i is the injection pressure during the grouting stage.

Stiff clay ($C_u > 90$ kN/m²)

37. Originally the technique of anchoring in stiff clay was simply to auger a hole to the required depth, home the cable and grout the fixed anchor length using the tremie method (see Fig. 9(a)). Anchorages of this type, however, are usually of low capacity since an adhesion of only 0·3–0·35 C_u may be mobilized at the grout/clay interface with the dilute cement grouts currently being used.

38. This situation can be improved by injecting irregular gravel into the augered holes over the fixed anchor length. Following this stage a small casing, fitted with a non-recoverable point, is driven by percussion through the gravel, thus forcing a proportion of the gravel to penetrate the surrounding clay. The cable is then homed inside the casing, the point is displaced and the gravel injected with cement grout as the casing is withdrawn. This technique increases the effective diameter of the grouted length in the fixed anchor zone and gives a more intimate and rough fixed anchor/clay interface (see Fig. 9(b)).

39. In this way a small end restraint component and a larger coefficient of adhesion may be incorporated into the empirical design rule.

$$T_f = \pi DL(0{\cdot}6\text{–}0{\cdot}75)C + \frac{\pi}{4}(D^2 - d^2)N_c C_u \qquad (6)$$

(side resistance) + (end resistance)

where D is the diameter of the fixed anchor (180–250 mm), d is the diameter of the shaft (130–150 mm), L is the length of the fixed anchor (3·1–7·6 m) and $N_c = 9$.

40. With the gravel placement anchor, safe working loads of up to 30 t ($S_f = 2$–$2{\cdot}5$) have been mobilized to date, and in cohesive soils, which are particularly susceptible to deterioration under water action, it is noteworthy that non-aqueous grouts may be used.

41. More recently further improvements have been brought about by the introduction of the multi-underreamed fixed anchor (see Fig. 9(c)). In this method an expanding brush underreamer is used to form a series of enlarged cavities or bells at close centres in the fixed anchor zone of the augered borehole (see Fig. 10). Thereafter the cable or rod is homed, centralized and grouted in the usual manner by the tremie method.

42. As a result of field tests on this type of anchor it is considered that the maximum side resistance is mobilized when failure occurs in the clay along a cylindrical surface linking the extreme points of the bells, and since this clay has not been disturbed by the construction procedure the full undrained cohesion is mobilized. Equation (7) represents the empirical design rule used for this type of anchor and a shaft adhesion component has been added since the use of cement grout is normally considered to be the cheapest way of filling the void immediately above the fixed anchor.

$$T_f = \pi DLC_u + \frac{\pi}{4}(D^2 - d^2)N_c C_u + \pi dl C_a \quad . \quad . \quad (7)$$

(side resistance) + (end resistance) + (shaft adhesion)

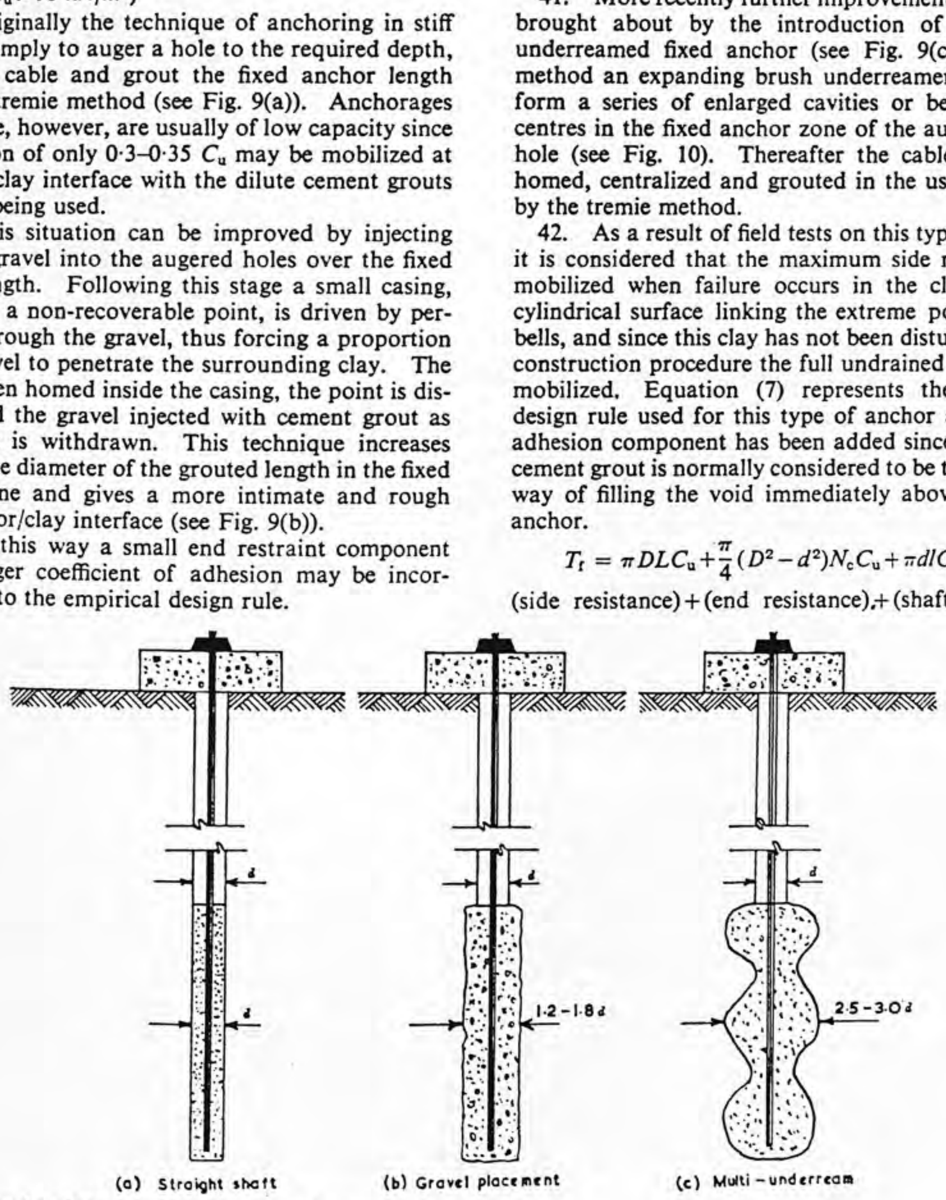


Fig. 9. *Augered anchorages in cohesive soil: main types*

Fig. 10. Section of multi-underreamed fixed anchor excavated from London clay at Vauxhall Bridge

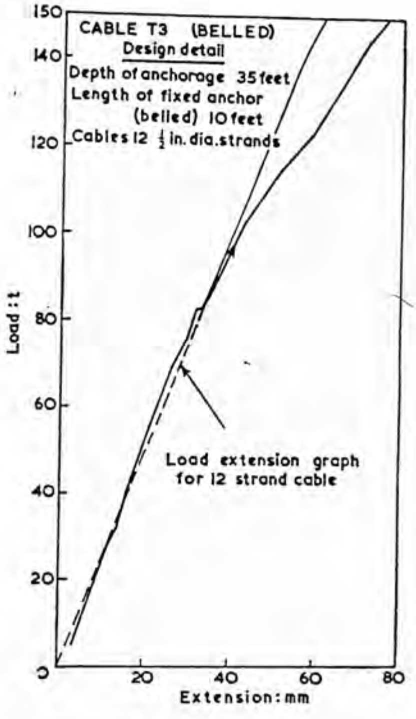


Fig. 11. Load–extension graph for multi-underreamed anchor formed in Keuper marl at Moseley Road, Birmingham

where D is the diameter of the bell (350–400 mm), d is the diameter of the shaft (130–150 mm), L is the length of the fixed anchor (3·1–7·6 m), l is the length of the shaft (1·5–3 m), $N_c = 9$ and $C_a = (0{\cdot}3\text{–}0{\cdot}35)C_u$.

43. Anchors of this type have already been designed and constructed to resist safe working loads of 60 t ($S_f = 3\text{–}3{\cdot}5$) although, where the clay adjacent to the fixed anchor zone contains open joints which can readily absorb flushing water during the underreaming stage, the value of C_u in equation (7) is reduced by half in the absence of test anchor data. This reduction does not apply to closed fissures, i.e. where the fractures are not visibly open.

Stiff to hard chalk

44. In grades III, II and I[7] of chalk safe working loads up to 100 t ($S_f = 1{\cdot}5\text{–}2$) have been mobilized, where the construction technique is simply to drill a hole of small diameter (76–102 mm), home the cable and grout the fixed anchor zone by the tremie method.

45. Variability of the chalk on any one site is the main problem for the anchorage contractor when attempting to optimize the design and construction, and rubbly chalk with soft zones of fissured material can change to stiff unfissured 'rock' chalk within a few yards. Test anchors pulled to failure at Reading and Ramsgate have established that the ultimate resistance to withdrawal, due primarily to skin friction, may be in the range of 214–1072 kN/m² of fixed anchor, and hence an assessment of the variability on any site is extremely important. In this connexion penetration test results from a site investigation can highlight potential problems and it is noteworthy that at Reading, where a stiff rubbly chalk ($N = 20$) changed with depth to hard blocky chalk ($N = 80$), the equivalent ultimate skin friction values increased from 214 to 805 kN/m². Thus on this particular site, although the standard penetration test values were used primarily to illustrate variability and not engineering behaviour, the test anchors provided a useful correlation and δ_{skin} (kN/m²) in equation (8) was replaced by 10N to give a more economical design.

$$T_f = \pi D L \delta_{skin} \quad . \quad . \quad . \quad . \quad (8)$$

where D is the diameter of the fixed anchor (102 mm) and L is the length of the fixed anchor (3·0–9·2 m).

46. In view of the limited data currently available on chalk, anchor design arrangements are now being made on a contract in Watford to pull 24 anchors to failure in an attempt to correlate load holding capacity with the engineering properties of the chalk.

Keuper marl—weathering zones I and II[8]

47. As in the case of chalk there is little information available on the design of anchors in Keuper marl. Pull-out tests on straight shafted anchors (90 mm dia.) formed in stiff to very stiff friable fissured marl at Charles Street, Leicester have indicated ultimate shaft

adhesion values ranging from 172 to 247 kN/m² (C_u=287–527 kN/m²). At this site equation (9) was used to estimate ultimate load holding capacity and 170 temporary anchors (working load = 30 t) were successfully installed with a factor of safety of 1·6.

$$T_f = \pi DL\, 0{\cdot}45\, C_u \quad . \quad . \quad . \quad (9)$$

48. More recently two multi-underreamed anchors have been constructed at Moseley Road, Birmingham, in marl similar to that already described. Test loads of 150 t were applied without any sign of failure (see Fig. 11). In view of these results it is now considered that safe working loads of up to 100 t (S_f=2–2·5) may be mobilized in marl where C_u is greater than 190 kN/m². At Moseley Road a straight shafted anchor (127 mm dia.) mobilized an average adhesion of 214 kN/m² at failure.

Safety factors

49. In the field of prestressed soil anchors factors of safety must be applied to the design of the individual anchors and the anchorage soil structure system.

50. Normally these factors are estimated, but in prestressed anchorage work post-tensioning in the field pretests the anchor thus ensuring its safety. In this way many of the estimated values can be checked to give measured factors of safety.

Individual soil anchors

51. The various safety factors recommended for current use are detailed with the following notation.

- T_b minimum breaking load of the steel cable
- T_f failure load of the grouted fixed anchor
- T_t maximum allowable test load to which an anchor can be temporarily subjected in order to check its capacity
- T_w working load of the anchor
- S_b factor of safety against breaking the cable
- S_f factor of safety against bond failure between the grouted fixed anchor and the adjacent ground

52. In multi-anchor systems where progressive failure must be prevented, the minimum factor of safety normally used against failure of the anchor is 1·6. Thus if for some unforeseeable reason an anchor completely fails during service, the adjacent anchors are capable of resisting the additional imposed load. Careful checks made on the tensile steel and top anchorage components guarantee this safety S_b for each anchor where the working load of the cable T_w does not exceed 62½% of its ultimate tensile strength T_b.

53. Since the local soil properties are not normally known with the degree of accuracy applicable to the steel components, a higher safety factor S_f is used for fixed anchor design to cover the uncertainties. A value of 2 is common to temporary and permanent anchors although in the case of permanent anchors in stiff clay S_f is increased to 3–3·5 to keep prestress losses within acceptable limits.

54. To establish a measured factor of safety against withdrawal of the anchor it is necessary to apply a temporary test loading on site. However, the allowable test load T_t is limited by the elastic limit of the steel cable and the maximum recommended test load is equal to 80% T_b. Thus, for a cable working at 62·5% T_b, the maximum measured safety factor which can be provided is $S_f = T_t/T_w = 1{\cdot}28$.

55. Every anchor should be tested to 80% T_b and representative anchors (1 in 10 say) should be constructed with extra cable where T_w = 50% T_b to give a measured S_f = 1·6.

56. In order to check and possibly optimize the fixed anchor design at the beginning of the contract a minimum of three test anchors pulled to failure is recommended where the fixed anchor length is varied, and the cable is designed in each case to ensure that failure occurs at the fixed anchor/soil interface.

Anchor soil structure system

57. When the geometry of the anchorage system has been decided, the stability of the whole system has to be checked to see whether the chosen anchor lengths are sufficient or not for a given factor of safety. A factor of 1·5 is customary, but as in all designs the choice is based on how accurately the relevant characteristics are known, whether the system is temporary or permanent and the consequences if failure occurs. A low factor of safety calls for a careful assessment of soil properties, accurate calculations and a sound theory. A type of stability analysis which can be used for retaining wall tie-backs is given in ref. 9.

Post-tensioning

58. During stressing it is usual to record the cable movement at the movable anchorage as the load is incrementally applied but, since the initial stretch of the cable at the jack ram may comprise fixed anchor displacement, cable elongation, wedge pull-in, bearing plate and structural movement, it will be appreciated that interpretation of such a load test requires skill and experience with the particular anchor system being used. In order to assess short term anchor performance therefore the following simplified procedure is recommended for consideration as a routine test.

(*a*) Test load anchor to 80% of the ultimate tensile strength of the cable, hold for five minutes and then reduce load to zero.

(*b*) Restress anchor to the required working load plus 10% and record cable movement at the ram as the load is incrementally applied. During this second loading cycle the load–extension graph obtained should compare closely with the estimated extension of the free length of cable. Lock off anchor at working load plus an allowance (usually 10%) for relaxation and pull-in of wedges.

(*c*) Check anchor load after 24 hours. If a loss of prestress in excess of 5% is recorded, restore to working load + 10% by shimming.

(*d*) Repeat (*c*).
(*e*) If a further loss of prestress is recorded, reduce anchor load until creep ceases. A safe working load for the anchor is then equal to 62·5% of the load showing no creep after 24 hours.

59. For special load tests where records of structural movement and fixed anchor displacement are required this procedure is used but with load/extension observations during the initial test loading cycle. In this case optical surveying techniques using a reference datum which is independent of the anchorage system are essential.

60. Broms[10] states that, if the spacing of the anchors is less than 2·5 m at any level, three anchors should be tested at the same time. However, Trofimenkov and Mariupolskii[6] show that the pull-out capacity of a deep screw pile formed in a sandy soil is unaffected by the presence of adjacent piles provided the spacing is not less than 1·5 *d*, where *d* is the fixed anchor diameter.

61. In the UK the minimum spacing is normally limited to 4 *d* but, if the loading intensity along a waling demands a closer spacing, alternate fixed anchors can be staggered by varying the anchor inclination. Even in this situation it is not normal practice in prestressed anchor work to load several anchors simultaneously since the routine post-tensioning procedure with its 24 hour loading check ensures that each anchor is making its proper contribution to the overall load. However, where a structural unit, such as a radial beam on the floor of a large stormwater tank, contains several anchors and must be loaded evenly all the anchors can be tensioned simultaneously.

Long-term behaviour

62. The majority of anchors currently constructed are prestressed to the designed working loads to minimize structural movements when the imposed loads are mobilized during service. Nevertheless structural movements and fixed anchor displacements do occur, and it is important to know what prestress fluctuations may be expected during the working life of the anchors.

63. Anchor forces may be checked after one day, then after one week and thereafter at monthly intervals if required in order to assess prestress fluctuations. By this form of routine check creep effects can be eliminated. Data on the long-term behaviour of soil anchors are limited but prestress losses due primarily to fixed anchor displacement are included in Table 1 as a guide when estimating realistic overloads.

64. The permissible variation in anchor force is usually 10% of the design value, but it is important to note that restressing should be carried out only after careful consideration. For example, in the case of a wall tied back using several rows of anchors in clay, a loss of prestress due to consolidation of the clay adjacent to the fixed anchor may be observed without accompanying movements of the retaining wall. In

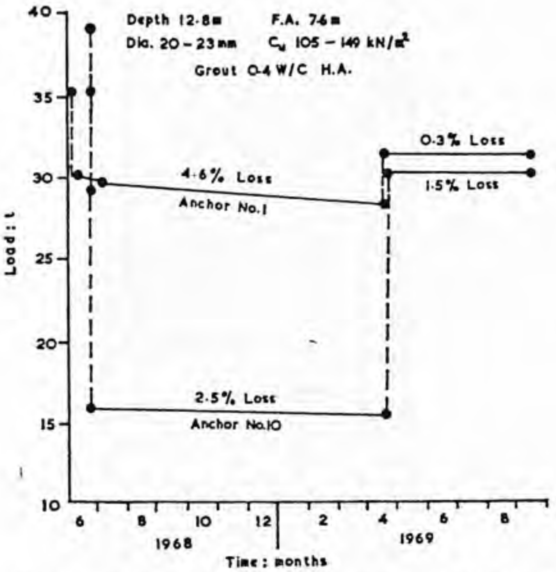


Fig. 12. Prestress loss–time graph for gravel placement anchor formed in London clay at Kilburn

these circumstances remedial measures may not be required.

65. At Kilburn the opportunity was taken to preload the anchors during the main construction period to minimize prestress losses during service. Fig. 12 shows the loss of prestress with time for two identical anchors Nos 1 and 10, preloaded to T_w and $T_w/2$ respectively for approximately ten months and then post-tensioned to the working load T_w plus 4%. After nine months' service the results are encouraging and further work on the effect of preloads in excess of T_w would be interesting especially in the more permeable and compressible soils.

Corrosion protection

Temporary anchors

66. Most anchors are of a temporary nature with a working life seldom in excess of two years. In these cases where the ground conditions are not hostile, cement grout around the fixed anchor section of the cable and a greased tape decoupling sheath over the elastic length form a reasonable protection. The movable head or top anchorage with protecting strands or bar may be sprayed or painted with a removable plastic coating.

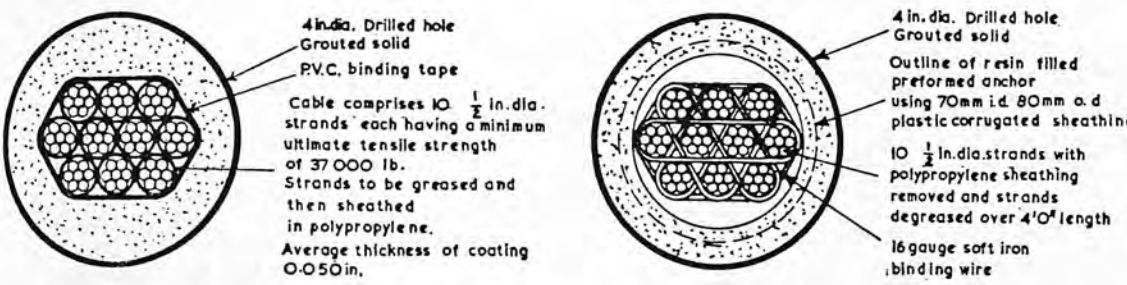
Permanent anchors

67. For permanent anchors or temporary anchors formed in a highly corrosive environment it is considered desirable to have a protective system which can be applied to the cable and inspected before homing. If the cable consists of several strands, it can be delivered to site fully protected and decoupled, where

Table 1. Long-term behaviour of injection anchors

Location	Soil	Fixed anchor length, *m*	Anchor diameter, *mm*	Prestress force, *t*	Number of anchors	Prestress loss due to fixed anchor displacement		Cable relaxation, %	Total prestress loss: mean, %	Accuracy of load measurement, %	Duration of observations, *weeks*	Remarks
						Mean, %	Range, %					
Vauxhall Bridge	Thames ballast ($\phi=40°$)	6·1	228	55	6	1	0–2	2	2	2	4	No prestress losses measured after four weeks and losses due to fixed anchor displacement appear to occur during the first 24 hours
Knightsbridge	Thames ballast	8·55	228 (estimated)	74	1	0	—	2	0	2	6	
Lambeth	Thames ballast ($\phi=35°$)	4·9	228 (estimated)	50	6	2	0–3	2	4	2	8	
Lambeth	London clay ($C_u=134$–168 kN/m^2)	10·7	356–406	55–66	8	4	0–10	2–4	6	2	26	Multi-underream anchor
Kilburn	London clay ($C_u=105$–149 kN/m^2)	7·6	203–228	30	1	5	—	2	7	2	43	Gravel placement anchor
Kilburn	London clay ($C_u=105$–149 kN/m^2)	7·6	203–228	16	7	2	1–4	2	4	2	43	Gravel placement anchor
Reading	Stiff rubbly/hard blocky chalk ($N=20$–80)	9·15	102	62	7	7	0–11	2–4	10	2	13	Straight shafted anchor

N.B. In all the situations quoted structural movements at the top anchorage were negligible, but in cases involving retaining wall tie-backs gains in prestress are more common since the wall normally yields slightly as the excavation proceeds. This wall movement can produce prestress gains of up to 20% which, for the periods of observation possible to date, overshadow the figures given.



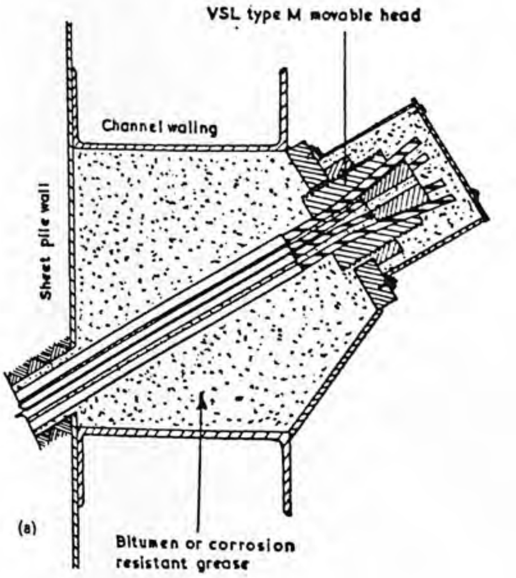
(a) SECTION BB (b) SECTION AA

Fig. 13. Typical cross sections of a fully protected restressable cable

it is greased and sheathed with extruded polypropylene under factory controlled conditions. Polypropylene has a high resistance to absorption of water and it resists attack by most inorganic acids, alkalis and salts and organic compounds. In addition, it is tough and abrasion resistant with the advantage of flexibility, and its use in fencing and for the covering of electric cables has been well established over the past ten years.

68. The thicker the plastic cover the longer the protection can be expected to be effective and the thickness specified depends largely on the environment and relative costs. For the majority of anchorage applications a coating 1·27 mm thick is considered adequate for corrosion protection and resistance to abrasion during anchor installation. In the unlikely event of severe mechanical damage during handling, repairs can be effected by the use of plastic tape and sealing materials such as epoxy resin.

69. The cable is fabricated on site using spacers (and where appropriate hole centralizers) and then the fixed anchor length of the cable is stripped of polypropylene and degreased before casting into a corrugated plastic tube using a high strength epoxy or polyester resin. These resins are specially recommended since they are the only materials which can match the corrosion protection offered over the elastic length of the cable and yet have sufficient adhesion to allow transmission of the imposed stress over the bond length of the fixed anchor without creep. On completion of the cable construction and a final inspection, the cable is homed and grouted in the normal way, to give a fully protected restressable anchor. Full-scale creep tests on a patented anchor of this type have been made up to working loads of 220 t with satisfactory results. Figs 13 (a) and (b) show typical cross sections of a protected cable over the fixed anchor and elastic lengths, respectively. When the anchor has been tensioned to the required load the top anchorage can be finally protected as shown in Figs 14 (a) and (b).



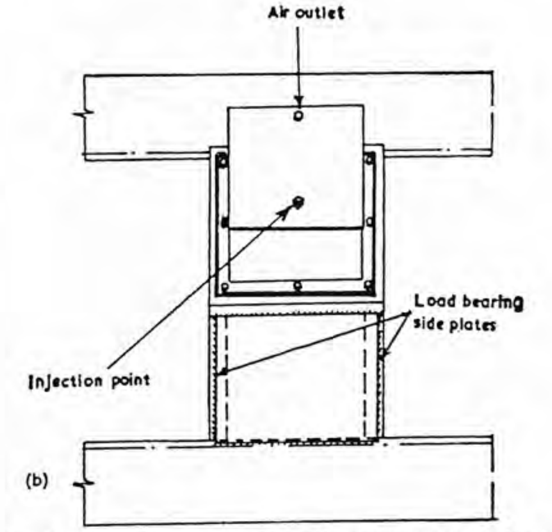


Fig. 14. Typical corrosion protection for permanent top anchorage

Future work

70. With the rapid development of anchor construction techniques for a wide range of soils there is a need for more test anchors taken to failure, especially on large contracts, since the results can be used to optimize the design and construction of the anchors on a particular site in addition to establishing actual factors of safety. In this way the validity of empirical design rules can be checked for the different soils encountered in anchorage work.

71. A minimum of three test anchors is recommended where the fixed anchor length L is varied, and for a particular site condition and anchor position an estimate of the magnitude of the side shear and end bearing components of the ultimate resistance to withdrawal may be obtained by plotting T_f against L.

72. With regard to long-term behaviour of individual anchors, variations of prestress with time should be recorded and it may be possible in due course to establish safety factors S_f related to period of service which will keep prestress fluctuations within acceptable limits. In this connexion the use of preloading techniques is worthy of further study.

73. In general site observations of anchor soil structure behaviour are required and full use should be made of field instrumentation to relate the performance of anchorage systems to the design assumptions.

References

1. McRostie G. C. and Schriever W. R. Frost pressures in the tie-back system at the National Art Centre Excavation. *Engng J.* Engineering Institute of Canada, 1967 (Mar.) 17–21.
2. Broms B. B. Methods of calculating the ultimate bearing capacity of piles. A summary. *Sols, Soils*, 1966 (18–19) 21–23.
3. Wild P. A. Tower foundations compacted. *Electl Wld*, 1961, **155** (Jan.) 36–38 and 66.
4. Littlejohn G. S. Recent developments in ground anchor construction. *Ground Engng*, 1968, **1** (3) 32–36 and 46.
5. Berezantzev V. G. *et al.* Load bearing capacity and deformation of piled foundations. *Proc. 5th Int. Conf. Soil Mech.*, **2**, 11–15.
6. Trofimenkov J. G. and Mariupolskii L. G. Screw piles used for mast and tower foundations. *Proc. 6th Int. Conf. Soil Mech.*, Montreal, 1965, **2**, 328–332.
7. Ward W. H. *et al.* Geotechnical assessment of a site at Mundford, Norfolk, for a large proton accelerator. *Géotechnique*, 1968, **18** (4) 399–431.
8. Chandler R. J. The effect of weathering on the shear strength properties of Keuper marl. *Géotechnique*, 1969, **19** (3) 321–334.
9. Ranke A. and Ostermeyer H. Contribution to the investigation of stability of multi-tied walls. *Bautechnik*, 1968, **10**, 341–350.
10. Broms B. B. Swedish tie-back systems for sheet pile walls. *Proc. 3rd Budapest Conf. Soil Mech.*, 1968, section 1–3, 391–403.

Anchored Diaphragm Walls in Sand—Some Design and Construction Considerations

G. S. Littlejohn, B.Sc., Ph.D., M.I.C.E., F.G.S., B. J. Jack, M.I.Struct.E., and Z. J. Sliwinski, M.I.C.E.

G. S. Littlejohn

B. J. Jack

Z. J. Sliwinski

BIOGRAPHIES

Dr Littlejohn is a graduate of the Universities of Edinburgh and Newcastle upon Tyne and is at present a consultant with Cementation Ground Engineering Ltd. He is responsible for research and development in ground engineering with particular reference to geotechnical processes.

For the last five years he has been concerned with the development of anchor design and construction techniques in a wide range of soils and soft rocks.

Mr Jack has been a Senior Design Engineer with Cementation Technical Services Division for the past seven years and his work has involved the design of marine structures and specialist foundation problems. For the last four years he has specialised in computer aided solutions to civil engineering problems and is now in charge of the Company's IBM 1130 computer which handles the technical computing requirements of the firm.

Mr Sliwinski graduated from Warsaw Technical University in 1928 and is a former director of Braithwaite Foundation Company where he was in charge of all overseas work.

He joined Cementation in 1964 and was later appointed Deputy Manager of the large diameter pile and diaphragm wall section. He is at present working in a consulting capacity with Cementation Piling and Foundations Ltd.

SUMMARY

The Paper introduces a method of estimating the anchor loads required to support a multi-tied continuous wall. It involves a procedure which calculates the position and magnitude of a resultant tie at any stage of excavation by treating the wall as a single tied structure.

Comparison of designs carried out by the Cementation method and experimental work indicates that the results obtained provide a good estimate of the horizontal forces. Also shown are the results obtained by the generally used methods.

The new method has the advantage of being a repetitive single-tied wall design, and it is amendable to varying soil strata which has always been a problem when implementing the trapezoidal method.

The Paper then describes the main design and stability considerations associated with trench excavation under bentonite, and gives recommendations covering the main requirements for tremie concrete for load bearing diaphragm walls.

Methods of estimating anchor location, overall stability and load carrying capacity with relevant safety factors, are illustrated. Anchor construction stages are described together with the post-tensioning procedures and corrosion protection normally recommended for sand anchors. Finally, the influence of prestressed tie-backs on the lateral movements and settlement of the retained soil mass is discussed.

The Paper is divided into four parts. Part I, Wall Design, is by Mr B. J. Jack, Part 2, Wall Construction, by Mr Z. J. Sliwinski and Parts 3 and 4, Anchor Design and Anchor Construction by Dr G. S. Littlejohn.

Introduction

Due primarily to the increasing tendency to design buildings with a number of basement floors, the formation level of the excavation being often at considerable depth below the foundations of the neighbouring properties, methods of temporary and permanent earth support have been developed in recent years to keep pace with the increased efficiency of modern construction.

The diaphragm process of constructing load-bearing walls in the ground prior to main excavation, based on the use of bentonite slurry to hold open the excavation until concrete has been placed, is such a method. The technique is of special value in built-up areas since diaphragm walls may be constructed in very close proximity to existing buildings, where other methods of piling and trenching may be ruled out by restrictions on access and noise, and where a wall with high structural efficiency and few joints is required. In addition, settlements of the soil surrounding the excavation are minimised due to the ability of the bentonite to reduce loss of ground during wall construction and the strength and stiffness of the wall itself. In some instances, the diaphragm wall may serve as the exterior wall for the permanent structure.

In many cases it is possible to achieve appreciably increased efficiency, as well as reducing settlements during the main excavation, by using prestressed soil anchors for supporting the wall. Anchors provide intermediate points of support at one or more levels, thereby reducing bending moments, with

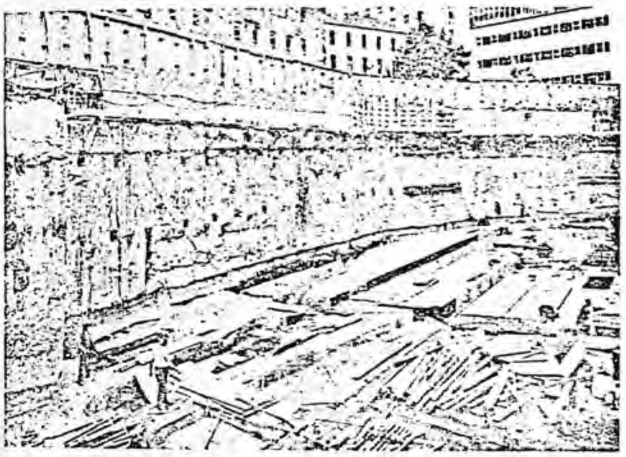


Figure 1 General view of anchored diaphragm wall. By courtesy of Trollope and Colls Ltd., — Guildhall Precincts Development.

consequent reductions in dimensions, reinforcement and depth of the toe of the wall. Interior struts can be eliminated which in turn brings quite large economic and constructional advantages. This is especially so in cramped excavations, in wide cuts or on sites where the contract programme calls for the use of efficient excavation and construction machinery. (Figure 1)

Although both diaphragm wall and soil anchor techniques have been proved since the late 1950's, there is little published information on walls supported by prestressed tie-backs. The purpose of this Paper is to discuss in some detail the main design and construction aspects associated with an anchored diaphragm wall in sand, which are likely to be considered by the practising engineer. The Paper also emphasises that whilst design procedures for excavations supported by soldiers and planking are readily available, the design of continuous walls supported by tiers of prestressed and inclined soil anchors entails a fundamental difference because construction methods play a more significant part in determining the forces and displacements which result in the wall and anchors.

1. WALL DESIGN

General Considerations

When studying the support of excavations the following topics have to be considered.

1) The type of soil to be retained.
2) The type of wall to be used.
3) The method of design.

The first stage is the collection and interpretation of soils data. In this connection the design engineer has to decide the type of soils investigation required to facilitate the design of the permanent structure and provide the construction engineer with sufficient information to build the work. These two separate requirements can quite often lead to different soils tests and affect the distribution of boreholes throughout the site. The role of the soils engineer in this kind of problem cannot be over-emphasised, he and the wall designer should co-ordinate from the initial concepts of the work right through to final design stage and even into the construction period.

Once the soils investigation has been carried out and test results are available, both for the permanent and temporary works construction, the design engineer must choose what type of wall is to be employed for the support of the excavation. At the moment a wide variety of methods is available, varying from planking and strutting through to the use of diaphragm walls which can be incorporated in the permanent structure. Having chosen the method or a number of methods of retaining the soil the design engineer is now faced with the problem of how to determine the structural stresses, tie bar forces, etc.

The design of walls to support deep excavations is an art rather than a science, but nevertheless it is essential that a logical approach is taken in the design. Shallow excavations can utilise cantilever or single-tied walls for which much valuable information has been supplied by Rowe(1) in his experimental work. By a careful study of the soil parameters, wall flexibility, etc. in these designs the engineer can make a fair estimation of the behaviour of the structure in practice. At present little direction is available on continuous multi-tied wall design, where cable anchors or struts are incorporated in the support system. For deep walls using planking and strutting the methods recommended are generally based on the "trapezoidal" pressure distribution which was originated by Terzaghi. This method was established from experiments carried out on an excavation in Berlin through cohesionless material supported by soldier piles and planking. It should be remembered, however, that the pressure distribution was based on an envelope enclosing assumed parabolic distributions at each stage of excavation, the size and shape being calculated equal to the magnitude and distribution of the actual measured strut forces. This trapezoidal pressure distribution is valid only under certain conditions since the actual earth pressures are a function of the degree and type of freedom for lateral expansion of the retained soil.

It is evident that the soil pressures existing behind contiguous piles, steel sheet piling and diaphragm walling will be different to those of a soldier pile construction as the earth support provided by the material to be excavated at any stage acts on the full wall face and not on small areas, which is the case when using soldier piles. The bearing pressure will be less, reducing the wall's lateral movement, thus affecting the re-distribution of earth pressure.

Design Parameters

Rowe's work on single tied walls has clearly shown that one of the most important factors in this type of design is the wall's flexibility, which tends to reduce the maximum bending moments and increase the tie bar forces through arching of the ground. It is quite logical that the smaller the deflection, the smaller will be the arching effect and it is reasonable to expect that where the wall continues below the excavation level, a high degree of fixity will produce a less effective span, a smaller wall deflection and less redistribution of soil pressures due to arching. It therefore seems inappropriate to use the "trapezoidal" method when designing walls of this type, because in these instances, a more triangular pressure distribution would be expected. If an extreme case is considered where a continuous diaphragm or sheet piled wall is constructed in the ground and struts are inserted at close centres as the excavation proceeds, little or no redistribution of soil pressures will take place and the resulting pressure diagram will be of a triangular form. The actual values will depend on the type of wall used. In the case of steel piling, where the wall is driven into the ground, the pressures before excavation at either side may be considered "at rest", whereas with diaphragm wall construction, where an excavation is opened up and the ground stabilised by the use of bentonite, some yielding of the soil occurs, tending to reduce the "at rest" pressures to those approaching the active state. Therefore, even within the field of continuous retaining walls, different parameters will have to be employed depending upon the type of construction, spacing of props etc.

The stress distribution, which governs the design, depends upon the walling material used, the method of construction and the centres at which the struts or ties are installed since it is the effective span at any one stage of construction which will determine the deflection, and consequently the re-distribution of soil pressures. One of the most important factors in this respect is the strength of the soil in front of the wall mobilising the passive pressure as this determines the additional dimension below excavation level making up the effective span.

In the case of multi-tied walls, as stage by stage excavation proceeds the passive pressure producing the effective span at any one stage can be assessed approximately, but as excavation proceeds past a particular stage the ground which was restraining the wall is removed and further deflection can take place. The amount of passive resistance built up at any intermediate stage, when removed, must be added to the active pressures on the wall, since in the previous stage it was subtracted to produce the resultant pressure diagram and hence the tie bar forces above. If this is done then the final soils distribution achieved will be of a triangular form with exaggerated "bumps" at each tie level, the amount of additional pressure at a tie being dependent upon the passive resistance built up at the installation stage of that tie.

With regard to the soil parameters which should be used in determining the passive pressure which is available at any particular stage of excavation, it is considered that the design should be evaluated for two conditions, firstly using the immediate or undrained soil parameters and secondly with the drained soil parameters, as it has been shown that the latter condition can arise in a very short time period. This approach, is therefore, applicable not only to the design of works of a permanent nature such as underpasses but also for temporary works where, as in basement construction, the wall is only exposed during the construction period. Since the drained parameters approach the ultimate strength of the soil the factor of safety when using these parameters can be reduced below that normally employed, and values of 1.1 to 1.2 are recommended for consideration.

Design Method

The following design method for multi-tied walls has been developed by Cementation to incorporate the effects of the temporary support produced by the passive pressure at intermediate excavation stages.

It involves a procedure which calculates the position and magnitude of a resultant tie at any stage of excavation by treating the wall as a single tied structure.

The method requires that the following assumptions are made:—

(1) The mobilising and resisting soil forces are those determined using Rankine's earth pressure theory;

(2) At failure there is a unique point of rotation in the plane of the wall; and

(3) The wall is only of sufficient length to mobilise a factor of safety of unity against rotation at any stage of excavation.

The first assumption is made to simplify the calculations, and is the usual one made when calculating earth pressures in the design office.

The second assumption, that the point of rotation occurs in the plane of the wall, enables the following simple procedure to be used in calculating the additional tie bar forces produced when the passive pressure is "transferred" to the active side during the next stage of excavation.

Consider the equilibrium conditions of the system shown in Figure 2 (a) and (b).

From Figure 2 (a):

$\Sigma H=0$ is satisfied when:

$$T_1=P_a'-P_p' \qquad (1)$$

$\Sigma M=0$ is satisfied when:

$$M_p=M_a \quad \text{(about position of } T_1\text{)}$$

From Figure 2 (b)

$\Sigma H=0$ is satisfied when:

$$T_1+T_2=P_a''-P_p''$$

$\Sigma M=0$ is satisfied when:

$$M_p''=M_a'' \quad \text{(about the centroid of } T_1 \text{ and } T_2\text{)}$$

In considering a resultant tie R_1 in Figure. 2 (b) acting at the centroid of T_1 and T_2,

then:

$$R_1=T_1+T_2=P_a''-P_p''$$

Substituting for T_1 from equation (1)

then $P_a'-P_p'+T_2=P_a''-P_p''$

hence $T_2=P_a''-P_p''-P_a'+P_p'$

which gives the amount of pressure transferred to T_2 when excavation proceeds to the point for the insertion of T_3, Figure 2 (c) shows this represented diagramatically.

The remaining problem now is to calculate the position and magnitude of R_1. As T_2 is unknown then both the position and magnitude of R_1 are unknown. By assuming one the other may be calculated and the initial assumption checked. Since the lever arm of R_1 to the position of factor of safety

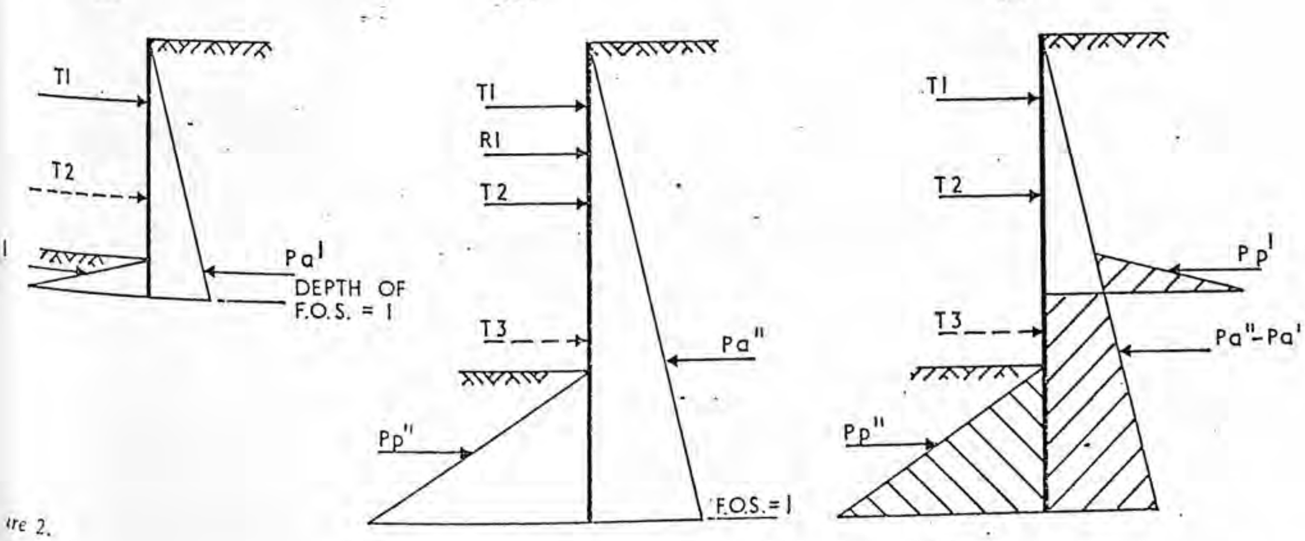
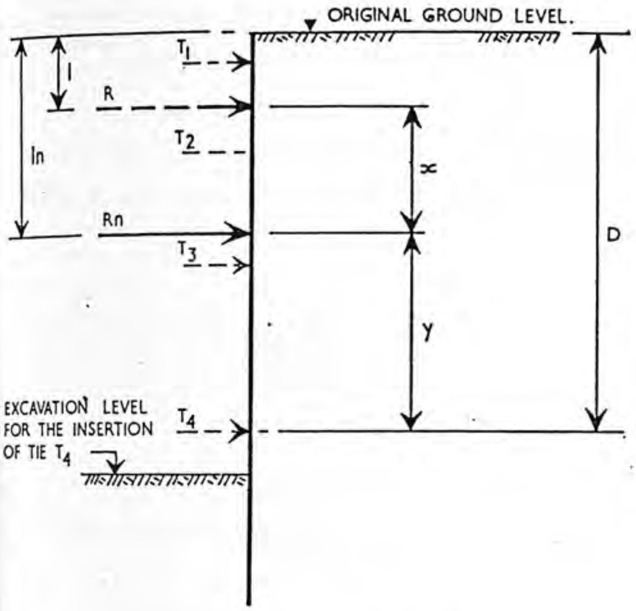


Figure 2.

of unity is usually small in comparison to its magnitude it is necessary that this dimension is calculated to a high degree of accuracy and the following iteration procedure is recommended to ensure a convergence upon the correct answer.

Figure 3 shows the case where the excavation level has been reduced to a position for the insertion of the fourth tie.



R — PREVIOUS RESULTANT TIE FORCE.
R_n — NEW RESULTANT TIE FORCE.
l — PREVIOUS RESULTANT TIE FORCE LEVEL.
l_n — NEW RESULTANT TIE FORCE LEVEL
$T_1, T_2, T_3,$ & T_4
INDIVIDUAL TIE FORCES.

FIG 3

Considering the equilibrium of the system.

$\Sigma H = 0$ is satisfied when:

$$T_4 = R_n - R$$

$\Sigma M = 0$ is satisfied when:

$$(R.x) - (T_4.y) = 0 \qquad (2)$$

where $y = D - x - 1$

Substituting in (2)

$$f(x) = (R.x) - T_4\,(D - x - 1)$$
$$= (R.x) - T_4D + T_4.x + T_4.1$$

Substituting in Newton — Raphson's iteration formula,

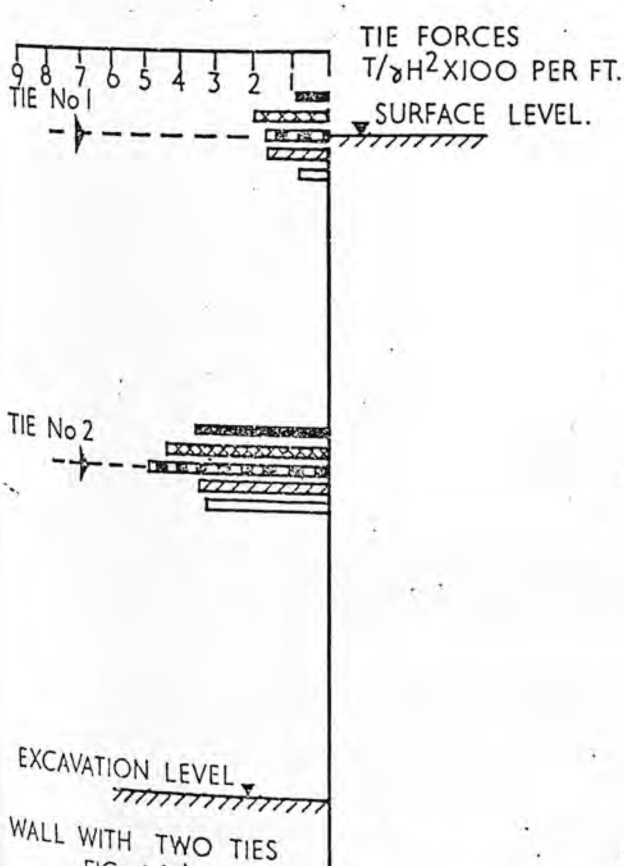
$$x_{n+1} = x_n - \frac{R.x - T_4D + T_4.x + T_4.1}{R + T_4}$$

where x_{n+1} is the new estimate of X
and x_n is the previous estimate of X

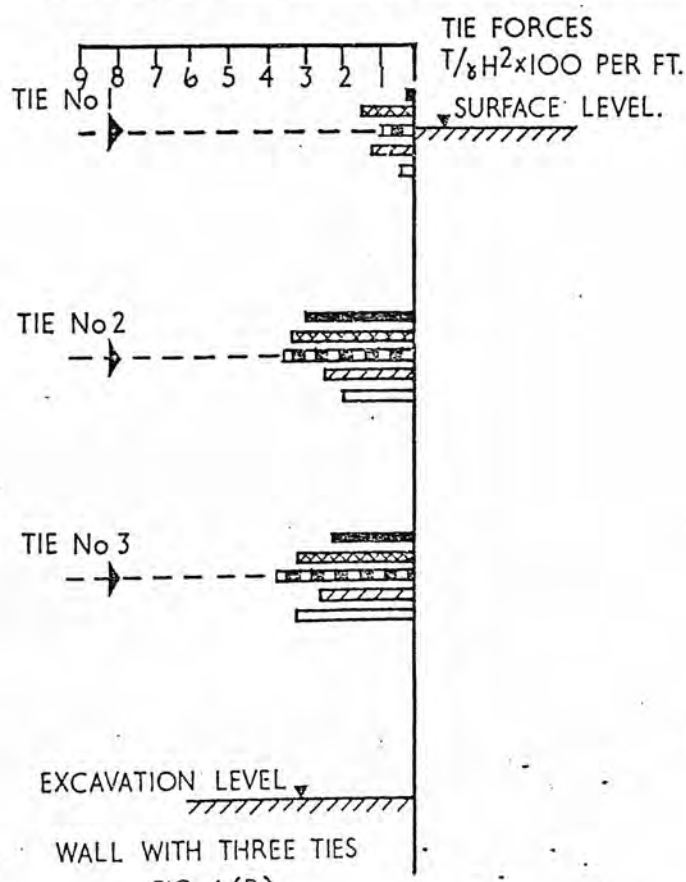
(This process is continued until the required accuracy is reached).

This method can now be followed through a typical design by considering the action of the wall as excavation proceeds. The first stage of excavation is usually a cantilever having an exposed height of the order of 2 to 5 metres. The bending moments in the wall under this condition can be determined by the usual methods, but it should be noted that the maximum bending moment occurs below the first tie level and not at the tie level as is often assumed when designing walls of this nature.

After the installation of the first tie, the second stage of excavation proceeds down to the level required for the insertion of the second tie. Under this condition a single-tied retaining wall exists for which, as mentioned previously, knowledge of the interaction of the soil and the wall flexibility is available. However, when the second tie is installed, very little information is available on the re-distribution of the soil pressures. In this design method, it is assumed that there is a point of rotation as in the single-tied wall but this point does not occur at a tie level but at some intermediate level between the first and second ties. Since this level cannot readily be deter-



WALL WITH TWO TIES
FIG 4 (A)



WALL WITH THREE TIES
FIG 4 (B)

mined its position has to be estimated, as outlined previously.

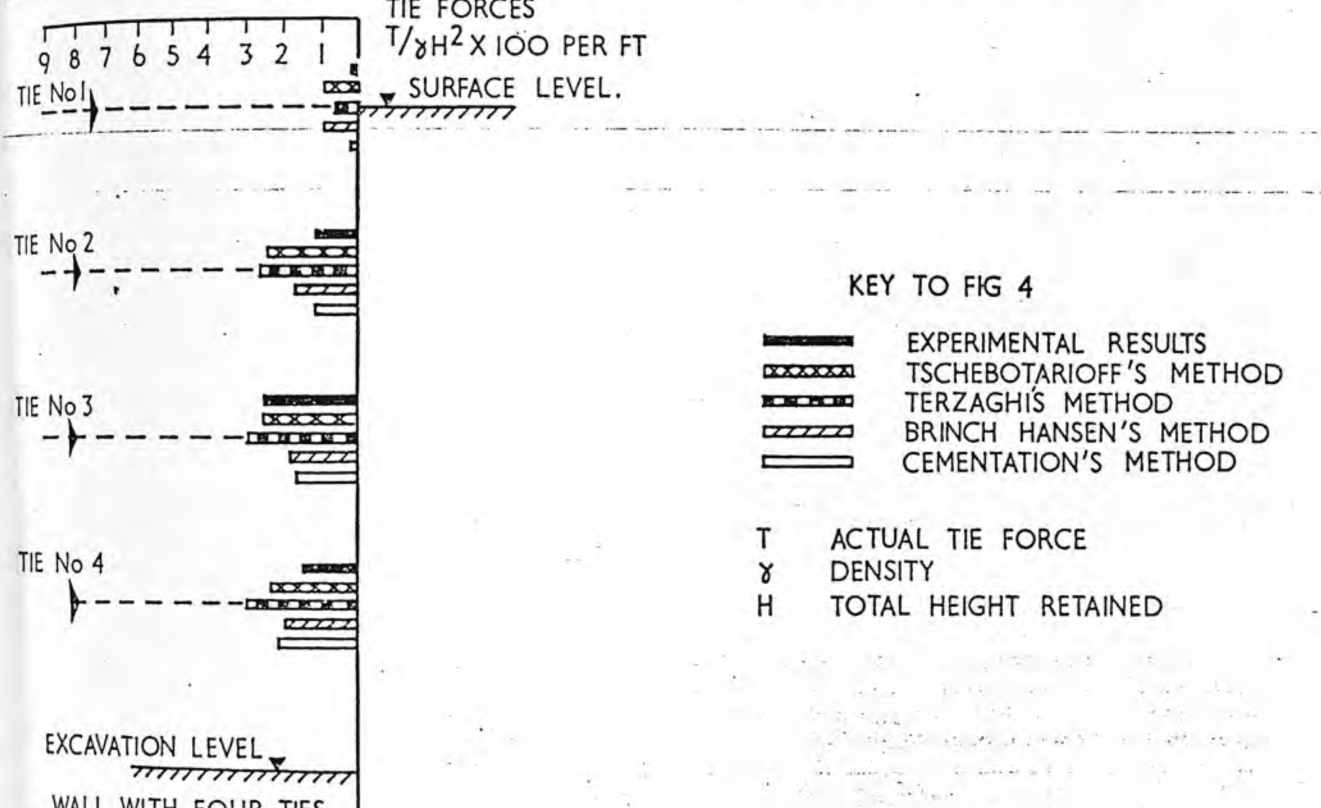
This procedure may be continued down through any number of ties.

Comparison with Existing Design Methods

Comparison of designs carried out by the Cementation method and experimental work indicates that the results obtained provide a good estimate of the horizontal forces and Figure 4 shows this method compared with model experiments carried out by Rowe and Briggs(2) for 2, 3 and 4 tied walls. Also shown are the results obtained for analyses carried out by the generally used methods.

Figure 5 shows the method compared with a full scale experiment and designs for this project submitted by various engineers.

Field evidence from other tests on full scale walls is encouraging, but modifications to the method will be built in if necessary in the form of flexibility coefficients when the complete results of full scale and model tests, currently being carried out by Cementation, are available.



WALL WITH FOUR TIES

FIG 4(C)

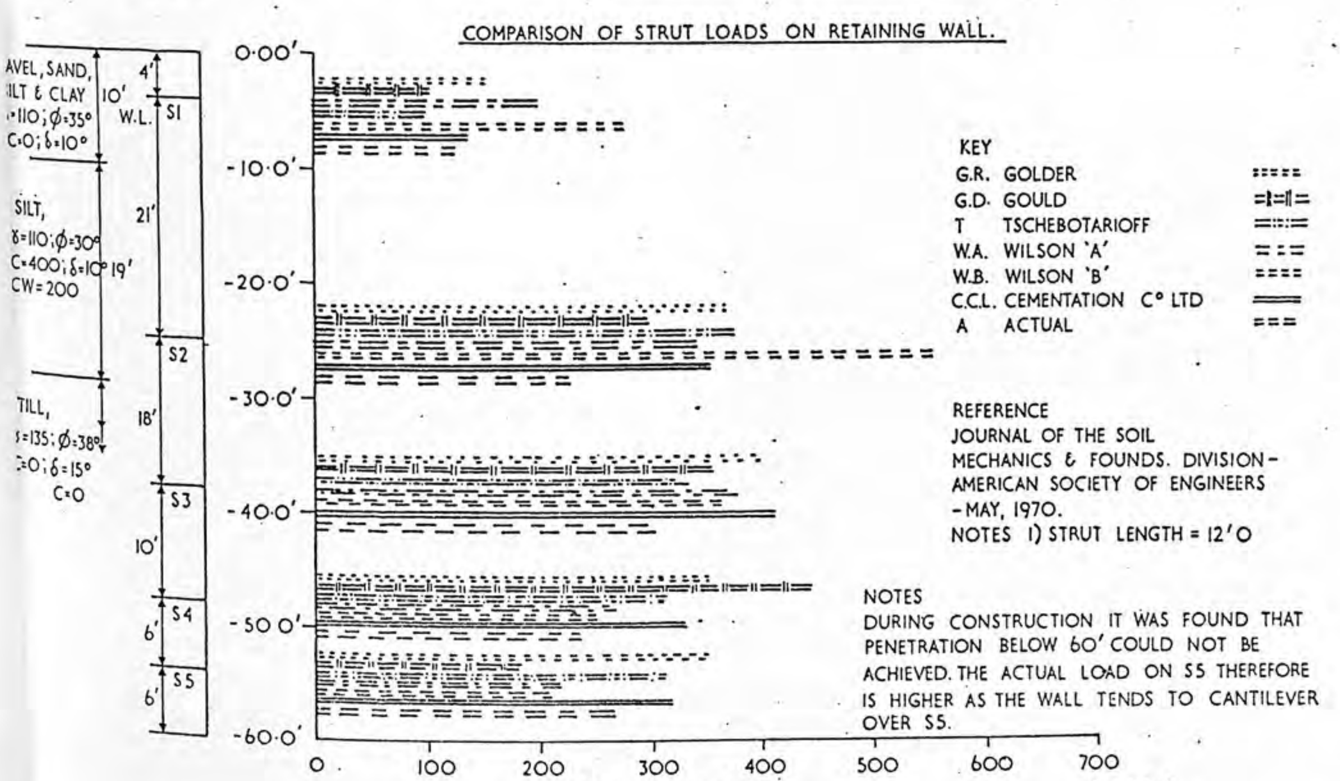


FIG 5.

It is considered that until a method of design is produced which takes into account the full interaction of the wall and the soil flexibilities then empirical methods of this nature must be used. The approach described takes account of varying soil strata, wall flexibility and method of excavation, and should enable the design engineer to make a better assessment of the wall's behaviour than by assuming a trapezoidal pressure distribution for a continuous wall, for which it was not originally envisaged.

The main advantages of the new method are listed below:—

(1) It is a repetitive single-tied wall design with which most engineers are familiar and should involve no difficulties in implementing;

(2) It is amenable to varying soil strata which has always been a problem when implementing the trapezoidal method;

(3) It allows struts or ties to be inserted at levels chosen by the engineer to take full advantage of the initial cantilevering abilities of diaphragm walls which is difficult, if possible at all when using the trapezoidal method;

(4) It allows the wall penetration to be calculated based on a rotational criteria as well as the direct sumation of horizontal forces used in the trapezoidal method; and

(5) Although the procedure is simple, it produces results which comply closely with current experimental data available.

The necessity of close co-ordination between the design, soils and construction engineers throughout all stages of the work is again emphasised because, whenever the problem of foundations is being studied, especially in the field of retaining walls, the design may have to be amended as the work proceeds. The limited number of boreholes which can be put down on any site is generally insufficient to provide a complete picture of the ground strata and tie bar positions and forces may have to be varied as construction takes place. For this purpose it is necessary for a quick and rapid design method to be available. It is noteworthy that Cementation have produced a computer program which can analyse continuous multi-tied walls supporting soils of varying characteristics which enable quick amendments to be made to the design based on the soil conditions exposed as the wall is being constructed.

2. WALL CONSTRUCTION

Plant for Excavation of Diaphragm Walls

The plant used for excavation of diaphragm wall trenches can be divided into two main groups:—

(i) First group — reverse circulation plant. The principle of machines of this type follow the drilling technique in which the rotary drilling bit loosens the ground, which is mixed with bentonite suspension and brought up by circulation of the fluid through a hollow drilling rod (or kelly). The bentonite suspension plays a double purpose of stabilising the excavation and conveying the soil to the surface.

ii) The second group uses tools which directly excavate the soil. The bentonite suspension is only employed to provide the support for the sides of the trench. There are many types of plant in this group:—

a) Mechanical diggers — either back or forward acting, can be used successfully for shallow depth (2.5-3 metres).

b) Bucket excavators of special construction (E.L.S.E.)

c) Machines using special trenching grabs.

The last type is most interesting and in recent years economical and efficient machines have been produced using grabs. The trenching grab is the width of the trench but its length is several times greater. The usual length is about 2 metres. Originally, trenching grabs were exclusively rope operated and this type is still widely used, but to improve efficiency, power closing grabs have now been developed. The power can be hydraulic (ram operated mechanism) or electric (motors in the grab). The superiority of power in closing grabs consists of speedier operation and the fact that the full weight of the grab effectively presses its jaw into the bottom of the trench during closing. In the rope operated method the weight is partly used to close the grab.

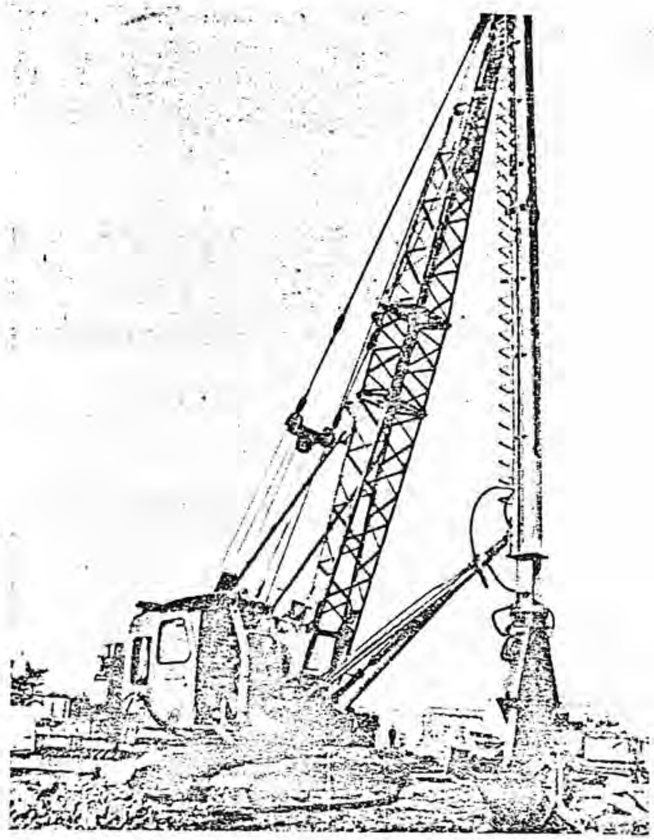


Figure 6

Further improvement in design consists of providing a kelly to which the grab is fixed. The kelly is guided above the ground and the purpose of the arrangement is:—

(i) to position and stabilize the grab above the trench in the minimum of time;

(ii) to guide the grab during lowering in a true vertical line; and

(iii) to provide additional weight to the grab.

Kelly-guided grabs are usually erected as an attachment to standard cranes forming an efficient unit. Figure 6 shows the Cementation Trencha Grab which can reach a very high output — 10 or even 15 sq.m per hr.

Trench Excavation

(a) Bentonite Suspension

The first recorded use of bentonite suspension to stabilize the side of a trench was for a wide excavation of a cut off wall with plastic fill in America in 1950. Later, about 1952-53, a narrow type of concrete diaphragm was executed in Italy. By 1954 a "slot" type excavation was an established procedure.

Bentonite is a special kind of clay (sodium form of Montmorillonite) and the properties of bentonite suspension were first studied about 1926 by Freudlich and later by Lorenz and Veder. These physical properties, which make bentonite useful in stabilising the sides of a trench during construction, may be summarised as follows:—

(i) Dispersion

Bentonite, like other clays, is essentially insoluble in water, but when mixed with water disperses under hydration much more easily than other clays. The dispersed particles are elongated, disc-like, about 2-3 microns thick and up to 300 microns in length. Even a low concentration of bentonite of a few per cent (by weight) readily forms a colloidial suspension, which in many respects behaves like a solution.

(ii) Thixotropy

The bentonite suspension exhibits thixotropic properties, i.e. it gels when undisturbed but becomes fluid when agitated by mechanical stirring. The most popular explanation of the thixotropy of bentonite is based on a theory that the clay particles have negative electrical charges on their planer surfaces and positive charges on the edges. In a state of gel the particles are orientated by electrical forces—a negative surface to positive edge—and form a "lattice structure", which produces the effect of a gel. When the suspension is agitated, the bonding forces are broken, the particles are orientated at random and the suspension becomes fluid. Left undisturbed, the "structure" is automatically rebuilt into a gel form.

(iii) **Capacity to form a filter cake.**

When placed over a filtering medium the bentonite suspension loses part of its water into the permeable material whilst the solid constituents form a "cake" of clay particles on the surface of the medium. This filter cake, of few millimetres thickness, provides a fairly resistant skin, which can be practically impervious.

When a bentonite suspension is introduced into an excavation where the strata are pervious, it will penetrate the soils. The depth of penetration depends on the excess hydrostatic pressure, the permeability of the strata and the viscosity of the fluid. When equilibrium is reached, the fluid forms a gel in the trench and in the penetrated zone. Under excess pressure a filter cake is formed on the sides of the excavation. As the digging tool enters the excavation it turns the gel into a fluid, but its disturbing action does not extend to the gel entrapped in the penetrated zone or break down the cake membrane on the sides.

This virtually impervious membrane allows the hydrostatic pressure of the suspension to act on the sides of the excavation, without raising the pore water pressure within the mass of the soil. This is one of the basic principles of the stabilizing action.

(b) Stability

The question of stability and equilibrium of forces under bentonite is often discussed and the names of Schneebeli, Morgenstern(4), Nash(5), Veder(7) and Elson(9) are well known. The subject of the stability created by bentonite was also the topic of one of the Speciality Sessions of the Seventh International Conference on Soil Mechanics and Foundation Engineering in Mexico 1969. At this session, the reporter, J. Florentin(10), said "The record of some three million square metres of walling already completed tends to show that a method of construction has been developed before a theory of stability, co-ordinated and acceptable to everybody, can be established." He added encouragingly "Should we not be reminded that aeroplanes were flying before aerodynamics existed."

The problem consists of expressing theoretically the state of equilibrium which must exist in practical cases, between earth pressure and the support offered by a bentonite suspension. In many cases it is not possible to balance the forces on the basis of the simple hydrostatic pressure of the suspension and earth pressure based on existing classical theories; hence the tendency of researchers to look for some secondary factors, which can contribute to the stability. Such factors often discussed are:—

(a) The shearing resistance of bentonite in gel form.
(b) An arching action of the soil in relation to the usually limited dimensions of an excavated panel.
(c) The resistance of the bentonite cake, which can act as reinforcement in both vertical and horizontal directions to the sides of the excavation.
(d) An increase in the shearing resistance of the zone of soil which the bentonite gel permeated beyond the cake.
(e) Electro osmotic forces.

In practice, Elson(9) estimates that the main stabilizing force is the hydrostatic pressure of the suspension, which accounts for 75 to 90 per cent of the stabilizing force. It is a matter of experience that, if any minor collapse of the trench does occur, this usually involves a zone of soil at shallow depth, for example, just beneath the concrete guide walls. It may well be, therefore, that classical theory overestimates the soil pressures in the lower part of a narrow trench.

Since no precise formulae exist at present for the calculation of the stability of an excavation under bentonite suspension, it is up to the specialist engineer to assess the conditions and use his own judgment, based on site experience.

The following procedure is proposed in the analysis of the problem. It is hoped that it will help the engineer in his appreciation, showing what are the essential assumptions and what is left as intelligent speculation.

(i) First the possible passive resistance of the bentonite suspension must be examined since this is the basic force which props the sides of the excavation. Neglecting the shearing resistance of the bentonite suspension, the basic stabilizing force can be expressed as a simple hydrostatic pressure which depends on the specific gravity (S.G.) of the suspension. The S.G. of a freshly mixed suspension of low concentration is only slightly higher than 1.0 but, when introduced into the trench, it soons entraps grains of soil and increases in density. Experience shows that, in average sandy conditions, the S.G. can rise to about 1,2 when grab excavation is used. (It is less if a reverse circulation system is used.) Table 1 illustrates typical S.G. values for bentonite mixes.

TABLE I

BENTONITE SPECIFIC GRAVITY RANGE

Contract	General Ground Conditions	Normal Bentonite Mix (%)	Specific Gravity Range
A	Made ground Cobbles, Clay	4.5	1.05–1.19
B	Sand & Gravel	7.5	1.10–1.24
C	Shale, Gravel, Sand, Sandstone	9	1.15–1.20
D	Cobbles in hard Clay	4.5	1.05–1.10
E	Sand/silt	6	1.15

Next it is necessary to check the level of bentonite which can be maintained at all times during the trench excavation and concrete placing stages. It is necessary to examine the strata for possible loss of bentonite, which the supply could not maintain without a lowering of its level in the trench. Very open strata, of permeability (Kw) more than 5cm/sec, can practically prevent the level being maintained at the required height. Similarly, cavities or unused drains can cause a sudden lowering of bentonite level.

The level of bentonite in relation to the ground water table is also important since the filter cake can only be formed if there is an excess of head in relation to ground water level. The practical lower limit is about 1.5 metres.

As a result of the above study, a pressure line of basic bentonite support can be drawn for both the minimum and maximum anticipated density, bearing in mind that the density of the suspension will increase from the lower to the upper limits of this range, as the excavation proceeds.

(ii) The second analysis refers to the expected active earth pressure. Employing the classical two dimensional theory of earth pressure, soil parameters such as ϕ and unit weight must be carefully established together with the water level and its variations. In addition surcharge loadings from adjacent foundations must be assessed when present in order to facilitate the calculation of the active pressure in terms of K_A. A pressure line can then be drawn to the same scale as the pressure of the bentonite.

(iii) A first comparison of the bentonite hydrostatic pressure and the active pressure lines should provide a general appreciation of stability.

If the line of bentonite pressure is at or above the line of active earth pressure, there is obviously nothing to worry about. If the deficiency of bentonite pressure is more than 25 per cent say, which is more than can be expected from the secondary factors, a further study or test is required. The recommended figure of 25 per cent is arbitrary and open to discussion. If the deficiency is less than 25 per cent a study of the influence of secondary factors is necessary.

The secondary factors have already been listed and it is now relevant to comment on their quantitative effects:—

(a) **Shearing resistance of bentonite in gel form** — Elson is of the opinion that this will not exceed 5 per cent of the stabilizing force. However, since the excavating tool disturbs the gel and can reduce its shear strength to negligible proportions, this factor can be omitted.

(b) **Arching action of the soil in relation to the limited dimensions of a panel.** This aspect can be very important and the analysis of a three dimensional wedge or longitudinal arch, as suggested by Schneebeli (1964)[(6)] is recommended.

(c) **The resistance of the bentonite cake.** This is an unknown quantity and difficult to introduce into the calculation. It is suggested that its influence be ignored.

(d) **An increase in shearing resistance of the zone into which the bentonite penetrated.** This aspect is treated in detail by Elson. He estimates that, in practice, it accounts for 10 to 25 per cent of the stabilising force when excavating in sands.

(e) **Electro-osmotic forces.** These forces are rather indefinite, but they have been analysed recently by Wielicka[(8)] (1967) and proved to be insignificant.

In most cases the introduction of approximate values for the above factors allow a balance to be established between the active and passive forces. In general, if the water level is some 1.5-2.0 metres below the ground level and the strata is not so pervious as to preclude the possibility of maintaining the suspension level, the excavation can be stabilised with bentonite. Certain difficulties can, however, occur with new hydraulic fills. If properly used the method can control running sands and silts.

Although a record of collapses does not exist, it is significant that most collapses occur in upper strata, which does not tie in with classical theory. Perhaps the lower strata possesses a better shearing resistance than is usually accepted, as suggested earlier, or the active earth pressure tends to be limited at depth.

Settlement of Adjacent Ground

The excavation of the soil from the trench under bentonite reduces the initial existing earth pressure at rest to the bentonite pressure, and consequently some movement can be expected.

If the support of the bentonite suspension approximates to the active pressure, then the active pressure in the ground will virtually be mobilised. Settlement of the adjacent ground will then depend mainly on the state of compaction of the soils. For compact sand, settlement could be almost zero up to 1/1000 of the supported height whereas for loose sand this fraction could rise to, say, 1/200. In practice however, the support of bentonite is above the theoretical active pressure of the soil and the rational approach would be to evaluate by how much it exceeds the active pressure and then to consider the strain created by the stress mobilised in the soil. Using a small scale model in the laboratory, Elson measured the settlement (Δs) and related it to the co-efficient of safety. For factor of safety 1.2, his graph shows a settlement of about 1/1000 of the supported height (H) and, for 1.05 Δs is less than H/500. The settlement will diminish away from the wall and should not extend beyond the formation of the "active wedge".

Combining practical experience with present limited theoretical knowledge, special precautions should be considered where adjacent buildings or structures are particularly sensitive to settlement and the foundation soil is loose. In such cases it is advisable to reduce the length of panels to a minimum of, say, 3 metres. In this way precautions taken by old trench diggers are followed and the theoretical factor of safety against mobilising the active soil wedge is increased by invoking the support offered at the ends of the panel due to arching within the soil. In the case of very severe conditions of loose strata and very heavy superimposed loads on adjacent ground, consideration can be given to chemical stabilisation before the trench is opened.

In most cases where the soil is moderately compact and where a standard length of panel of about 5 metres is adopted, the expected settlement will be negligible.

It may be concluded that the digging of a narrow trench, using conventional methods, to build a wall to protect a future excavation, is an established and successful method often used in the immediate vicinity of nearby buildings. Excavation under bentonite to install the wall offers much better conditions since it supplies instant support for the excavation, prevents influx of soil into the excavation and does not require pumping associated with the lowering of a ground water table. There are numerous examples of bentonite walls installed successfully in the proximity of tall buildings, even in relatively soft grounds.

Concrete for load bearing diaphragm walls

General Requirements

The requirement for finished concrete in the diaphragm wall does not vary substantially from concrete in other reinforced concrete structures.

The concrete for diaphragm walls is poured through bentonite by means of a tremie pipe and gradually, under gravity forces, displaces the bentonite fluid from the excavation. No mechanical means of compaction are used and for a successful concreting operation, it is imperative to take into consideration placing conditions at the stage of the mix design. The requirements for a mix, which can successfully be placed under bentonite (or water), can be summarised as follows:

(i) The consistency should be flowable to allow for gradual and complete filling of the excavation under gravity forces. The S.G. of the concrete is 2.3 approximately while the S.G. of bentonite suspension varies from 1 to 1.3 depending on the degree of contamination.

The more flow the concrete has the easier the tremie operation. There are, however, limits — too liquid a concrete may not be cohesive enough, or may show loss of strength.

(ii) The mix must be cohesive and must not segregate or bleed.

(iii) Setting time of the mix must be long enough to permit the operation of concreting to be completed without adverse effects on quantities already delivered.

(b) Mix design

The mix designer has to ensure that the above requirements are satisfied as far as possible, bearing in mind the final requirement of adequate strength, durability and impermeability.

Badly designed mixes which are too stiff or not cohesive can cause serious difficulties such as blocking the pipe, insufficient filling of ends and corners of the panel, segregation or mixing with bentonite.

The following are practical recommendations:—

1. Grading of the aggregate

In order to make a "flowable" consistency, the water has to be "trapped" within the aggregate. It has been found from experience that particles, which effectively oppose movement of water within the mix are particles below British Sieve Size No. 25 and these particles have to be in sufficient quantity. Normally, 20 to 30 per cent of this size makes good concrete.

In order to reduce the tendency for segregation, it is advisable to reduce the maximum size of aggregate to 19mm. The shape of the grading curve should show evenly graded aggregate. Gap graded concrete is prone to segregation, but a certain flattening of the curve between sieve No. 14 and 5mm can be advantageous. Gradings which have been successfully employed are illustrated in Figure 7.

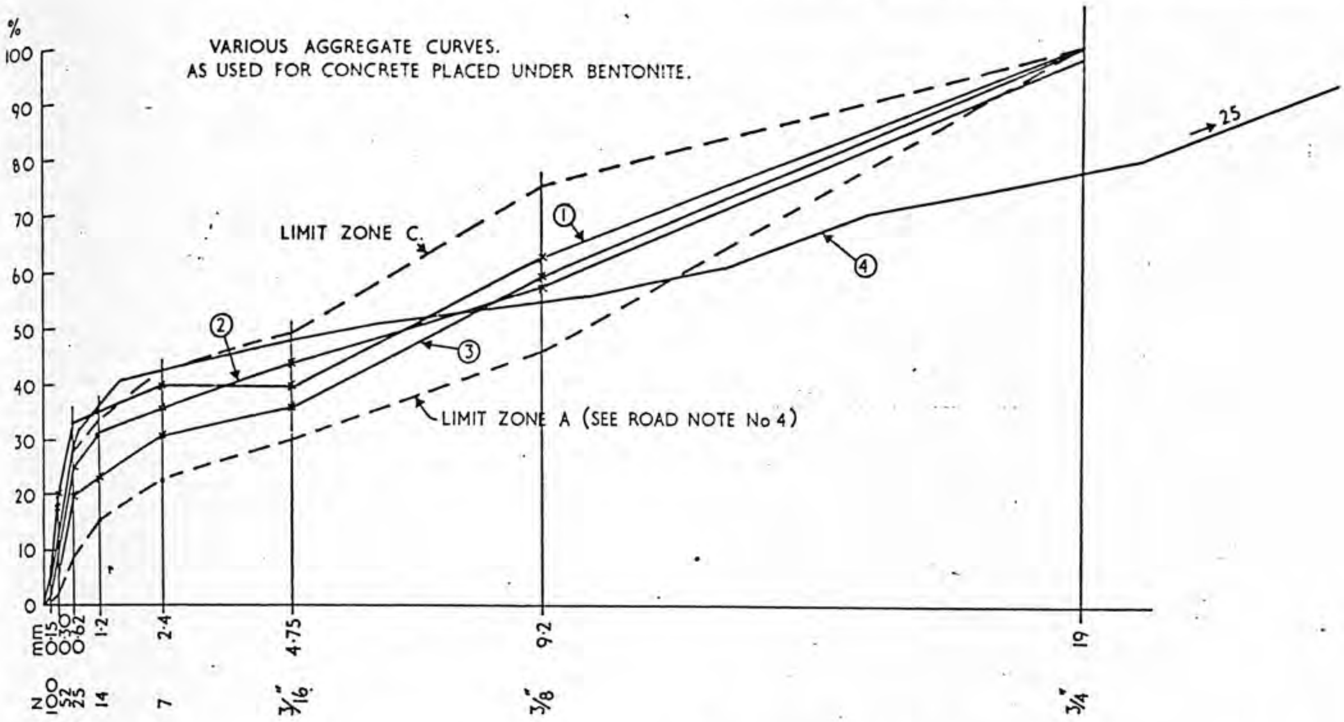


FIG 7.

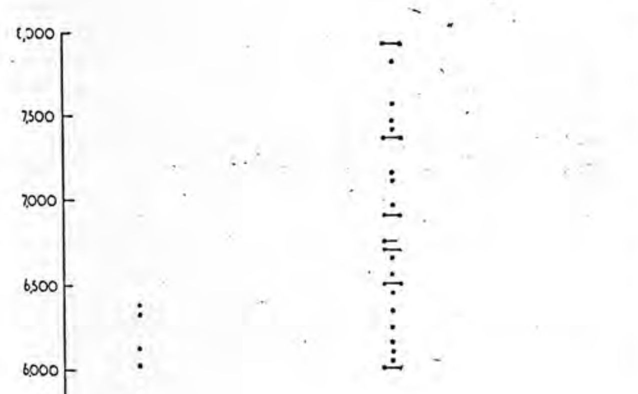


FIG 8.

2. Cement quantity

Because of the high quantity of fines and water, the cement content is high to ensure the required strength. In addition the cement particles are required to combine with the fines of the aggregate and water to produce the desired cohesion and flowability. In practice, the recommended minimum limit for a (21 N/mm²) 3000 p.s.i. concrete is 392 Kg/m³ per cubic yd. of concrete (however leaner mixes are also used).

The chosen cement quantity should also contain a factor of safety to cover the scatter of cube test results for flowable concrete and for local partial segregation, which can occur in spite of the best supervision.

3. Water quantity — plasticisers

The water quantity should be adequate to produce a consistency of 150-200 mm slump. To reduce the water requirements it is advisable to use plasticisers of a reputable type to make the mix more cohesive and permit a reduction of water content by 10 to 20 per cent. Reduction in water content not only increases the cube strength but also has a pronounced anti-bleed action. Also, it increases, albeit slightly, the specific gravity of the concrete, which helps to displace bentonite during placing and increases the resistance of concrete to erosion. On average, the water/cement ratio can be maintained at slightly above 0.5.

4. Retarders

The time of setting has to be checked against timing of the operation. Concrete setting too quickly (especially at high temperatures) can be very difficult to tremie. In such cases, retarders are advisable. It should be noted that a slight excess of water is less harmful than an insufficient quantity, which would produce stiff, non-flowable concrete. Similarly, an excess of fines is less harmful than a stony mix. However, none of the above recommendations should be exaggerated.

Examples of composition and cube test results are included in Figure 8 and it should be noted that excellent results have been obtained with very high slump concrete.

(c) Tremie Concreting.

The operation of concreting is simple but requires great care. It is important to check the concrete by slump tests and general appearance. This latter aspect should not be underemphasised since the human eye can be easily trained to detect bleeding, segregation and non-acceptable consistency.

Prior to concreting, the bottom of the trench should be cleaned of debris and bentonite suspension, which is contaminated beyond acceptable limits. The next step is the placing of stop ends true to position and verticality, followed by the placing of the reinforcing cage and finally by the tremie pipe.

Tremie pipes used for concreting are 10in. diameter steel pipes, and the required length is obtained by joining separate lengths of pipe. There is no standard length of pipe between joints, but to allow for simple adjustment of the tremie length during concreting, 2 metre lengths are recommended. The joints of the tremie must be watertight, easily disconnected and without projecting flanges, since these could foul the reinforcing cage. Experience shows that threaded pipe joints, carefully handled, are most practical.

The tremie pipe is placed in the centre of the panel, although two tremie pipes are sometimes placed in long panels. The bottom of the tremie rests firmly on the ground, and the funnel hopper is placed at the upper end. After assembly of the tremie a "plug" is placed in the tremie floating on the bentonite. The purpose of this plug is to separate the initial batch of concrete from the bentonite, which is in the pipe. As concrete is poured into the tremie, the plug travels down under the weight of concrete, until it reaches the bottom.

The tremie pipe is then lifted slowly from the bottom allowing the concrete to push the plug out. From this moment the end of the tremie must always be submerged in concrete and it is therefore important to check the level of concrete near the tremie and at the end of the panel regularly, after every delivery. A sounding weight of S.G. = 2, is recommended for this work.

During concreting, the top surface of the concrete will slope from a high point at the tremie to a low point at the ends of the panel. This slope indicates that the concrete is travelling upwards from the bottom of the tremie pipe in greater quantity near the pipe than at the ends. This must be associated with a horizontal movement on the concrete surface from the centre to the ends of the panels.

The more fluid the concrete and greater the value of submerged length in relation to the length of the panel, the less movement can be expected. For very short panels and a long submerged length, the filling of the trench will consist of a vertical movement without a horizontal component. This horizontal movement can influence the concreting operation in two ways:—

(i) the top concrete is gradually exchanged for fresh concrete and the time at which the initial set occurs is not as critical as in very short panels or piles; and

(ii) the impurities in the bentonite are gradually moved forward to the ends of the panel and this fact has to be considered when cut off level is reached.

Bond Stress

Prior to concreting, the steel is immersed in the bentonite suspension. The concrete rises, gradually displacing the suspension. The suspension cannot build a filter cake on the surface of the steel, as it is not pervious. It can adhere to the bars only by its cohesion which, for the concentration employed, would be approximately 2000 dynes/cm^2 or 215 N/m^2. Rising concrete, due to its granular composition and its inherent friction, exercises a sort of sweeping action which removes the bentonite suspension from the bar. On removal of stop end pipes placed as shuttering at the end of the panels, it can be observed that this sweeping action is efficient. If left longer than a few hours, it is almost impossible to remove the pipes because concrete adheres to the steel, which was submerged in bentonite. Experiments carried out and described in CIRIA Report 167, have shown that the bond stress, at no slip condition, of round mild steel bars was not materially affected, but for deformed bars, where employed, the bond stress is reduced. The probable explanation of the above is that deformed bars prevent efficient sweeping action and bentonite suspension is trapped under the deformations.

As a result of this report no adjustments are made for safe bond stress of plain bars but a reduction of 10-20 per cent is applicable to deformed bars.

3 ANCHOR DESIGN

Site Investigation

Of paramount importance is the provision of site investigation data which will facilitate anchor design and choice of anchor construction technique. The basic information required is illustrated in Table 2.

TABLE 2

Item	Data Required
Borehole	General Soil Profile Ground Water Level
Undisturbed Sample	Shear Strengths (ϕ, Cu, c' & ϕ') Density Consolidation and Compressibility Indices
Disturbed Sample	Mechanical Analysis Chemical Analysis
In-situ Test	Standard Penetration or Dutch Cone Readings Vane Test Results
Construction	Proximity of operations such as piling, blasting or freezing

In sands the friction angle (ϕ) combined with the effective overburden pressure enables the capacity of the anchor to be calculated since the resistance to pull-out of the anchor depends on the ground restraint which can be mobilised adjacent to the grout injection zone.

Grading samples are invaluable since they enable the permeability and therefore the groutability of the soil to be assessed, and in addition when the samples are used in conjunction with standard penetration tests to estimate relative density, then ϕ values can be determined if these are not already available.

Chemical analyses of the soil and groundwater are important since sulphate content and pH for example can dictate the type of cement grout and degree of corrosion protection.

Anchor Location

Since the waling level and spacing of the anchors is determined in the wall design, the location of the top anchor is fixed and only the inclination and length of the anchor remain to be calculated.

Anchor inclination is kept small and ideally should be less than 20° to the horizontal. In many cases however this is not possible due to the proximity of adjacent foundations, and values of 20° - 45° are common.

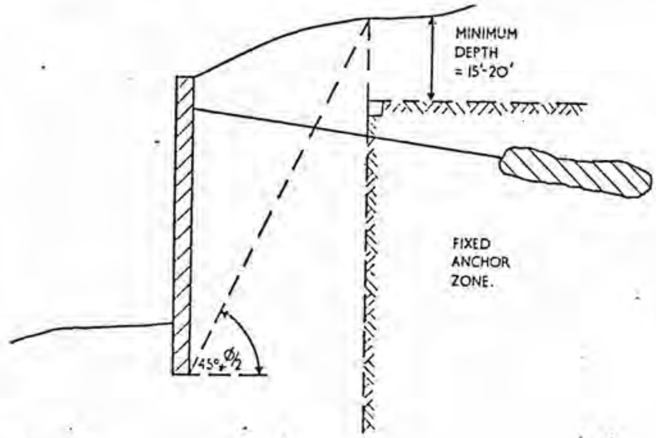


Figure 9

With regard to overall length, the fixed anchor must be embedded (a) deep enough to avoid the localised passive failure of the soil associated with the failure condition for shallow deadmen; and

(b) far enough away from the wall to ensure against a slip failure beneath the toe of the wall and beyond the fixed anchor zone, at a lower factor of safety than the design specification allows.

A minimum depth of 5-6 metres is normally considered sufficient to guarantee a deep seated failure condition at pull-out, and, for initial guide purposes only, the "free" anchorage length may be estimated with the help of the construction diagram shown in Figure 9.

Overall Stability

As a second step the stability of the whole system must be checked to ascertain whether the chosen anchor lengths are sufficient or not. Where the waling loads on the wall have been designed according to the principles outlined earlier it is assumed that the anchor prestress introduced will prevent slip planes occuring between the wall and the fixed anchor zone. In other words it is assumed that the prestressing of the anchors introduces a new state of stress in the retained soil mass where the normal stresses and consequently the shear strengths become large enough to prevent the mobilisation of sliding surfaces ahead of the fixed anchor zone. The sliding surfaces which are still possible will therefore pass beyond the fixed anchor zone. As an additional safety precaution the midpoints and not the ends of the fixed anchors are usually arranged along the sliding surface with the required safety factor, according to practice in Europe. It is noteworthy, however, that in the absence of detailed information on fixed anchor/soil interaction, the authors consider that the fixed anchor zone should be completely beyond the estimated slip plane.

The shape of the sliding surface which will occur for systems with only one row of anchors is known through the work of Kranz (1953)(13), Jelinek and Ostermeyer (1966)(14) and Ranke and Ostermeyer (1968)(15) and the procedure recommended for consideration is a modified Kranz method suggested by Locher in 1969 (Figure 10).

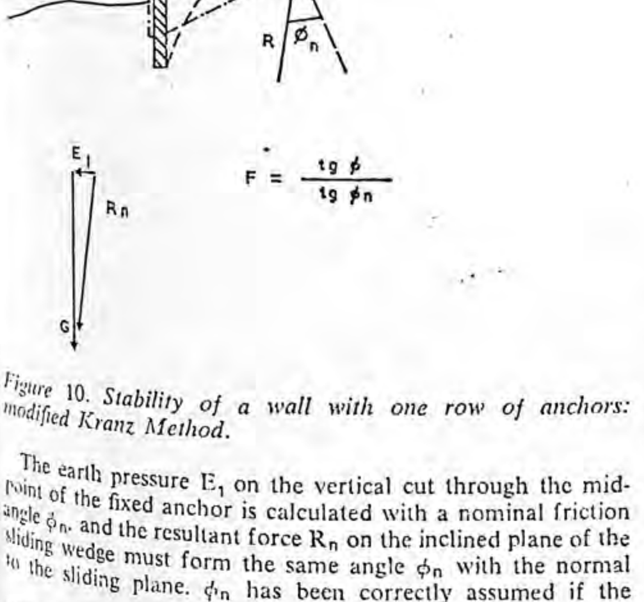


Figure 10. Stability of a wall with one row of anchors: modified Kranz Method.

The earth pressure E_1 on the vertical cut through the midpoint of the fixed anchor is calculated with a nominal friction angle ϕ_n, and the resultant force R_n on the inclined plane of the sliding wedge must form the same angle ϕ_n with the normal to the sliding plane. ϕ_n has been correctly assumed if the weight G and the forces E_1 and R_n are in equilibrium. If this is not the case then ϕ_n has to be altered and when equilibrium is achieved the factor of safety is defined as $F = \dfrac{\tan \phi}{\tan \phi_n}$ where ϕ is the actual angle of internal friction. This definition corresponds with the concept of partial safety factors as proposed by the late Professor Brinch Hansen. It is considered that the main attraction of this method is its simplicity, and although the forces acting are assumed to be concurrent it is considered that the value of F is a safe estimate since the stabilising passive resistance available from the embedded depth of soil within the excavation, is ignored in the calculation.

For systems with several rows of anchors the shape of the sliding surface is not known from experiments and the stability is evaluated using the circle or logarithmic spiral method. A logarithmic spiral has the property that the radius from the spiral centre to any point on the curve, forms a constant angle ϕ with the normal line to the curve. If a nominal friction angle of the soil ϕ_n is employed where $\tan \phi_n = \dfrac{\tan \phi,}{F}$ then the line of action of the resulting forces on each part of the sliding surface will pass through the spiral centre. None of the forces along the sliding line will therefore create a moment around this point, and they can therefore be neglected, when considering the equilibrium of moments around the point.

The safety factor F is correct, when the moments of the remaining weights and forces on the sliding body total zero. Figure 11 shows the principle, and again, by ignoring the passive resistance of the soil beneath the excavation when the moments produced by G_t and G_s balance, a conservative value is obtained of F equals $\tan \phi/\tan \phi_n$.

In both the stability analyses described the basic assumption is made that anchor prestress increases the shear strength of the sand sufficiently to displace the potential failure plane beyond the fixed anchor. Care should therefore be taken not

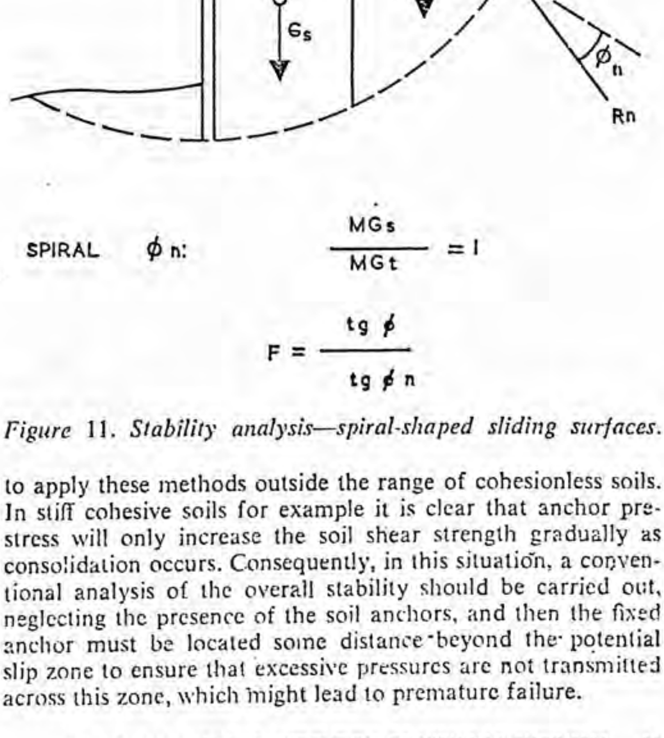


Figure 11. Stability analysis—spiral-shaped sliding surfaces.

to apply these methods outside the range of cohesionless soils. In stiff cohesive soils for example it is clear that anchor prestress will only increase the soil shear strength gradually as consolidation occurs. Consequently, in this situation, a conventional analysis of the overall stability should be carried out, neglecting the presence of the soil anchors, and then the fixed anchor must be located some distance beyond the potential slip zone to ensure that excessive pressures are not transmitted across this zone, which might lead to premature failure.

Load carrying capacity

In fine to medium sized sand where the permeability (K_w) ranges from 10^{-2} to 10^{-4} cm/sec, the fixed anchor formed consists of a smooth grout cylinder since the sand does not allow permeation of the dilute cement grout (Figure 12).

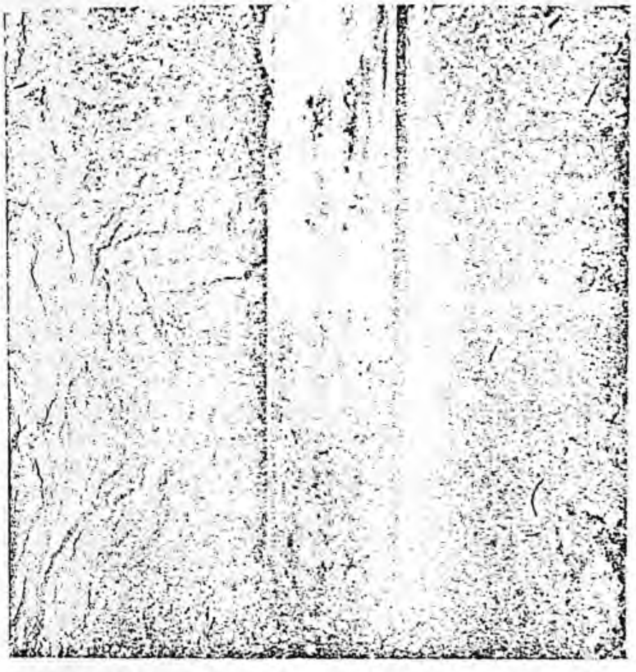


Figure 12

For this type of anchor equation 3 is commonly used by specialist contractors, to estimate the ultimate load carrying capacity.

$$T_f = L.\ n' \tan \phi \qquad (3)$$

(metres) 13-16.5 t/m

where L is the fixed anchor length (metres $n' = 13\text{-}16.5$ T/m and ϕ is the angle of internal friction. In equation 3, n' automatically takes account of the depth of overburden above the fixed anchor $h = 6.1\text{-}9.2$m fixed anchor diameter. $D = 180\text{-}200$mm and the range of fixed anchor lengths $L = 0.9\text{-}3.7$m over which the rule has been tested.

In general however, practising engineers require an empirical rule which relates anchor pull-out capacity with anchor dimensions and soil parameters. Equation 4 for vertical anchors is recommended for consideration.

$$T_f = A\bar{\gamma}\frac{(h+L)}{2}\pi DL \tan\phi + B\ \bar{\delta}\ h\frac{\pi}{4}(D^2-d^2) \qquad (4)$$

$$+ B\ \bar{\gamma}\ h\frac{\pi}{4}(D^2-d^2)$$

(side resistance) + (end resistance)

where A = ratio of the contact pressure at the fixed anchor/soil interface to the effective pressure of the overburden.

B = bearing capacity factor.

$\bar{\delta}$ = unit weight of soil overburden (submerged unit weight beneath the water table).

h = depth of overburden to top of fixed anchor.

L = length of fixed anchor.

D = effective diameter of fixed anchor.

d = effective diameter of grout shaft or column.

A normally lies within the range 1 - 2 but the actual value depends to a great extent on the anchor installation procedure i.e. drilling method and grout injection pressure. The bearing capacity factor B depends on the angle of shearing resistance of the soil and the ratio h/D. It is noteworthy that in compact fine to medium sand ($\phi = 35°$) values of 1.4 and 31 for A. and B. respectively have been measured where rotary percussive drilling techniques have been employed with grout injection pressures of 350 KN/m² to give a ratio $L/D = 27$.

Where anchors are formed in fine cohesionless soil using cement grout, safe working loads are usually limited to 40 tons.

Factors of Safety

Having established the ultimate load holding capacity of the anchor using either equation 3 or 4 it is necessary to apply a factor of safety to guarantee the performance of the individual anchor. In multi-anchor systems where progressive failure must be prevented, the minimum factor of safety (S_f) normally employed is 1.6. Since the local soil properties are not normally known with the degree of accuracy associated with the steel components of the cable and top anchorage, a value of 2 is common for fixed anchor design in cohesionless soil both for temporary and permanent works.

In order to check and possibly optimise the fixed anchor design at the beginning of the contract a minimum of three test anchors pulled to failure is recommended where the fixed anchor length is varied and the cable is designed in each case to ensure that failure occurs at the fixed anchor/soil interface.

With regard to overall stability a factor of safety $F = 1.5$ is customary, but as in all designs the choice is based on how accurately the relevant characteristics are known, whether the system is temporary or permanent and the consequences if failure occurs.

4 ANCHOR CONSTRUCTION

Construction Stage

The method which is employed for anchorages in sand entails a number of working operations as follows:—

(1.) A casing, 50-150 mm nominal diameter, is driven through the wall and the retained soil mass to the desired depth, using rotary, rotary-percussive or vibrodriving techniques.

Anchor hole formation is aided by various flushing techniques. In sands and gravels, for example, water flushing widens and cleans the hole and ensures a better bond at the grout-soil interface.

(2.) The cable, which consists of high tensile strands, wires or bar is homed, the length of cable above the fixed anchor being decoupled from the ground by some form of sheathing.

(3). Grout, consisting of neat cement and water, is injected into the hole under pressure as the casing is withdrawn over the fixed anchor length. The hole is then topped up with grout and allowed to set following complete withdrawal of the casing. Grout W/c ratios ranging from 0.4 to 0.65 are recommended and the injection pressures may vary from 30 to 1000 KN/m².

Care should be taken not to exceed the theoretical overburden pressure since this could cause fissuring in the ground and possibly lead to ground heave at the surface as well as possible damage to existing anchors. During the grouting stage therefore, a careful note of injection pressure is required together with grout consumption.

Where the ground is variable high alumina cement is often employed since it enables the anchor to be tensioned within 24 hours. Consequently, if the ground conditions have deteriorated locally without being observed, the tensioning stage will indicate a reduced capacity, and remedial measures can be taken immediately.

(4.) Within six hours of grouting, the grout column filling the hole is flushed back using air and water, to within 1.5 metres, say, of the top of the fixed anchor.

(5.) When the fixed anchor has hardened (minimum crushing strength of 28 N/mm² is normally specified), the cable is post-tensioned to the desired load.

Thus the anchorage is based on grout injection and consists basically of a cable which is bonded into a grouted zone of alluvium (the fixed anchorage). The rest of the cable is encased in a protective sheath to prevent the cable from coming into contact with the surrounding ground and also to provide a safeguard against corrosion.

Post Tensioning

The post-tensioning operation pre-tests the anchor, thus ensuring its safety. To establish a measured factor of safety against withdrawal of the anchor it is necessary to apply a temporary test loading on site. However, the allowable test load (T_t) is limited by the elastic limit of the steel cable, and the maximum recommended test load is equal to 80 per cent of the breaking load (T_b). Thus, for a cable working at 62.5 per cent T_b the maximum measured safety factor which can be provided is $S_m = T_t/T_w = 1.28$, where T_w is the working load.

Every anchor should be tested to 80 per cent T_b and representative anchors (1 in 10 say) should be constructed with extra cable where T_w = 50 per cent T_b to give a measured $S_m = 1.6$.

In fine to medium sized sand fixed anchor displacement during initial tensioning is fairly common and this should not be associated with failure, since some relative displacement at the grout/sand interface may be necessary to mobilise load, (Figure 13). In these circumstances the load carrying capacity of the anchor is established from a second tensioning cycle. The fixed anchor movement should then be simply due to small elastic deformations provided that the working load is not greater than 80 per cent of the maximum test load.

It should be emphasised that these test loads are only applied for short periods but experience has shown that the decrease in anchor carrying capacity under long term loading is relatively modest for most cohesionless soils i.e. loss of prestress due to fixed anchor displacement is not greater than 5 per cent.

Corrosion Protection

Where ground conditions are not hostile and the working life is less than two years, a greased tape decoupling sheath over the elastic length is normally specified with the normal grouting procedure which gives a cement grout cover over the fixed anchor. On completion of the stressing stage the top or movable anchorage and the protruding cable may be painted with a removable plastic coating such as "stripceal" (Figure 14).

For permanent anchors individual strands making up the cable can be greased and sheathed with extruded polypropylene

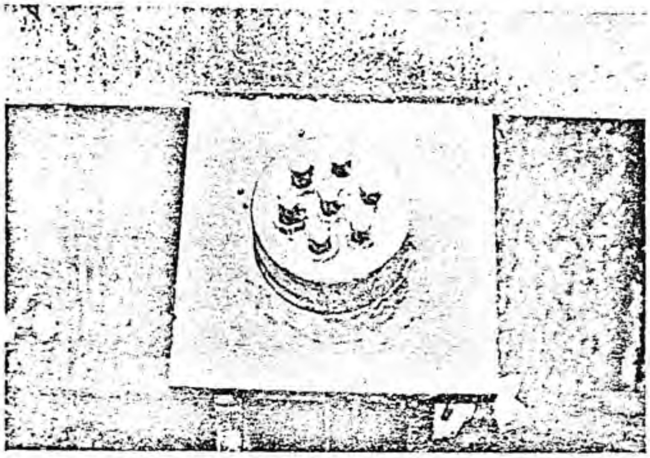


Figure 14.

under factory controlled conditions. The fixed anchor length of the cable is then stripped and degreased before casting into a corrugated plastic tube using a high strength synthetic resin.

This fully protected restressable cable is homed and grouted in the normal way and after stressing the top anchorage can be enclosed by a steel or rigid plastic cover filled with grease or bitumen.

Full scale sustained load tests have been carried out on resin bonded strand anchors of this type (Figure 15) and the results obtained from a test anchor prestressed to 220 tons (70 per cent U.T.S.) are illustrated in Figure 16. Creep of the lower end of the fixed anchor amounted to 0.09 mm which occurred within 25 days whilst the movement at the jack amounted to 0.9 mm after 30 days. This latter creep represents a loss of prestress in the laboratory system of less than 5 per cent due to cable relaxation and partial debonding in the fixed anchors. In practice, where the "free" or elastic length is commonly 30 ft., the loss of prestress would be 1 per cent.

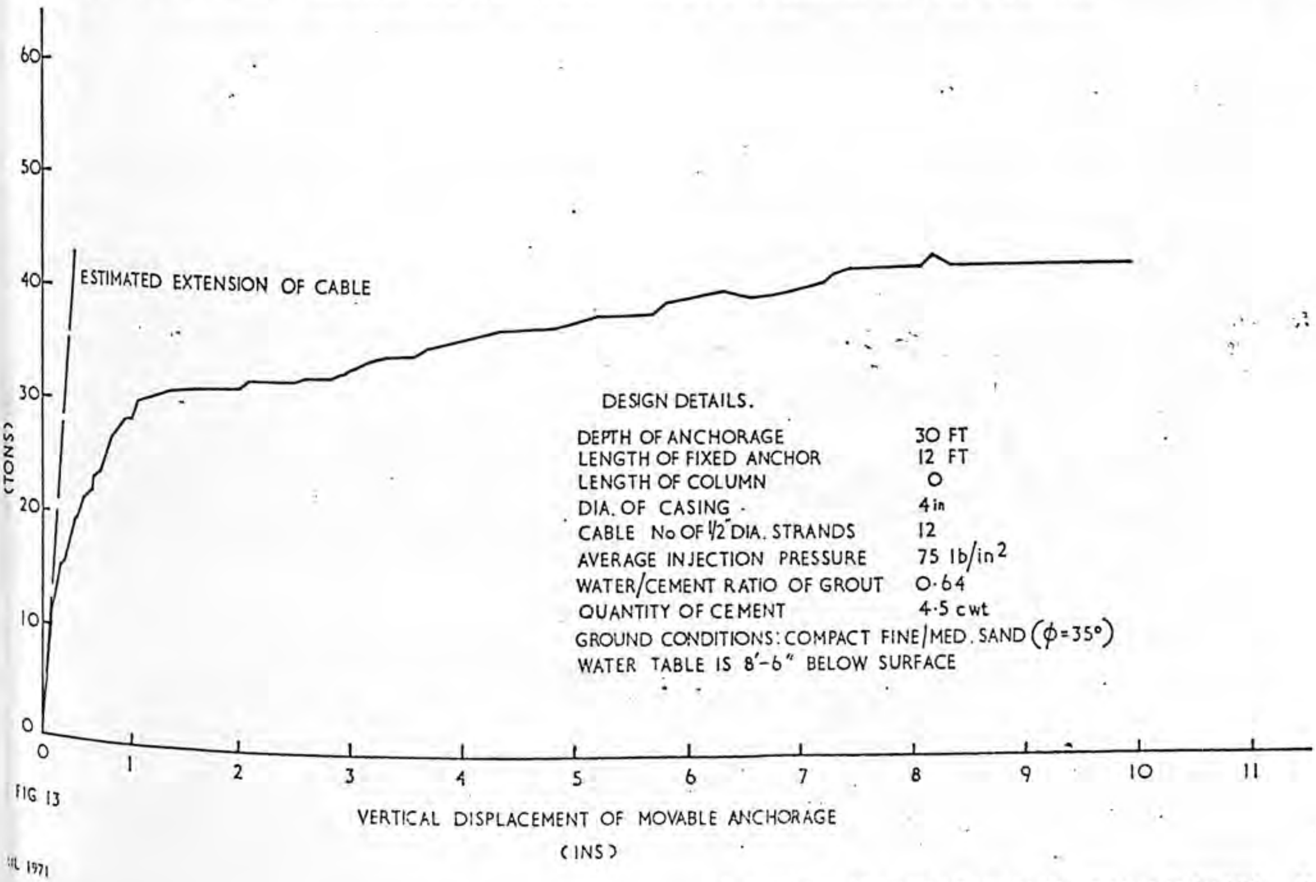


FIG 13

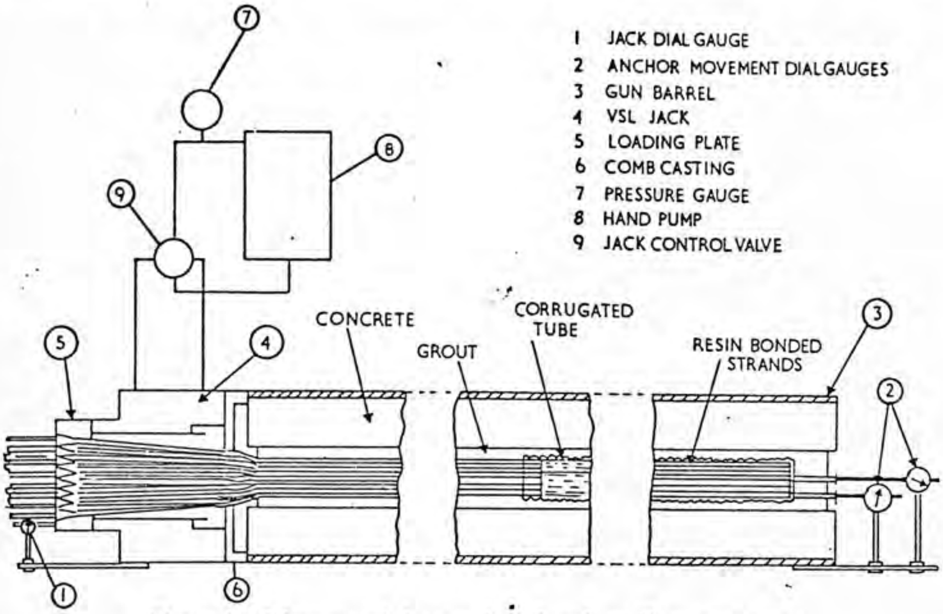


Figure 15. Diagram of the gun barrel and ancillary equipment.

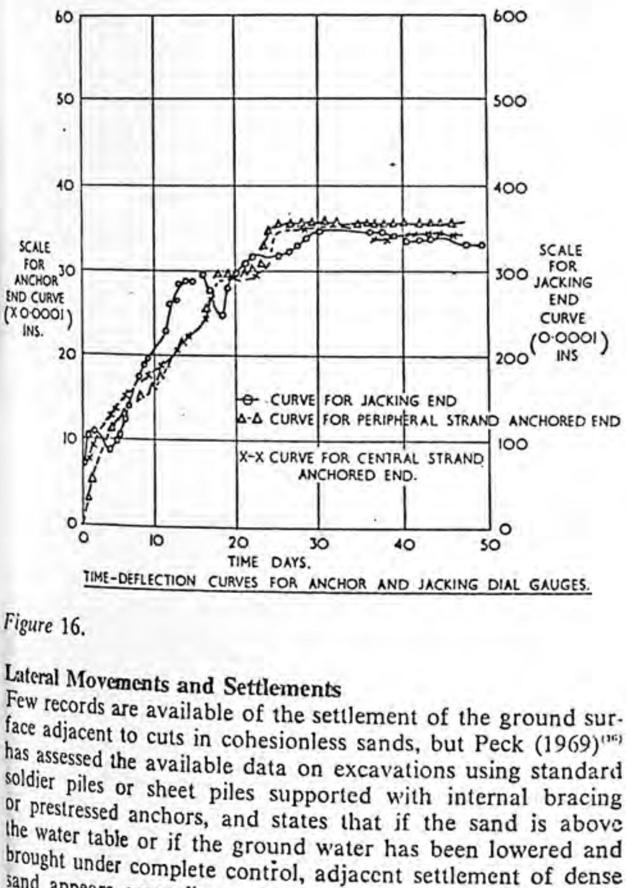


Figure 16.

Lateral Movements and Settlements

Few records are available of the settlement of the ground surface adjacent to cuts in cohesionless sands, but Peck (1969)[16] has assessed the available data on excavations using standard soldier piles or sheet piles supported with internal bracing or prestressed anchors, and states that if the sand is above the water table or if the ground water has been lowered and brought under complete control, adjacent settlement of dense sand appears generally to be inconsequential. However the settlements associated with loose sands and gravels may be of the order H/200 at the edge of the excavation, diminishing to negligible values within a distance of 2 to 3 H from the excavation.

With regard to the influence of anchor prestress on lateral displacement, a tie back system for bracing the soldier-pile walls of an excavation 11.3 metres deep in dense sands overlain by a layer of loose sand is shown in Figure 17a (Rizzo et al 1968). The tie backs consisted of driven H-piles prestressed to approximately 50 per cent of the load calculated for the condition of active earth pressure. The wales through which the tiebacks transferred their forces to the soldier piles were located at the third points of the height of the wall, and the soldier piles and the upper set of tiebacks were installed at the bottom of an initial excavation 3 metres deep.

The lateral displacement of the wall at the end of the excavation period is shown in Figure 17b. Subsequent movements were negligible. The soldier piles moved inward by amounts decreasing with depth from a little over 50 mm at the ground surface to about zero at the bottom of the cut, the horizontal displacement of the top anchor being about 25 mm.

At another section in the same cut excavated to 6.4 metres, a similar bracing geometry was used but the anchors were prestressed to 110 per cent of the load calculated on the basis of earth pressure at rest. With this prestress the horizontal displacements of the top and bottom anchors were only about 0.5 mm and 0.25 mm respectively.

These results were achieved with average workmanship, but it should be noted that inferior workmanship can easily lead to larger settlements than those inevitably associated with a given type of construcion and a given soil.

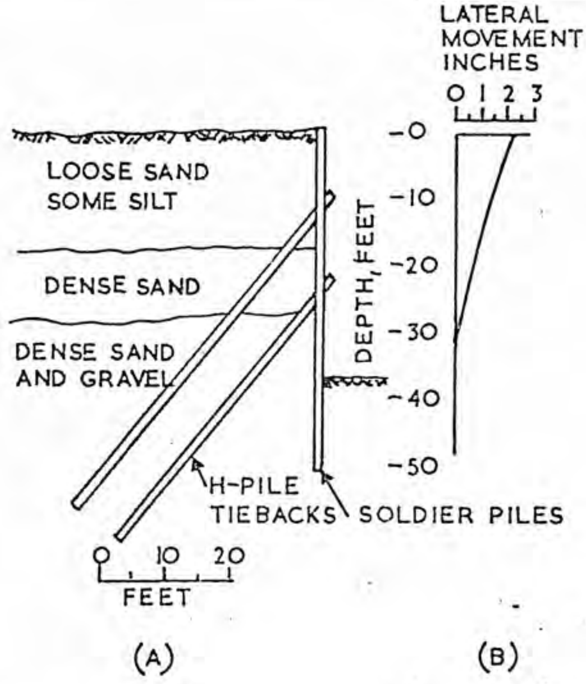


Figure 17.

Published data on ground movements associated with anchored diaphram walls in sand is limited, but two interesting case histories have recently been described by Maestre[17] and Vander Linden[18] at the Speciality Session No 14 of the Seventh International Conference on Soil Mechanics and Foundation Engineering in Mexico. In both cases lateral measured movements of the walls were only a few millimetres.

As Peck has suggested the absence of settlement records at least suggests the absence of serious settlements.

Conclusions

(1.) For an anchored or strutted continuous wall, a design procedure is now available which takes account of soil properties, wall flexibility, wall construction procedure and main excavation stages. Observations of the performance of models and field structures to date indicate that the new method gives realistic anchor loads.

(2.) The design method has been programmed for a computer and if different soil conditions, from those assumed, are encountered on site, the design can be quickly amended panel by panel.

(3.) Where specialist companies within one group possess the expertise and experience to construct both the soil anchors and the diaphram wall, the programme can be employed to produce a quick optimum solution for the design and construction of the anchored wall.

(4.) Excavation under bentonite to install a diaphragm wall offers much better conditions compared with conventional trench excavation methods, since the bentonite supplies immediate support for the excavation and does not require pumping associated with the lowering of a ground water table.

(5.) Soil anchors, employed for the support of braced cuts, eliminate the need for interior struts, which in turn brings quite large economic and constructional advantages.

(6.) The use of a stiff continuous diaphragm wall supported by prestressed anchors can greatly reduce lateral displacements of the wall and therefore settlements of the retained soil mass during the main excavation.

(7.) More use should be made of field instrumentation to observe the performance of anchored walls where the site observations are related back to design assumptions.

References

(1) Rowe, P. W. (1956) *Sheet Pile Walls at Failure,* Proc. I.C.E. Part 1, Vol. 5, p. 276.

(2) Rowe, P. W. and Briggs, A. (1961) *Measurements on Model Strutted Sheet Pile Excavations.* 5th Int. Conf. on Soil Mech. and Found. Eng. Vol. II, pp. 473-78.

(3) Golder, H. Q. (1970)
Gould, J. P.
Lambe, T. W.
Tschebatarioff, G. P.
Wilson, S. D.
Predicted Performance of Braced Excavation. Proc. Amer. Soc. of Civ. Engs. (S.M. & F.E. Div.) Vol. 96, No. SM3 (May)

(4) Morgenstern, N. R. and Amir-Tahmasseb, I. (1965) *The stability of slurry trenches incohesionless soils.* Geotechnique 15, No. 4, 387-395.

(5) Nash, J. K. T. L. and Jones, G. K. (1963) *The support of trenches using fluid mud.* Proc. Symp. Grouts and Drilling Muds in Engineering Practice, pp. 177-180. London: Butterworth.

(6) Schneebell, G. 1964. *La stabilité des tranchees profondes forées en présence de boue,* Houille blance, 19:7:815-820.

(7) Veder, C. (1963) *Excavations of trenches in the presence of bentonite suspensions for the construction of impermeable and load bearing diaphragms.* Proc. Symp. Grouts and Drilling Muds in Engineering Practice, p. 181. London: Butterworth.

(8) Wielicka, H. and Malasiewicz, A. (1967) *Wplyw zjawisk fizyko chemicznych na Wlasnosci zawievin ilowych stosowanych do glevienia waskprzestrzennyeh wykopow.* Archiwum Hydrotechniki 1967.

(9) Elson, W. K. 1968, *Experimental investigation of the stability of slurry trenches.* Geotechnique, XVIII, No. 1, pp. 37-49.

(10) Florentin, J. (1969) *Les Parois Moulees Dans Le Sol.* Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering Speciality Session 14 Mexico 1969. Vol. 3, pp. 507-12.

(11) Courteille, G. (1969) *Actron stabilisatrice des suspensions thixotropique sur le parois de fouilles.* Communication due 14me Session Speciale Mexico 1969. Vol. 3. pp. 507-12.

(12) Road Research Laboratory (1950) *Design of Concrete Mixes Road Note No. 4.* H.M.S.O., London.

(13) Kranz, E. (1953) *Ueber die Verankerung von Spundwänden.*—W. Ernst & Sohn, Berlin.

(14) Jelinek, R. and Ostermeyer, H. (1966) *Verankerung von Baugrubenumschliessungen.*—Vortrage der Baugrundtagung 1966 in Munchen, Deutsche Gesellschaft für Erd— und Grandbau e.V. Essen.

(15) Ranke, A. and Ostermeyer, H. (1968) *Contribution to the investigation of stability of multi-tied walls.* Bautechnik, 10, 341-50.

(16) Peck, R. B. (1969) *Deep Excavations and Tunnelling in Soft Ground.* Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering — Mexico.

(17) Maestre, M. (1969) *Incident following overloading of an anchored diaphragm wall.* Speciality Session No. 14. Proc. 7th Int. Conf. on Soil Mechanics and Foundation Engineering. — Mexico.

(18) Vander-Linden, J. (1969) *Controle des mouvements horizontaux d'une paroi moullèe dans le sol avec ancrages précontraints.* Speciality Session No. 14 Proc. 7th International Conference on Soil Mechanics and Foundation Engineering — Mexico.

PAPER 15

A case history study of multi-tied diaphragm walls

G. S. LITTLEJOHN, BSc, PhD, MICE, FGS, *Lecturer in Geotechnics, Department of Engineering, University of Aberdeen*

I. M. MACFARLANE, BSc, FICE, FGS, *Director, Trocoll Cementation Engineering Ltd*

The Paper is concerned with full-scale performance of multi-tied diaphragm walls, designed according to an empirical method of analysis first introduced in 1971. At the present time this design method appears unique in that it takes account of the continuous wall construction and excavation stages and the procedure is amenable to varying soil strata. Problems encountered and lessons learned are detailed, and where possible records of wall movement are analysed in relation to design assumptions. The Paper indicates that wall deflexions and bending moments, which occur as excavation proceeds, follow a pattern similar to those predicted by the new method. In addition the results suggest a triangular pressure distribution for a semi-rigid diaphragm wall, as distinct from the trapezoidal distribution commonly assumed. In all the case studies presented, no significant vertical or horizontal wall movements have been monitored during the construction period. The Authors submit that the empirical design method results in economical structures exhibiting satisfactory performance in the field.

INTRODUCTION

An empirical repetitive single-tied wall design method has recently been introduced[1] for the analysis of multi-tied diaphragm walls. This method may be used in the design office and the theoretical background, together with results from small-scale tests, are discussed in detail by James and Jack in Paper 6 of this Conference.

2. At the present time the method appears unique in that it takes account of the continuous wall construction, excavation and anchoring stages, and the procedure is applicable to varying soil strata.

3. Certain basic assumptions are made in the new method, e.g. that the soil pressure distribution is of triangular form and the wall yields progressively as excavation proceeds. In addition, a point of contraflexure in the wall is assumed to occur at a point where the factor of safety is unity against overturning. The purpose of the full-scale monitoring studies described is to check the validity of the basic assumptions with particular regard to bending moment profiles, and where possible to monitor overall movements in order to confirm satisfactory performance of this anchored diaphragm wall system in the field.

4. Case histories have been chosen which are representative with respect to depth of excavation, number of anchor levels and variation in ground conditions. Problems encountered and lessons learned are detailed, and where wall deflexions have been monitored these are analysed in relation to the design assumptions. The diaphragm wall at Victoria Street, London, which is described by Hodgson in Paper 7 of this Conference, represents an additional case study for this design method.

CASE STUDY NO. I: GUILDHALL PRECINCTS REDEVELOPMENT, LONDON

5. For the Guildhall precincts redevelopment, the main contractor was Trollope and Colls Ltd and specialist contractors were Cementation Piling and Foundations Ltd and Cementation Ground Engineering Ltd.

6. A plan of the site is shown in Fig. 1. The ground conditions comprised 6–8·8 m of gravel overlying London Clay. The effective excavation depth was 10·4 m and the soil was retained by a diaphragm wall 0·51 m thick and anchored at two levels into the gravels (see Figs 2 and 3).

7. To facilitate the study of full-scale results and comparison with design assumptions it was considered that information was required on contact pressures, wall displacements and anchor loads at various stages during the excavation.

8. The installation of in situ strain gauges and earth pressure cells in diaphragm walls formed under bentonite is expensive and difficult, and since contact pressures and wall displacements are interrelated it was decided that the most convenient and robust approach would be to record the displacement profile of the wall at each construction stage using a sensitive inclinometer. Designed to operate inside an aluminium extruded duct measuring 44·5 mm square internally, the inclinometer for this investigation had a gauge length of 152 mm, sensitivity of 10″ and a

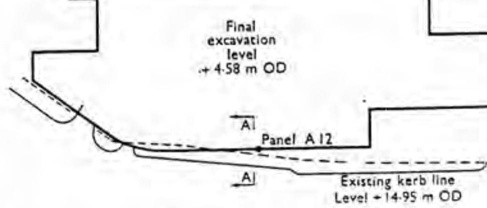


Fig. 1. Site plan—Guildhall

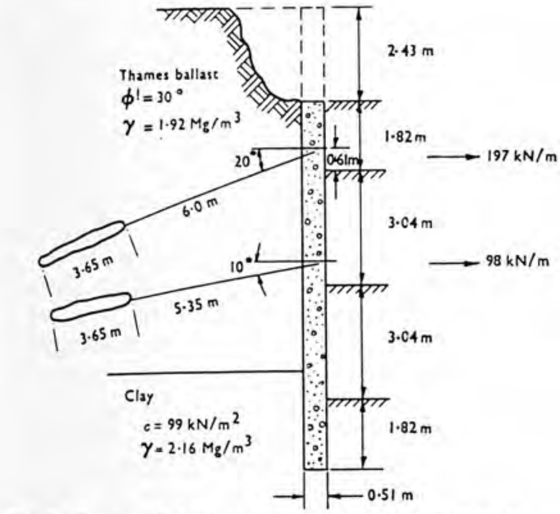


Fig. 2. Section AA through panel A12—Guildhall

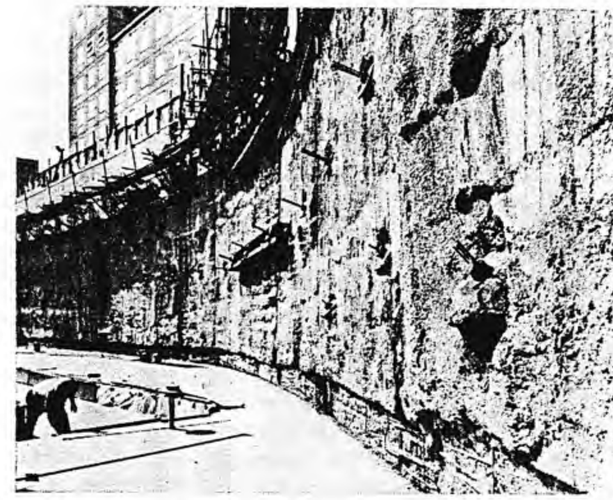


Fig. 3. General view of anchored diaphragm wall—Guildhall

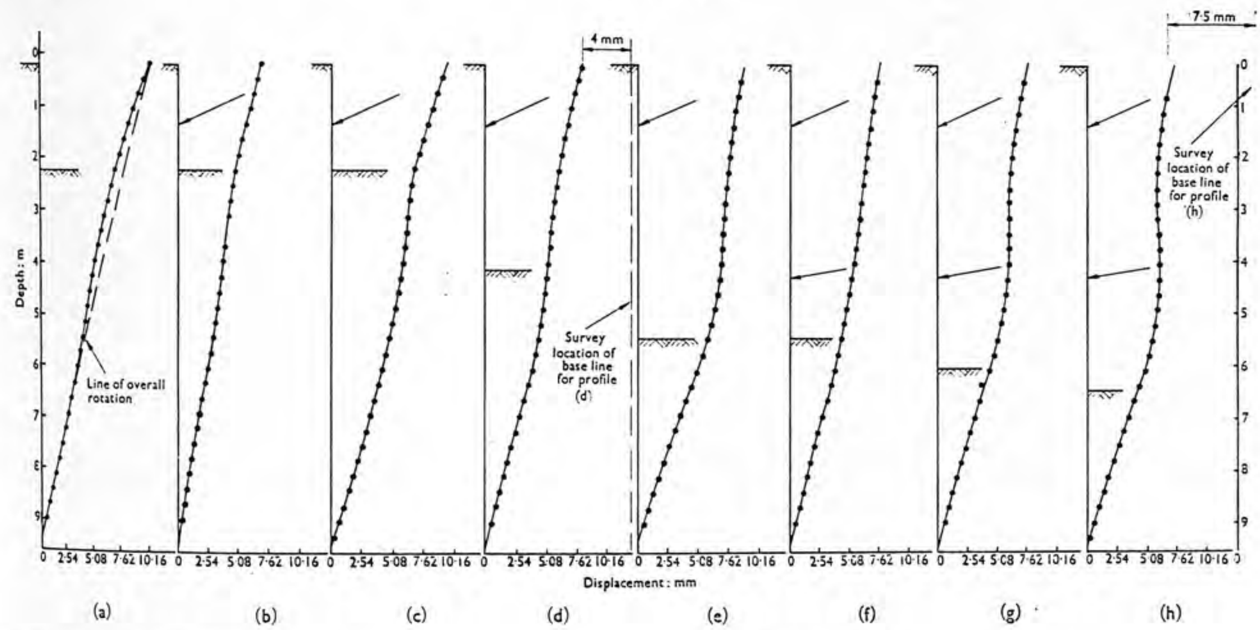


Fig. 4. (above) Displacement profiles of panel A12 at various construction stages

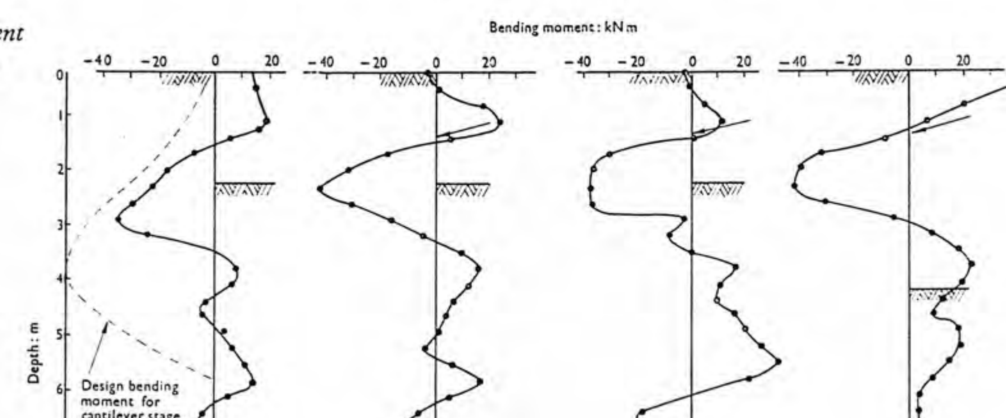


Fig. 5((a)–(h)). Bending moment profiles of panel A12 at various construction stages (moments based on 0·30 m strip) range of $\pm 3°$. Recording errors would tend to cancel over the 9·7 m length of the duct, but for the unlikely extreme case of summation of errors a total error of 1 mm was estimated.

9. To monitor anchor prestress, load cells consisting of steel annuli of 76 mm internal diameter, 9 mm wall thickness and 76 mm length were constructed, and located between the load bearing plate and the anchor stressing head. To measure axial strain four electrical resistance gauges (two axial and two circumferential) were fixed onto the outer surface, diametrically opposite each other to eliminate bending effects.

10. With regard to overall movements of the wall a permanent base line, measured accurately by invar tape, was established at a remote distance from the excavation and was transferred by triangulation to a temporary base line on site. A theodolite reading to 1″ was employed throughout. By further triangulation wall stations were located. A geodetic level was used to measure any wall settlements. Three general surveys were carried out during this investigation thus enabling the overall horizontal and vertical movements at two stages to be computed.

11. Panel A 12 (Fig. 1) was chosen to be monitored to ensure minimal effects from corners, and displacement profiles were measured along the neutral axis of the wall at points between two vertical rows of anchors. All displacement profiles (see Fig. 4) were plotted relative to the toe of the wall, no account having been taken of the overall displacement of the wall, except where this movement was measured during a general survey (see profiles (d) and (h) of Fig. 4). It should be emphasized that full explanations for the sequence of profiles obtained cannot be given authoritatively without accurate information on the absolute location of each profile, but valuable data on bending moments can be obtained by graphical differentiation of the displacement profile gradients (see Fig. 5).

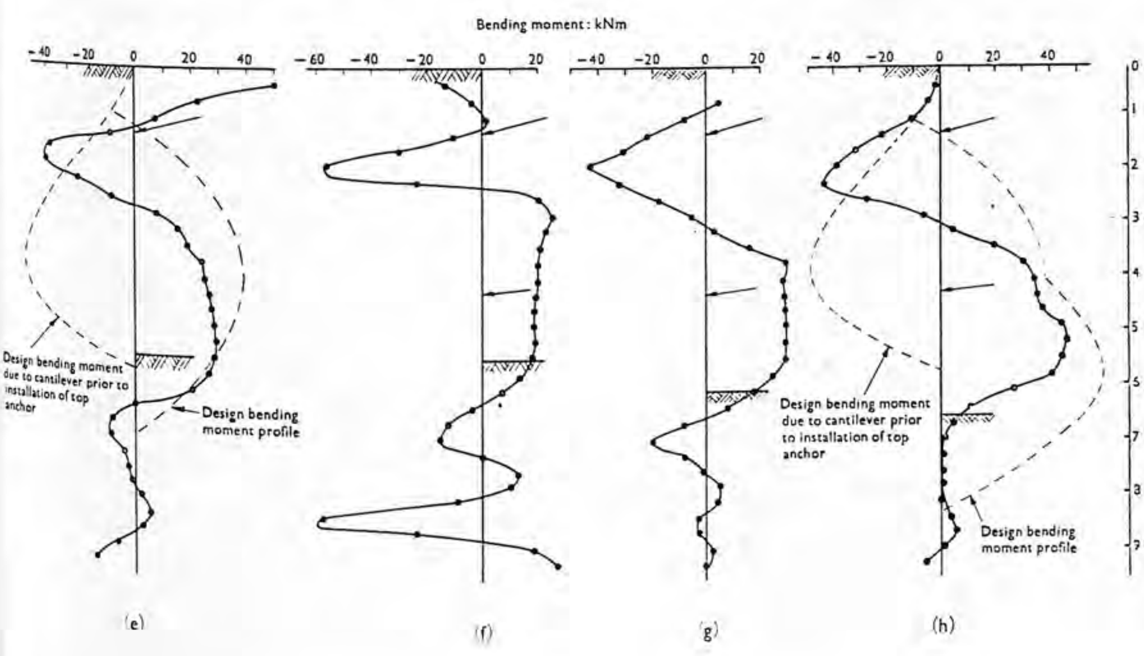
Discussion of results

12. In general the displacement profiles given in Fig. 4 display a logical progression and give rise to confidence in the inclinometer data. Profile (a) shows an overall rotation towards the excavation together with a superimposed cantilever action above the excavated depth. The maximum differential displacement (10 mm) between the crest and toe of the wall occurred during this initial cantilever stage of construction, giving an overall rotation of about 3·5 minutes of arc i.e. a slope of 1/970. The corresponding moment diagram curve ((a) of Fig. 5) appears to have been unduly affected by the stiffening effect at crest level due to the guide wall, and the peak of the curve occurs at the same elevation as the base of the guide wall.

13. The effect of stressing the top anchors is shown in profile (b) of Fig. 4, where the wall has been drawn back towards its original profile and the cantilever action has been reduced. The difference between profiles (b) and (c) corresponds to a time lapse of four days and indicates the magnitude of the movement set up within the gravels to mobilize the necessary restraint. During this period there has been an overall movement towards the excavation and a bulging deflexion below excavation level. This is further accentuated in profile (d).

14. Bending moment profiles (e)–(h) of Fig. 5 follow a pattern similar to that predicted by the design method. In particular (g) and (h) appear to confirm conclusively the bending moment profile that is assumed.

15. It was considered mathematically unsatisfactory to further differentiate the bending moment curve twice in order to obtain the resultant pressure distribution, but use of a sixth order polynomial resulted in a distribution which was approximately triangular. Together with the good agreement between design and measured bending moment profiles at the later construction stages, this reinforces the assumption of triangular pressure as opposed to the trapezoidal distribution normally assumed.



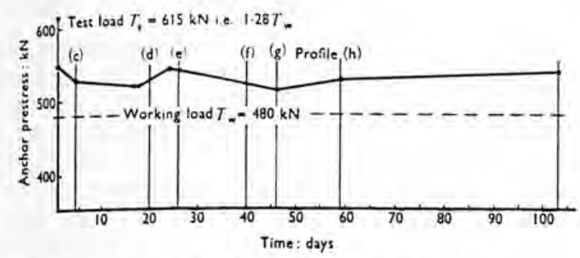


Fig. 6. Long term behaviour of anchor A12/T2

16. From the general survey the overall displacements monitored (profiles (d) and (h) of Fig. 4) show a displacement into the retained soil mass and some conflict appears to exist between these wall movements and the inclinometer profiles. Although the results were checked the apparent error was probably due to movement of the base line joining the fixed stations. In future the use of deep boreholes as fixed datum points is recommended to facilitate the monitoring of overall wall displacements at each construction stage.[2]

17. Vertical settlements measured showed no appreciable changes throughout the project, changes being scattered and of the order of 0·25 mm.

18. The upper anchors on panel A12 were monitored over a period of 103 days, and Fig. 6 shows that the prestress force remained remarkably constant throughout. These results suggest that there were no significant relative movements between the fixed anchor zone and the wall.

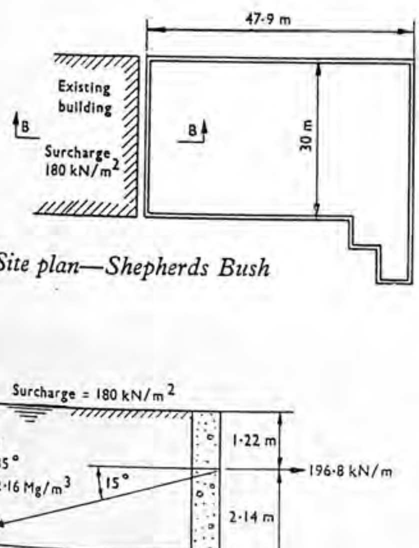


Fig. 7. Site plan—Shepherds Bush

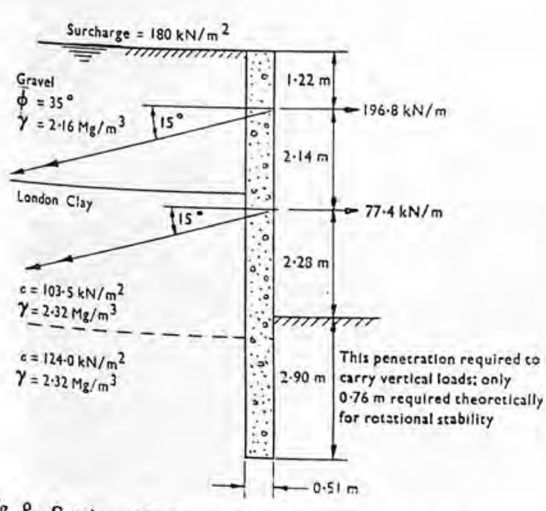


Fig. 8. Section BB through wall—Shepherds Bush

CASE STUDY NO. 2: NEW TELEPHONE EXCHANGE, SHEPHERDS BUSH, LONDON

19. For the new telephone exchange at Shepherds Bush, London, the main contractor was F. G. Minter and specialist contractors were Cementation Piling and Foundations Ltd and Cementation Ground Engineering Ltd.

20. Site conditions and soil parameters are shown in Figs 7 and 8. The site plan indicates a heavy surcharge from the existing multi-storey telephone exchange immediately behind the wall, which necessitated the adoption of panel lengths ranging from 4·5 m down to 2·2 m at this site.

21. For this diaphragm wall the total excavated depth was 10 m and generally the upper anchors were taken into the gravel and the lower ones into the clay. In spite of the close proximity of the existing exchange (see Fig. 9) no movement or distress was observed within this building during the construction period. However, the external re-entrant corner exhibited small lateral displacements (10–20 mm) into the excavation, and this led to groundwater seepage at local panel joints. The joints were subsequently sealed by cement grout but it is clear that the stability of external re-entrant corners is more critical than a normal straight section of diaphragm walling and special measures pertaining to wall design and the location and testing of anchors are recommended as follows.

(*a*) Reinforcement should be continuous in the diaphragm wall around the corner of re-entrant panels.
(*b*) A temporary capping beam should be cast if practicable around the top of the re-entrant corner and extended for two panels on either side, and should remain in position until the permanent floors or supports are installed.
(*c*) The grouted or fixed anchor zone of one row of cables must be outside the zone of influence of the active wedge of the wall parallel to this row of cables.

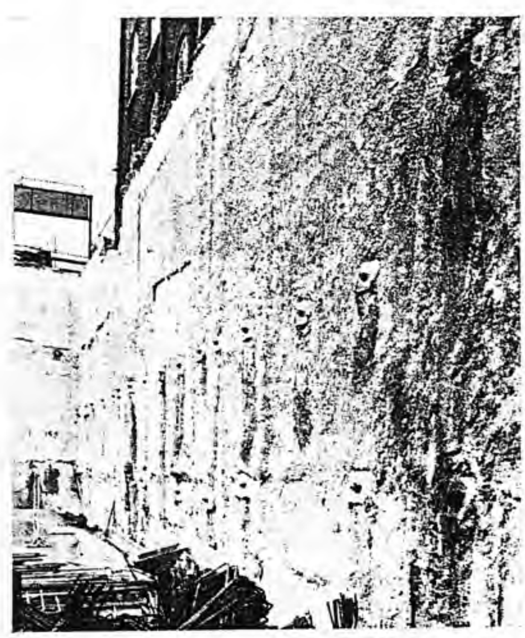


Fig. 9. Close-up view of anchored diaphragm wall —Shepherds Bush

(*d*) When an individual 24 hour check is carried out on an anchor supporting a re-entrant corner, there is a possibility of interaction with adjacent anchors still to be stressed. In this situation all anchors concerned (usually within three panels on either side of the corner) should be tested during the same period, preferably on one day, even if the 24 hour check has already been carried out on some of the anchors in question.

(*e*) Until more information is available on full-scale behaviour of external re-entrant corners, all anchors within three panels on either side should remain re-stressable until the permanent floors or supports are installed.

CASE STUDY NO. 3: CPF BUILDING, SINGAPORE

22. The CPF building was the first diaphragm walling contract to be executed in Singapore, and the case history is included in this Paper simply to illustrate a successful application overseas in difficult ground conditions. The anchorage contractor was Cementation Ground Engineering Ltd.

23. The site plan and panel section for this excavation are shown in Figs 10 and 11. Typical of the waterfront area of Singapore, ground conditions were highly variable (loose sands, soft marine clays, soft to stiff clay, sandstone and shales) with many old river inlet channels.

24. In such a situation it is rare for any anchored diaphragm wall enquiry to be accompanied by a comprehensive site investigation report. For example it is unlikely that the boreholes will be adequate in number, depth or disposition over the site area, and rarely are samples available for inspection. The designer is therefore faced with making engineering judgements on the stratification

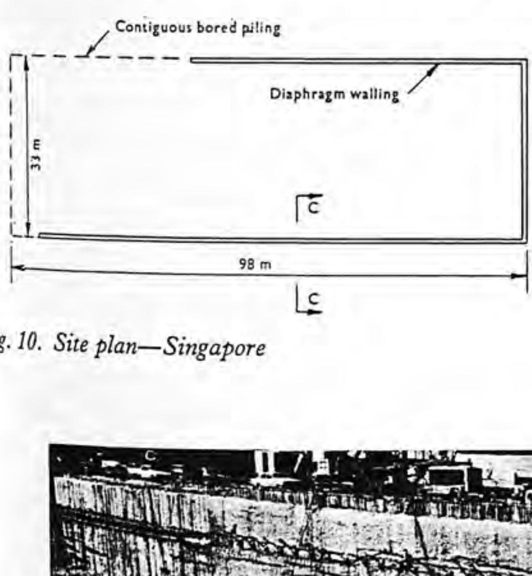


Fig. 10. Site plan—Singapore

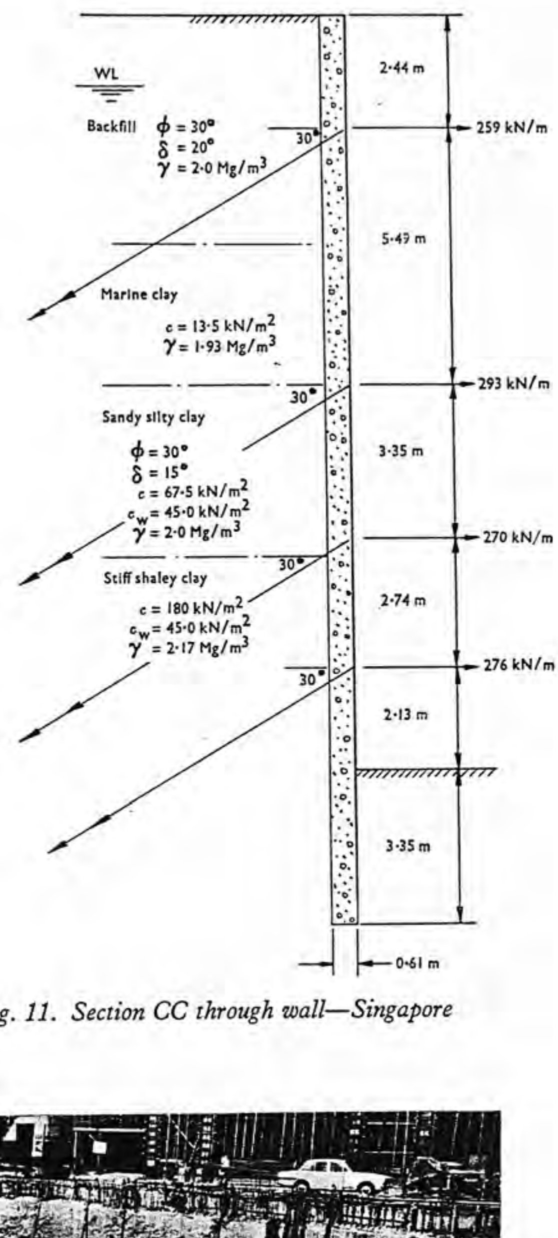


Fig. 11. Section CC through wall—Singapore

Fig. 12. General view of anchored diaphragm wall—Singapore

characteristics and strength parameters of the ground mass not just where results are available from boreholes, but also between boreholes spaced a hundred or more metres apart. The economics of the anchored wall construction and its field problems are intimately and inextricably tied to these fundamental judgements, which must be made before any design calculations can proceed.

25. For this type of project a construction investigation is necessary, in which continued observations are used to check information previously obtained, so that as the picture of the ground conditions becomes better defined anchor positions and forces may be varied if required. For this purpose it is necessary for a rapid design method to be available, and it is noteworthy that a computer program has been produced which enables design amendments to be made quickly to take account of significant differences in strata and soil conditions as these become exposed on site. This facility proved invaluable at Singapore. However, bearing in mind that the program is now readily available to UK engineers for local and overseas contracts, care must be taken to ensure that site investigations yield appropriate or relevant data of adequate quality before the computer program is invoked to execute the calculations.

26. With regard to wall construction, the panel thickness was 0·61 m and up to four rows of anchors were installed as excavation progressed downwards to a maximum depth of 16·8 m. The presence of soft clays and a high water table led to a reduction in length of wall panels from 4·5 to 3·5 m in certain areas, these panels in general being excavated down into stiff or very stiff clay and sometimes founded on the hard shale. Where the hard shale was above final formation level the wall had to be underpinned in short sections and in these areas the contact zone between the underside of the wall and the shale was found to be quite clean, the bentonite having been swept clear by the tremied concrete.

27. Throughout the construction period no distress was observed on adjacent roads and footpaths although these were heavily laden with contractor's plant, supply lorries and the dense Singapore traffic (see Fig. 12).

CASE STUDY NO. 4: KEYBRIDGE HOUSE, VAUXHALL, LONDON

28. Figure 13 shows a plan of a large double basement excavation for Keybridge House, a new telecommunications centre situated in a built-up area, most of the surrounding buildings being old with shallow foundations. Of particular concern was the close proximity of the Waterloo line, supported on viaduct arches, and St Anne's Church, where the diaphragm wall passed adjacent to the church foundations along two sides of the old building.

29. The main contractor for this project was Taylor Woodrow Ltd and specialist contractors were Cementation Piling and Foundations Ltd and Cementation Ground Engineering Ltd.

30. A diaphragm wall, 0·61 m thick and anchored at three levels, was adopted to retain 14·5 m of soil as shown in Fig. 14. Wall panel 9 was chosen to be monitored because it was fairly remote from corners and on a straight section of wall. It became evident during the excavation that the top of the clay dipped sharply in the region of panel 9, possibly indicating an old river channel. The

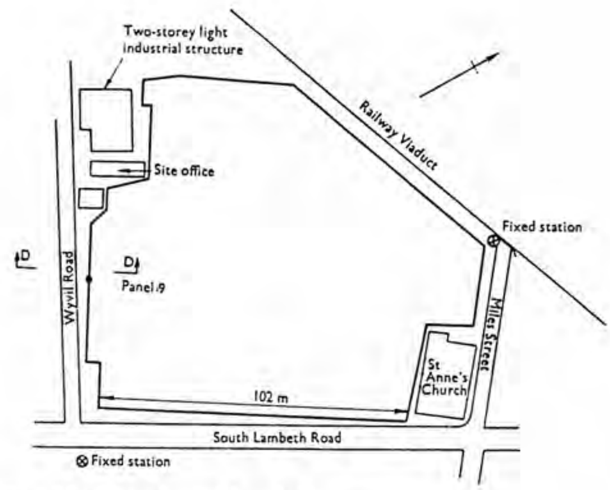


Fig. 13. Site plan—Vauxhall

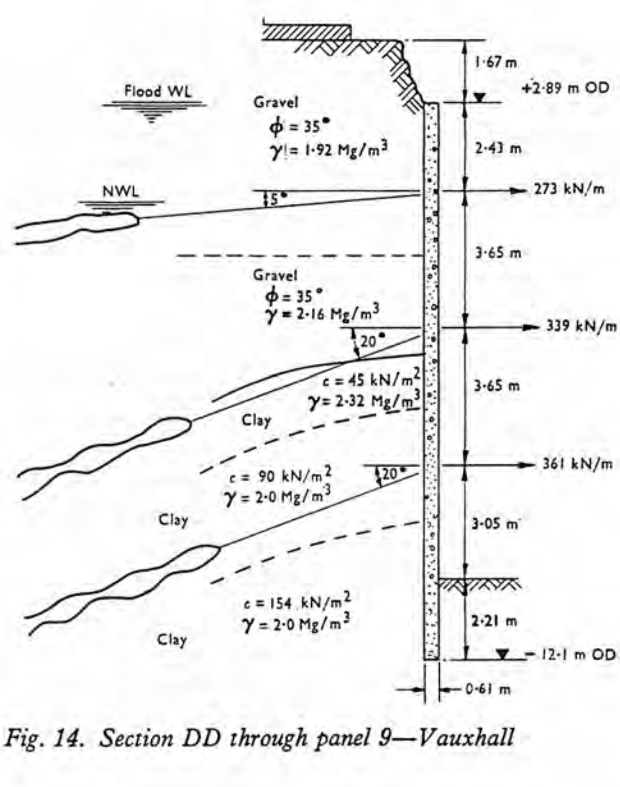


Fig. 14. Section DD through panel 9—Vauxhall

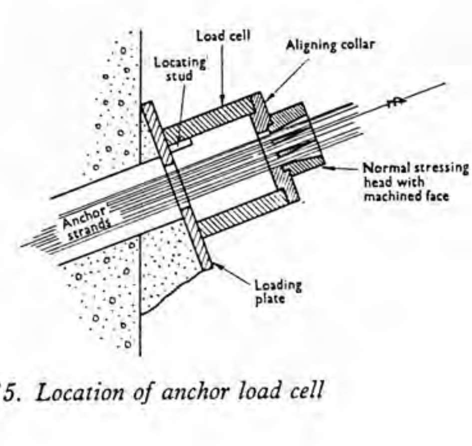


Fig. 15. Location of anchor load cell

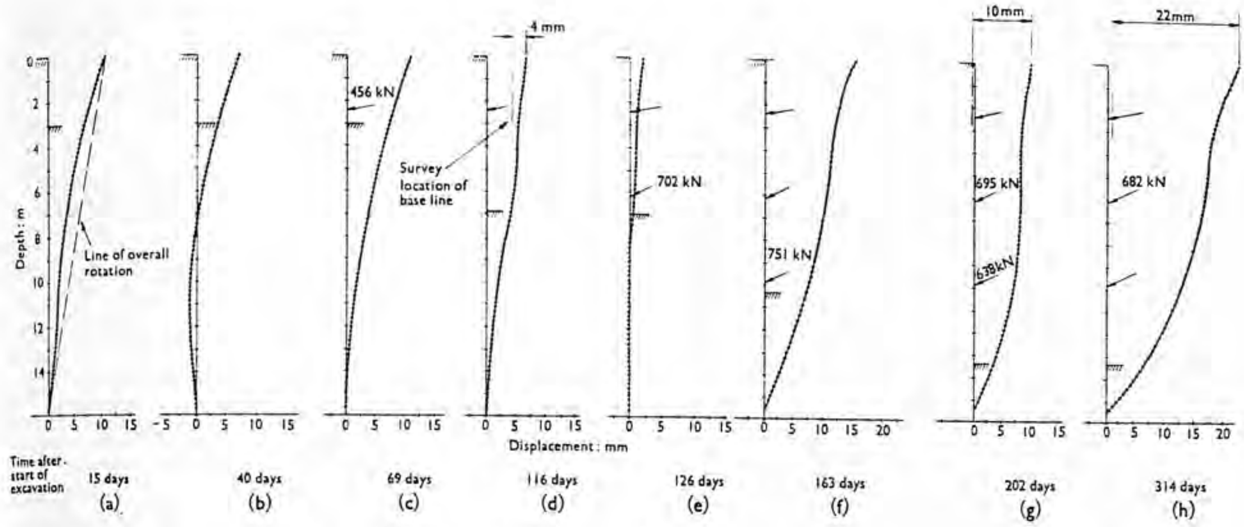


Fig. 16. Displacement profiles of panel 9 at various construction stages

presence of such channels, which appear to prevail in abundance in this district of London, necessitated considerable lengthening of certain cables in order to reach suitable strata for the grouted fixed anchor zone.

31. At panel 9, wall deformations were measured midway between two vertical rows of anchors. The method was similar to that described for Guildhall, except that an improved inclinometer[3] was employed (range $\pm 5°$, gauge length 200 mm and sensitivity 15″) and the base line distances from fixed stations to one wall station (see Fig. 13) were measured using a Tellurometer MA100 or Geodimeter in conjunction with a Kern DKM3 optional theodolite. These general surveys were supplemented by measurements of the vertical position of each station using a Kern GK23 level and staff, and closing errors indicated an accuracy of about 0·3 mm.

32. Anchor loads were measured using improved load cells fitted with eight electrical resistance strain gauges (four axial and four circumferential) to reduce effects of load eccentricity, and improved alignment of the loading system was also obtained by the use of a machined collar between the stressing head and load cell (see Fig. 15).

Discussion of results

33. Figure 16 illustrates the wall displacement profiles relative to the toe of the wall, the overall displacements being included only at the time of a general survey.

34. Profile (a) of Fig. 16 shows that an overall rotation of the wall occurred at an excavation depth of 3·05 m, similar to that of panel A12 at Guildhall. Slight bending is indicated between the excavation depth and 9·0 m, i.e. ± 3 m about the upper level of the London Clay.

35. It is clear from both panels monitored over the cantilever stage (Figs 4 and 16, profile (a)) that the displacement and overall rotation of the wall represent a large proportion (about 50%) of the corresponding movements at full excavation, thus illustrating the need for early support if wall movements are to be kept to a minimum.

36. Movement of diaphragm walls and associated basement heave are closely related to the method of support and manner of excavation. With anchored diaphragm walls the position of the top row of cables is governed by a balance between increase of the initial cantilever moments in the wall and limitation of the resultant inward movement towards the excavation. In the terraced areas of London where gravels overlie London Clay, the first row of anchors is usually located 3–4 m below ground level and the resulting inward deflexion of the wall is reduced to some extent when subsequent anchors are installed and tensioned. However, if movement of the wall must be kept to an absolute minimum, the first row of anchors must be located as close to ground level as possible; the limiting depth is usually 1·5 m, as with shallower anchorages there is a risk of local ground failure occurring behind the wall on tensioning the cables, with associated wall damage.

37. Profile (b) of Fig. 16 was monitored when all anchors had been installed and stressed to 450 kN for one week, except for one anchor immediately adjoining the inclinometer duct. Nevertheless this wall panel has been drawn back with apparent toe rotation, although the bending in the region below 12 m indicates resistance by the stiffer clay at toe level to the upper displacement of the wall by the anchors.

38. After a further 29 days when all upper level anchors were stressed to 450 kN, the wall deflexion (profile (c)) had reverted to the shape of profile (a). No major change in prestress load was monitored.

39. Following excavation to 6·8 m it was observed in profile (d) that below a depth of 6·5 m the deflexions were identical to those of profile (c), and an inward toe displacement towards the excavation of at least 2·25 mm occurred due to consolidation of the clay on the cut side. With the development of beam action below anchor level, the cantilever action within the upper 6 m almost disappeared. The degree of bending at this stage was very low, the maximum deflexion being only 0·6 mm with respect to the line of overall rotation.

40. With two levels stressed a further wall displacement towards the excavation occurred (profile (e)) and the deflexions within the upper 6 m were identical to those of profile (d), leading again to low bending moments.

41. Following excavation to 10·4 m and the stressing of all anchors the differential displacement between the upper anchor levels increased from 0·77 mm to 2·04 mm. Thus transition from profile (e) to profile (f) involved a further overall rotation of about 5 minutes of arc.

42. At the final excavation stage further rotation was indicated (profile (g)) and also toe displacement (about 1·5 mm) into the excavation. It would appear from this profile that further excavation caused outward bulging at and below lower anchor level and rotation about the toe and some point between the two upper anchors. This latter effect led to a reduction of differential displacement between crest and toe.

43. At this time the general survey indicated overall displacements of 10 mm and 0·5 mm into the excavation for the crest and toe respectively.

44. Vertical settlement readings indicated that the crest of the panel had moved down 12·2 mm, probably due mainly to the total vertical load component of the three levels of inclined anchors, equivalent to 432 kN/m^2.

45. After the final design stage was reached a delay of three months occurred in the construction programme, and profile (h) of Fig. 16 shows that differential displacement between crest and toe doubled, although the central anchor load exhibited only a slight loss of prestress. This indicates possible consolidation of the highly stressed soil surrounding the fixed anchor, or more likely that overall movement of the retained soil mass containing the anchors occurred. The total displacement of the crest was estimated to be 22·0 mm.

46. For the final two profiles the calculated bending moment curves are given together with the design values for the corresponding stage of excavation (see Fig. 17). Design values for the initial cantilever condition are also shown, where a modulus of 24 500 MN/m^2 ($3{\cdot}5 \times 10^6$ lb/in^2) and the full width of the wall (0·61 m) was used in calculating the second moment of area.

47. It can be observed that the magnitudes of the bending moment maxima are in good agreement with the design values although some variations in moment distribution occur. The magnitudes of peak bending moments measured are less than the design values by about 23%, although the magnitude of the design cantilever moment is similar to that created by the stressing of the upper row of anchors. However, it should be noted that the measured moments relate to the normal groundwater level whereas the design curves have been established on the basis of flood level (see Fig. 17).

48. Bending moment discontinuities at anchors occurred at slightly lower levels than the design elevations, the divergence being up to one metre. This anomaly is thought to be a function of anchor inclination and overdig before anchor installation. The moment curve above the upper anchor level is similar to one period of a sinusoidal waveform. The probable explanation for this phenomenon, as in the case at Guildhall, is the influence of the guide wall at the rear of the crest which had a depth of 1·3 m.

49. Comparison of the deformation profiles recorded up to final excavation stage indicates that bending moments were at maximum values at this stage.

50. With regard to overall behaviour the profiles indicate that more efficient anchoring was obtained with the gravel anchors and the panel exhibited rotation about the upper anchor regions. For the two anchors successfully monitored on the panel, a drop in load occurred over a period of six months. These reductions were small, being 20 kN (2·8%) for level 2 and 100 kN (12·7%) for level 3.

51. In general, crest displacements did not exceed 10 mm during the construction stage, and even at a later date the displacement was limited to 21·5 mm which corresponds to a ratio of displacement to excavated height of 1/620, and hence no noticeable settlements are likely. There is a dearth of information on acceptable movements associated with anchored walls but Jennings[4] has produced some interesting observations (see Table 1) for some large

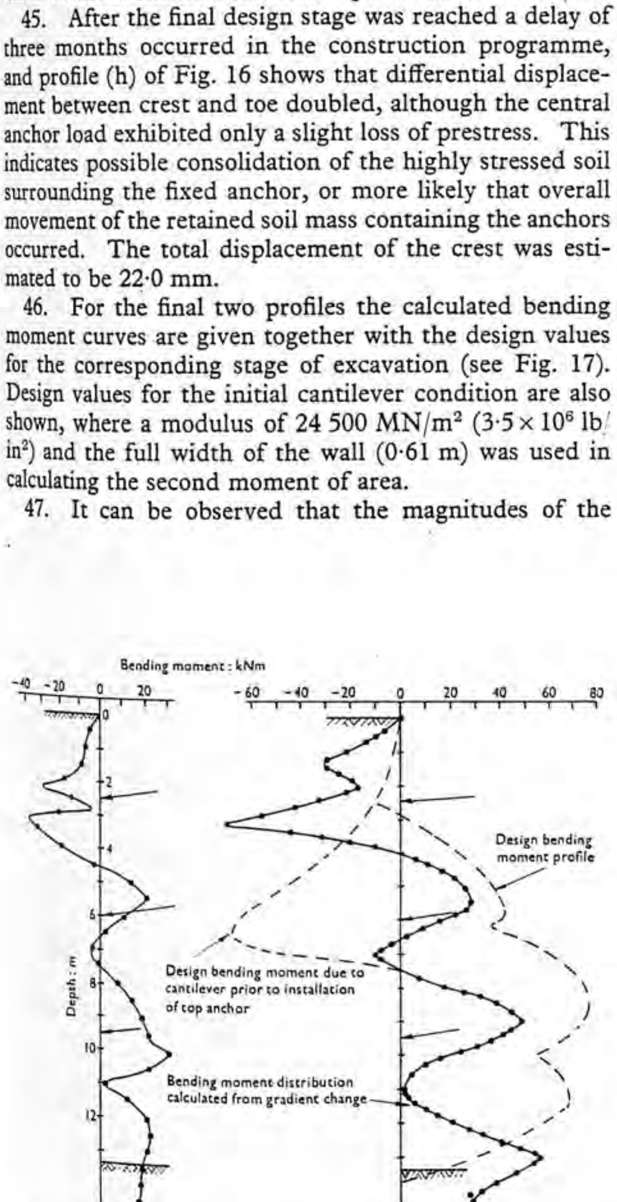


Fig. 17. Bending moment profiles of panel 9 at final excavation stage (moments based on 0·30 m strip)

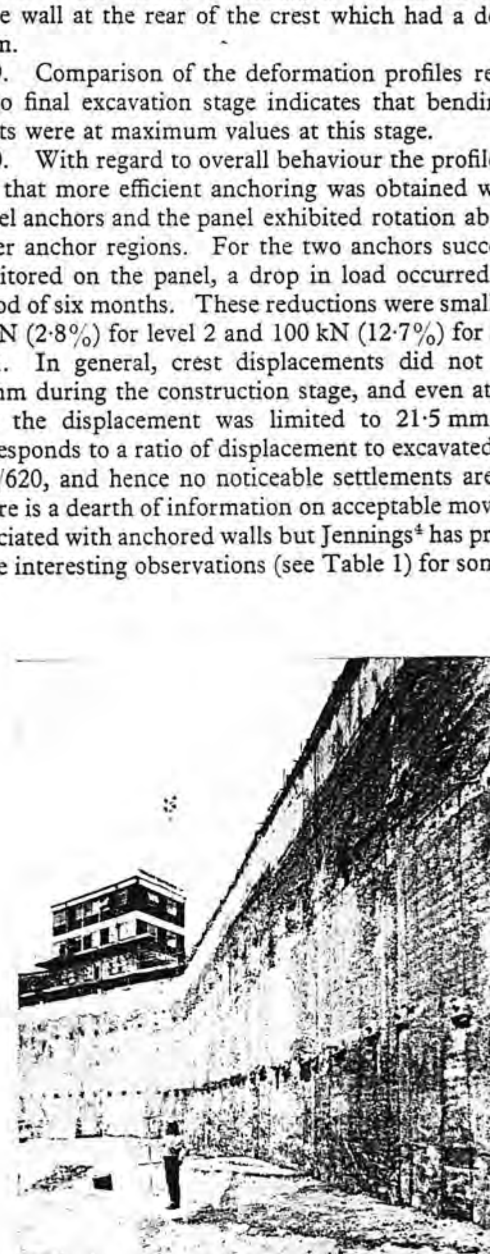


Fig. 18. General view of anchored diaphragm wall—Vauxhall

Table 1

Predominant soil supported	Depth of excavation H, m	Crest displacement Δ, mm	$\frac{\Delta}{H}$	Remarks
Firm fissured clay	14·7	76	1:194	Damage to services in the street and buildings across the street
Firm fissured clay	14·7	38	1:388	Acceptable movement
Firm fissured clay	22·9	38	1:600	Acceptable movement
Very stiff fissured clay	14·7	19	1:773	Acceptable movement
Soft jointed rock	18·3	25	1:730	Acceptable movement

deep cuts in South Africa. From the results it would appear that crest displacements should be limited to 40 mm otherwise severe damage to adjacent services or structures may result. At Vauxhall, although the movements were acceptable, it must be emphasized that the total crest displacement more than doubled in the four month period immediately following the final excavation stage, and clearly the consolidation of the clay on the excavation side was aided by the much reduced drainage path length.

CONCLUSIONS

52. Full-scale monitoring studies indicate that wall deflexions and bending moments, which occur as excavation proceeds, follow a similar pattern to those predicted by the empirical design method. In addition the results suggest a triangular pressure distribution which is contrary to the trapezoidal distribution assumed in the design of deep strutted excavations.

53. In view of the logical progression displayed by the displacement profiles formed using inclinometer data, it is recommended that monitoring studies of the type described should continue but investigations should also be carried out on a variety of similar construction techniques, e.g. on strutted diaphragm walls and multi-tied contiguous bored piled walls.

54. Finally, with regard to all the case studies presented no significant vertical or horizontal movements of walls have been monitored during the construction period. The Authors therefore submit that the empirical design method results in economical structures exhibiting satisfactory performance in the field, always provided that those design procedures are supported by an adequate site investigation programme.

REFERENCES

1. Littlejohn G. S. *et al.* Anchored diaphragm walls in sand. *Ground Engng*, 1971, **4**, Sept., 14–17, Nov., 18–21; 1972, **5**, Jan., 12–17.
2. Burland J. B. and Moore J. F. A. The measurement of ground displacement around deep excavations. *Field Instrumentation in Geotechnical Engineering*. Butterworths, London, 1974, 70–84.
3. Phillips S. H. E. and James E. L. An inclinometer for measuring the deformation of buried structures with reference to multi-tied diaphragm walls. *Field Instrumentation in Geotechnical Engineering*. Butterworths, London, 1974, 359–369.
4. Jennings J. E. Discussion on Deep excavations and tunnelling in soft ground. *Proc. 7th Int. Conf. Soil Mech., Mexico*, **3**, 1969, 331–335.

Ground anchors at Devonport Nuclear Complex

by G. S. LITTLEJOHN*, BSc, PhD, CEng, MICE, FGS, and C. TRUMAN DAVIES†

THE FIRST STAGE of the construction of the Nuclear Submarine Complex at Devonport has resulted in one of the largest and most interesting anchoring contracts ever undertaken in this country. Two dry docks, separated by a central promontory housing workshops, offices and other support facilities, are to be constructed in an existing basin approximately 140m square, situated in HM Dockyard.

The site is immediately underlain by a series of geosynclinal sediments of Upper Devonian age, dominated locally by hard grey-blue banded slate, known as "shillet". Numerous thin quartzitic greywacke layers occur, together with the occasional thin igneous intrusion. The rock surface dips at 3.5 deg on average from north east to south west across the site, and the uppermost 0.5-1.2m of bedrock are weathered and extremely fissile. The rock is tightly and strongly folded, due to its participation in the American orogenic period.

The basin was formed as part of a major extension to the Dockyard, between 1896 and 1907 and is surrounded by mass concrete retaining walls founded directly on unweathered slate bedrock. The minimum depth of these walls is 18m and around the north west corner, because of the bedrock dip, they reach a depth of 30m.

The Stage I contract, to produce a dredged and dewatered "hole" approximately 18m deep (15m below normal dock water level), involves the construction of a cellular steel sheet pile cofferdam and stabilisation of the existing basin walls against overturning.

The consulting engineers decided that the most suitable method, on technical and economic grounds, of improving the wall stability was to drill inclined holes from the top of the walls angled as near the heel of the wall as was thought practicable, into the underlying slate so that 333 No. 2 000 kN anchors could be installed. The design required that these anchors should not be more than 2.5m apart and that around the north west corner reduced centres of 1.0m would be necessary. The additional force provided by the anchors was considered sufficient to stabilise the east and the shallower sections of the north and west walls, but the north west corner, where the anchors were already at their minimum centres, still presented a problem.

Because the bedrock dips under this corner it was also judged that there was a risk of the toe of the north wall and underlying anchored mass of slate sliding bodily towards the centre of the basin when the full hydrostatic head was imposed. The problem of providing restraint to the north wall was solved by casting a thick mass concrete block near the centre of the basin, anchoring this concrete to the bedrock with 142 No. 2 000 kN vertical anchors and connecting the anchored block to the wall by means of a mass concrete thrust slab (Fig. 1). As stability of the walls is essential before dewatering commences both the placing of the mass concrete and the installation and stressing of the anchors had, of necessity, to be carried out under 15m of water.

Design aspects

(a) Overall stability

The assessment of the overall stability of an anchor is carried out to ensure that failure of the rock mass surrounding the anchor does not occur.

On this site an inverted cone of rock with an included angle of 90 deg was considered to fail and since the load is transferred from the tendon to the rock by bond the position of the apex of the cone was chosen at the middle of the grouted or fixed anchor length. The uplift capacity is normally equated to the weight of the specified rock cone, and where the ground is beneath the water table, the submerged weight of rock is used. The effect of groups of anchors involving interaction is to produce a flat vertical plane at the interface of adjoining cones (Fig. 2) and, as the spacing for a single line of anchors reduces a simple continuous wedge failure is ultimately assumed. No account was taken of the overburden pressure acting on the bedrock or the shear strength of the rock.

Little information is available on the safety factors employed when analysing the weight of rock in the assumed pull-out zone, but it is known that some designers apply safety factors of 1.6 to 2.0 whilst others equate the weight of rock to the required working load and assume that other rock parameters ignored in the calculation, e.g. shear strength, will produce a sufficiently large safety factor in the design as a matter of course. After discussion with the engineers the design allowed for the working load to equal the weight of rock.

Therefore based on a factor of safety of 1.0 and using a submerged density of $1.28 Mg/m^3$ ($80 lb/ft^3$) it was calculated that the depth through rock to the middle of the fixed anchor should not be less than 12m. The spacing between anchors was specified at 1.0, 1.5 or 2m, depending on the calculated magnitude of the overturning forces. At a spacing of 1m, the depth of alternate anchors was increased by 2m to spread the zone of load transfer over a greater thickness of rock and thereby improve resistance to laminar failure.

(b) Rock/grout bond

The straight shaft anchor relies mainly on the development of bond or shear in the region of the rock/grout interface to transfer the load to the surrounding rock, and estimation of the magnitude and distribution of the bond strength mobilised along the fixed anchor is without doubt a major problem facing the design engineer.

In very soft rock it is normal to assume a uniform distribution of bond stress but for rocks of high elastic modulus the distribution varies considerably. In spite of this knowledge it is current practice to assume an equivalent uniform distribution for bond stress or skin friction when designing the fixed anchor, i.e.

$$L = \frac{T_w}{\pi D . \delta_{skin}}$$

where $L =$ fixed anchor length
$D =$ fixed anchor diameter
$T_w =$ working load
$\delta_{skin} =$ working bond stress

Where shear strength tests are carried out on representative samples of the rock mass, the maximum average bond stress at the fixed anchor rock interface should not exceed the minimum shear strength divided by a suitable safety factor (normally not less than 2). In the absence of shear strength data or field pull-out tests, the ultimate bond stress (in the case of massive rocks) is often taken as one tenth of the unconfined compressive strength[1], up to a maximum value of $\delta_{skin} = 4.2 N/mm^2$ ($600 lb/in^2$), where the core crushing strength is equal to or greater than $42 N/mm^2$ ($6\,000 lb/in^2$). Bearing in mind the lack of relevant geotechnical data, an apparent safety factor of 3 is usually applied and the working bond stress is limited to $1.4 N/mm^2$ ($200 lb/in^2$). A minimum fixed anchor length of 3m is also generally recommended. In the absence of compressive strength data, but where 100 per cent core recovery is recorded the working bond stress is sometimes reduced to $0.7 N/mm^2$ ($100 lb/in^2$) at the initial design stage, prior to test anchors.

At the Nuclear Complex site, the geotechnical report included descriptions of the "shillet" at fixed anchor level, core recoveries ranging from 80-100 per cent were recorded but no strength data was provided. However, the conclusions and recommendations of a report[2] on test anchors for the new Frigate Complex nearby were made available to the specialist contractor. With reference to anchor design this report suggested that differences in the angle of bedding planes of the "shillet" had no significant effect upon the load carrying capabilities of the bedrock, and further that to give a factor of safety of 3 against failure at the rock/grout interface the allowable working bond stress should be $0.6 N/mm^2$. A factor of safety of 3 against shear failure at the rock/grout interface was specified for the Nuclear Complex anchors working at 2 000 kN and consequently a bond stress of $0.6 N/mm^2$ was employed to calculate a fixed anchor length of 8m for a 140mm dia hole.

*G. S. Littlejohn, Geotechnics Research Group, Department of Engineering, University of Aberdeen.
†C. Truman Davies, General Manager, Ground Anchors Ltd., Strood, Rochester, Kent.

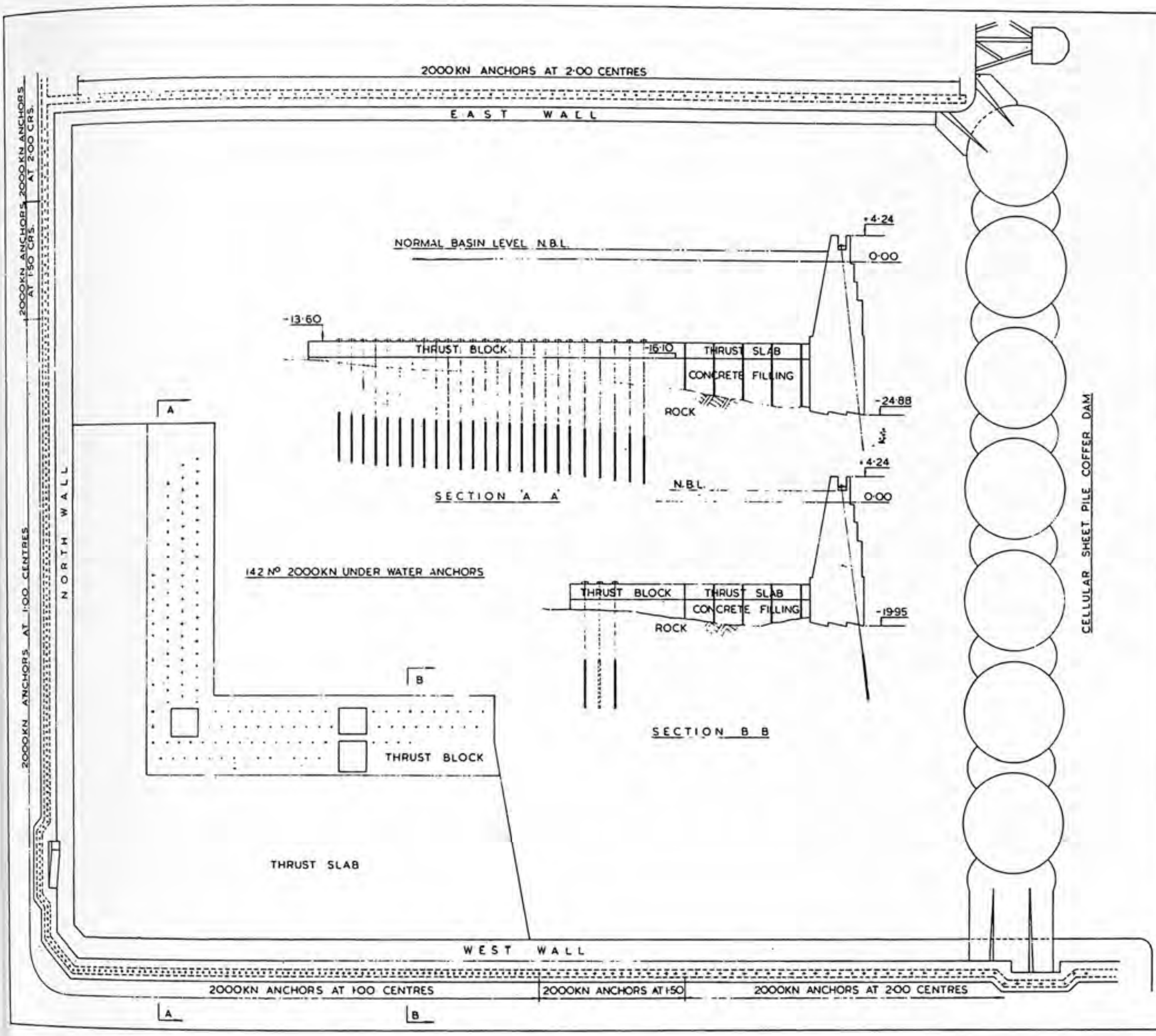


Fig. 1. Layout of anchors for the Devonport Nuclear Complex, Stage I

(c) Tendon/grout bond

Little information is readily available on this subject in relation to rock anchors, but the Australian Code (1973)[3] stipulates a maximum bond stress of 2.1N/mm^2 (300lb/in^2) for a clean strand tendon, which is close to the 2.2N/mm^2 recommended by CP 110 (1972)[4] for deformed bar embedded in concrete of characteristic strength equal to 30N/mm^2. This part of the design is not normally considered critical since the fixed anchor length, necessary to mobilise sufficient resistance at the rock/grout interface, usually allows a large safety factor against failure of the tendon/grout bond.

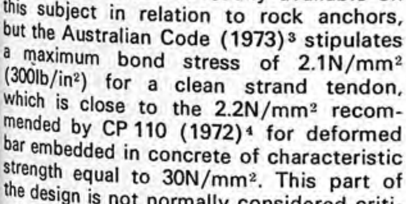


Fig. 2. Interaction of inverted cones in a stability analysis

(d) Tendon

A factor of safety of 1.6 against failure of the tendon was specified, giving a working load equal to 62.5 per cent of the ultimate tensile strength of the strands. To keep the borehole size and therefore drilling costs to a minimum, 15.2mm dia low relaxation "Dyform" strand was adopted with an ultimate strength of 300kN. At the initial design stage 11 strands were proposed for a 140mm borehole, but after discussions with the engineers it was agreed that the working stress in the tendon should be reduced in view of possible future standards for permanent anchors.

As a result of increasing the tendon size from 11 to 12 strands the working stress was reduced to 55 per cent UTS, the area of steel in the tendon being increased to 14.2 per cent of the borehole area. For permanent anchors it is considered that the area of steel should not exceed 15 per cent in order to ensure adequate spacing between strands and an outside grout cover of 5-10mm. Consequently the same hole diameter of 140mm was used to accommodate the new tendon.

(e) Test anchor

In order to check the design value pertaining to bond at the rock/grout interface and also monitor short term behaviour of the proposed anchor system prior to the contract, a test anchor was constructed according to the specified procedure for the contract but with a fixed anchor of only 3m. The hole, 140mm dia, was drilled through 27m of concrete wall and 16.0m of "shillet" and the tendon of 13 Dyform strands homed and grouted with neat cement grout ($w/c = 0.45$) during one day.

Initial stressing took place after nine days using a PSC MonoGroup jack type S335 capable of stressing the complete tendon in one operation. The first stage of the test was to stress the anchor to 800kN in 200kN increments at 5 min intervals. This was followed by destressing in similar decrements, tendon extensions being recorded at each load.

This cycle was repeated three times and the results are shown in Fig. 3. The anchor was then reloaded to 800kN and held for 53 hours, during which time monitoring was carried out at regular intervals but no further extension was observed. For a

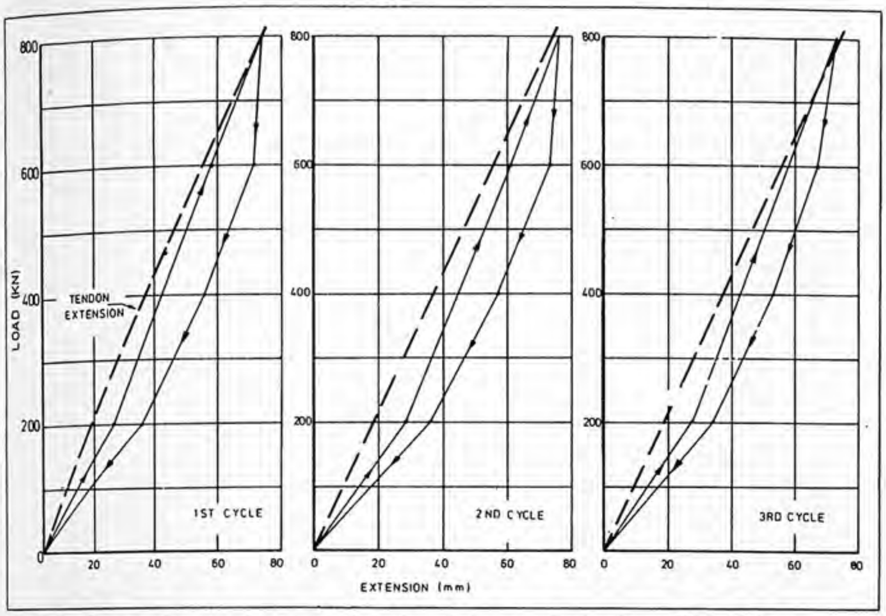


Fig. 3. Load-extension graphs for trial anchor during cyclic loading which indicates absence of permanent set

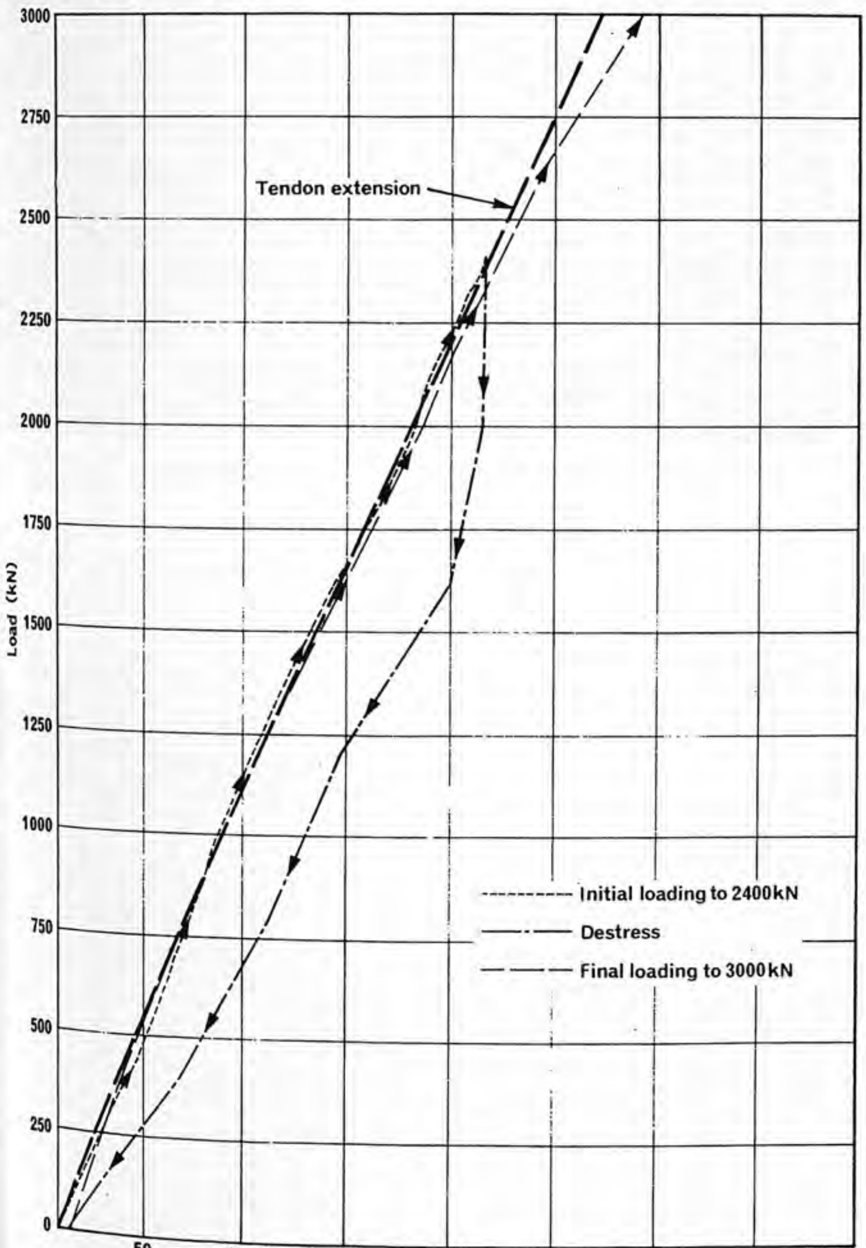


Fig. 4. Load-extension graph for trial anchor confirming a safety factor of 4

working bond stress of 0.6N/mm² at the rock/grout interface it was therefore concluded that the anchor behaved satisfactorily under cyclic loading conditions and no problem in relation to long term behaviour was highlighted.

Following this stage the load was applied in 200kN increments up to 2 400kN when the full extent of jack travel had been reached. The anchor was then destressed in 400kN increments, extensions being monitored at each increment of loading and unloading. At the end of this period when the anchor was completely destressed an apparent permanent set of 8mm was recorded.

Loading was resumed in 400kN increments until a load of 2 000kN was reached and the anchor automatically locked off. The jack was reset to give the required extra travel and loading was continued in 200kN increments to the maximum of 3 000kN (Fig. 4).

It was considered that the test anchor behaved satisfactorily throughout the test and the results indicated a minimum factor of safety of 4 with respect to fixed anchor length. Assuming an equivalent uniform distribution of bond the test imposed bond stresses of 1.61 and 2.27N/mm² at the tendon/grout and rock/grout interfaces, respectively.

Construction aspects—Wall anchors

An existing redundant services trench 1.2m wide and 2.0m deep, running around the existing basin provided the location for a heavily reinforced anchor beam, into which the load distribution plates and guide tubes were cast prior to drilling.

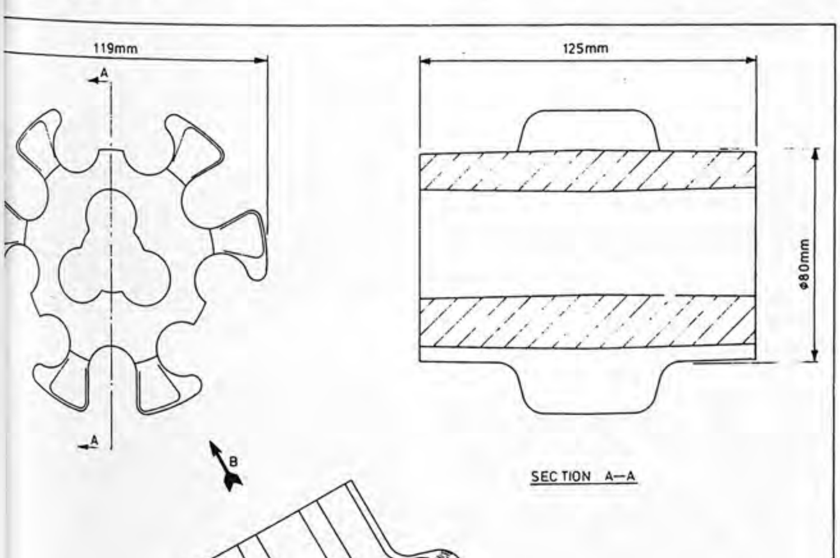
(a) Drilling

The 140mm anchor holes were drilled using Holman Voltrac rigs and Mission or Holman hammers fitted with button bits. Holes were inclined at 7 to 15 deg from the vertical and the maximum permissible deviation of the anchor zone centre line from the required position was specified at 1 in 80; this degree of accuracy was essential to avoid drilling into the soft material behind the wall. Until it had been demonstrated satisfactorily that the drilling could be kept within the allowed tolerance the initial holes were checked for deviation, at regular intervals, using a single shot photographic borehole inclinometer.

Although the Specification for the construction of the existing walls called for the removal of all weathered "shillet" and excavations for the bases of the walls to be taken a further 600mm into unweathered rock, soft zones were encountered during the drilling of certain anchors, mainly around the deep north west corner section. These zones of softened material occurred directly under the wall and were made evident by loss of air flush, difficulties in withdrawing the drill string or collapses into the hole obstructing the homing of the tendon. In all cases they were stabilised by injection and subsequent redrilling.

The delay between drilling and grouting should always be kept to a minimum and on this contract drilling of the fixed anchor zone, cable homing and subsequent grouting always took place on the same day. Each hole was overdrilled by 0.60m to act as a sump for drilled debris which might be left in spite of air and water flushing.

5. *Protection of inividual strands on site using a specially developed greasing sheathing machine*



6. *GA125 tendon spacer/centraliser*

(b) Tendon fabrication

The tendons were fabricated in a large covered area in which the strand delivered to site was also stored. The tendon comprised twelve 15.2mm Dyform strands laid parallel and since the anchors are permanent it was specified that the free length of individual strands should be greased and sheathed with plastic under factory controlled conditions. Protective systems from strand manufactureres are now readily available but suffer one major disadvantage, in that over the fixed anchor length it is necessary to remove the protection and carefully degrease the individual wires of each strand to permit efficient transmission of the load by bond.

To eliminate the laborious and inherently risky job of attempting to completely remove a graphited bituminous grease, which has been designed to resist easy removal, a machine has recently been developed to grease each individual strand and apply a protective plastic sheath (1.5mm wall thickness) only over the free length, where it is required (Fig. 5). The machine, which was designed for site operation, requires a power source equivalent to 12kVA.

It consists of a thermostatically controlled tank, to heat the grease to a suitable viscosity, a small air compressor running a pneumatic pump which pumps the grease under pressure to the greasing head, and a drive unit to feed the strand from its coil through the greasing head into the plastic tube. In operation, the end of the plastic tube is clamped into the greasing head and the strand is fed through to provide the projecting or stressed length. The pump is then started and the strand is greased under pressure before entering the plastic. In this way the free length is treated until the stressed length is clear of the end of the pre-cut plastic tube. The tube is then released and the fixed anchor length is fed past the head and the strand is cut by the integral disc cutter.

When 12 strands had been prepared in this way, the tendon was assembled, including a 25mm flexible tremie tube, and coupled to a purpose-made nose cone at the leading end of the fixed anchor; the anchor head was also fixed in place. In order to ensure a reasonable cover of grout of 5-10mm over the fixed anchor length, and to encourage efficient distribution of bond stress, specially developed spacer/centraliser units were attached at 2m centres (Fig. 6). These units, made of high density polythene, were designed to give a minimum outside grout cover of 10mm. It is considered that the use of parallel strands, properly spaced and centralised over the fixed anchor, is superior to the more traditional method of unravelling strands which gives a rather congested zone of individual wires with random spacing and cover.

(c) Tendon installation

With the longest tendons (45m) weighing over 700kg and almost every tendon being more than 30m long, it was necessary to find an efficient way to transport them around the site and lower them into the anchor holes. A machine built for the contract comprised a large hydraulically powered drum, mounted vertically on a two wheel trailer which could be pulled around the site by a Fordson tractor (Fig. 7).

On site the drum, which has a variable drive in both directions, was driven to the tendon preparation shed. A special yoke was then attached to the head of the tendon which was wound on the drum. The tendon was taken to the anchor hole, the drum reversed, and the tendon lowered slowly into the hole, until the anchor head was resting on the load distribution plate with the tendon hanging free in the hole. Once the tendon was in position, the hole was flushed with fresh water, after which the hole was left filled with water.

(d) Grout injection

The neat cement grout used in the injection stage was mixed in a Colcrete colloidal mixer. This patented method of mixing is based on the colloidal mill principle which produces an intense shearing action. The following advantages of the method were considered important at this site.

(i) Production of a grout that is virtually immiscible with water.

(ii) For any given water/cement ratio the fluidity of the grout is increased thereby permitting the use of lower water/cement ratios.

On this contract a 28 day minimum crushing strength of 27N/mm² was specified but a neat grout ($w/c = 0.45$) using a Rapid Hardening cement was chosen giving 42N/mm² at 28 days, so that the specified strength could be reached at an earlier time. As a result stressing was normally carried out 7-10 days after grouting, when the grout had reached 28N/mm² (4 000lb/in²) as established by grout cube tests. These test cubes were taken for each anchor and regular checks on the fluidity of the grout were also made as a further quality control during mixing.

Once mixed, the grout was pumped at a controlled rate to the bottom of the anchor hole via the tremie tube. Injection was continued until good quality grout emerged from the top of the anchor hole, and since all grouting took place in an open hole no pressure greater than the excess head of grout in the borehole was applied to the fixed anchor. It is noteworthy that today where plastic covered tendons are used, or where the free length is otherwise de-bonded, grouting is usually carried out in one operation for economical reasons, compared with the more traditional method of primary and secondary grouting of the fixed and free anchor lengths, respectively. It is most important to ensure that the set grout column has no contact with the anchor head, otherwise during stressing a compressive load will be generated in the grout and this "strut effect" may prevent proper loading of the fixed anchor zone.

(e) Stressing

Stressing of each anchor was carried out in one continuous operation using a PSC MonoGroup stressing jack (Fig. 8). The normal procedure was to record tendon movement at the jack ram as the load was incrementally applied, until the designed working load (T_w) was attained. The anchor was then locked off at the working load plus an allowance of approximately 5 per cent relaxation. During stressing, anchors were tested to 1.25 and 1.5 T_w as directed and to check for loss of prestress the residual loads in selected anchors were monitored 14 days after initial tensioning.

It is noteworthy that current practice recommended that all anchors should be tested to at least 1.25 T_w, and that one in ten pre-selected working anchors should be tested to 1.5 T_w, but recent recommendations[5,6,7] accept this practice for temporary anchors only and indicate that more rigorous testing is required for permanent anchors.

For important works involving permanent anchors in the future it is recommended by the authors that all anchors should be tested to at least 1.5 T_w.

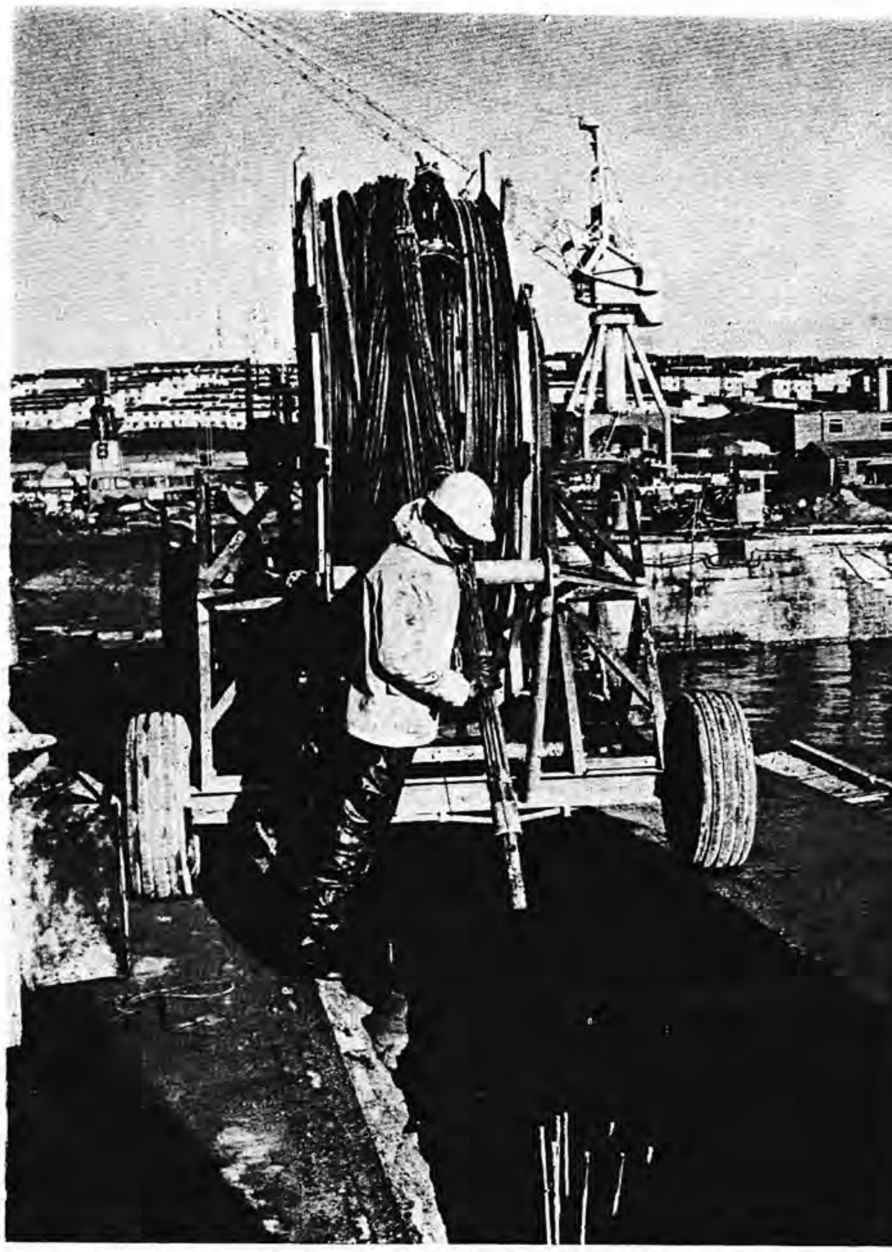


Fig. 7. Hydraulically operated tendon transporter for handling 45m long tendons

Underwater anchors

(a) Underwater concreting and stool placement

The initial operation, to reduce the level within the basin, was carried out by a dredger which also removed the weathered surface of the "shillet". Final cleaning of the surface, immediately prior to concreting was done with a conventional air lift, suspended from a floating crane. All the underwater concrete was placed through a tremie pipe, and in areas where the rock surface was uneven concrete was placed intially to bring it up to the underside of the 2m thick thrust block. L-shaped precast concrete units, 2m high, were placed along the line of each edge of the thrust block and at pour divisions, to act as permanent shutters and provide a level track for the underwater screed. Concrete was then placed within the units and screeded when the final level was reached.

Since the concrete was not reinforced it was necessary to provide a method of spreading the high loads that are imposed by the anchors. To achieve this end, and to assist in collaring the holes, a precast stool, 800mm square and 600mm high incorporating a 2 000kN anchor load distribution plate was placed at each anchor position. These stools were assembled in groups of three in frames on land, lowered into the water by crane and guided into the correct position by divers. The frame was fitted with levelling jacks to facilitate levelling and when set in position the individual stools were grouted to the surface of the thrust block. When the grout had set the frame was released.

(b) Anchor installation

To provide a stable working platform for drilling rigs and compressors use was made of a number of coupled Uniflotes.

g. 8. *An operator recording the tendon extension of a 2 000kN anchor during the* *urse of multistrand stressing*

sition was held by two hand winches d mooring ropes and the platform was trutted" against the existing walls. ter the initial positioning of the plat- m had been made in relation to the ain Contractor's setting out, the drill s set up and fixed in approximately the rrect position. The hammer and casing e were then coupled until 15m had en assembled, and the diver descended ng the drill tubes as a guide to "stool" el.

By means of a special communications met, the diver then directed the driller set up the rig vertically over the centre the stool.

Drilling, anchor fabrication, tendon plac- and grout injection were then carried using the same procedures, as for the ll anchors, except that an electrical ut level indicator probe was attached he tendon at a predetermined level. To id grout wastage and contamination of water which would effect visibility it decided to stop injection as soon as grout reached this level. Secondary ting was carried out after anchors been stressed.

(c) Diving chamber

The installation of high capacity anchors under 15m of water presented a unique problem—that of satisfactorily stressing the anchors. After considering possible alternative methods it became apparent that this must be done by divers and, because of the minimal underwater visibility in the basin, it was decided to use a diving chamber.

The main advantages were that the whole operation could be carried out in a dry environment and that there would be facilities available to engineers to witness the loading of the anchors.

The diving chamber, which was designed and built for this contract, was rectangular (internal dimensions approximately 2.1m x 1.5m x 1.8m high) and was suspended from a winch rope below a four Uniflote platform carrying all the necessary support systems. These consisted of a winch for lifting the chamber, a high pressure compressor set for aqualung charging, a low pressure compressor set supplying air to the chamber, the hydraulic pump for post tensioning and a control room.

The chamber accommodated two diver/stressmen and the observer, and from the roof a 3 000kN hydraulic stressing jack was suspended having its master controls on the surface and duplicate gauges and controls within the chamber. The chamber was also equipped with communications, lighting and an emergency air supply.

The stressing operation commenced with the chamber being roughly positioned over the anchor stool, by reference to surface setting-out points. The divers, wearing self-contained breathing apparatus, descended and entered the chamber, removed their breathing appartus and directed the chamber over the anchor. The anchor load was applied in the normal manner, operation of the jack being controlled from the surface, where tendon extensions and load increments were also recorded in a control room on the diving platform.

Service behaviour

Bearing in mind the importance of this contract, a specimen number of vibrating wire load cells have been installed at three wall locations. Each group of instrumented anchors straddles the location of an existing inclinometer station in order to facilitate future analyses of wall movement in relation to anchor loads.

The installation of load cells at Devonport represents part of a general field investigation by the Geotechnics Research Group into the service behaviour of soil and rock anchor systems. The specific objectives on this contract are:

(i) to provide data on long term behaviour of high capacity rock anchors with particular reference to prestress fluctuations, and

(ii) to monitor anchor loads during dewatering and subsequent construction stages at three wall locations, in order to check that behaviour of the complete anchor/wall system is satisfactory.

It is hoped that this work will provide an interesting case study in the future.

Acknowledgements

The authors are indebted to the Property Services Agency, Department of the Environment, and their consulting engineers, Sir Alexander Gibb and Partners for permission to publish this article. The main contractor for the civil work was Costain Civil Engineering Ltd., and the anchoring contractor was Ground Anchors Ltd.

References

1. *Littlejohn, G. S. (1972):* "Some empirical design methods employed in Britain". Part of questionnaire on rock anchor design. Geotechnics Research Group, Department of Engineering University of Aberdeen (unpublished technical note).
2. *Universal Anchorage Co. Ltd. (1972):* Report on Rock Anchor Tests for Frigate Complex, HM Dockyard, Devonport. Report No. 189. Universal Anchorage Co. Ltd., Egerton Street, Farnworth, Bolton (unpublished).
3. *Standards Association of Australia (1973)* "Ground Anchorages" SAA Prestressed Concrete Code CA35, Section 5, pp. 50-53.
4. *British Standards Institution (1972)* "The Structural use of Concrete" CP 110, Part 1, BSI, 2 Park Street, London.
5. *Bureau Securitas (1972):* Recommendations regarding the design, calculation, installation and inspection of ground anchors. Editions Eyrolles, 61 Boulevard Saint-Germain, Paris—Ve. (Ref. TA 72).
6. *Littlejohn, G. S. (1973):* "Ground Anchors Today— A Foreword". *Ground Engineering 6*, 6, pp. 20-22.
7. *Ostermayer, H. (1974):* "Construction Carrying Behaviour and Creep Characteristics of Ground Anchors". Proc. of Conf. on Diaphragm Walls and Anchorages, Instn. Civ. Engrs, London.

Rock anchors - state of the art

by G. S. LITTLEJOHN, BSc, PhD, MICE, MIStructE, FGS,
and D. A. BRUCE, BSc, AMICE, FGS

Originally published in 'Ground Engineering' magazine between May 1975 and May 1976

FOUNDATION PUBLICATIONS LTD. 1977.

7, Ongar Road, Brentwood, Essex, CM15 9AU, England

Part 1: Design

INTRODUCTION

THE HISTORY OF prestressed rock anchors dates from 1934, when the late Andre Coyne pioneered their use during the raising of the Cheurfas Dam, in Algeria. Since then, the employment of rock anchors in dam construction has become world wide, and several million tons of working capacity have been successfully installed. Rock anchors have also been used for many years to ensure the safety of large underground excavations and the stability of natural and artificial rock slopes.

In recent years the range of applications has widened considerably due in part to the success achieved by soil anchors in tying-back retaining walls, holding down dock floors, and pile testing. Now, largely as a consequence of the success of anchors in these new applications, rock anchors are expected to perform without difficulty, even when installed in relatively poor quality weathered or laminated rock.

In addition there is a trend towards higher load capacities for individual and concentrated groups of anchors. For the higher dams in vogue today, prestressing of the order of 200t/m may be required, necessitating individual anchors of capacity well in excess of 1 000 tonnes. In the field of suspension bridges concentrated groups of anchors with a working capacity of 6 000t are already being seriously considered, and design loads of 15 000t are anticipated in the future. Even in strong competent rocks, these high prestress levels are demanding engineering judgements in areas where no relevant precedents exist.

Bearing these points in mind, the authors believe that there is a growing need to establish and employ reliable design formulae and realistic safety factors together with relevant quality controls and testing procedures.

The first article in this state-of-the-art review, therefore, considers design procedures relating to overall stability, grout/rock bond, tendon/grout bond, and tendon, along with the choice of safety factors. The second article deals with the practicalities of installation, construction and quality control, whilst the third examines testing and stressing procedures§.

The purpose of this general appraisal is to describe current practice in relation to rock anchors by drawing on the experience gained in various countries over the past 30 years. Experimental and theoretical studies in the fields of reinforced and prestressed concrete are also included where relevant. It is hoped that the information provided will be of direct benefit to anchoring specialists but, at the same time, the series of articles are intended as a basis for discussion since points are highlighted concerning the validity of the basic design assumptions, and the lack of knowledge of full-scale anchor performance.

DESIGN—AN INTRODUCTION

A grouted rock anchor may fail in one or more, of the following modes:

(*a*) by failure within the rock mass,
(*b*) by failure of the rock/grout bond,
(*c*) by failure of the grout/tendon bond, or
(*d*) by failure of the steel tendon or top anchorage.

Therefore in order to establish the overall safety factor for the anchor each of the above phenomena must be considered in turn.

Broadly speaking, present design criteria may be classified into two equally unsatisfactory groups. On the one hand there are the procedures based on the classical theory of elasticity. Clearly, the validity of results derived from, for example, photoelastic or finite element techniques dependent on such a theory, is questionable when dealing with a heterogeneous rock mass. On the other hand, anchor parameters are frequently selected by, at best, crude empirical rules or trial and error methods, and at worst, by pure guesswork. The gap between these two extremes is still very real, despite a growing awareness of the problems, as witnessed by the recent appearance of standards or draft codes on ground anchors in several countries.

The main design concepts are now reviewed with respect to the four failure modes listed above, but it should be emphasised that these concepts relate primarily to prestressed cement grout injection anchors.

UPLIFT CAPACITY OF THE ROCK ANCHOR SYSTEM

Design procedures

This section deals with methods currently used in practice to estimate the anchor depth required to ensure that the working load will be resisted safely without failure occurring in the rock mass. The methods described apply to anchors which have been constructed in a vertical or steeply inclined downwards direction.

In the case of single anchors, most engineers assume that, at failure, an inverted cone of rock is pulled out of the rock mass (Fig. 1). The uplift capacity is normally equated to the weight of the specified rock cone, and where the ground is situated beneath the water table, the submerged weight of rock is used. The depth of anchor calculated in this manner may, of course, be reduced where it can be demonstrated by test anchors that the working force can be otherwise achieved safely.

The effect in groups of anchors is the production of a flat, vertical plane at the interface of adjoining cones (Fig. 2). As the spacing for a single line of anchors reduces further, a simple continuous wedge failure in the rock is assumed. This approach has been employed by many engineers in practice and is described by Parker (1958), Hobst (1965), Littlejohn (1972) and Hilf (1973).

However, although the shape of the fail-

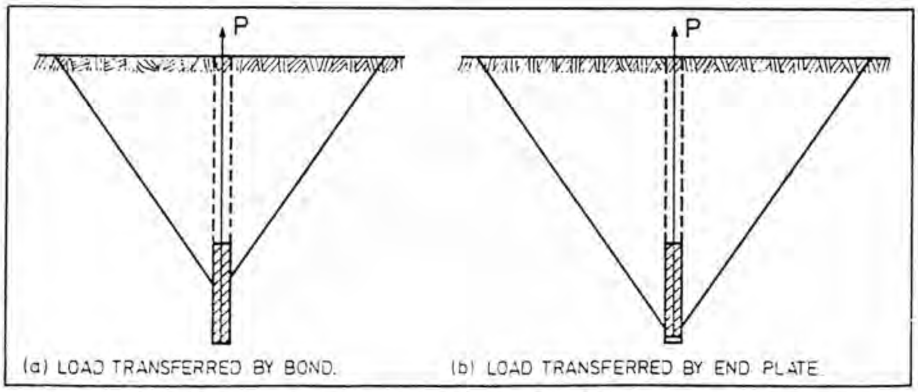


Fig. 1. Geometry of cone, assumed to be mobilised when failure occurs in a homogeneous rock mass

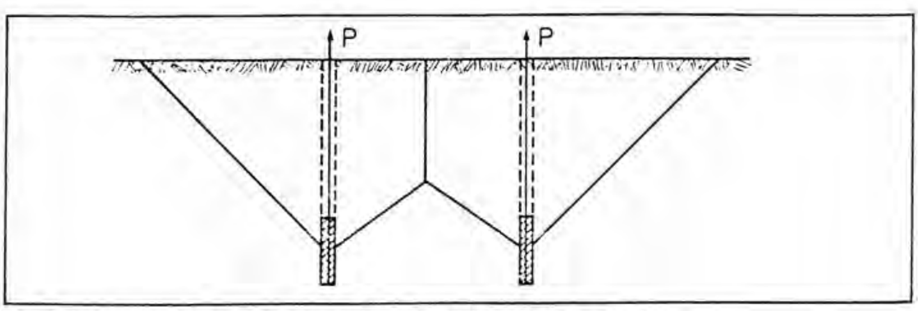


Fig. 2. Interaction of inverted cones in an overall stability analysis

TABLE I—GEOMETRIES OF ROCK CONE RELATED TO FIXED ANCHOR WHICH HAVE BEEN EMPLOYED IN PRACTICE

Geometry of inverted cone		Source
Included angle	Position of apex	
60 deg 60 deg	Base of anchor Base of anchor	Canada—Saliman & Schaefer (1968) USA—Hilf (1973)
90 deg 90 deg 90 deg 90 deg 90 deg 90 deg 90 deg 90 deg 90 deg	Base of anchor Base of anchor Base of anchor Base of anchor Base of anchor Base of anchor Base of rock bolt Base of anchor Base of anchor where load is transferred by end plate or wedges	Britain—Banks (1955) Britain—Parker (1958) Czechoslovakia—Hobst (1965) USA—Wolf et al (1965) Canada—Brown (1970) Australia—Longworth (1971) USA—Lang (1972) USA—White (1973) Germany—Stocker (1973)
90 deg 90 deg 90 deg 90 deg	Middle of grouted fixed anchor where load is transferred by bond Middle of anchor Middle of anchor Middle of anchor	Germany—Stocker (1973) Britain—Morris & Garrett (1956) India—Rao (1964) USA—Eberhardt & Veltrop (1965)
90 deg 90 deg 90 deg	Top of fixed anchor Top of fixed anchor Top of fixed anchor	Australia—Rawlings (1968) Austria—Rescher (1968) Canada—Golder Brawner (1973)
*60 deg-90 deg *60 deg-90 deg	Middle of fixed anchor where load is transferred by bond Base of anchor where load is transferred by end plate or wedges	Britain—Littlejohn (1972)
90 deg 60 deg	Top of fixed anchor or Base of anchor	Australia—Standard CA35 (1973)

*60 deg employed primarily in soft, heavily fissured or weathered rock mass

ure volume is widely agreed, its position with respect to the grouted fixed anchor length (socket) varies somewhat in practice. This aspect is illustrated by Table I, which contains examples drawn from anchor designs in various countries. Another feature which is widely appreciated, but receives little attention is that a solid, homogeneous rock mass is seldom encountered, and so, in the vast majority of cases, modifications to the simple cone approach should be made by experienced rock mechanics engineers.

In connection with this "weight of rock" method of calculating the ultimate resistance to withdrawal, little data are available on the safety factors employed. However, it is known that safety factors of 3 and 2 have been used by Schmidt (1956) and Rawlings (1968) respectively, while most recently a factor of 1.6 was employed for anchors at the Devonport Nuclear Complex by Littlejohn and Truman-Davies (1974). In current practice the factor of safety is reduced to unity on many occasions on the basis that certain rock parameters, e.g. shear strength, otherwise ignored in the design will give rise to a sufficiently large factor of safety as a matter of course. This bonus of shear strength is, of course, greatly reduced when anchors are installed in highly fissured "loose" rock masses, especially those with much interstitial material or high pore water pressure. This point was recognised by Hobst (1965) when he presented the formulae given in Table II for calculating the depth of the cone; in these

τ = shear strength of rock (tonnes/m^2)
F = factor of safety against failure (F = 2-3 customary)
s = spacing of anchors (metres)
ϕ = angle of friction across fractures in rock mass
γ = specific gravity of rock (tonnes/m^3)

Note that the shear strength is considered in dealing with anchors in homogeneous rock, whereas rock weight is the dominant parameter when dealing with fissured rock masses. In Britain, the shear strength parameter is usually ignored in practice (thus erring conservatively) since quantitative data on the fracture geometry and shear strength of the rock mass are seldom available at the design stage. In this connection it is noteworthy that Klopp (1970) found in typical Rhine Slate, that elevated hydrostatic and seepage pressures could reduce the shear strength of mylonitic zones to about 20 per cent of the "ideal" laboratory dry value, and occasionally to as low as 4 per cent of this figure.

TABLE II—DEPTH OF ANCHOR FOR OVERALL STABILITY (after Hobst, 1965)

Rock type	Formula for depth of cone: One anchor	Formula for depth of cone: Group of anchors
"Sound" homogeneous rock	$\sqrt{\dfrac{F.P}{4.44\,\tau}}$	$\dfrac{F.P}{2.83\,\tau\,.\,s}$
Irregular fissured rock	$\sqrt[3]{\dfrac{3F.P}{\gamma\pi\tan\phi}}$	$\sqrt{\dfrac{F.P}{\gamma\,.s.\tan\phi}}$
Irregular submerged fissured rock	$\sqrt[3]{\dfrac{3F.P}{(\gamma-1)\,\pi\tan\phi}}$	$\sqrt{\dfrac{F.P}{(\gamma-1).s\,.\tan\phi}}$

Other engineers confirm that rock shear strength generally contributes a major component of the ultimate pull-out resistance. Brown (1970) states that the ultimate capacity of an anchor, in homogeneous, massive rock, is dependent on the shear strength of the rock and the surface area of the cone, which for a 90 deg cone is proportional to the square of the depth of embedment i.e. $4\pi h^2$. Usually a maximum allowable shear stress is specified, acting over the cone surface e.g. 0.034N/mm^2 (Saliman and Schaefer, 1968)). Hilf (1973) advocates that regardless of rock type a value of 0.024N/mm^2 may be allowed and specifies a safety factor of 2 on a test load displacement of up to 12mm. Values in excess of 0.024N/mm^2 may be used if verified by field tests.

Experimental evidence

In general, there is a dearth of data on anchor failures in the rock mass but a set of tests which provides some results on the overall stability aspect is presented by Saliman and Schaefer (1968) who describe the failure of grouted bars on the Trinity Clear Creek 230kV transmission line. Four tests were carried out on deformed reinforcement bars grouted into 70mm diameter holes to a depth of 1.52m in sediments, largely shale. In all cases failure occurred when a block of grout and rock pulled-out; the propagation of cracking to the rock surface gave an indication of the cone of influence (Fig. 3). Assuming a bulk density of 2Mg/m^3 for the rock, back analysis of the failure loads indicates very conservative results—safety factors on the pull-out load between 7.4 and 23.5—if the apex of the 90 deg cone is assumed at the mid-point of the anchor length, but lower factors—0.9 to 2.9—for a cone with the apex at the base.

However, in laminated dolomite in which Brown (1970) installed shallow test anchors, the shape of the pull-out zone could not be observed, although the extensive area over which the rock surface was uplifted around certain anchors suggested failure along a horizontal bedding plane (laminar failure).

Spacing

Rock failures of this mode Brown thought to be restricted to shallow anchors, but in current practice, fear of laminar failure or excessive fixed anchor movement during service has led to the adoption of staggered anchor lengths even at great depths for closely spaced anchors. In unfavourable conditions, for example where rock bedding planes occur normal to the anchor axis, the purpose of staggered lengths is to reduce the intensity of stress across such planes at the level of the fixed anchors.

It is thus evident that whilst a major factor in the choice of anchor depth is the size of rock cone or wedge to be engaged, the possibility of laminar failure may also influence the designer's choice of length in closely spaced anchor groups.

The South African Recommendations (1972) suggest that in the case of a "concentrated" group, where the fixed anchors are spaced at less than 0.5 x the fixed anchor length apart, the stagger between alternate anchors should be 0.5 x the fixed anchor length. This compares with a stagger of 0.25 x the fixed anchor length recommended for the Devonport Nuclear Com-

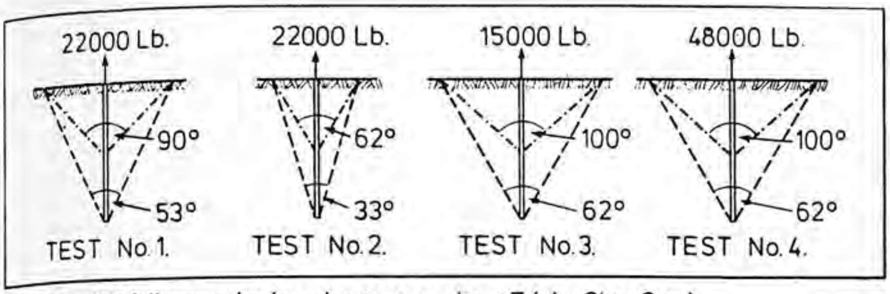


Fig. 3. Possible failure modes based on test results at Trinity Clear Creek
(after Saliman and Schaefer, 1968)

plex by Littlejohn and Truman-Davies (1974) where 2 000kN anchors were spaced at 1m centres. Another method to dissipate load within the rock mass, is simply to install anchors at different inclinations as in the design by Soletanche (1968) for the Zardesas Dam, Algeria. In some other countries a minimum distance between anchors is stipulated. Broms (1968), reviewing Swedish practice, confirmed a minimum spacing of 2.5m, whilst the Czech Standard (1974) recommends 1.5m, one consideration being to reduce "inter-hole grouting", although this phenomenon is not necessarily a disadvantage in practice.

It is noteworthy that these guide rules or approaches are based on experience and engineering judgement, and not on an intimate knowledge of stress distribution around the anchor.

Remarks

With regard to uplift capacity no experimental or practical evidence and only very little theoretical data substantiate the methods currently used (Table I) to calculate the ultimate resistance to pull-out of individual, or groups of anchors. Indeed, there would appear to be results (Saliman and Schaefer (1968) and Brown (1970)) which indicate that failure in a rock mass does not generally occur in the form of an inverted 90 deg cone or wedge. However, it is reassuring to know that most designs are likely to be conservative in adopting a cone method with no allowance for the shear strength of the rock mass.

Nevertheless, some standardisation on safety factors for temporary and permanent anchors is desirable together with agreement on what allowances should be made for surcharge due to unconsolidated overburden and the effect of upper layers of weathered rock.

In general, effort should now be expended, in the form of field testing in a wide range of rock materials and masses which have been carefully classified, in order to study the shape and position of the rock zones mobilised at failure. Such programmes should accommodate single anchors and groups tested over a range of inclinations. Only in this way can anchor design in relation to overall stability be optimised both technically and economically.

BOND BETWEEN CEMENT GROUT AND ROCK

Introduction

Most designs to date concerning straight shaft fixed anchors have been successfully based on the assumption of uniform bond distribution over the fixed anchor surface area. In other words it has been generally accepted that the bond developed is merely a function of fixed anchor dimensions and applied load.

However, recent experimental and theoretical analyses have indicated that the character of the bond to the rock is more complex, and reflects additional parameters which often give rise to a markedly non-uniform stress distribution. Thus, in many cases the assumed mechanism of load transfer in the fixed anchor zone may be grossly inaccurate. For example, the situation could well arise where, for a high capacity anchor, the level of bond stress at the loaded (or proximal) end may be extremely high, possibly approaching failure, whereas the more distal parts of the fixed anchor may in effect be redundant. Clearly, such a situation will have a bearing on overall stability analyses, the interpretation of anchor extensions, and long-term creep behaviour.

Design criteria are reviewed relating to the magnitude and distribution of bond, fixed anchor dimensions, and factors of safety. For comparison, the results of relevant theoretical and experimental investigations are presented.

Fixed anchor design

The straight shaft anchor relies mainly on the development of bond or shear in the region of the rock/grout interface, and as described by Littlejohn (1972) it is usual in Britain to assume an equivalent uniform distribution of bond stress along the fixed anchor. Thus the anchor load, P, is related to the fixed anchor design by the equation:

$$P = \pi d L \tau \quad \text{.................} (1)$$

where L = fixed anchor length
d = effective anchor diameter
τ = working bond stress

This approach is used in many countries e.g. France (Fargeot, 1972), Italy (Mascardi, 1973), Canada (Coates, 1970), and USA (White, 1973).

The rule is based on the following simple assumptions:

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

Where shear strength tests are carried out on representative samples of the rock mass, the maximum average working bond stress at the rock/grout interface should not exceed the minimum shear strength divided by the relevant safety factor (normally not less than 2). This approach applies primarily to soft rocks where the uniaxial compressive strength (UCS) is less than 7N/mm², and in which the holes have been drilled using a rotary percussive technique. In the absence of shear strength data or field pull-out tests, Littlejohn (1972) states that the ultimate bond stress is often taken as one-tenth of the uniaxial compressive strength of massive rocks (100 per cent core recovery) up to a maximum value τ_{ult} of 4.2N/mm², assuming that the crushing strength of the cement grout is equal to or greater than 42N/mm². Applying an apparent safety factor of 3 or more, which is conservative bearing in mind the lack of relevant data, the work-

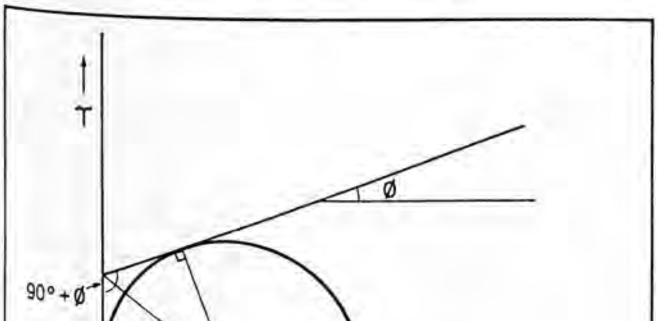


Fig. 4. Relationship between shear stress and uniaxial compressive strength

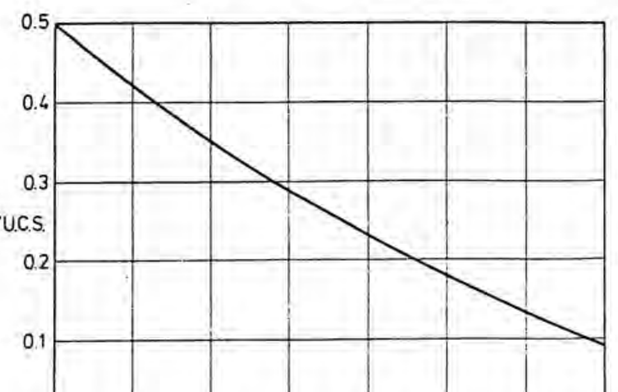


Fig. 5. Effect of ϕ on $\tau_{ult.}/UCS$ ratio

ing bond stress is therefore limited to 1.4N/mm².

In this connection it is noteworthy that Coates (1970) allows a maximum working bond of 2.45N/mm² but with a safety factor of 1.75, which indicates a value of τ_{ult} of 4.3N/mm². In some rocks, particularly granular, weathered varieties with a relatively low ϕ value, the assumption that τ_{ult} equals 10 per cent rock UCS may lead to an artificially low estimate of shear strength (Figs. 4 and 5). In such cases, the assumption that τ_{ult} equals 20-35 per cent UCS may be justified.

As a guide to specialists, bond values, as recommended throughout the world for wide range of igneous, metamorphic and sedimentary rocks, are presented in Table III. Where included, the factor of safety relates to the ultimate and working bond values, calculated assuming uniform bond distribution. It is common to find that the magnitude of bond is simply assessed by experienced engineers; the value adopted for working bond stress often lies in the range 0.35 to 1.4N/mm². Koch (1972) suggests bond stresses in this range for weak, medium and strong rock (Table III), and the Australian Code CA 35—1973 states that a value of 1.05N/mm² has been used in a wide range of igneous and sedimentary rocks, but confirms that site testing has permitted bond values of up to 2.1 N/mm² to be employed.

In this connection the draft Czech Standard (1974) concludes that since the estimation of bond magnitude and distribution is a complex problem, field anchor tests should always be conducted to confirm bond values in design, as there is no efficient or reliable alternative. Certainly, a common procedure amongst anchor designers is to arrive at estimates of permissible working bond values by factoring the value of the average ultimate bond calculated from test anchors, when available. Usually the recommended safety factor ranges from 2 to 3, but is frequently lower in very competent rocks, and higher in weaker, fissured, or weathered varieties.

The degree of weathering of the rock is

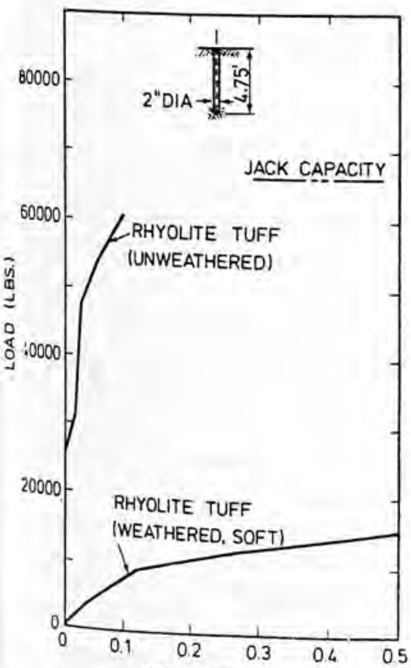


Fig. 6. Effect of weathering at Currecanti Midway Transmission Line

(after Saliman and Schaefer, 1968)

TABLE III—ROCK/GROUT BOND VALUES WHICH HAVE BEEN RECOMMENDED FOR DESIGN

Rock type	*Working bond (N/mm²)*	*Ultimate bond (N/mm²)*	*Factor of safety*	*Source*
Igneous				
Medium hard basalt		5.73	3-4	India—Rao (1964)
Weathered granite		1.50-2.50		Japan—Suzuki et al (1972)
Basalt	1.21-1.38	3.86	2.8-3.2	Britain—Wycliffe-Jones (1974)
Granite	1.38-1.55	4.83	3.1-3.5	Britain—Wycliffe-Jones (1974)
Serpentine	0.45-0.59	1.55	2.6-3.5	Britain—Wycliffe-Jones (1974)
Granite & basalt		1.72-3.10	1.5-2.5	USA—PCI (1974
Metamorphic				
Manhattan schist	0.70	2.80	4.0	USA—White (1973)
Slate & hard shale		0.83-1.38	1.5-2.5	USA—PCI (1974)
Calcareous sediments				
Limestone	1.00	2.83	2.8	Switzerland—Losinger (1966)
Chalk—Grades I-III	0.01N (N=SPT in blows/0.3m)	0.22-1.07	1.5-2.0 (Temporary) 3.0-4.0 (Permanent)	Britain—Littlejohn (1970)
Tertiary limestone	0.83-0.97	2.76	2.9-3.3	Britain—Wycliffe-Jones (1974)
Chalk limestone	0.86-1.00	2.76	2.8-3.2	Britain—Wycliffe-Jones (1974)
Soft limestone		1.03-1.52	1.5-2.5	USA—PCI (1974)
Dolomitic limestone		1.38-2.07	1.5-2.5	USA—PCI (1974)
Arenaceous sediments				
Hard coarse-grained sandstone	2.45		1.75	Canada—Coates (1970)
Weathered sandstone		0.69-0.85	3.0	New Zealand—Irwin (1971)
Well-cemented mudstones		0.69	2.0-2.5	New Zealand—Irwin (1971)
Bunter sandstone	0.40		3.0	Britain—Littlejohn (1973)
Bunter sandstone (UCS > 2.0N/mm²)	0.60		3.0	Britain—Littlejohn (1973)
Hard fine sandstone	0.69-0.83	2.24	2.7-3.3	Britain—Wycliffe-Jones (1974)
Sandstone		0.83-1.73	1.5-2.5	USA—PCI (1974)
Argillaceous sediments				
Keuper marl		0.17-0.25 (0.45 C_u)	3.0	Britain—Littlejohn (1970) C_u=undrained cohesion
Weak shale		0.35		Canada—Golder Brawner (1973)
Soft sandstone & shale	0.10-0.14	0.37	2.7-3.7	Britain—Wycliffe-Jones (1974)
Soft shale		0.21-0.83	1.5-2.5	USA—PCI (1974)
General				
Competent rock (where UCS > 20N/mm²)	Uniaxial compressive strength—30 (up to a maximum value of 1.4N/mm²)	Uniaxial compressive strength—10 up to a maximum value of (4.2N/mm²)	3	Britain—Littlejohn (1972)
Weak rock	0.35-0.70			Australia—Koch (1972)
Medium rock	0.70-1.05			
Strong rock	1.05-1.40			
Wide variety of igneous and metamorphic rocks	1.05		2	Australia—Standard CA35 (1973)
Wide variety of rocks	0.98			France—Fargeot (1972)
	0.50			Switzerland—Walther (1959)
	0.70			Switzerland—Comte (1965)
		1.20-2.50		Switzerland—Comte (1971)
	0.70		2-2.5 (Temporary) 3 (Permanent)	Italy—Mascardi (1973)
	0.69	2.76	4	Canada—Golder Brawner (1973)
	1.4	4.2	3	USA—White (1973)
		15-20 per cent of grout crushing strength	3	Australia—Longworth (1971)
Concrete		1.38-2.76	1.5-2.5	USA—PCI (1974)

TABLE IV—ROCK/GROUT BOND VALUES WHICH HAVE BEEN EMPLOYED IN PRACTICE

Rock type	*Working bond (N/mm²)*	*Test bond (N/mm²)*	*Ultimate bond (N/mm²)*	s_m *(test)*	s_f *(ultimate)*	*Source*
Igneous						
Basalt	1.93		6.37		3.3	Britain—Parker (1958)
Basalt	1.10	3.60				USA—Eberhardt & Veltrop (1965)
Tuff	0.80					France—Cambefort (1966)
Basalt	0.63	0.72				Britain—Cementation (1962)
Granite	1.56	1.72				Britain—Cementation (1962)
Dolerite	1.56	1.72				Britain—Cementation (1962)
Very fissured felsite	1.56	1.72				Britain—Cementation (1962)
Very hard dolerite	1.56	1.72				Britain—Cementation (1962)
Hard granite	1.56	1.72				Britain—Cementation (1962)
Basalt & tuff	1.56	1.72				Britain—Cementation (1962)
Granodiorite	1.09					Britain—Cementation (1962)
Shattered basalt		1.01				USA—Saliman & Schaefer (1968)
Decomposed granite		1.24				USA—Saliman & Schaefer (1968)
Flow breccia		0.93				USA—Saliman & Schaefer (1968)
Mylontised porphyrite	0.32-0.57					Switzerland—Descoeudres (1969)
Fractured diorite	0.95					Switzerland—Descoeudres (1969)
Granite	0.63	0.81				Canada—Barron et al (1971)
Metamorphic						
Schist	0.31					Switzerland—Birkenmaier (1953)
All types	1.20					Finland—Maijala (1966)
Weathered fractured quartzite	1.56	1.72		1.1		Britain—Cementation (1962)
Blue schist	1.52	1.67		1.1		Britain—Cementation (1962)
Weak meta sediments	1.10	1.23		1.1		Britain—Cementation (1962)
Slate	0.43					Britain—Cementation (1962)
Slate/meta greywacke	1.57	1.73		1.1		Britain—Cementation (1962)
Granite gneiss	0.36-0.69					Sweden—Broms (1968)
Folded quartzite	0.51					Australia—Rawlings (1968)
Weathered meta tuff		0.29				USA—Saliman & Schaefer (1968)
Greywacke	0.34					Germany—Heitfeld & Schaurte (1969)
Quartzite	0.93-1.20	1.02-1.32		1.1		Britain—Gosschalk & Taylor (1970)
Microgneiss	0.95					Italy—Mantovani (1970)

Rock	Working bond (N/mm^2)	Test bond (N/mm^2)	Ultimate bond (N/mm^2)	s_m (test)	s_f (ultimate)	Source
Metamorphic—contd.						
Sericite schist	0.05					Italy—Berardi (1972)
Quartzite/schist	0.10					Italy—Berardi (1972)
Argillaceous & calcareous schist	0.63					Italy—Berardi (1972)
Slate	0.95	1.24		1.3		Switzerland—Moschler & Matt (1972)
Highly metasediments	0.83	1.08		1.3		USA—Buro (1972)
Slate & greywacke	1.08	1.40		1.3		Germany—Anon (1972)
Various metasediments			1.57			Germany—Abraham & Porzig (1973)
Micaschist/biotite gneiss	0.53	0.80		1.5		USA—Nicholson Anchorage Co. Ltd. (1973)
Slate	0.60	0.90	1.80	1.5	3.0	Britain—Littlejohn & Truman-Davies (1974)
Sound Micaschist	1.74	2.16				USA—Feld & White (1974)
Micaschist	0.52-0.74			1.24		USA—Feld & White (1974)
Very-poor gneiss & mud band	0.07					USA—Feld & White (1974)
Carbonate sediments						
Loamy limestone		0.63				Italy—Berardi (1960)
Fissured limestone & intercalations	1.08	1.19		1.1		Britain—Cementation Co. Ltd. (1962)
Limestone	0.65					Switzerland—Muller (1966)
Poor limestone	0.32					France—Hennequin & Cambefort (1966)
Massive limestone	0.39-0.78					France—Hennequin & Cambefort (1966)
Karstic limestone	0.54					France—Hennequin & Cambefort (1966)
Tertiary limestone	1.00		2.83		2.8	Switzerland—Losinger & Co. Ltd (1966)
Limestone			4.55-4.80			Switzerland—Ruttner (1966)
Marly limestone	0.03-0.07 (average) 0.21-0.36 (measured)					Italy—Berardi (1967)
Limestone			0.27			USA—Saliman & Schaefer (1968)
Limestone	0.28					Italy—Berardi (1969)
Dolomitic limestone			1.80			Canada—Brown (1970)
Marly limestone	0.39-0.94					Italy—Berardi (1972)
Limestone	0.26					Italy—Berardi (1972)
Limestone/puddingstone	0.44					France—Soletanche Co Ltd (1968)
Limestone	1.18	1.42		1.2		USA—Buro (1972)
Chalk			0.70			Britain—Associated Tunnelling Co Ltd. (1973)
Dolomite		1.66				Canada—Golder Brawner (1973)
Dolomitic siltstone	0.43					USA—White (1973)
Limestone & marly bands	0.37	0.55		1.5		Italy—Mongilardi (1972)
Arenaceous sediments						
Sandstone	1.44	1.58		1.1		Britain—Morris & Garrett (1956)
Hard sandstone	1.42	1.56		1.1		Britain—Cementation Co. Ltd. (1962)
Bunter sandstone	0.95	0.98		1.03		Britain—Cementation Co. Ltd. (1962)
Sandstone	0.76	0.84		1.1		Britain—Cementation Co. Ltd. (1962)
Sandstone	0.74					Czechoslovakia—Hobst (1968)
Sandstone	0.31	0.40	1.73	1.29	5.6	USA—Drossel (1970)
Sandstone	0.80					USA—Thompson (1970)
Poor sandstone	0.40					Germany—Brunner (1970)
Good sandstone	1.14					Germany—Brunner (1970)
Sandstone & Breccia	0.38					France—Soletanche (1968)
Sandstone		0.95				Australia—Williams et al (1972)
Bunter sandstone	0.60	1.20		2.0		Britain—Littlejohn (1973)
Sandstone	1.17					Australia—McLeod & Hoadley (1974)
Argillaceous sediments						
Shale	0.62					Canada—Juergens (1965)
Marl	0.10	0.28		2.8		Italy—Berardi (1967)
Shale	0.30		0.63		2.1	Canada—Hanna & Seaton (1967)
Very weathered shale			0.39			USA—Saliman & Schaefer (1968)
Shale	0.13-0.24					USA—Koziakin (1970)
Grey siltstone	0.62					Britain—Universal Anchorage Co. Ltd. (1972)
Clay marl	0.14-0.24	0.21-0.36		1.5		Germany—Schwarz (1972)
Shale	0.62					Canada—McRosite et al (1972)
Argillite	0.82					Canada—Golder Brawner (1973)
Mudstone	0.63	0.88		1.4		Australia—McLeod & Hoadley (1974)
Miscellaneous						
Bedded sandstone & shale	0.25-0.50					Italy—Beomonte (1961)
Porous, sound goassamer	1.57	1.72		1.1		Britain—Cementation Co. Ltd. (1962)
Shale & sandstone	0.07	0.10		1.5		USA—Reti (1964)
Soft rocks	0.75					Sweden—Nordin (1966)
Sandstone & shale	1.82					Poland—Bujak et al (1967)
Siltstone & mudstone	1.65					Australia—Maddox et al (1967)
Fractured rock (75 per cent shale)						
Poor			0.24			USA—Saliman & Schaefer (1968)
Average			0.35			USA—Saliman & Schaefer (1968)
Good			0.75			USA—Saliman & Schaefer (1968)
Limestone & caly breccia	0.20-0.23					Italy—Berardi (1972)

a major factor which affects not only the ultimate bond but also the load-deflection characteristics. Fig. 6 shows the results produced by test anchors in rhyolite tuff, of both sound and weathered varieties. No data are provided on grout or rock strengths but it is significant that the equivalent uniform bond stress at maximum jack capacity is scarcely $0.1N/mm^2$. For design in soft or weathered rocks there are signs that the standard penetration test is being further exploited. For example, Suzuki *et al* (1972) state that for weathered granite, the magnitude of the bond can be determined from the equation:

$$\tau_{ult} = 0.007N + 0.12 \ (N/mm^2)$$
where N = number of blows per 0.3m ....(2)

Similarly, Littlejohn (1970) illustrates a correlation between N and ultimate bond for stiff/hard chalk, as follows:

$$\tau_{ult} = 0.01N \ (N/mm^2) \ ....(3)$$

In grades III, II and I of chalk, he observed a range of τ_{ult} of $0.21-1.07N/mm^2$ based on test anchors pulled to failure.

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

Table IV contains data abstracted from reports of rock anchor contracts throughout the world. In addition to the working, test, and ultimate bond values, the measured and designed safety factors are provided where available. In certain cases, the fixed anchor diameter has been inferred, to facilitate analysis of the data, as published.

It will be noted that, even for one rock type, the magnitude of bond used in practice is extremely variable. There are many reasons for this, the most important of which are:

(*i*) Different designers use different bond values and safety factors, which may be related to type of anchor and extent of the anchor testing programme.

(*ii*) "Standard" values for a certain rock type have often been modified to reflect local peculiarities or irregularities of the geology.

(*iii*) Factors related to the construction techniques e.g. drilling method, flushing procedure, and grout pressure will influence the results obtained. (The effect of these aspects will be discussed in Part 2—Construction.)

On the whole however, it would appear that the bond values employed are to a degree consistent with rock type and competency.

Fixed anchor dimensions

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

With regard to the choice of anchor diameter several considerations may be taken into account:

(*i*) Type and size of tendon,

(*ii*) The relation of diameter to perimeter area of fixed anchor and hence the anchor capacity, assuming uniform bond,

(*iii*) Ratio of steel area to cross-sectional area of borehole for efficient bond distribution and corrosion protection,

(*iv*) Drilling method and rig to be used, and

(*v*) Nature of rock in the anchor zone and presence of unconsolidated overburden, if any.

The authors find from a survey of several hundred commercial anchor reports that no direct relationship may be observed bearing in mind the range of anchor types, but that most anchors conform to the trend indicated in Table V1.

TABLE V—FIXED ANCHOR LENGTHS FOR CEMENT GROUTED ROCK ANCHORS WHICH HAVE BEEN EMPLOYED OR RECOMMENDED IN PRACTICE

Fixed anchor length (metres)		Source
Minimum	Range	
3.0		Sweden—Nordin (1966)
3.0		Italy—Berardi (1967)
	4.0- 6.5	Canada—Hanna & Seaton (1967)
3.0	3.0-10.0	Britain—Littlejohn (1972)
	3.0-10.0	France—Fenoux et al (1972)
	3.0- 8.0	Italy—Conti (1972)
4.0 (very hard rock)		South Africa—Code of Practice (1972)
6.0 (soft rock)		South Africa—Code of Practice (1972)
5.0		France—Bureau Securitas (1972)
5.0		USA—White (1973)
3.0	3.0- 6.0	Germany—Stocker (1973)
3.0		Italy—Mascardi (1973)
3.0		Britain—Universal Anchorage Co. Ltd. (1972)
3.0		Britain—Ground Anchors Ltd. (1974)
3.5 (chalk)		Britain—Associated Tunnelling Co. Ltd. (1973)

TABLE VI—APPROXIMATE RELATIONSHIP BETWEEN FIXED ANCHOR DIAMETER AND WORKING CAPACITY

Capacity, kN	Diameter, mm
200— 1 200	50—100
1 000— 3 000	90—150
3 000— 4 500	150—200
4 500—14 000	200—400

The third and fourth considerations will be dealt with in Part 2—Construction, but it is noteworthy that where corrosion protection is important, the South African Code (1972) stipulates that the fixed anchor diameter should be equal to the outside diameter of the tendon plus at least 12mm. This approach has also been discussed by FIP (1972) who recommend a grout cover to the tendon of 5mm, and 5-10mm for temporary and permanent rock anchors, respectively.

With regard to the amount of steel which should be placed in an anchor borehole there is a scarcity of information although Littlejohn and Truman-Davies (1974) suggest that the steel should not exceed 15 per cent of the borehole cross-sectional area.

Theoretical evidence

Studies of the stress distribution around a cylindrical anchorage in a triaxial stress field have been carried out by Coates and Yu (1970), using a finite element method. Figs. 7a and 7b show the typical anchor geometry and the model employed to calculate approximately the stress induced by an anchor loaded either in tension or compression. The authors show that the shear stress (i.e. bond) distribution, is dependent on the ratio of the elastic moduli of the anchor material (E_a) and the rock (E_r). Fig. 8 shows the variation of the shear stress along the interface of an anchor of length equal to six times its radius for E_a/E_r ratios of 0.1, 1 and 10. The smaller this ratio the larger is the stress calculated at the proximal end of the anchor; higher values of the ratio are associated with more even stress distributions. It is also apparent that for $E_a/E_r > 10$, i.e. for very soft rocks, it is reasonable to assume that the bond is evenly distributed along the anchor, and that the anchor design may be based accurately and directly on the shear strength of the weaker medium.

For anchors subjected to tensile loading the shear stresses in turn induce tensile stresses in the rock, which reach a maximum value at the proximal end of the anchorage. Fig. 9 illustrates the rapid dissipation of the tensile stresses radially at the distal end of the fixed anchor. For a 1 500kN capacity anchor in a 75mm diameter hole, the maximum tensile stress is estimated to be about 145N/mm² at the proximal end of the fixed anchor in rock, whilst at the opposite end, this stress is 48N/mm², provided, of course, that the rock can sustain these stresses. It seems probable that cracking will occur, and the magnitude of the maximum tensile stress decrease, as it transmits radially outwards, reaching an equilibrium position if the rock remains in position. The propagation of such cracks due to large tensile stresses acting parallel to the anchor axis possibly accounts in part for the anchor creep frequently observed to occur for a period of time after stressing. Deformation measurements adjacent to such anchors would provide useful information in this respect.

With regard to the magnitude of E_a, Phillips (1970) quotes a value of 2.1×10^4 N/mm² for a neat grout of water/cement ratio 0.4 and Boyne (1972), using a 0.35 water/cement ratio expansion grout, obtained a value of 1.0×10^4N/mm². Therefore, before the uniform bond distribution can be assumed, the rock must have an elastic modulus in the range $0.1\text{-}0.2 \times 10^4$ N/mm². Using a statistical relationship derived by Judd and Huber (1961), which relates rock compressive strength to elastic modulus:

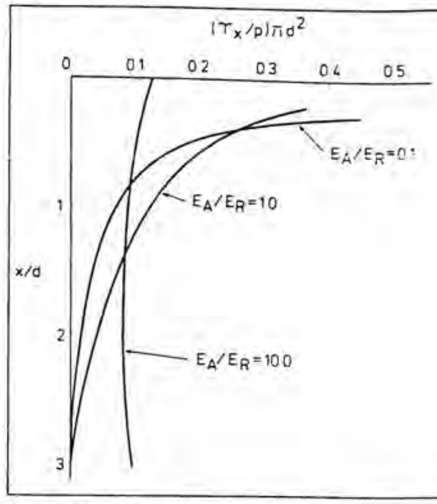


Fig. 8. Variation of shear stress with depth along the rock/grout interface of an anchor (after Coates and Yu, 1970)

$$\text{UCS} = \frac{E}{350} \quad \ldots\ldots\ldots\ldots\ldots (4)$$

Phillips estimates therefore that the compressive strength of the rock in this case should be significantly less than 6N/mm².

However, the majority of rock anchors to date have been installed in rocks giving values for the ratio E_a/E_r of between 0.1 and 1, and for which, according to Fig. 8, the bond distribution is markedly non-uniform. Indeed, for anchors in these rocks of compressive strength in excess of 6N/mm², stress concentrations at the proximal end are most likely, having a magnitude possibly 5-10 times the average stress level.

Although less satisfactory from a theoretical point of view, anchors in strong rocks at present represent less of a problem in practice, since a large safety factor can be accommodated without significantly increasing the cost. However, for the accurate design of high capacity anchors, insufficient attention has been paid to the high stresses at the proximal end, and in particular to the effect of debonding on

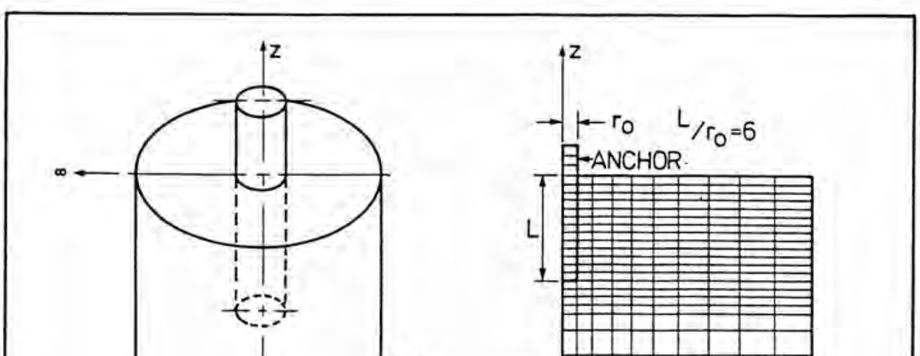


Fig. 7. The geometry of the rock anchor studies: (a) definition of axes; (b) finite element model (after Coates and Yu, 1970)

stress distribution. In this context Phillips (1970) suggests three possible approaches:

1. Following debonding, the restraint imposed by the rock on the uneven rock-grout interface causes dilation. Additional anchorage movement is only possible through further shear failure of the grout, giving a possible stress distribution as shown in Figs. 10a and 10b.
2. The residual bond stress, when considered alone, and ignoring dilation, will depend on the magnitude of "ground pressure" acting normal to the interface. This will probably vary over the debonded length and it may be less than the grout shear strength (Fig. 10c). If it is greater than the grout shear strength, the stress distribution will revert to that of Figs. 10a and 10b.
3. It is probable that the stress distribution will vary with applied load possibly as shown in Figs. 10d, 10e and 10f. This presumes an initial stress distribution similar to the theoretical stress distribution (Fig. 10b). At large loads, virtually the whole of the anchor is debonded and the stress is distributed according to the amount of relative movement and the degree of dilation or frictional shear strength mobilised (see Fig. 10f).

It should be emphasised however that these approaches are hypothetical and experimental work is required to confirm their validity in relation to rock anchor design.

Experimental evidence

In Italy much valuable experimental research has been conducted, principally by Berardi, into the distribution of stresses both along the fixed anchor and into the rock. In 1967 he reported on tests to determine the distribution of fixed anchor stresses and concluded that the active portion of the anchor is independent of the total fixed anchor length, but dependent on its diameter and the mechanical properties of the surrounding rock, especially its modulus of elasticity.

Figs. 11a and 11b are typical diagrams which illustrate the uneven bond distribution as calculated from strain gauge data. Both anchors were installed in 120mm diameter boreholes in marly limestone ($E = 3 \times 10^4$kN/m^2; UCS = 100N/mm^2 approximately). Other results show that the bond distributions are more uniform for high values of E_{grout}/E_{rock}, non-uniform for low values of this ratio i.e. for rock of high elastic modulus, thus confirming the predictions of Coates and Yu.

Muller (1966) produced interesting results in Switzerland on the distribution of shear stress along the 8m fixed anchor of a 220 tonne BBRV anchor (Fig. 12). From readings obtained during stressing, he concluded that the load was not uniformly distributed to the rock over the length of the fixed anchor. For example, at a load of 55 tonnes the force was transmitted uniformly over the proximal 5.55m, implying an average bond of 0.22N/mm^2. At 185 tonnes however, load was recorded over the lower 4.1m of the tendon with apparent debonding of the tendon over the upper 3.9m. About 30 tonnes was resisted by the bottom of the anchor, but between points A and C, (Fig. 12) the average bond stress was about 0.98N/mm^2. At 280 tonnes, a comparison of theoretical and measured anchor elongations suggested that total debonding of the tendon had occurred, and that all the load was resisted by the foot of the fixed anchor. The values

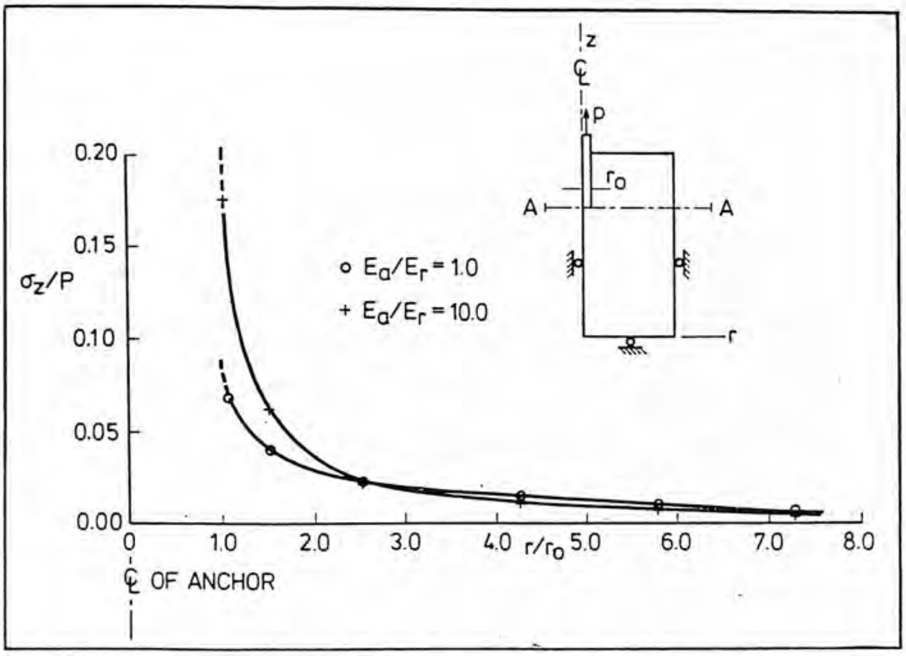


Fig. 9. Variation of tensile stress in the rock adjacent to the end of a tension anchor
(after Coates and Yu, 1970)

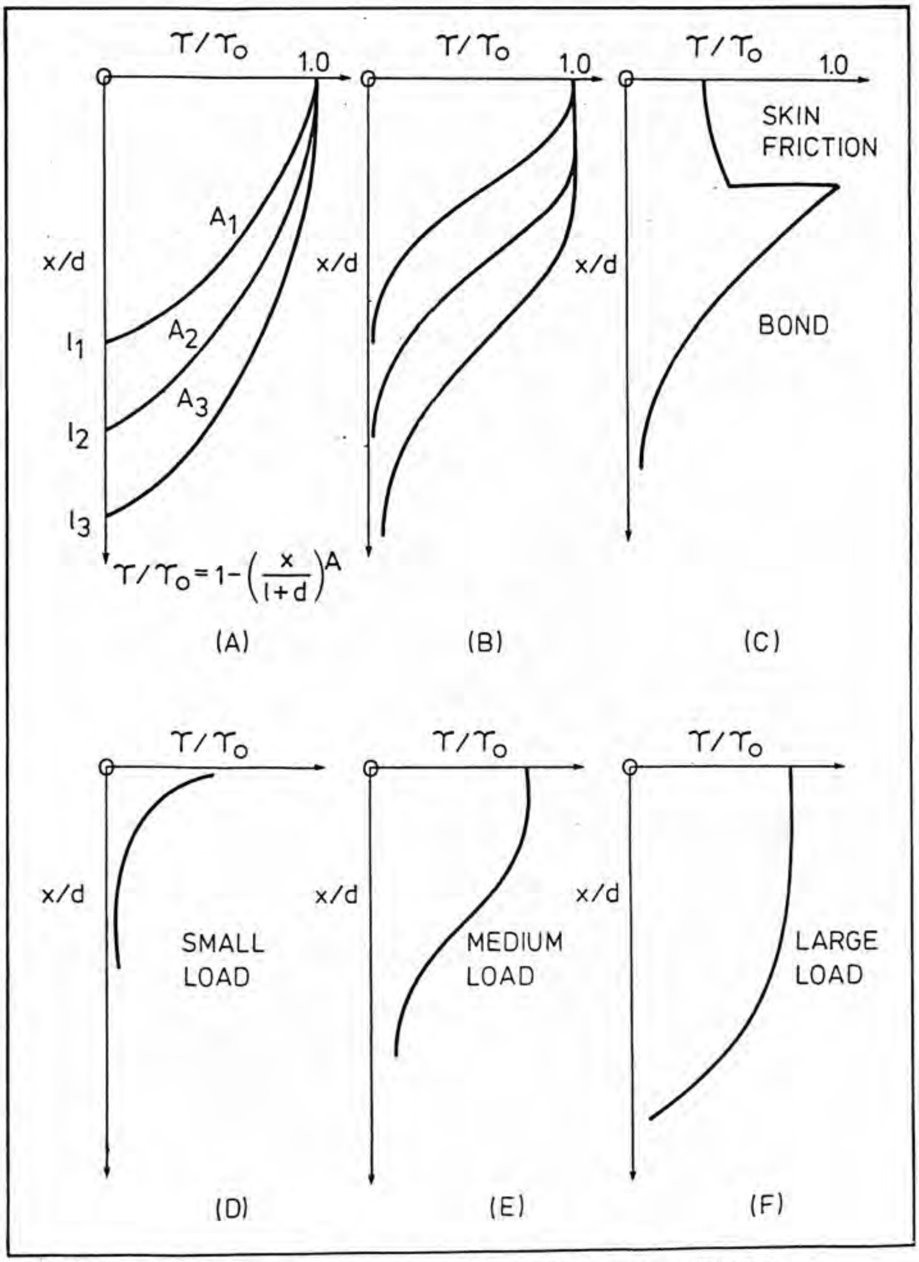


Fig. 10. Hypothetical stress distributions around a partly debonded anchorage
(after Phillips, 1970)

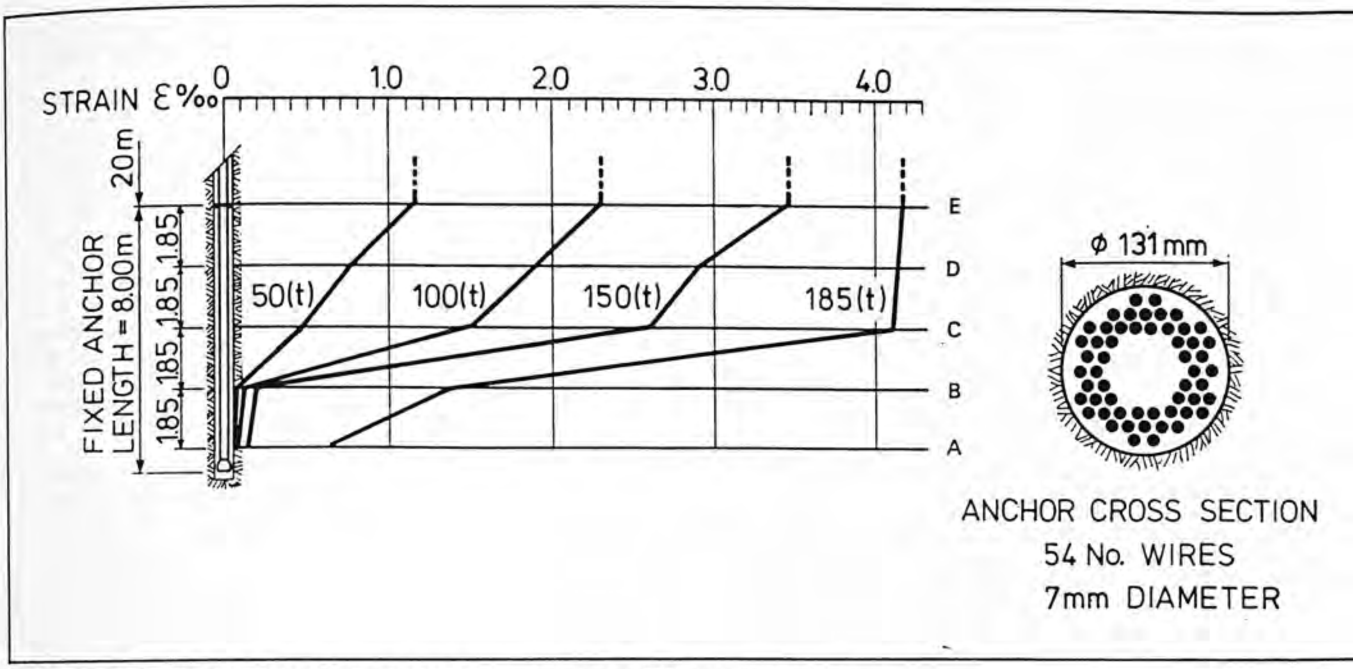


Fig. 12. Strain distribution along tendon in fixed anchor zone of a 220t capacity anchor

(after Muller, 1966

for bond strength quoted above compare with an average value, based on uniform distribution, of about 0.65N/mm², which is well below both the actual value at 185 tonnes and the grout shear strength.

Decoupling, equivalent to an addition in free length of 2.2m, has also been reported by Eberhardt and Veltrop (1965), during the stressing of a 1 300t capacity test anchor installed in basalt (fixed anchor length = 11.5m, diameter = 406mm).

Remarks

From mathematical, laboratory and field evidence, the distribution of the bond mobilised at the rock/grout interface is unlikely to be uniform unless the rock is "soft". It appears that non-uniformity applies to most rocks where E_{grout}/E_{rock} is less than 10.

In the case of high capacity anchors evidence exists that partial debonding in the fixed anchor occurs, and that debonding progresses towards the end of the anchor as the load is increased. Information is scarce concerning the conditions where debonding is serious.

Since the validity of the uniform distribution of bond which is commonly assumed by designers is clearly in question, it is recommended that instrumented anchors should be pulled to failure in a wide range of rock masses whose engineering and geological properties can be fully classified, in order to ascertain which parameters dictate anchor performance. In this way it should be possible in due course to provide more reliable design criteria.

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

The prior knowledge of certain geological and geotechnical data pertaining to the rock is essential for the safe, economic design of the anchor and correct choice of construction method. The authors believe that the following geotechnical properties should be evaluated during the site investigation stage, in addition to the conventional descriptions of lithology and petrography: quantitative data on the nature orientation, frequency and roughness of th major rock mass discontinuities; shea strength of these discontinuities; and com pressive and shear strength of the roc material.

Also, particularly in the softer rocks weatherability and durability should be as sessed, especially on samples drawn fron the level of prospective fixed anchor zones It is realised that the determination of th modulus of elasticity is rather involved an expensive, particularly for rock masses However, as the influence of this para meter on anchor performance has alread been demonstrated, efforts should be mad whenever possible, to obtain a realisti value.

The ground water regime is also of prim importance, especially the position of th water-table, and the groundwater rate c flow, pressures and aggressivity. It shoul be noted that the ratio of anchor length t discontinuity spacing determines the rela tive importance of intact material an rock mass properties in any one case. Fo example, where the fracture spacing i relatively large, the rock material propertie will be the dominant controls of, for ex ample, drillability and rock/grout bonc However, this is rarely the case, and th properties of the rock mass are usuall crucial, particularly in the assessment c the overall stability of the anchor system.

The extent of the site investigatio should be determined by the importance c the contract, and the potential difficultie and risks inherent in its execution. In sit anchor tests should be carried out whereve possible to clarify design proposals.

Bearing in mind that anchors are ofte installed at very close centres it would ap pear in site investigation that a "construc tion" stage is required where drill logs penetration rates, grout consumptions an check pull-out tests are monitored i order to highlight "difficult" or change rock conditions. These terms need to b defined in order to avoid legal problem and the question is important wheneve doubt about anchor competence exists.

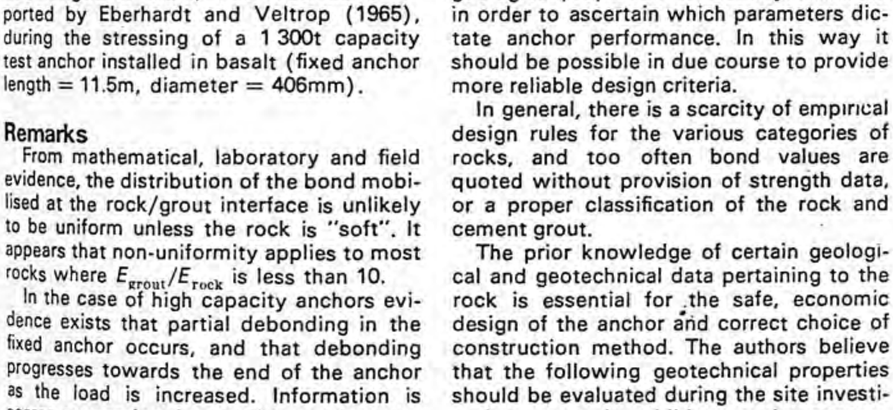


Fig. 11. Distribution of bond along fixed anchor length

(after Berardi, 1967)

BOND BETWEEN CEMENT GROUT AND STEEL TENDON

Introduction

Little attention has been paid to this aspect of rock anchor design, principally because engineers usually consider that the fixed anchor length chosen with respect to the rock/grout bond ensures more than adequate tendon embedment length.

However, as has been demonstrated in the section dealing with rock/grout bond§, little standardisation or uniformity of approach is apparent related to the grout/tendon bond, and the rather simple design assumptions commonly made are in contradiction to certain experimental observations.

In this section, the mechanisms of bond are discussed and anchor design procedures employed in practice are reviewed. Bearing in mind the scarcity of information pertaining to anchors, data abstracted largely from the fields of reinforced and prestressed concrete are also presented, which relate to the magnitude and distribution of bond.

The mechanisms of bond

It is widely accepted that there are three mechanisms:

1. *Adhesion.* This provides the initial "bond" before slip, and arises mainly from the physical interlocking (i.e. gluing) of the microscopically rough steel and the surrounding grout (Fig. 13). Molecular attrac-

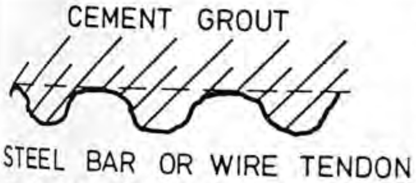


Fig. 13. Magnified view of interface between grout and steel

tion is also thought to act. Adhesion is considered to disappear when slip comparable with the size of the micro indentations on the steel occurs.

2. *Friction.* This component depends on the confining pressure, the surface characteristics of the steel, and the amount of slip, but is largely independent of the magnitude of the tendon stress. The phenomena of dilatancy and wedge action also contribute to this frictional resistance as radial strains are mobilised where the longitudinal strain changes.

3. *Mechanical interlock.* This is similar to micro mechanical locking, but on a much larger scale, as the shear strength of the grout is mobilised against major tendon irregularities, e.g. ribs, twists.

An idealised representation of these three major bond components is shown in Fig. 14. For short embedment lengths the adhesive component is most important, but

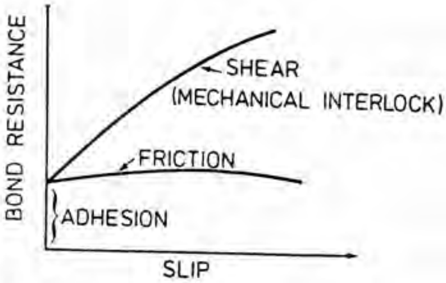


Fig. 14. Idealised representation of major components of bond

for longer lengths, all three may operate—adhesion failure occurring initially at the proximal end and then moving progressively distally to be replaced by friction and/or mechanical interlock. Frictional and interlocking resistances increase with lateral compression and decrease with lateral tension. Clearly, the grout shear strength and the nature of the tendon surface, both micro-and macroscopically, are major factors in determining bond characteristics.

Fixed anchor design

It is common in practice to find embedment lengths for bars, wires and strands quoted as equivalent to a certain number of diameters, as this method ensures a maximum value of apparent average bond stress for each type of tendon. The transmission length is the length required to transmit the initial prestressing force in a tendon to the surrounding grout or concrete.

In Britain, the following general recommendations may be followed, based on CP 110, 1972 and information supplied by Bridon Wire Ltd (1968).

Wire

(*i*) For a bright or rusted, plain or indented wire with a small off-set crimp e.g. 0.3mm off-set, 40mm pitch, a transmission length of 100 diameters may be assumed when the cube strength of the concrete or grout at transfer is not less than 35N/mm².

(*ii*) For a wire of a considerable crimp e.g. 1.0mm off-set, 40mm pitch, a bond length of 65 diameters may be assumed for the above conditions.

(*iii*) Galvanised wire provides a poor bond, less than half that of comparable plain wire.

(*iv*) It may be assumed that 80 per cent of the maximum stress is developed in a length of 70 diameters for the conditions mentioned in (*i*) and in a length of 54 diameters for the conditions mentioned in (*ii*).

Strand

(*i*) From the available experimental data, the transmission length for small diameter ordinary strand is not proportional to the diameter of the tendon. Table VII gives values of transmission length for strand working at an initial stress of 70 per cent ultimate in concrete of strength 34.5-48.3 N/mm².

TABLE VII—TRANSMISSION LENGTHS FOR SMALL DIAMETER STRAND

Diameter of strand (mm)	*Transmission length (mm)*	*(diameters)*
9.3	200 (±25)	19-24
12.5	330 (±25)	25-28
18.0	500 (±50)	25-31

N.B.—Range of results given in brackets.

(*ii*) Tests in concrete of strength 41.4-48.3N/mm² with Dyform compact strand at 70 per cent ultimate show an average transmission length of 30-36 diameters.

According to the results of an FIP questionnaire (1974) national specifications vary considerably for transmission lengths, the most optimistic being those of the United Kingdom. It is accepted that compact strand e.g. Dyform, has transmission lengths 25 per cent greater than those for normal 7-wire strand, and that sudden release of load also increases the transmission length. (An additional 25 per cent is recommended in Rumania).

Bar

(*i*) With regard to permissible bond stresses for single plain and deformed bars in concrete, Table VIII illustrates the values stipulated by the British Code for different grades of concrete. These values are applied to neat cement grouts on occasions.

(*ii*) For a group of bars, the effective perimeter of the individual bars is multiplied by the reduction factors below

No. of bars in group	*Reduction factor*
2	0.8
3	0.6
4	0.4

It is important to note that no information is provided in the Code on group geometry e.g. minimum spacing, where the reduction factors should be employed. In addition no guidance of any kind is given for groups of strands or wires.

With reference to minimum embedment lengths, Morris and Garret (1956) have calculated from stressing tests on 5mm diameter wires that the minimum necessary embedment is just over 1m. Golder Brawner Assocs. (1973) found that although the grout/strand bond is higher than expected from tests on single wires due to "spiral interlock", the value drops rapidly if the embedment length is less than 0.6m. Results from Freyssinet anchors with spacers have shown that each strand can withstand about 156-178kN with 0.6m embedment. Since the capacity of such strand is usually in the range 178-270kN, Golder Brawner Assocs. conclude that no strand of a rock anchor logically needs an embedment length in excess of 1.5m. However, for other reasons, a length of 3m is usually considered the minimum acceptable.

Data abstracted from papers describing rock anchor contracts is presented in

TABLE VIII—ULTIMATE ANCHORAGE BOND STRESSES

Type of bar	Characteristic strength of concrete (f_{cu}, N/mm²) 20	25	30	40+
	Maximum bond stress, N/mm²			
Plain	1.2	1.4	1.5	1.9
Deformed	1.7	1.9	2.2	2.6

Tables IX, X and XI for bar, wire and strand, respectively. In all the calculations, except where otherwise noted, the bond is assumed uniform over the whole tendon embedment zone, which is taken as equal to the length of the fixed anchor.

Bearing in mind the relatively small number of values, comments are limited to the following:

(*i*) There would appear to be a greater degree of uniformity on values chosen for the working bond between strand and grout, than for the bond developed by bars and wires with grout. The value of the bond (up to 0.88N/mm²) for 15.2mm strand is slightly higher overall than that for 12.7mm strand (up to 0.72N/mm²), and in both cases there is a trend towards a reduction of the bond with an increase in number of strands.

(*ii*) The actual safety factor against failure of the grout/tendon bond is usually well in excess of 2.

(*iii*) The average bond developed by bars, especially deformed types, is on average significantly higher than that developed by strands or wires. Also the presence of deformities increases the bond magnitude by up to 2 times with respect to plain bars.

Distribution of bond

Much of the work to investigate the distribution of bond along grout/steel interfaces has been carried out in the United States in connection with prestressed and reinforced concrete. Gilkey, Chamberlin

TABLE IX—GROUT/BAR BOND VALUES WHICH HAVE BEEN EMPLOYED OR RECOMMENDED IN PRACTICE

Bar tendon	Embedment (m)	Load (kN)	Working load (N/mm²)	Test bond (N/mm²)	Ultimate bond (N/mm²)	Remarks	Source
Plain			1.2-1.9			Design criteria: bond dependent	Britain—CP110(1972)
Deformed			1.7-2.6			on concrete	
Square twist			5.25			Design criteria	Britain—Roberts (1970)
Ribbed			7.0				
Plain					1.38	Short embedment test	Canada—Brown (1970)
Plain and threaded end					2.62	Bond dependent on embedment and grout tensile stress	
Deformed bar	30d					Design criteria: "solid" rock	Canada—Ontario Hydro (1972)
Deformed bar	40d+					Design criteria: "seamy" rock	
20 No. 20mm dia plain	2.5	1750		0.56		Test anchor	Italy—Berardi (1960)
20mm dia ribbed and threaded	2.2			1.1		Test anchor	Italy—Beomonte (1961)
with end nut	3.9			0.6		Test anchor	
	2.2			1.2		Test anchor	
	2.2			0.9		Test anchor	
25mm dia deformed bar	0.2				2.7	Test anchor	Canada—Brown (1970)
	0.4				5.0	Test anchor, Bond for deformed bar=5 x bond for plain bar	
	0.6						
25.4mm dia square	1.83	289		1.98		Test anchor	USA—Salisman & Schaefer (1968)
25.4mm dia plain	0.06	52			10.1	Tests: for each pair the first	Australia—Pender et al (1963)
	0.06	53.5			11.2	test conducted at 28 days.	
	0.12	52			5.5	and the second at 90 days	
	0.12	67			7.0		
	0.18	63			4.4		
	0.18	117			8.1		
	0.36	139			4.9		
	0.36	148			5.1		
28mm dia plain	5.9	160		0.31		Test anchor: bond known to be	Italy—Berardi (1967)
	11.0	160		0.16		much higher locally	
28 mm dia plain		400	0.76			Design criteria	Switzerland—Comte (1971)
28.6mm dia plain	0.91	220		2.72		Test anchor	USA—Drossel (1970)
30mm dia plain		320	3.0	7.2		Test at bar UTS	Switzerland—Muller(1966)
31.8mm dia high tensile	1.83	605	3.3			Commercial anchor	USA—Wosser et al (1970)
31.8mm dia and thread	1.2	700		5.74		Test anchor	USA—Drossel (1970)
31.8mm dia Dywidag & locknut	8.5	545	0.64			Commercial anchor	USA—Oosterbaan et al (1972)
35mm dia mild steel	6.1	360	0.54			Anchor pile	Canada—Jaspar et al (1969)
35mm dia plain	6.1	505		0.75		Test anchor	Canada—Barron et al (1971)
35mm dia plain	6	700	1.06			Commercial anchor	USA—Feld et al (1974)
43mm dia mild steel	12.2	610	0.37			Anchor pile	Canada—Jaspar et al (1969)
44mm dia plain	0.35				4.7	Test anchor	Canada—Brown (1970)
	0.6						

TABLE X—GROUT/WIRE BOND VALUES WHICH HAVE BEEN EMPLOYED OR RECOMMENDED IN PRACTICE

Wire tendon	Embedment (m)	Load (kN)	Working bond (N/mm²)	Test bond (N/mm²)	Ultimate bond (N/mm²)	Remarks	Source
Plain	100d					Design criteria	Britain—CP110 (1972) also CP115 (1969)
Crimped	65d						
Groups of 5, 6, 7mm	-		0.5-1.0			Design criteria	Switzerland—Comte (1971)
e.g. from 14 No. 5mm	1.7-3.4	350					
to 54 No. 7mm	6-12	2470					
Plain/Crimped			1.035			Design criteria	Australia—Standard CA 35 (1973)
12 No. 5mm	1.8	280	0.53			Commercial anchor	Switzerland—Pliskin (1965)
24 No. 5mm	4.0	500	0.33			Commercial anchor	Poland—Bujak et al (1967)
37 No. 5mm	1.0	1540			2.62	Test anchor	Britain—Morris et al (1956)
	2.44	700	0.49			Commercial anchor	
	2.44	950	0.67			Commercial anchor	
40 No. 5mm	5.0	850	0.27			Commercial anchor	Switzerland—Birkenmaier (1953)
102 No. 5mm	4.0	2000	0.32			Commercial anchor	India—Rao (1964)
24 No. 6.4mm	18.3	670	0.076			Commercial anchor	USA—Reti 1964)
90 No. 6.4mm HT	9.14	3750	0.23			Commercial anchor	USA—Thompson (1970)
90 No. 6.4mm	9.14	11570	0.70			Commercial anchor	Canada—Golder Brawner (1973)
12 No. 7mm	2.5	515	0.78			Commercial anchor	Switzerland—Pliskin (1965)
12 No. 7mm	7.5	450	0.23			Commercial anchor	Brazil—da Costa Nunes (1969)
12 No. 7mm	2.5	600	0.93			Commercial anchor	Africa—Anon (1970)
12 No. 7mm	4.0	500	0.47			Commercial anchor	Italy—Berardi (1972)
34 No. 7mm	7.6	1300	0.23			Commercial anchor	Australia—Rawlings (1968)
35 No. 7mm	3.5	1380	0.51			Commercial anchor	Switzerland—Ruttner (1966)
	0.6				2.26	Test anchor	
	1.5				2.0	Test anchor	
72 No. 7mm galvanised HT	5.2	2740	0.33			Commercial anchor	Australia—Maddox et al (1967)
33 No. 7.62mm	3.5	1700	0.62			Commercial anchor	Germany—Anon (1972)
12 No. 8mm	2.9	675	0.77			Commercial anchor	Switzerland—Pliskin (1965)
12 No. 8mm					0.47	Test anchor	Britain—Bundred (1973)
18 No. 8mm		1450		0.7		Test anchor:	Switzerland—Walther (1959)
27 No. 8mm		2250		0.7		at wire UTS	
24 No. 8mm	4.5	1725		0.46		Test anchor	Switzerland—Möschler et al (1972)
33 No. 8mm	3	1255		0.69		Test anchor	
4 No. 15.2mm	6.5	500	0.47			Commercial anchor	Italy—Berardi (1972)
6 No. 15.2mm	8.4	870	0.36			Commercial anchor	
9 No. 13-16mm	3.5	700	0.54			Commercial anchor	Germany—Anon (1972)

and Beal (1940) discuss in general terms the bond characteristics of bars during pull-out. As the load increases progressive slip at the proximal end occurs, and the location of the maximum intensity of bond stresses moves towards the distal end. The total resistance continues to increase primarily because the length of the tendon which has passed its maximum resistance does not release entirely but exerts a residual resistance or drag acting concurrently with the adhesive bond in the region of maximum bond stress. Fig. 15 is an ideal-

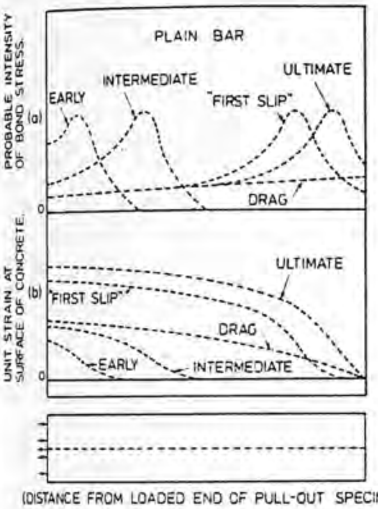


Fig. 15. Qualitative variation of (a) bond stress, (b) total tensile stress, during a pull-out test
(after Gilkey, Chamberlain & Beal, 1940)

ised diagram showing the progressive nature of bond distribution at successive stages of a test. Curves *(a)* represent intensities of bond stress between the bar and concrete. Curves *(b)* may be considered as stresses in bar, at successive points along the specimen. It should be recognised that for curves *(b)*, the intensity of bond stress at any point (rate of change of stress in the bar) is represented by the slope of the curve, with respect to the axis of the specimen, at that point. Bond is what makes stress transfer possible and can be present only in a region of changing stress in the steel or the concrete.

Considering Fig. 15, it is apparent that for a plain bar pull-out test:

(i) Bond resistance is first developed near the proximal end of the bar, and only as slight slip occurs are tensions and bond stresses transmitted progressively distally.

(ii) The region of maximum intensity of bond stress moves away from the proximal end as the pull increases. Between the proximal end and the region of maximum bond stress there is a fairly uniform frictional or drag resistance of greatly reduced intensity.

(iii) "First slip" occurs only after the maximum intensity of bond resistance has travelled nearly the full length of the specimen and has approached the distal end of the bar.

(iv) After appreciable slip, the primary adhesive resistance disappears and the bar offers a frictional or drag resistance throughout its entire length, amounting to perhaps half the ultimate total resistance attained.

In Britain, Hawkes and Evans (1951) were able to conclude from pull-out tests that the distribution of bond obeys an exponential law of the form:

TABLE XI—GROUT/STRAND BOND VALUES WHICH HAVE BEEN EMPLOYED OR RECOMMENDED IN PRACTICE

Strand tendon	*Em-bed-ment (m)*	*Load (kN)*	*Working bond (N/mm^2)*	*Test bond (N/mm^2)*	*Ultimate bond (N/mm^2)*	*Remarks*	*Source*
Any type			<2.1			Design criteria	Australia—Standard CA35 (1973)
Any type					3.105	Design criteria	USA—PCI (1974)
4 No. 12.7mm	4.3	495		0.72		Test anchor	Switzerland—Sommer et al (1974)
	1.8	495		1.71		Test anchor	
8 No. 12.7mm	5.0	1020	0.64			Commercial anchor	USA—White (1973)
8 No. 12.7mm	6.0	1120	0.59			Commercial anchor	USA—Feld et al (1974)
8 No. 12.7mm	3.6	1350	0.71			Commercial anchor	
9 No. 12.7mm	5.2	860	0.46			Commercial anchor	Canada—Hanna et al (1967)
12 No. 12.7mm	4.5	1410	0.65			Commercial anchor	Switzerland—Pliskin (1965)
12 No. 12.7mm	6.1	1200	0.41			Commercial anchor	Canada—Juergens (1965)
12 No. 12.7mm	5.2	1565	0.63			Commercial anchor	Canada—Hanna et al (1967)
12 No. 12.7mm	6.1	1335		0.46		Test anchor	Canada—Barron et al (1971)
12 No. 12.7mm	6.5	1360	0.44			Commercial anchor	Canada—McRostie et al (1972)
16 No. 12.7mm	6.1	1760	0.45			Commercial anchor	Canada—Golder Brawner (1973)
54 No. 12.7mm	11.3	7010	0.29			Commercial anchor	USA—PCI (1974)
N.B.—In the following, no distinction is made between "Normal" (15.4mm) and "Dyform" (15.2mm) strand. All results are calculated using the smaller diameter.							
4 No. 15.2mm	3.0	500	0.88			Commercial anchor	Britain—Universal Anchorage Co. Ltd. (1972)
6 No. 15.2mm	7.3	520	0.25			Commercial anchor	USA—Chen et al (1974)
8 No. 15.2mm	6.1	1500		0.64		Test anchor	USA—Nicholson Anchorage Co. Ltd. (1973)
10 No. 15.2mm	6.7	2150	0.67			Commercial anchor	Australia—Williams (1972)
10 No. 15.2mm	6.1	1900	0.65			Commercial anchor	Australia—McLeod et al (1974)
12 No. 15.2mm	8	2000	0.44			Commercial anchor	Britain—Littlejohn et al (1974)
13 No. 15.2mm	3	3000		1.61		Test anchor	
12 No. 15.2mm	6.5	1950	0.52			Commercial anchor	Switzerland—Pliskin (1965)
18 No. 15.2mm	7.6	4330	0.66			Commercial anchor	Canada—Golder Brawner (1973)
18 No. 15.2mm	7.6	2825	0.43			Commercial anchor	USA—Schousboe (1974)
18 No. 15.2mm	7.62	3770		0.58		Test anchor	
19 No. 15.2mm	15	3740	0.28			Commercial anchor	USA—Feld et al (1974)

$$\tau_x = \tau_0 \, e^{-\frac{Ax}{d}} \qquad \text{...........(5)}$$

where τ_x = bond stress at a distance x from the proximal end
τ_0 = bond stress at the proximal end of the bar
d = diameter of the bar
A = a constant relating axial stress in the bar to bond stress in the anchorage material

Assuming the applied tensile load, P, is equal to the sum of the total bond stress multiplied by the surface area of the tendon, Phillips (1970) extended the above theory as follows:

$$P = \int_0^L \pi d \, \tau_x . dx = \frac{\pi d^2 \tau_0}{A}\left(1 - e^{-\frac{AL}{d}}\right) \qquad \text{..............(6)}$$

between the limits $x = o$ and $x = L$, where L is the length of the fixed anchor. The length of the anchor will depend upon the axial distance required to transfer the load across the interface (transmission length L_0).

At $x = L_0$, τ_x approaches 0 and from (5) it can be seen that Ax/d approaches infinity, giving

$$P = \frac{\pi d^2 \tau_0}{A} \qquad \text{..............(7)}$$

Substituting equation (5) into equation (7) gives

$$\frac{\tau_x}{P}(\pi d^2) = A e^{-\frac{Ax}{d}} \qquad \text{..............(8)}$$

Equations (5) and (8) are represented graphically in Figs. 16 and 17 which show the variation of shear stress along the anchorage and its dependence upon the constant A. The greater the value of A, the larger the stress concentration at the free or proximal end of the anchor. The smaller the value of A the more evenly the stresses are distributed along the length of anchor.

Although values for A have been obtained for steel anchorages embedded in concrete—Hawkes and Evans give $A = 0.28$—insufficient information exists at pre-

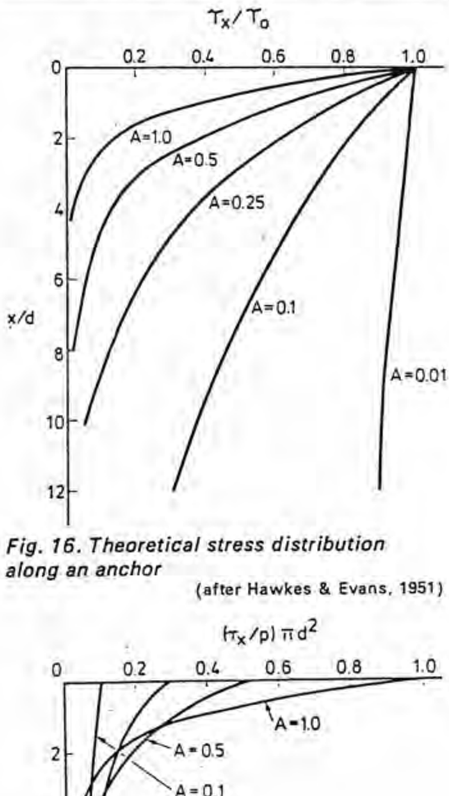


Fig. 16. Theoretical stress distribution along an anchor
(after Hawkes & Evans, 1951)

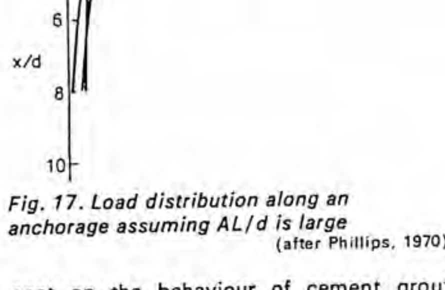


Fig. 17. Load distribution along an anchorage assuming AL/d is large
(after Phillips, 1970)

sent on the behaviour of cement grout anchors in rock to provide meaningful values for A. It is reassuring however, to find that the results in Fig. 17 are very similar to the results of Coates and Yu (1970) in Fig. 8 with E_a/E_r proportional to $1/A$, which suggests that the basic approach of Hawkes and Evans is applicable to rock anchors.

Magnitude of bond

Bars. In a rigorous investigation of the bond between concrete and steel bars, Gilkey, Chamberlin and Beal (1940) emphasise the following major points relevant to rock anchors:

1. Contrary to accepted belief, bond resistance is not proportional to the compressive strength of a standard cured concrete, there being some increase in bond but a reduction in the ratio of bond resistance to the ultimate compressive strength as the strength of concrete increases, especially for the higher strengths. To be specific, for the weaker concretes (UCS $< 21N/mm^2$) bond increases with the compressive strength. However, as the concrete strength exceeds this value, the increase in bond resistance becomes less, and within the strong concrete range i.e. UCS $> 42N/mm^2$, no added bond allowance is justified for added strength of concrete.
2. The bond developed by added length of embedment is not proportional to the additional length. The shorter the embedment, the greater is the average unit bond stress that can be developed by a plain bar. Therefore doubling the length of embedment as a means of increasing the anchorage does not actually double the amount of tension that the bar can resist by bond. On the other hand, additional embedment does add to the sum total of bond resistance.
3. Variations in age and type of curing seem to alter bond resistance much less than they alter the compressive strength of the concrete, bearing in mind that the strongest concrete gives the higher bond, but the weakest concretes have the highest ratio of bond to compressive strength.

Little information is available on the effect of spacing but Chamberlin (1953) conducted a series of tests with various types of bars to determine the effect of spacing on bond magnitude. For clear spacings of 1*d* and 3*d* differences in bond were not significant.

Wires and strand. Based on results obtained from almost 500 pull-out tests, Stocker and Sozen (1964) conclude:

(*a*) Due to the helical arrangement of the exterior wires, strand rotates while slipping through a grout channel, but the increase in bond is not significant. (Anderson *et al* (1964) also observed rotation of strand of about 15 deg during pull-out tests.)

(*b*) Bond magnitude increases by approximately 10 per cent per $6.9N/mm^2$ of concrete compressive strength, in the range $16.6\text{-}52.4N/mm^2$.

(*c*) Results from pull-out tests subjected to externally applied lateral pressures ranging from $0\text{-}17.25kN/m^2$ indicate a linear increase of bond strength with lateral pressure. In connection with this, concrete shrinkage is clearly important.

Effect of rust on bond

Gilkey *et al* (1940) also investigated the effect of steel surface conditions on bonding properties and found that:

(*i*) Deep flakey rust on bars, following 6-8 months exposure, lowers the bond, but wiping the loosest rust off finally produces a surface that will develop a bond equal to or greater than that which the bar would have developed in the unrusted condition.

(*ii*) Slightly rusted bars, following up to three months exposure, developed greater bond than unrusted or wiped rusted bars.

(*iii*) The loose powdery rust which appears on bars during the first few weeks of ordinary exposure has no significant effect on the bond properties of bars.

These findings have also been confirmed by Kemp *et al* (1968) for deformed bar, and Armstrong (1948), Base (1961) and Hanson (1969) for wire and strand.

Remarks

Some designers consider the question of grout/tendon bond in anchor systems to present no problems, as the design at the rock/grout interface is more critical. Therefore any embedment length accommodating that interface automatically ensures a high factor of safety at the tendon/grout interface. A factor of safety of at least two is allowed by other designers.

While there is an appreciable amount of information available concerning the mechanism of bond transfer in the field of reinforced and prestressed concrete, it is considered that much more study is required in the field of rock anchors. The mode of failure of a tendon bonded into the grout of an in situ rock anchor may be dissimilar to that of the tendon pull-out test used in concrete technology and from which most data are obtained. In the former case the grout is usually in tension, whereas during a standard bond test, part, at least, of the surrounding concrete is in compression. In rock anchors, therefore, the mechanism of bond action depends on the respective elastic moduli of the steel and grout.

Little work has been done on multi-unit tendons with respect to their bond distribution. The use of spacers and centralisers, leading possibly to decoupling, also warrants investigation.

In general, recommendations pertaining to grout/tendon bond values used currently in practice for rock anchors commonly take no account of the length and type of tendon, tendon geometry, or grout strength, and for these reasons it is still advisable to measure experimentally the embedment length for known field conditions.

TENDON

Introduction

Accurate data on the mechanical properties of tendon components are readily available, but the choice of type of tendon and safety factors to be employed against rupture still demand assessment and judgement by the designer, especially in countries not covered by a code relating to anchors.

Tendons may be formed of bar, wire or strand. The latter two have distinct advantages with respect to tensile strength, ease of storage, transportation and fabrication. Bars, however, are more readily protected against corrosion and in the case of shallow, low capacity anchors, are often easier and cheaper to install.

Largely as a result of recent developments in prestressing equipment and techniques, the use of strand appears to be increasing in popularity. A recent survey by FIP (1974) also confirms that strand tends to be more popular than wire, and the use of strand is now accepted even in countries where the basic material cost is greater. It is now widely recognised that the smaller the diameter of the tendon, the less is the cost of the material per unit of prestress force, but direct cost comparisons for the supply of tendon material in any country can be misleading since the real cost of the tendon also reflects cost of fabrication, installation and stressing.

Tendon characteristics

With regard to general characteristics it is of value to know that in Britain the production of prestressing tendons is governed by BS 4486:1969 (Cold Worked High Tensile Alloy Steel Bar), BS 2691: 1969 (Steel Wire), BS 3617:1971 (7 Wire Strand) and BS 4757:1971 (19 Wire Strand).

Following publication of CP 110:1972, permissible stresses are quoted in terms of the specified characteristic strength which is the guaranteed limit below which not more than 5 per cent test results fall, and none of these is less than 95 per cent characteristic strength. For wire and strand, the specified minimum strength is taken as the characteristic strength, which for practical purposes is termed 100 per cent fpu.

At home and abroad it is common to find tendon stresses quoted in such terms as elastic limit, 0.1 per cent proof stress and 0.2 per cent proof stress. Therefore to facilitate understanding and comparisons, some reconciliation is required between these terms and characteristic strength. In this connection it is noteworthy that in the preamble to the French Code (Bureau Securitas 1972) the term *Tg* is identified and defined as the elastic limit, measured as the 0.1 per cent proof stress, i.e. that point at which the permanent elongation reaches this value. The same note draws attention to the fact that this limit should not be confused with the 0.2 per cent proof stress adopted in the British Codes. Based on the advice of wire metallurgists the authors understand that the 0.1 per cent proof stress varies from 3-5 per cent below the 0.2 per cent proof stress which is defined as 87 per cent fpu in CP 110. Taking the average figure of 4 per cent below 0.2 per cent proof stress, then a 0.1 per cent proof stress is equivalent to 83.5 per cent fpu. This correlation may be employed when comparing safety factors in subsequent tables.

With respect to the values of elastic modulus quoted subsequently, it is known that an error of 5 per cent is possible, although the majority of results are within three standard deviations from the mean. Knowledge of this possible variation can be very important when interpreting load-extension graphs and for the same reason relaxation characteristics of tendons should be assessed carefully. Both aspects are detailed in Part 3 of this review, but it is of general interest to know that relaxation loss is a function of the logarithm of time.

For example, the loss after one hour is

TABLE XII—TECHNICAL DETAILS OF BRITISH PRESTRESSING BARS

Item	*Unit*	*Bar diameter (mm)*									*Remarks*
		20*	22	25*	28	32*	35	40*	4x32	4x40	
Sectional area	mm^2	314.2	380.1	490.9	615.8	804.3	962.1	1256.6	3217	5026	In each case, the characteristic tensile strength is $1000N/mm^2$
Minimum breaking load	kN	325	375	500	625	800	950	1250	3200	5000	

*Recommended sizes

50-60 per cent of that at 100 hours, which in turn is about 80 per cent of that at 1000 hours. The loss at 1000 hours is also about half that at 5-8 years. Relaxation loss depends on the initial stress in the tendon and production history, and whilst tendons of exceptionally low relaxation properties can be produced, the anchor designer should remember that little advantage will be gained through their use, if for example creep in the ground is likely to be large in comparison.

1. *Bars*. CP 110 (1972) quotes detail supplied by McCalls Macalloy Ltd. (1969) on typical British bars in use (Table XII). The modulus of elasticity is about 165 000 N/mm², although CP 110 suggests a value of 175 000N/mm².

With regard to relaxation Antill (1965) found that the load loss for a typical alloy steel bar, initially stressed to 70 per cent UTS is about 4 per cent at 1 000 hours, and double that at 100 000 hours. For comparison the performance of bars relative to other tendon components is shown in Fig. 18. This information is provided for designers bearing in mind that CP 110 advises that an "appropriate allowance for relaxation" be made "for sustained loading conditions".

The use of bar anchors is very popular in Germany and North America, where bar sizes are available from 6.4mm (No. 2 bar) to 25.4mm (No. 8 bar) in steps of 3.2mm, and thereafter to 35.8mm (No. 11 bar) in slightly larger increments.

Bars tend to be used as tendons in fairly short low-medium capacity anchors mainly in single bar situations, but are increasingly used in certain sophisticated forms in Germany, where compression tubes and elaborate end bearing devices are incorporated. Groups of up to four bars have been used on occasions, but larger groups are rare although Berardi (1960) successfully used 20 No. 20mm plain bars for a 1 750kN test anchor.

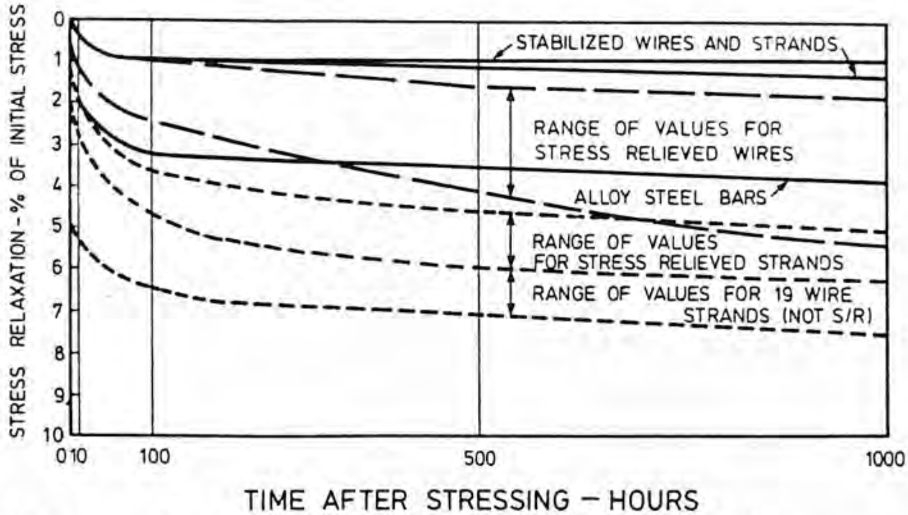


Fig. 18. Relaxation of British tendons at 20°C from initial stresses of 0.7 UTS
(after Antill, 1965)

2. *Wires*. Prestressing wire is manufactured from cold drawn plain carbon steel, and in a few countries, for example Germany, quenched and tempered (oil hardened and tempered or oil hardened) varieties predominate.

The ultimate tensile strength is inversely proportional to wire diameter, but also depends on the method of manufacture, and the steel specifications of the country concerned.

The major properties of British wire are summarised in Table XIII. CP 110 indicates that a typical value for the elastic modulus of wire and small diameter strands is 200 000N/mm².

It is noteworthy that Shchetinin (1974) reveals that Soviet industry produces wires of capacity 1 375-1 865N/mm² to meet the Soviet Code GOST, 7348-63. A popular choice for anchors is 5mm wire (1 670 N/mm²), with elastic modulus 184 000 N/mm² and 6.8 per cent relaxation at 1 000 hours. Wire tendons are recommended by Shchetinin on the basis that they eliminate suspected torsional and bending problems of strand anchors.

In general, tendons comprise between 10 and 100 wires (5-8mm diameter), dependent on the required anchor capacity, but 660 No. 5mm wires were employed at the Cheurfas Dam by Soletanche in 1934.

3. *Strand*. All strand is made from cold drawn plain carbon steel wire in Britain and seven wire strand is by far the most popular. Seven wire strands are stress relieved after stranding to produce a "normal relaxation" type, in two grades—regular

TABLE XIII—TECHNICAL DETAILS OF BRITISH PRESTRESSING WIRE

Wire diameter (mm)	*Characteristic strength (N/mm²)*	*Remarks*
Mill coil (BS 2691 Sect. 4)		
5.0*	1570*	Average E(N/mm²) = 192 000
	1670	0.2 per cent stress = 75 per cent specified
	1720	minimum strength.
4.5*	1620*	Average relaxation at 1 000 hours from
4.0*	1570*	70 per cent = 8 per cent ultimate at
	1670	20 deg C
	1770	
3.25*	1670	
	1720*	
	1770	
3.0	1670	
	1720*	
	1770	
2.65	1770	
	1870	
2.0	2020	
Prestraightened—Normal relaxation (BS 2691, Sect. 2)		
8.0	1470	Average E(N/mm²) = 201 000
	1570	
7.0*	1470	0.2 per cent proof stress = 85 per cent
	1570*	specified minimum strength.
	1670	
6.0	1470	Average relaxation at 1 000 hours from
	1570*	70 per cent = 3.8 per cent ultimate at
	1670	20 deg C
5.0*	1570*	
	1670	
	1720	
4.5	1570*	
	1670	
	1720	
4.0*	1570*	*Preferred sizes
	1670	
	1770	

TABLE XIV—TECHNICAL DETAILS OF BRITISH PRESTRESSING STRAND

Dia (mm)	*Minimum breaking load (kN)*	*Average E. (N/mm²)*	*Average relaxation at 1000 hrs from 70 per cent ultimate at 20 deg C (per cent)*	*Remarks*
Regular: normal relaxation (BS 3617 Sect. 2)				
6.4	44.5	198 000	5.6	The load at 1.0 per cent extension or
7.9	69.0	198 000	(7	0.2 per cent proof load ≡ 89 per cent
9.3	93.5	198 000	maximum)	actual breaking load (average).
10.9	125.0	198 000		The load at 0.01 per cent proportional
12.5*	165.0	198 000		limit ≡ 73 per cent actual breaking
15.2*	227.0	198 000		load. (average)
Regular: low relaxation (BS 3617 Sect. 3)				
9.3	93.5	200 000	1.1	The load at 1.0 per cent extension ≡
10.9	125.0	200 000	(2.5	89 per cent actual breaking load
12.5*	165.0	200 000	maximum)	(average).
15.2*	227.0	200 000		The load at 0.01 per cent proportional limit ≡ 80.5 per cent actual breaking load (average)
Super: normal relaxation				
9.6	102.5	197 000	5.5	Load at 1.0 per cent extension ≡ 85
11.3	138.0	197 000	(7	per cent actual breaking load
12.9*	184.0	197 000	maximum)	(average).
15.4*	250.0	197 000		Load at 0.01 per cent proportional limit ≡ 76 per cent actual breaking load (average)
Super: low relaxation				
9.6	102.5	198 000	1.15	Load at 1.0 per cent extension ≡
11.3	138.0	198 000	(2.5	90 per cent actual breaking load
12.9*	184.0	198 000	maximum)	(average).
15.4*	250.0	198 000		Load at 0.01 per cent proportional limit ≡ 79 per cent actual breaking load (average)
Dyform				
12.7*	209.0	198 500	1.1	Load at 1.0 per cent extension ≡
15.2*	300.0	196 500		92 per cent, 92 per cent and 91 per
18.0*	380.0	195 100		cent actual breaking load, respectively (average) Load at 0.01 per cent proportional limit ≡ 85 per cent, 85 per cent and 83 per cent actual breaking load, respectively (average)

*Preferred sizes

and super. "Low relaxation" strand is produced by a patented stabilisation process of applying a tensile stress to the strand during the stress relieving process. Again two grades are available. In addition, strand may be subjected to a compacting, or "dyforming" process whereby about 20 per cent more of the nominal cross-sectional area is occupied by steel, with respect to ordinary strand, and so higher loads can be sustained. Such strand also has low relaxation properties.

The mechanical properties of British seven wire strand are summarised in Table XIV, bearing in mind that the values of minimum breaking load and elastic modulus may vary by up to 8 per cent and 5 per cent respectively.

Nineteen wire strand is available in diameters of 22.2, 25.4, 28.6, and 31.8mm, with minimum breaking loads of 503.1, 658.9, 823.6 and 1 002.0kN respectively.

In general, tendons comprise between four and 20 strands (12.7 and 15.2mm diameter) but 54 No. 12.7mm strands were used for 7 010kN capacity anchors in the Interstate Highway 1-96 retaining wall, Detroit, USA.

Allowable stresses and safety factors

In Britain, the vast majority of anchor tendons are designed with a working stress of 62.5 per cent fpu i.e. a factor of safety against failure of 1.6. However, in recent years several publications have suggested that whilst this approach is acceptable for temporary anchors (working life less than two years), the design stress for permanent tendons should be reduced to 50 per cent fpu, giving a safety factor of 2 and permitting a larger test overload. Since the French Code (1972) is widely recognised as one of the most authoritative documents on ground anchors published to date, it is encouraging to observe that for temporary and permanent anchors, the Bureau Securitas recommend design forces of $0.75Tg$ and $0.6Tg$, respectively. As shown earlier, Tg is the elastic limit which is equivalent to 83.5 per cent fpu and thus it may be calculated that the French recommendations are almost identical to those presented in recent British publications.

To provide a more general picture, Table XV shows recommendations made in Codes of Practice, and by practising engineers, throughout the world. For convenience, only permanent anchors have been considered.

It would appear that there is a definite trend towards raising both the measured and ultimate safety factors, to 1.5 and 2.0 respectively; this is undoubtedly an encouraging feature, at a time when larger capacity anchors are being installed, often in poor quality rock. In such conditions a large test overload is considered necessary for security, and this can only be achieved by a reduction in the level of working stress to about 50 per cent ultimate. Otherwise, permanent set may be induced in the tendon.

Tables XVI-XVIII have been prepared from data provided in papers describing this aspect of anchor design. A number of examples are quoted for each type of tendon for purposes of illustration and discussion.

The average working stress is highest for wires and lowest for bars; the safety factor against rupture of the tendon is thus in the inverse relation. Testing to 1.5 times the working stress seems at present to be the exception rather than the rule, and commonly contract anchors are over-prestressed by an amount thought equivalent

TABLE XV—ALLOWABLE STRESSES AND SAFETY FACTORS WHICH HAVE BEEN RECOMMENDED FOR ANCHOR TENDONS

Working stress (per cent)	*Test stress (per cent)*	*Measured safety factor*	*Ultimate safety factor*	*Remarks*	*Source*
50	75	1.5	2	With respect to (w.r.t.) characteristic tensile strength	Britain—Littlejohn (1973)
<50	70	1.5	>2	w.r.t. characteristic tensile strength	Britain—Mitchell (1974)
50	75-80 80	1.5-1.6	2	w.r.t. characteristic tensile strength	Britain—Ground Anchors Ltd. (1974) Britain—CP 110 (1972)
<60	<90	1.5	1.75	w.r.t. yield strength	Germany—DIN 4125 (1972)
70	77	1.1	1.43	"Swissboring SA" BBRV anchors	Switzerland—Descoeudres (1969)
<69	≤90	>1.3	>1.45	w.r.t. yield strength	Switzerland—Draft recommendations (1973)
<70	<95	1.36	1.43	w.r.t. 0.1 per cent residual elongation	France—Fargeot (1972)
60				w.r.t. the elastic limit	France—Adam (1972)
53-66	80	1.2-1.5	1.5-1.9	w.r.t. ultimate tensile strength	France—Fenoux et al (1972)
≤60		1.3	2	w.r.t. elastic limit	France—Bureau Securitas (1972)
			1.5-2	w.r.t. elastic limit	Italy—Mascardi (1972)
65 or			1.54	w.r.t. ultimate tensile strength	Finland—Laurikainen (1972)
85				w.r.t. elastic limit	Finland—Laurikainen (1972)
59 or			1.69	w.r.t. ultimate tensile strength	Czechoslovakia—Voves (1972)
71				w.r.t. elastic limit	Czechoslovakia—Voves (1972)
<57	<69	>1.2	>1.75	w.r.t. ultimate tensile strength	Czechoslovakia—Draft Standard (1974)
60	80	1.33	1.67	w.r.t. ultimate tensile strength	Canada—Golder Brawner Assocs. (1973)
50	75	2	1.5	w.r.t. ultimate tensile strength	USA—White (1973)
55-60		1.1	1.7-1.9	w.r.t. ultimate tensile strength	Brazil—da Costa Nunes (1971)
90				w.r.t. elastic limit	Brazil—da Costa Nunes (1971)
65 or		1.1	1.54	w.r.t. ultimate tensile strength	South Africa—Parry-Davies (1968)
80				w.r.t. yield strength	South Africa—Parry-Davies (1968)
≤70		>1.2	≥1.43	w.r.t. ultimate tensile strength	South Africa—Johannesburgh (1968)
≤70			≥1.43	*Wires* w.r.t. ultimate tensile strength	South Africa—Code (1972)
≤67			≥1.49	*Bars* w.r.t. ultimate tensile strength	South Africa—Code (1972)
or 80		1.25	≥1.49	*Bars* w.r.t. 0.2 per cent proof strength	South Africa—Code (1972)
	75			w.r.t. ultimate tensile strength	Australia—Koch (1972)
<60			≥1.67	w.r.t. ultimate tensile strength	Australia—Code CA 35 (1973)
50	80	1.6	2	w.r.t. ultimate tensile strength	New Zealand—Irwin (1972)

Measured safety factor = Test load / Working load

Ultimate safety factor = Failure load / Working load

TABLE XVI—DESIGN STRESSES AND SAFETY FACTORS WHICH HAVE BEEN EMPLOYED IN PRACTICE FOR BAR TENDONS

Bar	*Working stress (per cent ultimate)*	*Test stress (per cent ultimate)*	*Measured safety factor*	*Ultimate safety factor*	*Source*
28mm Lee Macalloy	70	—	—	1.43	Britain—Banks (1955)
32mm Macalloy	56	84	1.5	1.79	Britain—Jackson (1970)
32mm hollow	54	64	1.2	1.85	Sweden—Nordin (1968)
35mm	50	75	1.5	2	USA—Drossel (1970)
22mm HS	47	52	1.1	2.1	USA—Koziakin (1970)
HS bars	—	—	1.5	—	USA—Wosser et al (1970)
35mm Bauer	44	54	1.2	2.27	USA—Larson et al (1972)
27mm Dywidag	55	58	1.06	1.82	Japan—Construction Ministry (1964)

TABLE XVII—DESIGN STRESSES AND SAFETY FACTORS WHICH HAVE BEEN EMPLOYED IN PRACTICE FOR WIRE TENDONS

Wire	*Work stress (per cent ultimate)*	*Test stress (per cent ultimate)*	*Measured safety factor*	*Ultimate Safety factor*	*Source*
5mm	64	74	1.36	1.57	Britain—Morris et al (1956)
7mm	63	69	1.1	1.59	Britain—Gosschalk et al (1970)
7mm	66	79	1.2	1.52	Switzerland—VSL (1966)
8mm	68	82	1.2	1.47	Switzerland—VSL (1966)
8mm	50	65	1.3	2.0	Switzerland—Moschler et al (1972)
6.4mm	60	—	1.08	1.67	Canada—Golder Brawner Assocs. (1973)
6.4mm	60	70	1.17	1.67	USA—Eberhardt et al (1965)
7mm	60	62	1.03	1.67	Australia—Rawlings (1968)

TABLE XVIII—DESIGN STRESSES AND SAFETY FACTORS WHICH HAVE BEEN EMPLOYED IN PRACTICE FOR STRAND TENDONS

Strand	*Working stress (per cent ultimate)*	*Test stress (per cent ultimate)*	*Measured safety factor*	*Ultimate safety factor*	*Source*
15.2mm	55	61	1.1	1.82	Britain—Ground Anchors Ltd. (1973)
15.2mm	58	80	1.37	1.71	France—Soletanche (1968)
12.7mm	48	57	1.2	2.1	Switzerland—VSL (1966)
12.7mm	30	73	2.43	3.3	Switzerland—Sommer et al (1974)
12.7mm & 15.2mm	60	—	—	1.67	Canada—Golder Brawner (1973)
12.7mm	65	80	1.23	1.54	Canada—Golder Brawner (1973)
15.2mm	50	80	1.6	2.0	Canada—Golder Brawner (1973)
12.7mm	52	78	1.5	1.93	USA—White (1963)
12.7mm	60	80	1.33	1.67	USA—Buro (1972)
15.2mm	59	79	1.34	1.69	USA—Schousboe (1974)
12.7mm	60	85	1.42	1.67	Australia—Langworth (1971)

TABLE XIX—PITCH OF TENDON SPACERS IN THE GROUTED FIXED ANCHOR ZONE

Pitch of tendon spacers (m)	*Remarks*	*Source*
0.5	Cheurfas Dam	USA—Zienkiewicz et al (1961)
0.5	3m fixed anchor	Czechoslovakia—Hobst (1965)
0.8	Multi-wire tendons	France—Cambefort (1966)
0.6	VSL anchors	Switzerland—Losinger SA (1966)
0.8-1.6	Multi strand tendons	Britain—Littlejohn (1972)
2.0	Multi strand tendons	Italy—Mascardi (1972)
0.5-2.0	Dependent on "stiffness" of tendon system	Germany—Stocker (1973)
1.5-2.0	Conenco (Freyssinet) anchors	Canada—Golder Brawner Assocs. (1973)
1.8	7.3m fixed anchor	USA—Chen et al (1974)
2.0	8m fixed anchor (12 No. 15.2mm strands)	Britain—Littlejohn et al (1974)
0.5	Multi-wire tendons	USSR—Shchetinin (1974)

to long term load losses—usually 10 per cent.

Tendon spacers

Spacers are used in both the free and fixed sections of multicomponent tendons. In the free length they may serve to centralise the tendon with respect to the borehole but their main function is to prevent tangling or rubbing of the individual bars, wires or strands. This is particularly important in long, flexible tendons, where, if the tendon is allowed to lose its design geometry, load may be dissipated through friction in the free length during stressing. In addition, extremely high stress concentrations may develop, particularly just under the top anchor head, where rupture of individual elements can easily occur. Spacers in this part of the anchor are hollow cored and between 4-8m apart.

In the grouted fixed anchor zone the spacers encourage effective penetration of grout between the tendon units, thereby ensuring efficient transmission of bond stress. In addition the spacer units should be designed to centralise the tendon in the borehole to (*a*) avoid contamination of tendon e.g. clay smear, and (*b*) give adequate cover of grout for corrosion protection and good grout bond at the borehole interface.

Spacers in this zone may also be used in conjunction with intermediate fastenings to form nodes or waves, in order to provide a more positive mechanical interlock between the tendon and surrounding grout. Whilst this method gives a tendon geometry which allows adequate penetration and cover of grout, it is important to note that the practice of unravelling strands followed by bushing of the wires gives a random geometry which cannot guarantee efficient load transfer.

With reference to the pitch of spacers, Table XIX gives an indication of the distances which have been employed in practice. In general it would appear that little work has been carried out on the influence of pitch or spacer design on load transfer in the fixed anchor zone.

Remarks

Whilst tendons are produced to a high standard and reliable minimum breaking loads are specified for use by the designer, few load/extension tests have been carried out on long tendons (10-30m) which are comparable in size to the free anchor lengths used in practice. Since interpretation of anchor load/displacement characteristics can be quite controversial in practice, particularly in the case of strand, it would be of value to know if long strand tests give E values which are significantly different from those obtained using short gauge lengths of 0.61m. The influence of tendon curvature, and splaying of multi-component tendons near the top anchor head on stress/strain behaviour also requires clarification in view of the dearth of published information, at present. Top anchor heads will be discussed in Part 3 of this review.

GENERAL CONCLUSIONS

Although rock anchors have been used for over 40 years, it is difficult to justify technically certain aspects of contemporary design. Progress in the development and rationalisation of design has been slow, largely due to the scarcity of reliable laboratory and field experimental data relating directly to rock anchors.

As a result, practising engineers have been obliged to make reference to values and methods employed with apparent success in earlier designs, without fully appreciating or understanding their accuracy or reliability. ~~Bearing~~ this in mind, it is perhaps understandable that the majority of designs are overconservative in certain aspects, if not in all. This dilemma is becoming increasingly acute now that engineers are being requested to design for circumstances where no exact precedents exist.

In view of the inconsistencies between theory and practice which have been highlighted in this design review, it is considered that more attention should be directed towards studies in the following areas:

1. *Uplift capacity*. There is little justification for the inverted cone method of assessing the ultimate resistance of withdrawal of the rock anchor system. However, until full-scale field tests are carried out to study modes of failure in relation to the geotechnical properties of rock masses, the present method of design, where rock shear strength is ignored, must be persevered with, as it is basically very conservative. Nevertheless, some standardisation on acceptable modes of failure, safety factors and allowances for unconsolidated overburden is now required.
2. *Fixed anchor*. A uniform distribution of bond stress is assumed in the vast majority of anchor designs, although this approach is only valid in the case of soft rock. In hard rock, the stress distribution is non-uniform, the highest stresses being mobilised at the proximal end of the grouted fixed anchor zone. The ratio E_{grout}/E_{rock} has a major influence on stress distribution, although the authors find that rock masses are seldom classified in sufficient detail for other potentially important parameters to be highlighted. The phenomena of debonding in rock anchors is not well understood, although it has undoubtedly been significant in certain high capacity anchors described. Values for the magnitude of bond at the grout tendon interface are usually abstracted from publications relating to reinforced and prestressed concrete. However, it should be noted that the boundary conditions existing in conventional bond tests, may be wholly different from these present in the rock anchor situation.

References

Abraham, K. and Porzig, R. (1973): 'Die Felsanker des Pumpspeicherworkes Waldeck II'' Baumaschine und Bautechnik, 20 (6), pp. 209-220.
Adam, M. (1972): Reply to FIP questionnaire.
Anderson, G. F., Rider, J. H. and Sozen, M. A. (1964): "Bond Characteristics of Prestressing Strand" Prog. Report 13 of Project 1HR-10, Illinois Coop. Highway Res. Prog., Eng. Exp. Stn., Univ. Illinois, June, 157 pp.
Anon (1970): "Rock Stabilisation in the Vicinity of the Intake Tower, El Atazar" Report on test cables. Source unknown 8pp.
Anon (1972): "Sea of Anchors and Bolts Keep Powerhouse Cavern Shipshape". Eng. News Record, July 20, pp. 39-40.
Antill, J. M. (1965): "Relaxation Characteristics of Prestressing Tendons". Civ. Eng. Trans. Inst. Eng. Aust., 7 (2) pp. 151-159.
Armstrong, W. E. I. (1949): "Bond in Prestress concrete". J. Inst. Civ. Engng., *33*, pp. 19-40.
Associated Tunnelling Ltd. (1973): "Report on Ground Anchor Tests— Thames Flood Prevention". Unpublished (14 pp.) Lowton St. Mary's near Warrington, England.
Banks, J. A. (1955): "The Employment of Prestressed Techniques on Allt-na-Lairige Dam". 5th Int. Cong. on Large Dams, Paris, 2, pp. 341-357.
Barron, K., Coates, D. F. and Gyenge, M. (1971): "Artificial Support of Rock Slopes". Dept. of Energy, Mines and Resources, Mines Branch, Ottawa. Research Report R 228 (Revised), 145 pp.
Base, G. D. (1961): "An Investigation of the Use of Strand in Pretensioned Prestressed Concrete Beams". Research Report No. 11, 12 pp., Cement and Concrete Assoc., London.
Beomonte, M. (1961): "Criteri Peril Calcalo e la Posa in Opera di Bulloni di Ancoraggio". Geotecnica, *1* (Feb.), pp. 5-13.
Berardi, G. (1960): "Ricerche Teoriche e Sperimentali Sugli Ancoraggi in Roccia". Geotecnica, *6*, 12 pp.
Berardi, G. (1967): "Sul Comportamento Degli Ancoraggi Immersi in Terreni Diversi". Univ. Genoa. Inst. Constr. Sc. Series III, No. 60, 18 pp.
Berardi, G. (1969): "Sui Problemi Geotechnici Dellacittà di Genova con Particulare Riguardo Alle Pareti Rocciose". Rivista Italiana di Geotecnica, *1*, 21 pp.
Berardi, G. (1972): "Predimento per la Bonifica e Sostegno dei pendii a Mezzo di Chiodatura ed Ancoraggi". 3rd Conf. on Problems of Foundation

Engineering. Turin, 16 March, 60 pp. 16 Nov.

Birkenmaier, M. (1953): "Vorgespannte Felsanker". Schweizerische Bauzeitung, 71 (47), pp. 688-692.

Boyne, D. M. (1972): "Use of Skin Friction Values in Rock Anchor Design", BSc(Hons) thesis, Engineering Dept., University of Aberdeen.

Bridon Wire Ltd. (1968): "Wire and Strand for Prestressed Concrete".

Bridon Wire Ltd. (1971): "Wire and Strand for Prestressed Concrete". Section No. 1, Edition No. 5.

Bridon Wire Ltd. (1971): "Dyform Prestressing Strand". Section No. 2, Edition No. 3. Bridon Wire Ltd., Doncaster, England.

British Standards Institution (1972): "The Structural Use of Concrete". CP 110, Part 1, BSI, 2 Park Street, London.

Broms, B. B. (1968): "Swedish Tieback Systems for Sheet Pile Walls". 3rd European Conf. S.M. & F.E.—Budapest, pp. 391-403.

Brown, D. G. (1970): "Uplift Capacity of Grouted Rock Anchors". Ontario Hydro Research Quarterly, 22 (4), pp. 18-24.

Brunner, H. (1970): "Praktische Erfahrungen bei den Hangsicherungen in Neuenhof", Strasse und Verkehr, 9, pp. 500-505.

Bujak, M., Glab, W., Moraczewski, K. and Wolski, W. (1967): "Preventative Measures Against the Rock Slide at Tresna Dam Site". 9th Int. Cong. on Large Dams, Istanbul, 1, pp. 1027-1035.

Bundred, J. (1973): "Insitu Measurements of Earth Pressure and Anchor Forces for a Diaphragm Retaining Wall". Symp. on Field Instr. in Geotech. Eng., London, pp. 52-69.

Bureau Securitas (1972): "Recommendations Regarding the Design, Calculation, Installation and Inspection of Ground Anchors". Editions Eyrolles, 61 Boulevard Saint-Germain, Paris-Ve (Ref. TA 72).

Buro, M. (1972): "Rock Anchoring at Libby Dam". Western Construction (March), pp. 42, 48 and 66.

Cambefort, H. (1966): "The Ground Anchoring of Structures". Travaux, *46* (April-May), 15 pp.

Cementation Co. Ltd. (1962): "Stressing". Technical Report No. 10 (unpublished), Croydon, England.

Chen, S. C. and McMullan, J. G. (1974): "Similkameen Pipeline Suspension Bridge". Proc. ASCE, Transportation Eng. J. *100* (TEI), pp. 207-219.

Coates, D. F. (1970): "Rock Mechanics Principles". Department of Energy, Mines and Resources. Mines Monograph No. 874, Ottawa.

Coates, D. F. and Yu, Y. S. (1970): "Three Dimensional Stress Distributions around a Cylindrical Hole and Anchor". Proc. 2nd Int. Conf. on Rock Mechanics, Belgrade, *2*, pp. 175-182.

Comte, C. (1965): "L'utilisation des ancrages en Rocher et en Terrain Meuble". Bull. Tech. de la Suisse Romande, *22* (Oct), pp. 325-338.

Comte, C. (1971): "Technologie des Tirants". Institute Research Foundation Kollbrunner/Rodio, Zurich, 119 pp.

Conti, N. (1972): Reply to FIP questionnaire.

Da Costa Nunes, A. J. (1969): "Anchorage tests in clays for the Construction of Sao Paulo Subway". 7th Int. Conf. of the Int. Soc. for Soil Mech. and Found. Eng., Mexico, Paper 15-2, pp. 120-125.

Da Costa Nunes, A. J. (1971): "Metodas de Ancoragens". Parts (1) and (2), Associaçao dos Antigos Alumos da Politecnica. 50 pp.

Descoeudres, J. (1969): "Permanent Anchorages in Rock and Soils". 7th Int. Conf. of Int. Soc. Soil Mech. and Found. Eng., Mexico, Paper 15-17 pp. 195-7.

Deutsche Industrie-Norm (1972): Verprebanker für Vorübergehende Zwecke im Lockergestein: Bemessung, Ausführung und Prüfung. DIN 4125, Blatt 1, 1972.

Drossel, M. R. (1970): "Corrosion Proofed Rock Ties Save Building from Earth Pressures". Constr. Methods, May, pp. 78-81.

Eberhardt, A. and Veltrop, J. A. (1965): "Prestressed Anchorage for Large Trainter Gate". Proc. ASCE, J. Struct. Div., *90* (ST6), pp. 123-148.

Fargeot, M. (1972): Reply to FIP questionnaire.

FIP (1972): "Draft of the Recommendations and Replies to FIP Questionnaire (1971)". FIP Subcommittee on Prestressed Ground Anchors.

FIP (1973): "Final Draft of Recommendations". FIP Subcommittee on Prestressed Ground Anchors.

FIP (1974): "Questionnaire on Choice of Tendon". FIP Notes 50, pp. 4-8, Cement and Concrete Assoc., 52 Grosvenor Gardens, London SW1W 0AQ.

Feld, J. and White, R. E. (1974): "Prestressed Tendons in Foundation Construction". Proc. Tech. Session on Prestressed Concrete Foundations and Ground Anchors, pp. 25-32, 7th FIP Congress, New York.

Fenoux, G. Y. and Portier, J. L. (1972): "Lamise en Précontrainte des Tirants". Travaux, *54* (449-450), pp. 33-43.

Gilkey, H. J., Chamberlin, S. J. and Beal, R. W. (1940): "Bond between Concrete and Steel". Reproduced in Eng. Report No. 26, Iowa Eng. Exp. Station, Iowa State Coll. Ames (1956), pp. 25-147.

Golder Brawner Assocs. (1973): "Government Pit Slopes Project III: Use of Artificial Support for Rock Slope Stabilisation". Parts 3.1-3.6, Unpublished, Vancouver, Canada.

Gosschalk, E. M. and Taylor, R. W. (1970): "Strengthening of Muda Dam Foundations using Cable Anchors". Proc. 2nd Cong. Int. Soc. Rock Mech., Belgrade, *3*, pp. 205-210.

Ground Anchors Ltd. (1974): "The Ground Anchor System". (28 pp.) Reigate, Surrey.

Hanna, T. H. and Seeton, J. E. (1967): "Observations on a Tied-back Soldier-pile and Timber-lagging Wall". Ontario Hydro Res. Qtly., *19* (2), pp. 22-27.

Hanson, N. W. (1969): "Influence of Surface Roughness of Prestressing Strand on Bond Performance". J. Prestr. Conc. Inst., *14* (1), pp. 32-45.

Hawkes, J. M. and Evans, R. H. (1951): "Bond Stresses in Reinforced Concrete Columns and Beams". Structural Engineer, *29* (12), pp. 323-327.

Heitfield, K. and Schauerte, E. (1969): "Badentung der Geologischen Verhältnisse für die Böschungssicherung am Ehemaligen Hofolder Tunnel (Sauerland)". Z. Deutsch Geol. Ges., *119*, pp. 270-284.

Hennequin, M. and Cambefort, H. (1966): "Consolidation du Remblai de Malherbe". Revue Generale des Chemins de Fer (Feb), 9 pp.

Hill, J. W. (1973): Reply to Aberdeen questionnaire (1972), unpublished.

Hobst, L. (1965): "Vizepitmenyek Kihorgonyzasa". Vizugi Kozlemenyek, *4*, pp. 475-515.

Hobst, L. (1969): "Stabilizace Svahů Pred petim". Invényrské Stavby, *9-10*, pp. 353-359.

Irwin, R. (1971): Reply to FIP questionnaire.

Jackson, F. S. (1970): "Ground Anchors—the Main Contractor's Experience". Supplement on Ground Anchors, *The Consulting Engineer*, May.

Japanese Government, Construction Ministry (1964): "Foundation,Treatment of Kawamata Dam". Proc. 8th Int. Cong. on Large Dams, Edinburgh, *1*, pp. 187-207.

Jasper, J. and Shtenko, V. (1969): "Foundation Anchor Piles in Clay—Shales". Canadian Geot. Journal, 6, pp. 159-175.

Johannesburg City Engineer's Dept. (1968): "Cable Anchors; Design Procedures". Unpublished Memorandum (4 pp.).

Judd, W. R. and Huber, C. (1961): "Correlation of Rock Properties by Statistical Methods". International Symposium on Mining Research.

Juergens, R. E. (1965): "Cables Grouted into Bedrock Brace High Retaining Walls". Construction Methods, April, pp. 172-176.

Kemp, E. L., Brezny, F. S. and Unterspan, J. A. (1968): "Effect of Rust and Scale on the Bond Characteristics of Deformed Reinforcing Bars". J. Amer. Conc. Inst., *65* (9), pp. 743-756.

Klein, K. (1974): "Draft Standard for Prestressed Rock Anchors". Symposium on Rock Anchoring of Hydraulic Structures, Vir Dam, November 7-10, pp. 86-102.

Klopp, R. (1970): "Verwendung Vorgespannter Felsanker in Geklüftetem Gebirge aus Ingenieur Geologischer Sicht". Der Bauingenieur, *45* (9), pp. 328-331.

Koch, J. (1972): Reply to FIP questionnaire.

Koziakin, N. (1970): "Foundations for US Steel Corporation Building in Pittsburg, Pa". Civ. Eng. and Pub. Works Review, Sept. pp. 1029-1031.

Lang, T. A. (1961): "Theory and Practice of Rock Bolting". Trans. Am. Inst. Min. Eng., *220*, pp. 333-348.

Larson, M. L., Willette, W. R., Hall, H. C. and Gnaedinger, J. P. (1972): "A Case Study of a Soil Anchor Tie-back System". Proc. Spec. Conf. on Performance of Earth and Earth Supported Structures, Purdue Univ., Indiana, *1* (2), pp 1341-1366.

Laurikainen (1972): Reply to FIP questionnaire.

Littlejohn, G. S. (1970): "Anchorages in Soils—Some Empirical Design Rules". Supplement on Ground Anchors, The Consulting Engineer (May).

Littlejohn, G. S. (1972): "Some Empirical Design Methods Employed in Britain". Part of Questionnaire on Rock Anchor Design, Geotechnics Research Group, Department of Engineering, University of Aberdeen (unpublished Technical Note).

Littlejohn, G. S. (1973): "Ground Anchors Today—A Foreword" *Ground Engineering*, *6* (6), pp. 20-23.

Littlejohn, G. S. and Truman-Davies, C. (1974): "Ground Anchors at Devonport Nuclear Complex". *Ground Engineering*, *7* (6), pp. 19-24.

Longworth, C. (1971): "The use of Prestressed Anchors in Open Excavations and Surface Structures". Australian Inst. Mining and Metallurgy (Illwarra Branch), Symposium on Rock Bolting, 17-19 Feb., Paper No. 8, 17 pp.

Losinger and Co. (1966): "Prestressed VSL Rock and Alluvium Anchors". Technical Brochure, Berne, 15 pp.

Losinger and Co. (1972): Reply to FIP questionnaire.

McCalls Macalloy Ltd. (1969): "Macalloy Prestressing Stressing Procedures". Publications 46 (July), 1st Edition, 24 pp.

Macleod, J. and Hoadley, P. J. (1974): "Experience with the Use of Ground Anchors". Proc. Tech. Session on Prestressed Concrete Foundations and Ground Anchors. pp. 83-85, 7th FIP Congress, New York.

McRostie, G., Burn, K. and Mitchell, R. (1972): "The Performance of Tied-back Sheet Piling in Clay". Can. Geotech. J. 9, pp. 206-218.

Maddox, J., Knistler, F. and Mather, R. (1967): "Foundation Studies for Meadowbank Buttress Dam". Proc. 9th Int. Cong. on Large Dams, Instanbul, *1*, pp. 123-141.

Maijala, P. (1966): "An Example of the Use of Long Rock Anchors". Proc. 1st Int. Cong. Int. Soc. Rock Mech., Lisbon, *3*, pp. 454-6.

Mantovani, E. (1970): "Method for Supporting Very High Rock Walls in Underground Power Stations". Proc. 2nd Int. Cong. Int. Soc. Rock Mech., Belgrade, *2*, pp. 157-165

Mascardi, C. (1973): "Reply to Aberdeen questionnaire (1972)".

Mitchell, J. M. (1974): "Some Experiences with Ground Anchors in London". ICE Conference on Diaphragm Walls and Anchors", London (Sept.), Paper No. 17.

Mongilardi, H. (1972): "Consolidamento di una Scarpata Mediante Ancoraggi Pro fondi". Strade, *11*, pp. 669-678.

Morris, S. S. and Garrett, W. S. (1956): "The Raising and Strengthening of the Steenbras Dam". (and discussion) Proc. ICE Pt. 1, Vol. 5, No. 1, pp. 23-55.

Moschler, E. and Matt, P. (1972): "Felsanker und Kraftnessanlage in der Kaverne Waldeck II". Schweizerische Bauzeitung, *90* (31), pp. 737-740.

Muller, H. (1966): "Erfahnungenmit Verankerungen System BBRV in Fels—und Lockergesteinen". Schweizerische Bauzeitung, *84 (4)*, pp. 77-82.

Nicholson Anchorage Co. Ltd. (1973): "Rock Anchor Load Tests: for Sheet Pile Bulkhead for the General Reinsurance Corp. Project, Steamboat Road, Greenwich, Conn., USA". Unpublished Report (8 pp.).

Nordin, P. O. (1968): "Insitu Anchoring". Rock Mech. and Eng. Geol. 4, pp. 25-36.

Ontario Hydro (1972): "Provisional Specification M 388 and Standard Specification M285, Toronto, Canada.

Oosterbaan, M. D. and Gifford, D. G. (1972): "A Case Study of the Bauer Earth Anchor". Proc. Spec. Conf. on Performance of Earth and Earth Supported Structures, *1* (2), pp. 1391-1400.

Parker, P. I. (1958): "The Raising of Dams with Particular Reference to the Use of Stressed Cables". Proc. 6th Cong. on Large Dams, New York, Question 20, 22 pp.

Parry-Davies, R. (1968): "The use of Rock Anchors in Deep Basements", Ground Engineering Ltd., Johannesburgh, South Africa, unpublished paper.

PCI Post-Tensioning Committee (1974): "Tentative Recommendations for Prestressed Rock and Soil Anchors". PCI, Chicago, USA, 32 pp.

Pender, E., Hosking, A. and Mattner, B. (1963): "Grouted Rock Bolts for Permanent Support of Major Underground Works". J. Inst. Eng. Aust., *35*, pp. 129-150.

Phillips, S. H. E. (1970): "Factors Affecting the Design of Anchorages in Rock". Cementation Research Report R48/70 Cementation Research Ltd., Rickmansworth, Herts.

Pliskin, L. (1965): "Ancrages Précontraints dans le Rocher Système Freyssinet". Bull. Tech. de la Suisse Romande, *16* (7 Aug.), pp. 253-7.

Rao, R. M. (1964): "The Use of Prestressing Technique in the Construction of Dams". Indian Concrete Journal, August, pp. 297-308.

Rawlings, G. (1968): "Stabilisation of Potential Rockslides in folded Quartzite in Northwestern Tasmania". Engineering Geology, *2* (5), pp. 283-292.

Rescher, O. J. (1968): "Aménagement Hongrin—Léman—Soutènement de la Centrale en Cavernes de Veytaux par Tirants en Rocher et Béton Projeté". Bull. Tech. de la Suisse Romande, *18* (7 Sept.), pp. 249-260.

Reti, G. A. (1964): "Slope stabilised by Anchored Retaining Wall". Civ. Eng. (NY), *34* (4), pp. 49-53

Roberts, N .P. (1970): "Deformed Bars in Concrete". Concrete (July), pp. 306-8.

Ruttner, A. (1966): "Anwendung von Vorgespannten Felsankern (System BBRV) bei der Erhöhung der Spullersee—Telsperren". Schweizerische Bauzeitung, 77 (47), pp. 773-777.

Saliman, R. and Schaefer, R. (1968): "Anchored Footings for Transmission Towers". ASCE Annual Meeting and National Meeting on Structural Engineering, Pittsburg, Pa., Sept. 30-Oct. 4, Preprint 753, 28 pp.

Schmidt, A. (1956): "Rock Anchors Hold TV Tower on Mt. Wilson" Civil Engineering, 56, pp. 24-26.

Schousboe, J. (1974): "Prestressing in Foundation Construction". Proc. Tech. Session on Prestressed Concrete Foundations and Ground Anchors, pp. 75-81. 7th FIP Congress, New York.

Schwarz, H. (1972): "Permanent Verankerung Einer 30m Hohen Stützwand in Stuttgarter Tonmergel durch Korrosions—Geschützte Injektionsanker, System Duplex". Die Bautechnik, 9, pp. 305-312.

Shchetinin, V. V. (1974): "Investigation of Different Types of Flexible Anchor Tendons (Rock Bolts) for Stabilisation of Rock Masses". Gidrotekhicheskoe Stroitel' Stuo, *4* (April), pp. 19-23.

Soletanche Entreprise (1968): "La Surrelevation du Barrage des Zardezas sur l'oued Saf-Saf". Unpublished Report, 4 pp.

Sommer, P. and Thurnherr, F. (1974): "Unusual Application of VSL Rock Anchors at Tarbela Dam, Pakistan". Proc. Tech. Session on Prestressed Concrete Foundations and Ground Anchors, pp. 65-66. 7th FIP Congress, New York.

South African Code of Practice (1972) "Lateral Support in Surface Excavations". The South African Institution of Civil Engineers, Johannesburg.

Standards Association of Australia (1973): "Prestressed Concrete Code CA35—1973, Section 5—Ground Anchorages". pp. 50-53.

Stocker, M. A. and Sozen, M. (1964): "Investigation of Prestressed Reinforced Concrete for Highway Bridges, Part V: Bond Characteristics of Prestressing Strand". Univ. Illinois Eng. Res. Station, Bulletin 503.

Stocker, M. F. (1973): Reply to Aberdeen questionnaire.

Suzuki, I., Hirakawa, T., Morii, K. and Kanenko, K. (1972): "Developpments Nouveaux Dans les Fondations de Pylons pour Lignes de Transport THT du Japon". Conf. Int. des Grande Réseaux Electriques à Haute Tension, Paper 21-01, 13 pp.

Thompson, F. (1970): "The Strengthening of John Hollis Bankhead Dam". Civ. Eng. (NY), *39* (12), pp. 75-78.

Universal Anchorage Co. Ltd. (1972): "Report on Rock Anchor Tests for Frigate Complex HM Dockyard, Devonport". Report No. 189, Universal Anchorage Co. Ltd., Egerton Street, Farnworth, Bolton, unpublished.

Universal Anchorage Co. Ltd. (1972): "Ground Anchors Simplify Excavation for Telephone Exchange Foundations". Contract Journal, April 6.

Voves, B. (1972): Reply to FIP questionnaire.

Walther, R. (1959): "Vorgespannte Felsanker". Schweizerische Bauzeitung, *77* (47), pp. 773-7.

White, R. E. (1973): Reply to Aberdeen questionnaire.

Williams, A. F. and Muir, A. G. (1972): "Stabilisation of a Large Moving Rock Slide with Cable Anchors". Report of County Roads Board of Victoria, Melbourne.

Wolf, W., Brown, G. and Morgan, E. (1964): "Morrow Point Underground Power Plant". Source unknown.

Wosser, T. and Darragh, M. (1970): "Tie-backs for Bank of America Building Excavation Wall". Civ. Eng. (NY), *40* (3), pp. 65-67.

Wycliffe-Jones, P. J. (1974): "Personal communications.

Zienkiewicz, O. C. and Gerstner, R. W. (1961): "Stress Analysis of Prestressed Dams". Proc. ASCE, Journal of the Power Division, *87* (PO1), Pt. 1, pp. 7-43.

Part 2: Construction

INTRODUCTION

IRRESPECTIVE OF THE care and conservatism applied to the design§ of an anchor system, thoughtless or careless constructional procedures can cause rock anchors to fail at very low loads. The majority of failures seem to be related to the grouting stage although some bond failures have clearly been due to poor tendon preparation. On a few occasions the drilling and flushing techniques may have been incorrect. Fortunately, failures have not occurred too often and these have usually been highlighted at the stressing and testing stage.

It is significant that although the technology of drilling and grouting can be highly complex, site techniques on the whole are left to skilled and experienced specialists, and close on-site inspection by supervising engineers has been relatively uncommon to date. Thus, rock anchoring after 40 years is still regarded as an art. Whilst it is appreciated that the highly variable ground conditions encountered in practice, giving rise to a large number of construction techniques, add to the mystique of anchoring, nevertheless it seems that the time is overdue for certain guidelines on construction practice to be presented for consideration by civil engineers.

The second part of this review discusses anchor construction techniques related to drilling, flushing, water testing, tendon preparation and installation, grouting and finally corrosion protection§. Since anchor construction is sensitive to poor workmanship emphasis is placed on quality control and close on-site supervision.

Aspects of anchor stressing and testing will be reviewed in the third and concluding part of this series of articles.

DRILLING

Introduction

In practice drilling rates often dictate anchor production rates and therefore influence in a major way overall costs. As a result major decisions to be taken by anchor specialists before each contract include

(i) The selection of the most suitable and efficient drilling method, and
(ii) The prediction of penetration rates.

With respect to choice of drilling method, the rock type, rate and scale of drilling operations, availability of plant, hole geometry and labour and drilling costs must all be assessed.

The prediction of drilling rates involves careful study of machine characteristics, bit and flushing medium properties as well as rock and borehole parameters. It is considered that a prior knowledge of drilling rates provides a sound basis for evaluating the feasibility of planned operations and for selecting alternative operational procedures if necessary.

The range and selection of drilling equipment and methods are described briefly, together with guide information on the prediction of drilling rates. The latter is perforce qualitative, simply because insufficient research has yet been conducted —or published—on the determination of "rock drillability indices". Drilling tolerances are mentioned in relation to current rock anchor practice.

Drilling methods

The major mechanical drilling systems in use are rotary, percussive and rotary percussive. Each system is characterised by the manner in which the bit attacks the rock, and a simple comparative analysis of the mechanics of various drilling systems can often reveal the inherent limitations of each and indicate the most promising system for a specific type of rock. For example a rock of high compressive strength, regardless of its abrasive properties, is likely to respond well to the crushing/chipping action of a percussion bit. On the other hand, a rock classified as hard because it is highly abrasive, but which is weakly bonded, may respond to percussive action more like a ductile material than a brittle one. For such a rock a percussion bit would do inferior work compared with a wear-resistant rotary drag bit. A current rule of thumb for the applicability of drilling methods for different rock categories is based on the resistance of rock to penetration, as shown in Table I.

Rotary drills

A rotary drill imparts two basic actions through the drill rod and bit into the rock —(i) axial thrust (a static action), and (ii) rotational torque (a dynamic action).

The resultant force applied to the rock is increased until rock fracture is induced and each machine has a point where an optimum axial thrust interrelated with the available torque can achieve a maximum penetration rate for a particular rock. Operating below the optimum thrust decreases the penetration and imparts a noticeable polishing or grinding action to the bit. Operating above the optimum thrust requires high rotational torque, and stalling of the machine is likely.

In general, rotary drills have a higher torque output than either percussive or rotary-percussive drills and require higher thrust capabilities. Types of machines and operating practice are described in detail in a US Army Report [1964].

Where specified, most core drilling is carried out using diamond bits which are available in two main forms—(a) "Surface set" bits with individual diamonds set in a metal matrix, and (b) "Impregnated bits" with fine diamond dust incorporated in a matrix.

The diamonds used for the surface set bits vary in both quality and size. Choice is governed by the rock to be drilled, but it can be summarised that "the harder the rock, the smaller the size and the higher the quality of the diamonds". Dixon and Clarke (1975) give specific recommendations on size of diamonds in bits related to type of rock. It is noteworthy that tungsten bits are less costly than diamond bits but are not regarded as suitable for drilling in very hard rocks.

When drilling with surface set diamond bits, Paone *et al* [1968] have shown that the most significant parameters affecting penetration rates are thrust and rotation speed of the drill, and the rock compressive strength, hardness, and quartz content.

Diamond drilling is not commonly employed in anchoring, partly for economic reasons, and partly due to the smoothness of the hole it creates, thereby leading to poorer rock-grout bond characteristics. Borehole roughness is undoubtedly increased by using percussive methods, but to date this does not appear to have been quantified.

For anchor construction in soft rock formations, such as stiff-hard clays and

TABLE I. APPLICATION OF DRILLING SYSTEMS

Method	*Resistance to penetration of rock*			
	Soft	*Medium*	*Hard*	*Very hard*
Rotary-drag bit	X	X		
Rotary-roller bit	X	X	X	
Rotary-diamond bit	X	X	X	X
Percussive	X	X	X	X
Rotary-percussive	X	X	X	

(After Paone, Under and Tandanand, 1968)

TABLE II. DRILLING METHODS AND EQUIPMENT RELATED TO GROUND CONDITIONS

Basic method	Percussive	Percussive	Percussive	Rotary	Rotary
Drill string	Standard coupled rods, separate anchor	Coupled rods also act as anchor	Coupled drill tubes and rods used simultaneously from same drive adapter. Atlas Copco Overburden Drilling method	Coupled flight augers	Standard rotary drilling tubes
Drilling machine	Wagon drill with drifter or crawler drill with independent rotation drifter. Compressed air powered.	Wagon drill with drifter or crawler drill with independent rotation drifter. Compressed air powered.	Special independent rotation drifter mounted on heavy wheeled chassis or crawler. Compressed air powered.	Standard auger drill capacity of torque and thrust dependent on hole size and depth. Diesel/ hydraulic power. Chassis powered wheel or crawler designed for drilling of shallow angle holes. Wheeled or skid mount possible.	Rotary rod drill or diamond drill. Performance about 2.7m.kN torque, 50kN thrust 0-500 r.p.m. Diesel/hydraulic power. Chassis powered wheel or crawler designed for drilling of shallow angle holes. Wheeled or skid mount possible.
Anchor	Muti-wire strand or single bar.	Special coupled rods	Multi-wire strand and single bar.	Multi-wire strand most common. Single bar also possible.	Single bar most common as in Bauer system. Multi-wire strand possible where ground is self-supporting.
Flushing medium	Normally air but water could be used.	Invariably water but air occasionally useful.	Water. Air used very rarely.	None	Water. Air used very rarely.
SUITABLE STRATA	Self-supporting rock only. Few metres of overburden possible with aid of stand pipe.	All materials.	All materials provided drill tubes are uncoupled when rock is encountered and drilling continued alone with rods.	All self-supporting soft material such as clay and chalk. Not rock. Not collapsible material such as sand and gravel unless casing is used.	All soft materials such as clay, sand and gravel. Also soft and medium rocks. Not hard rock.

(Modified after Mawdsley, 1970)

marls, augers are often employed. They fall into three broad categories:

(1) standard continuous flight augers for normal open/hole drilling,
(2) continuous flight augers with hollow couplings to permit water, bentonite or cement grout to be pumped into the bottom of the hole, and
(3) hollow stem augers with a removable centre bit to facilitate sampling through the centre of the auger during the drilling stage, and subsequently to permit homing of the tendon prior to withdrawal of the auger. Augers are available which can accept the standard U4 sampler tube, and on occasions this drilling method can be very attractive from a quality control point of view.

In general, a wide range of drill bits is available from auger tool manufacturers but experience is required in making the correct choice in practice. For example, a tungsten tipped finger bit is normally suitable for moderate to hard formations such as hard shale, siltstone, and soft decomposed sandstone whilst a fishtail bit is often ideal for boring clean holes through soft shale and stiff/hard clay.

Percussive drills

Percussive drills penetrate rock by the action of an impulsive blow, usually from a chisel or wedge-shaped bit: repeated application of a high intensity short duration force crushes or fractures rock when the blow is sufficiently large. Torque, rotational speed, and thrust requirements are significantly lower for percussive systems than they are for rotary or rotary percussive systems.

Hammer drills, in which the hammer remains at the surface, are used for drilling holes up to 125mm in diameter. Down-the-hole tools, (DTH) in which the hammer is always immediately above the bit, are used mainly for hole diameters ranging from 120 to 750mm.

Penetration rates of percussive drills are shown by Ryd & Holdo [1956] to be proportional to the rate at which energy is supplied by the reciprocating piston.

Rotary-percussive drills

These drills impart three actions through the drill bit:

(*i*) axial thrust of lower magnitude than that of a rotary drill,
(*ii*) torque, lower than a rotary drill but much higher than a percussive drill, and
(*iii*) impact.

The rotation mechanisms may be powered by the impact mechanism or by a separate motor, and the mechanism of rock failure is considered by White [1965] to combine the characteristics of both rotary and percussive mechanisms.

Choice of drilling method

The method of drilling is chosen primarily with respect to

(*a*) the type and capacity of the anchor, and hence the diameter and depth of the hole,
(*b*) the nature of the rock material and mass,
(*c*) the borehole surface roughness requirements,
(*d*) the accessibility and topography of the site,
(*e*) the availability and suitability of the flushing medium, and
(*f*) the drilling rate.

A guide to the choice of drilling method is given by Mawdsley [1970] who considers that in the majority of projects the most important factors affecting choice are the type of anchor and the strata to be drilled (see Table II). Parker [1958] writes that for holes up to 100m dia. and 60m in length percussive methods are preferable for most rock conditions. For deeper holes, which put a severe strain on percussive equipment, or poorer ground conditions, rotary methods are recommended. McGregor [1967] summarises in general terms the relation between rock type and diameter as shown in Fig. 1, and emphasises the differences (see Figs., 2 and 3) when drilling in soft friable rocks and variable strata. Where the rock has alternating hard and soft (collapsible) zones the use of a rotating eccentric bit has proved a successful innovation in recent years since it underreams the rock permitting the use of a uniform size of casing, as opposed to the more traditional use of telescopic casing which gradually reduces in size with increasing depth.

It is noteworthy that one of the disadvantages of the down-the-hole hammer (DTH) was illustrated recently at Muda Dam in Malaysia where two very expensive hammers were jammed at depth. Normally, the down-the-hole hammer is less prone to jamming than the ordinary percussive drill but when it does, the financial consequences are greater.

Drilling equipment

Irrespective of the method of drilling, there are certain desirable characteristics which are common to most rigs used in ground anchoring work. For instance, Mawdsley [1970] recommends the following items.

The rig should have powered traction so that it can easily be moved and positioned for each hole. When site floor conditions are bad the rig should be mounted on crawler tracks. An exception to the above is when the rig is mounted on another piece of equipment which is itself movable, for example, a floating pontoon.

The centre of gravity of the rig should be as low as possible as many anchor holes are drilled at shallow angles. The necessary drilling thrusts cannot be applied safely unless the rig is stable.

The rig should be capable of drilling at any angle from horizontal to vertical and should be able to perform as many drilling methods as possible e.g. rotary and auger.

In the view of the authors, the following practical aspects may also merit consideration:

Noise: It is noticeable that there has been a recent swing away from the use of percussive or rotary percussive drills, to rotary drills in built-up areas. This is primarily due to noise restrictions and a noise level of 75dBA at 15m is now specified in urban areas. In 5-10 years it is anticipated that rotary percussive drifters will be banned in built-up areas. In future planning therefore it is recommended that consideration should be given to hydraulically powered rigs.

Nevertheless, whilst percussive drills continue to be employed it is important for engineers to appreciate that exposure to high noise levels, usually above 90dBA, for extended time periods can produce physiological damage to the ear. On many construction sites, particularly in the UK, warnings of this potential hazard to drillers seem in the main to go unheeded.

Versatility: All rigs should be designed to accommodate a rotary head, rotary percussive, drifter, vibrodriver and down-the-hole hammer. Where high production is required, mechanical handling of drill rods and casing could be advantageous and use of drill racks, rod-changing units and hydraulic positioners merits consideration.

Prime movers: All prime movers to operate rigs should be "built-in" to give a compact, independent unit. For the vast majority of anchor applications a power supply of 50-60 h.p. is considered sufficient.

Mast movements: A sub-mast is required capable of rotating 90 deg. in elevation i.e. vertical to horizontal. The main mast, attached to the sub-mast through a turntable/sliding carriage, should be capable of rotating 180 deg. in plan.

The ability to (*a*) position the toe of the main mast at the hole location, (*b*) hold the main mast at any level from 0-2m above the ground is considered important.

Hoist and feed rating: Bearing in mind possible use of vibrodrivers in the future to cope with unconsolidated ground overlying rock, a maximum feed rate of 10m/min may be desirable. A satisfactory hoist rate is 3m/min.; acceptable hoist capacity = 35kN; and acceptable feed capacity = 25kN.

Ideally, pressure gauges giving a measure of torque and feed capacity during drilling should be incorporated in the rig. These gauges could be monitored by an experienced driller or engineer to highlight changes in the strata, and thereby improve quality control.

Exhaust pollution: In the future, attempts should be made to design and specify prime movers which emit "clean" exhaust.

In spite of the above recommendations, it is noteworthy that for anchors installed directly into rock the traditional wagon drill with a percussive hammer may still provide the most economical solution in some circumstances.

In general, the correct choice of a drilling method and machine for an anchoring project is a critical factor in the eventual successful completion of a project and therefore the greatest care should be exercised in making that choice.

Drilling rates

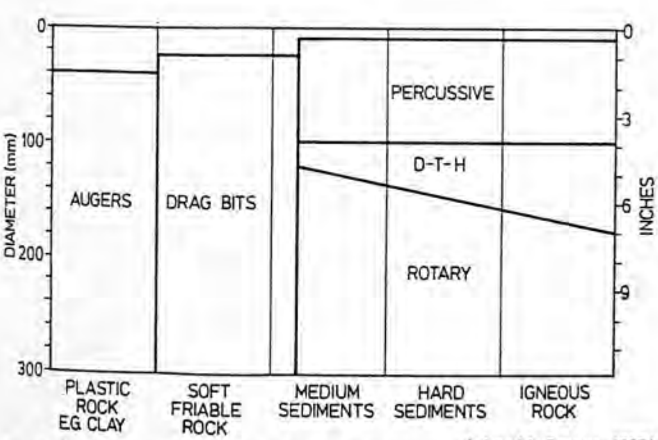
Since the rate of drilling holes in rock depends on the nature of the material drilled and the drilling machine, it is desirable to have as much knowledge as possible on both the rock and the machine.

Regardless of origin, all rocks may possess complex secondary structures, banding or foliation, and the degree of fracturing and weathering, and bedding of the rock mass can affect the physical properties and the drillability of the rock. Consequently, although average or typical properties can be established for sound, unweathered specimens of rocks, in practice each site tends to be evaluated individually, and purely geological classifications of rocks offer little help in grouping rocks according to drillability. On the other hand classifying rocks on the basis of their physical properties, such as compressive and tensile strength, Young's modulus, scratch and impact hardness, toughness and others, is a major factor in establishing a suitable drillability scale. Nevertheless, no definite conclusion has been reached as to which are the most useful physical parameters to determine, and no single property correlates perfectly with drilling rate, although rock compressive strength remains a popular and useful parameter in the hands of the specialist.

Most recently, van Ormer [1974] has attempted to relate penetration rate to rock mass and material properties, and considers texture (porous to dense fine), hardness (1-10 on the Moh scale), breaking characteristics (brittle to malleable) and geological structure (solid to laminated). In each case the first named in the range sustains a faster drilling rate than the other extremes. Table III summarises the data pertaining to hardness, and the drilling rate for various rocks relative to 1.0 (for solid, homogeneous Barre Granite) is shown in Table IV. The latter table does not take into account the secondary structure of the rock mass—the influence of which, it is claimed, is best determined from experience. Differences between measured and predicted drilling rates based on physical properties of the rock are probably due to the ever present variation of these properties throughout the length of hole. Although rock material and mass anistropy is known to affect drillability, little work has been carried out to quantity its influence. In view of its importance however some effects are sumarised by van Ormer in Table V.

Whilst solid formations should provide good drilling, seamy, broken formations induce slow rates as tedious, careful supervision is necessary to avoid loss of flushing capacity, loss of drill string, and bit sticking.

From the standpoint of the anchor contractor, one of the simplest procedures at present for predicting penetration rates, particularly in percussive and rotary-percussive drilling, is to determine the coefficient of rock strength of the rock to be drilled. The test, which was first described by Protodiakonov [1962] and subsequently modified by the U.S. Bureau of Mines (Paone *et al*, 1968), consists basically of fracturing rock samples by impacting them with a falling weight. The resulting damage is measured by screening the broken sample. The test is relatively simple, does not require elaborate equipment and



(after McGregor, 1967)

Fig. 1. Preferred methods of drilling different classes of rock and at different hole diameters. Depth of hole generalised

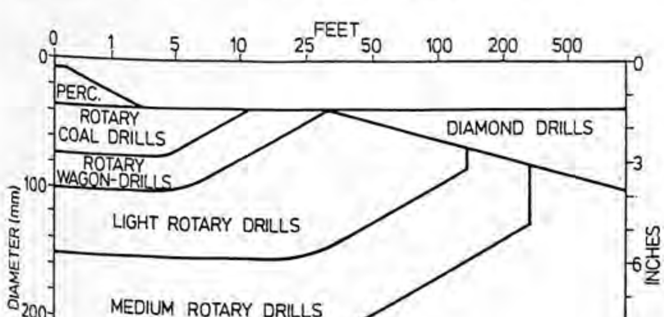


Fig. 2. Preferred methods in soft friable rocks (after McGregor, 1967)

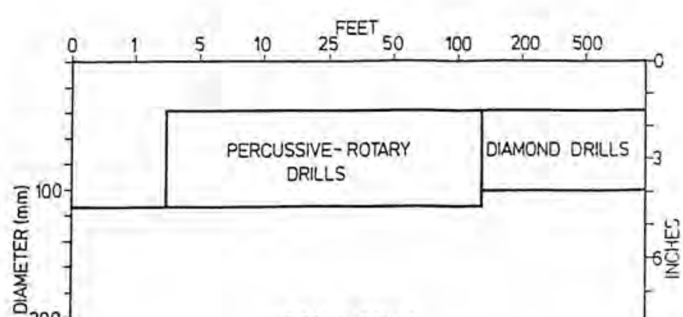


Fig. 3. Preferred methods in variable strata (after McGregor, 1967)

one man can carry the apparatus into the field and make several determinations in one day. Good results have been obtained in correlating field penetration rates with the coefficient of rock strength for rotary-percussive drills (Unger & Fumanti, 1972) and for percussive drills (Schmidt, 1972).

One major disadvantage, however, in using only the coefficient of rock strength for prediction is that no account is taken of drill power and machine characteristics.

Penetration rates, particularly for percussive drills, are a function of the air pressure supplied to the drill, the condition of the drill and the type and condition of the bits. Other technical factors such as flushing medium and bit diameter are also important, but to date have received little investigation. Since these parameters are usually difficult to measure with any degree of precision, especially in the field, it is not surprising that some discrepancies between calculated and measured rates are evident.

As a result it is now widely appreciated that a step that considers energy output of the drill must be included to further refine the procedure for predicting drilling rates. Whilst much work remains to be tackled Paone *et al* [1968] in a detailed account have already suggested a method of estimating penetration rate based on the quantity of energy required to cut a unit volume of rock and the energy output of the drilling system. It is also noteworthy that Paone *et al* [1969] have suggested using the coefficient of rock strength to determine the energy required to remove a unit volume of rock.

Flushing

It is vital to remove particles from the bit quickly and efficiently. Energy expended on grinding such fragments obviously cannot be used for hole production; comminution of the fragments also increases wear of the bit.

Commonly used flushing media are air, water or "mud"—usually being a colloidal suspension of bentonite in water. A distinction is also drawn between normal and reverse flush circulation. In the former, the flush is introduced via the rods and bit, and returns to the surface between the rods and the hole wall. In the latter, the opposite situation occurs.

Of the media listed, air is probably the most efficient scavenger, water the best coolant and mud the best lubricant. Air is the commonest fluid used for surface drilling with percussive machines, and with drag-bit and roller-bit rotary drilling in quarries. Air is best used in dry ground, although it can be used in very wet conditions provided ample air is available but offers little advantage over wet drilling. Underground, and in confined spaces generally, air is unsatisfactory unless used in reverse circulation, because of the health hazard of dust particles. Rock-drilling in confined spaces such as tunnels is therefore normally restricted to wet or suction drilling, the latter being one example of reverse circulation.

Water flushing is the standard method used for drilling in sticky ground (i.e. where there is a small inflow of water into the hole from the rock, only sufficient to combine with the cuttings to form a paste or where there are clayey layers), for drilling under the water table at depth, and for diamond drilling. The quantity of water used is not excessive—usually less than 4 litres per minute for conventional

TABLE III. HARDNESS OF SOME ROCKS AND MINERALS

Mineral or rock	*Hardness*	*Sratch test*
Diamond	10.0	
Carborundum	9.5	
Sapphire	9.0	
Chrýsoberyl	8.5	
Topaz	8.0	
Zircon	7.5	
Quartzite	7.0	
Chert	6.5	Quartz
Trap rock	6.0	Quartz
Magnetite	5.5	Glass
Schist	5.0	Knife
Apatite	4.5	Knife
Granite	4.0	Knife
Dolomite	3.5	Knife
Limestone	3.0	Copper coin
Galena	2.5	Copper coin
Potash	2.0	Fingernail
Gypsum	1.5	Fingernail
Talc	1.0	Fingernail

Percussive drilling

Rotary drilling

(After van Ormer, 1974)

TABLE IV. DRILLING CHARACTERISTICS OF COMMON ROCKS

Characteristics	*Comparative drilling speed*	*Rock material*
Hardness— 1-2 Texture—Loose Breakage—Shatters	1.5 and up	Shales Schist Ohio Sandstone Indiana Limestone
Hardness— 3-4 Texture—Loose grained to granitoid Breakage—Brittle to shaving	1.0 to 1.5	Limestone Dolomites Marbles Porphyries
Hardness— 4-5 Texture—Granitoid to fine grained Breakage—Strong	0.6 to 1.0	Granite Trap Rock Most fine-grained igneous Most quartzite Gneiss
Hardness— 6-8 Texture—fine grain to dense Breakage—Malleable	0.5 and less	Hematite (fine-grained, grey) Kimberly chert Taconite

(After van Ormer, 1974)

Barre Granite is used as the standard for determining a comparative drilling speed of 1.0 because of its even texture, hardness, and consistent drilling.

TABLE V. EFFECT OF ROCK MASS STRUCTURES ON DRILLING RATES

Rock mass	*Nature of fractures*	*Drill rate*
Massive	——	Fast
Stratified	Perpendicular to drill rod; > 1.2m apart, clean	Fast Medium
Laminated	Perpendicular to drill rod; < 1.2m apart, clean	Medium
Steeply dipped	Small angle to drill rod, 1.2m apart, clean	Slow Medium
Seamy	Various inclinations to drill rod; close, open fractures	Slow

(After van Ormer, 1974)

anchor hole drilling. In spite of this wet drilling is often regarded as a messy and inconvenient method, whilst mud flushing is considered expensive and thought to require a great deal of preparation. Mud flushing is not common in rock anchor construction although it has been used successfully in France for open hole drilling through silts and sands overlying rock.

The type of flush employed may in cases improve the efficiency of hole formation. In weakly cemented sandstones for example, water flushing widens and cleans the hole and ensures a better bond at the grout/rock interface. However, in rock strata liable to deterioration from water action such as marls and chalks, water flushing where necessary should be kept to a minimum.

Regardless of the supposed efficiency of the flushing process, it is usual in anchor construction to leave a "sump" length for debris at the bottom of the borehole. In current practice, 0.3-0.7m is commonly added to the designed borehole length. After each hole has been drilled to its full depth and throughly flushed out in order to remove any loose material, the hole should then be sounded to ascertain whether "fall-in" or "blow-up" of material has occurred and whether it will prevent the anchor tendon reaching the required depth. If satisfactory the top of the hole should then be effectively plugged to prevent debris falling into it.

With regard to the logging of data relating primarily to ground water and flushing medium, it has been shown that local variations in ground conditions, over a few metres, can have marked effects on subsequent anchor performance—especially in soft rocks. Much qualitative data can be obtained on ground conditions by logging drilling rates and the degree of bit blocking, but a more sensitive record is often provided by observing changes in the amount and composition of flush return.

Other data relating to ground water, pressure and permeability can also be readily obtained if close liason is established and maintained with the driller. For example, the following should be noted:

(*i*) the depth at which ground water is first encountered in the hole,

(*ii*) any water added to the hole to assist drilling,

(*iii*) the level of water, and amount and diameter of casing in the boring at the end of the shift, and

(*iv*) the level of water when work recommences.

Alignment and deviation

In the drilling of rock anchor boreholes, it is important to maintain a true, straight hole, terminating in the expected, calculated position. Three causes of errors may be recognised:

(*a*) incorrect setting-up, with the drill pointing in the wrong direction at the start of drilling,

(*b*) misalignment, in which the drill is correctly lined up but the hole is out of line with the axis of the drill, and

(*c*) deviation in which the hole is started in the correct line but subsequently alters direction.

Correct setting-up of a drill is largely a matter of care and a good eye, but should always be aided by the use of a profile and spirit level. The use of a casing or drill rod guide plate at the base of the drill mast is advantageous.

Regardless of cause, misalignment is troublesome and can result in damage to the drill and string as well as causing jamming of the rods. Furthermore, McGregor [1967] notes that the rubbing of the rods on the wall of the hole may dislodge rock fragments whilst the resultant friction—especially in rotary drilling—can increase enormously the torque requirements. Resettlement of the rig when the drilling thrust is relaxed may also be a problem in soft ground and experience indicates that special care is required when drilling from free-floating platforms.

Deviation of the hole during drilling does not normally arise from a single circumstance. It may originate by using too thin rods, from excessive thrust, or by the bit following a fissure or other rock planar structure. Deviation is not usually a serious problem for DTH drills, but is exaggerated by the hole length in diamond drilling.

The above remarks have been primarily related to vertical downward-holes. With angled holes, the rods are apt to lie on the lower side of the hole and this has the effect of upturning the bit slightly. Hence angle holes often—but not invariably—tend to follow a shallow curve away from the vertical.

Wherever possible, drill holes should be planned so that they intersect the major rock discontinuities at as high an angle as possible. If this rule is not observed, then it is probable that a proportion of the holes will tend to deviate along the planes of the rock. In mica-schist for example, holes will follow the mica defined schistosity if originally drilled at, say, a 5 deg. angle to it.

It is therefore essential to set-up the drill with the greatest care and precision and to monitor the progress of the hole. It becomes progressively more difficult and costly to alter the direction of the hole after drilling has proceeded beyond a few metres.

Little guidance on maximum permitted deviations has appeared, but tolerances of 0° 28′ (Parker, 1958), 1° 10′ (Eberhard and Veltrop, 1965) and 0° 43′ (Littlejohn and Truman-Davies, 1974) may be compared with the less rigorous maximum of 2° 30′ permitted by the South African Code. Contractors often quote average deviations of 1 in 50 i.e. 1° 09′ and tolerances are usually relaxed in the fixed anchor zone. (Tolerance is measured as a deviation of anchor hole from the specified centre line divided by the length of drill hole).

A common method of inexpensively checking the deviation in a vertical hole is to lower a torch down it and observe by how much, if at all, the face is obscured at various depths. Alternatively, the deviation may be more accurately checked at regular intervals using a single-shot photographic or continuous reading borehole inclinometer.

TABLE VI. LIMITING FLOW RATES WHICH HAVE BEEN RECOMMENDED OR EMPLOYED TO DETERMINE THE NEED FOR WATERPROOFING

Source	Flow rate (recommended or employed)	Flow rate (gal/ft/min/atm)*
GERMANY		
(Brunner)	5 litres/min	0.00091
(Bomhard & Sperber)	1 litre/metre/min/10 atm	0.00670
SWITZERLAND		
(Moschler & Matt)	1 litre/metre/min/10 atm	0.00670
(Buro)	0.08 gal/ft/min/10 atm	0.00800
SOUTH AFRICA	0.075 gal/100 ft/min	0.00013
NEW ZEALAND	0.01 gal/ft/min	0.00167
AUSTRALIA	0.001 gal/in. dia/ft/min	0.00067
USA	0.001 gal/in. dia/ft/min (hole full plus 5 p.s.i.)	0.00063
MALAYSIA	0.003 gal/ft/min	0.00050
UK		
(Parker)	0.01 gal/ft/min	0.00167
(Falmouth)	0.25 gal/min	0.00021
(Devonport)	1 litre/metre/min/10 atm	0.00670

*Imperial units have been used in this table since the majority of references relate to contracts carried out prior to transfer to S. I. Units

WATER TESTING AND WATERPROOFING

On completion of drilling, the anchor borehole must be tested for "watertightness", since subsequent loss of grout from around the tendon in the fixed anchor zone is of prime importance in relation to efficient load transfer and corrosion protection. Reasonable threshold values for water loss or gain must be assessed which, when exceeded, dictate the need for waterproofing. In practice, it has been generally accepted that cement is not suitable for the treatment of fissures which are less than 250 microns wide although recent experimental studies suggest that the lower limit is closer to 160 microns for Ordinary and Rapid Hardening Portland Cements.

The authors believe that a logical approach is to establish the minimum width of fissure which will permit flow of cement at low pressure. The water flow per atmosphere which is caused by a single fissure of this width may then be specified as a threshold value which dictates the need for waterproofing.

It may be estimated that a single 160 micron fissure under an excess head of one atmosphere gives rise to a flow rate 3.2 litres/min (Littlejohn, 1975). It is therefore suggested that this order of flow should be considered as a reasonable threshold for water loss when Ordinary Portland Cements are employed in the neat cement grout. A lower fissure width of 100 microns gives a flow rate of 0.6 litres/min/atm. and this may be a more realistic threshold for minimal penetration when fine-grained cements are employed.

With regard to rock anchor practice, the magnitudes of water flow which have been permitted in various countries to date are listed in Table VI.

Clearly, great care must be taken in the interpretation of limiting flow rates, with particular regard to the length of section being tested. To avoid serious misinterpretation, it is recommended that permissible flow rates should be quoted simply in terms of litres/min/atm, no reference being made to flow per unit length of hole or stage.

In general, it is considered that water tests carried out over sections e.g. the fixed anchor, with the aid of packers are preferable to rate-of-fall tests carried out under atmospheric pressure from the surface, since more detailed information can be obtained over specific locations. Packer testing is not essential however and on many occasions rate-of-fall tests can be carried out more cheaply and quickly. In these situations packer testing may only be warranted if the acceptable water flows are exceeded.

On the practical side the hole must be thoroughly flushed with clean water from the bottom before testing, and during the test it may be of value to reduce the level of water in any adjacent holes so that any interhole connections may be more easily detected.

From a review of current world practice, it is clear that water-testing is not a routine procedure and even when waterproofing is carried out, generally acceptable water flows have not been established for rock anchor grouting. As a result, the following recommendations are presented for consideration.

(a) Waterproofing is required if leakage or water loss in an anchor borehole exceeds 3.0 litres/min/atm. The duration of the test should not be less than 10 minutes and in terms of the Lugeon coefficient the above flow is equivalent to 10*L*.

(b) Where there is a measured outflow or water gain (under artesion conditions) care should always be taken to counteract this flow by the application of a "backpressure" during the grouting stage. If the flow cannot be stabilised in this way waterproofing is required, irrespective of the magnitude of the water gain.

(c) Permissible flow is related to "excess head". Therefore the position of the water table in relation to the section being investigated must be established so that the driving or excess head inducing flow at the section may be calculated accurately. In fine fissures high applied pressures may induce turbulent flow, create high pressure gradients and open up the natural fissures. As a principle, changes in the local environment should be minimised and therefore the applied pressure inducing flow should be as small as possible.

(d) The flow rates in (*a*) are minimum values since they all pertain to single fissures. Clearly, larger limiting flow rates are acceptable if a number of fissures (thickness $<$ 160 microns) exist. This situation however must be confirmed by close examination of the borehole interface using a camera or close circuit television and/or multipacker injection tests.

In order to waterproof the hole against water loss, grout should be tremied into the hole from the base upwards. After a period of time (usually from 6 to 24 hours) the hole is redrilled and the water test repeated. The anchor construction proedure may only continue when the waterproofing criteria are satisfied. If the pregrouting is not successful on the first one or two occasions, then pressure grouting may be required to force the grout into the fissured rock mass and thereby stabilise the borehole wall against subsequent redrilling.

TENDON

Storage and handling

Longbottom and Mallett [1973] make a number of sound recommendations regarding this topic, on the basic assumption that anchor tendons must be protected against mechanical damage and severe corrosion on site.

Tendons must not be dragged across abrasive surfaces or be accessible to weld splash. Bars should be stored in straight lengths, and wires and strand in coils of diameter at least 200 times that of the tendon diameter. Kinked or twisted wire should be rejected, since experience has shown that bond and load/displacement characteristics can be adversely affected.

To avoid damage to protective sheathing, the ends of the tendon should be treated, after cutting to size, to remove very sharp edges. With respect to bars, care should be taken to protect the threads. Superficial damage to the threads can often be repaired by means of a file, but it is usually impracticable to recut or extend a bar thread on site because of the hardness of the steel.

Ideally, steel for anchor tendons should be stored indoors in clean, dry conditions. If this is impossible, the steel may be left outdoors for several months without serious corrosion, provided it is stacked off the ground and completely covered by a waterproof tarpaulin. Although the tarpaulin should completely cover the steel it should be fastened so as to permit circulation of air through the stack.

The humidity of the air, allied to possible atmospheric pollution (industrial and marine) is the major cause of corrosion during storage. There would appear to be little problem if the relative humidity is always less than 70 per cent, but severe corrosion occurs at levels in excess of 85 per cent. The worst conditions are experienced in marine tropical areas, where the average rate of corrosion is about three times that in a heavy industrial area in the UK. In such areas, wrappings should be impregnated with a vapour phase inhibitor powder, and in this case air through flow must be prevented.

Although it is known now that normal rusting actually improves the bond to grout, flakey, loose rust must be completely removed, and tendons which are severely pitted, particularly in the case of small diameter multi-wire strands, or at threaded sections of bars, should be rejected.

Fabrication

With respect to bar anchors, all threads must be thoroughly cleaned and lightly oiled, and it is important to ensure that bars are properly screwed into couplers, and that full thread engagement is obtained in nuts and tapped plates. To minimise corrosion, the tendon should not be left ungrouted for long after cleaning, especially if paraffin has been used.

Anchors with multi-strand or multi-wire tendons usually require more time for fabrication. If the strand is supplied already coated in PVC, then great care should be taken to degrease the intended fixed anchor length effectively, using solvents such as acetone, trichloroethylene or paraffin. Some contractors specify unravelling of the strand to facilitate effective cleaning; the wires are afterwards returned to their correct lay. This basic method is recommended and an efficient, if somewhat time-consuming refinement to the system has been developed by U.A.C. Ltd., who introduce small ferrules on to the central wire prior to relaying the strand. This produces nodes in each strand and undoubtedly increases the resistance to the strand-grout failure. Altrnatively, to eliminate the laborious and inherently risky job of attempting to completely remove a graphited bituminous grease which has been designed to resist easy removal, a machine has recently been developed (Littlejohn and Truman-Davies, 1974) to grease each individual strand and apply a protective plastic sheath only over the free length where it is required.

The fixing and location of spacers and centralisers must be done with care and precision, especially in the fixed anchor length where the tendon is usually formed into a roughly circular configuration with steel or polythene spacers and wire bindings. Attention should also be given to the bottom of the tendon and use of a sleeve or nose cone which will minimise the risk of tendon or borehole damage during homing is recommended.

Homing

Any method can be used provided that it will ensure that the tendon is lowered at a steady controlled rate. It is recommended that for heavy flexible tendons of total weight in excess of 200kg, mechanically operated pulleys or large drums (Littlejohn and Truman-Davies, 1974) be used to gradually unreel the tendon into the hole. It has been found that 200t capacity anchors, weighing about 16kg/m, are the largest that can be handled in restricted areas, e.g. dam crests, without elaborate handling equipment.

If the borehole grout is preplaced under water, grout dilution can occur if the tendon is lowered too quickly. The use of drums from which to unwind the tendon into the hole is preferable to the use of cranes, or (for vertical anchors) manhandling, as both these methods often create sudden bending of the tendon which may damage both steel and protection.

Immediately prior to homing, the tendon should be carefully inspected, and in certain situations the efficiency of the centraliser/spacer units may be judged by carefully withdrawing the tendon—prior to grouting—to observe damage or distortion, or the amount of smear.

In general the choice of the best methods of storage, handling, fabrication, and installation of anchor tendons is wholly an exercise in commonsense. Prestressing steel and fittings are valuable stores, and should be treated as such on site.

GROUTS AND GROUTING

The most common and lowest basic cost material used for fixing and protecting rock anchors is neat cement grout. The influence of certain grout parameters on bond development has already been noted (Littlejohn and Bruce, 1975) and information on grout mixes and grouting procedures as used in rock anchor practice is now reviewed, and recommended quality controls are discussed.

Grout composition

Cement

The type of cement used will obviously vary from contract to contract as dictated by ground conditions and the installation programme. Thus, while Ordinary Portland Cement (Type I) may suffice in many cases, a sulphate-resisting (Type II), or a rapid hardening variety (Type III) may be required. In Britain, Ordinary and Rapid Hardening Cements must comply with BS 12 and High Alumina Cement with the relevant clauses of BS 12 and 195. It is recommended that high alumina cement be restricted to short term test anchors, in view of the use of high water cement ratios often necessary for pumpability.

Since cement surface areas (and therefore particle sizes) are normally controlled by specification, the most likely deterioration in cement quality may be due to age or poor storage, when partial dehydration or carbonation may lead to particle agglomeration and reduction in post-mix hydration. Although large sizes may be removed by sieving, it is likely that better control may be exercised by insisting on fresh cement, and by careful storage. Ideally cement should not be stored on site for more than one month, and must be kept below 40 deg. C, under cover. Cement should be used in order of delivery.

Water

Water which is suitable for drinking (except for the presence of bacteria) is generally considered suitable for cement grout formulation. Water containing sulphates (> 0.1 per cent), chlorides (> 0.5 per cent), sugars or suspended matter e.g. algae must be considered technically dangerous. High chloride content should be particularly avoided where the steel tendon is in contact with the grout.

Where there is some doubt as to the quality of the water, a test on the lines of BS 3148 "Tests for water for making concrete" may be carried out.

Water-cement ratio (w/c)

The proportion of water to cement in a grout rather than the quality of water is the most important determinant of grout properties. Excess water causes bleed, low strength, increased shrinkage and poor durability. The extent to which these (and also fluidity) are related to the w/c ratio of an OPC grout is shown in Fig. 4.

Table VII has been prepared to illustrate a range of w/c values recently used or recommended throughout the world, for neat cement grouts. Most ratios are between 0.40 and 0.45 which gives a grout with sufficient fluidity to be pumped and placed easily in small diameter boreholes, and yet retains sufficient continuity and strength after injection to act as a waterproofing and/or strengthening medium.

Admixtures

The use of inert "fillers" such as ground quartz, limestone dust, fine sand, clay, and even sawdust, has long been common, particularly in Europe. The resultant mixes have been used primarily to waterproof

TABLE VII. RANGE OF W/C RATIOS RECENTLY USED OR RECOMMENDED

W/C ratio			Remarks	Source
Ordinary Portland	Rapid Hardening	High Alumina		
0.4			Anti-bleed admixture permitted	Maddox et al (1967)
	0.45		U.A.C. anchors	Anon (1969)
	0.3		Anchors in Keuper Marl	Cementation Co Ltd. (1969)
0.4	0.4	0.4	Recommendation	Mullett (1970)
0.4–0.45	0.4–0.45		Fluidifier permitted	Buro (1970)
≃ 0.45			"Intrusion aid" permitted	Thompson (1970)
0.4			—	Gosschalk and Taylor (1970)
	0.46		Expanding agent required	Barron et al (1971)
0.38–0.43			Recommendation	Conte (1971)
0.4	0.4	0.35	Recommendation	Littlejohn (1972)
<0.45	<0.45	<0.45	Recommendation	C.P. 110 (1972)
0.35–0.4			Admixtures permitted	Bureau Securitas (1972)
<0.45	<0.45		Recommendation	Mascardi (1972)
0.4	0.4		Recommendation	Ontario Hydro (1972)
⩽0.5			Recommendation	South African Code (1972)
0.36–0.44			Recommendation	Stocker (1973)
0.38–0.44	0.38–0.44		Recommendation	Hilf (1973)
0.45	0.45		Recommendation	White (1973)
0.4–0.5			Expanding agents or retarders permitted	Golder Brawner (1973)
	0.45		—	Littlejohn and Truman Davies (1974)
0.4–0.45	0.4–0.45	0.4–0.45	—	Ground Anchors Ltd (1974)

TABLE VIII. COMMON CEMENT ADMIXTURES FOR ANCHOR GROUTS

Admixture	*Chemical*	*Optimum dosage (% of cement by weight)*	*Remarks*
Accelerator	Calcium Chloride	1—2%	Accelerates set and hardening
Retarder	Calcium Lignosulphonate	0.2—0.5%	Also increases fluidity
	Tartaric acid	0.1—0.5%	May affect set strengths
	Sugar	0.1—0.5%	
Fluidifier	Calcium Lignosulphonate	0.2—0.3%	
	Detergent	0.5%	Entrains air
Expander	Aluminium powder	.005 — .02%	Up to 15% expansion
Anti-bleed	Cellulose Ether	0.2—0.3%	Equivalent to 0.5% of mixing water
	Aluminium Sulphate	up to 20%	entrains air

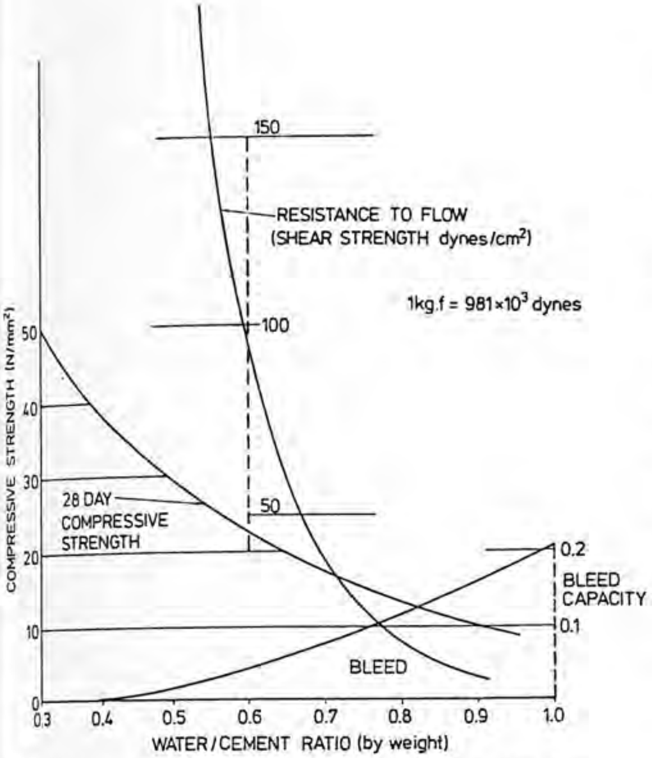


Fig. 4. (above). Effect of water content on grout properties

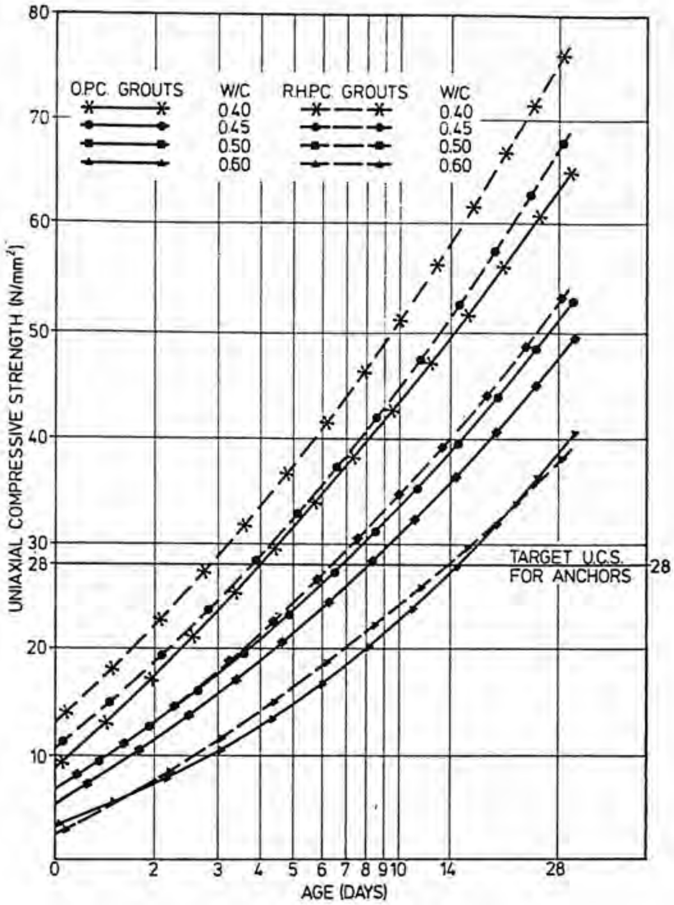


Fig. 5. (right). Gain in strength of set grouts

or consolidate boreholes prior to redrilling —a role in which neat cement grouts may be uneconomic. Such fillers are seldom employed however in grouts used for tendon bonding.

With respect to anchor grouts, chemical admixtures have often been employed particularly those to prevent shrinkage, to permit a reduction of the w/c ratio while ensuring fluidity, to accelerate or retard setting, and to prevent bleeding which in turn discourages corrosion. Table VIII lists common types of admixtures employed in grouts. Care should be taken however to ensure that the basic grout materials are compatible and except under carefully considered and controlled conditions, different types of admixture should not be included in the same grout. For example, admixtures such as calcium chloride should not be used with sulphate resisting, supersulphate or high alumina cement. Calcium chloride can also corrode steel in contact with the grout and to avoid this potential hazard the authors recommend that use of this admixture should be banned in anchor grouting.

Geddes and Soroka [1964] conclude that aluminium-based expanding agents improve grout workability while increasing the "confined" compressive strength (i.e. where expansion has been restrained on setting). This latter effect increases the bond capacity of the grout which has been illustrated experimentally by a reduction in bond transmission length. Leech and Pender [1961] have also favoured the use of aluminium powder in an amount of 0.005 per cent by weight of cement and they stipulate that bleeding was also inhibited. Pender *et al* [1963] advocate that a 2 per cent expansion of grout volume is desirable: this figure can be attained by using 0.002-0.005 per cent aluminium powder. However, a warning on the use of aluminium powder has been sounded by Moy [1973]. While confirming the findings of Leech and Pender, he emphasises the great sensitivity of grout mix properties to the amount of aluminium powder added —and its efficiency of dispersion and mixing. For example, slightly larger dosages of powder can give a markedly spongy and crumbly grout.

In Britain, some success has been achieved with calcium lignosulphonate as a grout fluidifier, when used at a concentration of 0.03 per cent by weight of cement. In this way a pumpable low w/c grout—0.3—can be satisfactorily produced for anchors, installed in water sensitive marls and shales.

In rock anchoring, grout bleed seldom receives consideration despite its great importance in corrosion protection. Antibleed additives based on cellulose ethers have been successfully employed (e.g. Maddox *et al*, 1967: 0.2 per cent by weight of cement), although slightly lower grout crushing strengths and higher initial grout viscosities result. They found from field tests that the final mix gave negligible settlement at the top of the tendon, and complete grout cover free from fissures or water filled lenses. Commercial products are readily available and Celacol M5000DS and Methocel 65HG4000 are recommended for consideration. Dosages are normally expressed as a percentage of the mixing water, rather than the cement, and vary according to the viscosity grade of the material. For example Celacol M5000DS and equivalent grades are normally added at a rate of 0.4-0.5 per cent by weight of water.

In general, considerable international agreement on the use of admixtures is apparent. For instance, the use of chloride bearing compounds is banned in Britain, Germany, France, Switzerland, Italy and the United States. CP 110 stipulates that admixtures may be permitted only when "experience has shown that their use improves the quality of the grout". Nitrates, sulphides, and sulphates are also banned, and total expansion should not exceed 10 per cent.

In Germany, the use of any additive is rare, and only those which increase workability of the grout are employed. Mascardi [1973] states that in Italy moderately expanding additives are used but air entraining or metallic expanding types are banned, as are rapid hardening agents.

Hilf [1973] considers that sand, and antibleed and expansion agents are acceptable in the United States, whereas White [1973] discourages the use of anything other than cement grouts. A very comprehensive survey of grout admixtures has been prepared by the American Concrete Institute [1971], and is recommended to the interested reader.

In summary, it may be concluded that the use of admixtures for grouts is still very much an art. Even the manufacturers have relatively little practical experience of their use for rock anchoring. Consequently, whenever a new mix is designed or adopted, the following must be recorded:

(*i*) water/cement ratio,
(*ii*) admixture concentration,
(*iii*) flow reading (through flowmeter, flow cone or viscometer),
(*iv*) crushing strengths (two cubes each) at 3, 7, 14 and 28 days, and
(*v*) notes on amount of free expansion or shrinkage, bleed and final setting time.

Even if the design is satisfactory, unless the cement and admixture is delivered on site ready mixed, very careful supervision of the grout mixing personnel is essential. Hence the general indication is that admixtures should be used only where absolutely necessary.

Grout crushing strength

Some grout properties have already been alluded to—pumpability, slight expansion on setting, a minimum w/c, and resistance to bleeding. In addition, the crushing strength requirements are of fundamental importance.

CP 110 states that grout used for prestressed concrete work must have a compressive strength in excess of 17N/mm² at 7 days. Normally higher strengths are specified for stressing, and Littlejohn [1972] finds that 28N/mm² is favoured in Britain. A survey of world practice reveals that this figure is in fact common in many countries, although Mascardi (Italy) feels that 35N/mm² is necessary (w/c $<$ 0.45) whilst PCI [1974] recommends a minimum value of 24N/mm².

It is noteworthy that Thompson [1970] describes how satisfactory anchors were installed at the John Hollis Bankhead Dam, Alabama, with a grout of 28 day strength of 17N/mm². However, this serves as a reminder that low strength grouts are only acceptable in rigid, competent rocks where "arching" mechanisms of the particulate grout can be mobilised, whereas high strength grouts are necessary in soft, yielding rocks.

In general a major disadvantage of cement grouts, even when admixtures are used, is the time required for the grout to develop full operational strength (see Fig. 5). Other problems are associated with its low tensile strength, brittle nature, and installation in adverse conditions. However, where time and bond length are not restricting factors—especially where large annular volumes are involved—no economic substitute to cement grout is available.

Mixing

The authors recommend that to ensure good practice, the following fundamental points should be observed.

1. The cement (and fillers where applicable) must be measured by weight.
2. Water should be added to the mixer before the cement (and fillers) and any admixtures should be added with great care usually during the latter half of the mixing time.
3. Although the mixing time depends on the type of mixer, the total time should not be less than 2 minutes according to CP 110.
4. Mixing by hand is to be strongly discouraged.

The equipment must be able to produce grout of uniform consistency, and should have two drums or tanks: one for mixing, the other for storage and delivery. In order to avoid heating of the grout, slow agitation only is permissible in the storage tank.

Rate of shear during mixing is particularly important and it is noteworthy that the most common type of grout mixer, comprising an impellor in a tank, combines two major effects which influence the efficiency of mixing—circulation and fluid shear. These are essentially incompatible, since a large slowly rotating impellor will produce a high circulating capacity and low shear rate, while a small rapidly rotating impellor will yield a high shear rate and low circulating capacity. For cement grouts of low w/c ratio shear rate is a critical factor in mixing and ideally impellor speeds of 1 500-2 000 rpm are required. In this connection an ideal type of mixer is the Colcrete double drum mixer which circulates the grout through a centrifugal pump. The grout is recirculated through a zone of high shear with sufficient impact to break down lightly bonded clusters or agglomerates, and provide maximum interdispersion of water and cement.

Where conventional paddle mixers are employed, field analysis indicates that the best results are obtained when the paddles are cut with slots, and where slotted baffle plates are fitted around the perimeter of the tank or drum.

Experience suggests that the actual mixing in the field is generally satisfactory, but that often the strainer between the two tanks is too small or easily clogged. In such cases, unstrained and lumpy grout overflows into the delivery tank and thence into the borehole. In addition exit points should be fitted at the base of tanks to avoid formation of cement cake at the bottom.

The use of rapid "snap-off" couplings permits the quick removal of obstructions which tend to form in bends of flexible pipes or at constrictions. It is noteworthy that rigid steel pipes do not allow the position of the obstruction to be quickly ascertained.

Finally it is an elementary yet important observation that a high standard of cleanliness of grout mixing and pumping equipment is usually associated with simpler and more efficient grouting operations.

Grouting methods

There are basically two distinct modes of anchor grouting, namely by two-stage or single-stage injection.

Two-stage grouting involves first injecting a "primary" mix to effect the bond between tendon and rock. After final stressing, a "secondary" phase is introduced, largely for the corrosion protection of the free length. In the one-stage system, both functions of the grout are simultaneously performed.

In two-stage injections the primary grout may be preplaced or postplaced with respect to the introduction of the tendon. Postplacing can be advantageous when dealing with large tendons and poor "slabby" rock, and is the only choice for very shallow or upwards-inclined anchors.

It is good practice to ensure that the primary grout extends for at least 2m above the designed fixed anchor length. This inhibits crack formation in the proximal end of the anchorage during stressing. Where the primary grout is preplaced, the tendon should be homed within 30 minutes of the injection. Even after the tendon has been correctly homed, problems have been experienced with grout/tendon bond development and opinions currently differ as to whether the tendon should be left static after homing (FIP, 1973) or vibrated (Standards Association, Australia, 1974).

Secondary grouting is usually accomplished with a mix of the primary composition although Mitchell [1974] recommends that to ensure complete freedom of tendon movement, an American practice of backfilling the free length with sand, sand and gravel, weak grout, or stone chippings, should be adopted.

At the present time the two-stage system is more common in practice, but has certain disadvantages:

(*a*) an additional interface is created at the top of the fixed anchor and is considered to be a prime target for corrosive agencies,

(*b*) the exact quantity and quality of the vital primary batch is difficult to judge without careful checking, and

(*c*) a two-stage method is intrinsically more time-consuming and laborious.

Single-stage methods are free from these problems. However it must be noted that unless the free tendon length is meticulously greased before sheathing all the load applied at the head will not be transmitted to the intended anchorage zone due to friction in the free anchor length.

On the practical side, before grouting commences, it is advisable to check the airtightness of all pipes involved, and the tremie pipe—flexible and usually 12-25mm in diameter—should be blown and flushed with water.

Both hole and tendon should be thoroughly water-flushed from the bottom upwards for at least 10 minutes prior to grouting. If the grout is to be postplaced the tremie pipe may be conveniently incorporated in the tendon, but terminating at least 150mm from the foot.

Grout should be tremied at a steady rate, and the pipe, if not incorporated in the tendon, may be withdrawn slowly during the operation. At no time must either the end of the tube be lifted above the surface of the grout or the level of grout in the pump storage tank be allowed to drop below that of the exit pipe, otherwise air may be drawn into the grout placed.

In the single-stage method or during the secondary phase of a two-stage injection, grouting should continue until grout of the same composition as that mixed has been emerging from the hole for at least 1 minute.

The Australian Code recommends that it is preferable to provide a standpipe during grouting so that grout shrinkage will occur in this pipe and not in the hole. In any case it is traditionally regarded as good practice, particularly in relation to dams, to "top up" anchor holes where necessary, a few days after the major grouting operation.

Grouting pressures

The general conclusion amongst specialist contractors is that high grout pressures are completely unnecessary for successful anchors in intact rock but useful for anchors in badly fissured rock. Analysis of the data received suggests that grouting

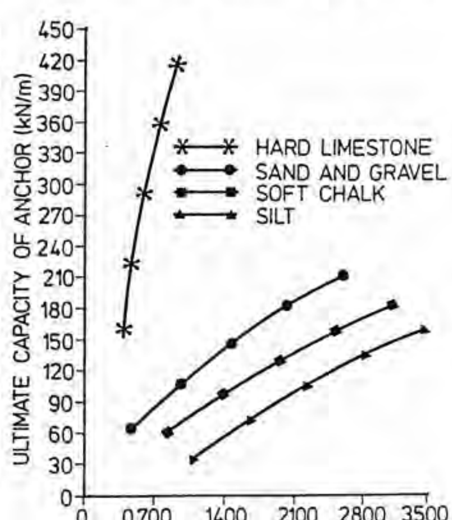


Fig. 6. Anchor resistance related to grouting pressure (after Soletanche, 1970)

TABLE IX. GROUTING PRESSURES RECOMMENDED AND USED FOR ROCK ANCHORS

Grouting pressure (N/mm^2)			Source
Upwards sloping anchors	Downwards sloping anchors	Either	
		3.5	Holz (1963)
		0.5—2	Broms (1968)
	0.1		Buro (1970)
		0.69	Irwin (1972)
0.3	0.2—0.3		Conti (1972)
0.207—0.345	0.345—0.522		Koch (1972)
		< 2.5	F.I.P. (1973)

pressures normally lie in the range 0.28—0.70N/mm².

A.T.C. Ltd. (1973) experimented with different injection pressures when grouting anchors in chalk and concluded that there is no real benefit in employing grouting pressures of the order of 4N/mm². Practical and economic considerations often set the maximum grouting pressure at 3N/mm², and for the subsequent contract anchors, a pressure of 2N/mm² was used. Fig. 6 illustrates the relation of grout pressure to averaged anchor capacity, claimed for Soletanche "Tamanchor" (I.R.P.) system anchors. Others grouting pressures which have been used in practice are shown in Table IX.

It can be summarised that permissible grouting pressures are largely a matter of conjecture. They depend on the circumstances and geology of the site and "rules of thumb" should be proven at each site by in situ water or grout pumping tests, before being put into general use. As a starting point the most common rule for permissible pressure appears to be 0.023 N/mm² per metre of overburden.

Quality control

Variations in grout properties arise from three principal causes:

(a) inadequate mixing,

(b) variations in grout materials quantities and quality, and

(c) apparent variations arising from the testing procedure.

In order to obtain a satisfactory basis for grout mix design it is essential, prior to any anchor contract, that methods of storage, batching, mixing and testing of materials be rigidly defined and adhered to.

Mixing of cement grouts

Contact between cement and water leads to a prolonged sequence of exothermic reactions leading to complete hydration and ultimately final setting of the cement-water paste. There are normally four stages to this reaction:

(i) an initial highly exothermic reaction lasting 5-10 minutes,

(ii) a dormant period lasting up to 2 hours during which there is a low rate of heat evolution,

(iii) an increasing rate of reaction leading to final set after 6 or more hours, and

(iv) a continuing decreasing rate of reaction after setting.

During the dormant period, a cement grout should maintain a consistent physical state, when its properties can be measured and predicted. In order to obtain this consistent physical state when the cement is added to the water, sufficient mechanical agitation must be induced to fully disperse the cement grains. To achieve this and at the same time avoid false sets, mixing for a period of 5-10 minutes is normally required. Under most field applications this should be achieved by agitation during storage, and pumping and placement after mixing.

Variation in grout quality

Variations of material quantities and qualities from those specified in the grout design are largely a reflection of the standards of site organisation, equipment and supervision and as such are difficult to quantify. Neville [1963] has attempted however to define the quality of concrete mixes by relating the coefficient of variation of cube strength to the degree of site control, and it is considered that these standards (Table X) could apply to cement grouts.

TABLE X. VARIATION OF CONCRETE STRENGTHS

Degree of site control	Coefficient of variation = *Standard deviation* / *mean strength*
Best laboratory control	5
Best site control	10
Excellent	12
Good	15
Fair	18
Bad	25

(after Neville, 1963)

The best possible results obtainable when site control' approaches laboratory precision should have a coefficient of variation of 10. This will require:

(a) Obtaining cement, fillers and chemical admixtures from a reliable source,

(b) Storage of cementitious materials under dry and constant conditions,

(c) Accurate determination and monitoring of moisture content of fillers,

(d) Use of cement in fresh condition,

(e) Weigh batching of all materials (meter for water is acceptable),

(f) Controlled water/cement ratio,

(g) Adequate mixing rate and time of mixing,

(h) Immediate pumping and injection of grout after mixing, and

(i) Rigid supervision of all operations.

In practice the cement grout is expected to fulfil the dual role of fixing the anchor to the rock and protecting it against corrosion, often in "aggressive" environments. It is surprising, therefore, that the only common method of checking quality is by crushing a nominal number of cubes after the anchors have been constructed. Furthermore, samples are often carelessly taken, or not taken for every anchor.

Additional measurements are therefore recommended which permit the quality of the grout to be assessed before the grout is injected, thereby pre-empting the possibility of potentially expensive and/or dangerous errors occurring.

Measurement of important grout properties

Accuracy of measurement of grout properties is an important factor in determining the variability of grout properties in the field. Some property measurements, such as bleed, have been developed principally as laboratory measurements, for example, Powers float test and the ASTM method (see Powers, 1968). In the field, levels of bleed above 0.5 per cent are relatively easily detected in any sample contained in a wide, low container, and in anchors the actual magnitude of bleed is less important than the fact of its existence.

Laboratory measurements of grout fluidity in terms of shear strength and viscosity are normally carried out with a rotating disc or coaxial cylinder viscometer. Two instruments which are commonly used in the field are the Colcrete flowmeter (which expresses fluidity in terms of horizontal slump) and the Portland Cement Association cone (in terms of flow time). Various specialists and researchers have calibrated these instruments in terms of standard grout parameters e.g. w/c ratio, but for particular grouts it is the authors' view that the most direct information on fluidity is still best obtained from field pumping tests. Nevertheless flowmeter and flow cone data can be useful in assessing efficiency of mixing.

Check measurements of water/cement ratio can be made on site by measuring the specific gravity of the grout using a Baroid mud balance (see Table XI). Hydrometers are not recommended since at low water/cement ratios larger errors are introduced due to the thixotropy and solid structure of the grout.

TABLE XI. CALCULATED SPECIFIC GRAVITIES OF WATER/CEMENT GROUTS

Specific gravity	*Water/cement ratio*
2.10	0.3
1.95	0.4
1.84	0.5
1.74	0.6
1.67	0.7
1.61	0.8
1.56	0.9

In most grouts the hydrogen ion concentration is of value as an indicator of chemical contamination; pH is therefore another parameter which can be a useful control in practice and where a large number of site tests are planned, a battery or mains pH meter can be used.

With regard to the strength development characteristics of cement grouts, Fig. 5 indicates the curing times required by a range of grout mixes made from Ordinary Portland and Rapid Hardening Cement to attain the minimum strength of 28N/mm² before stressing. The results were obtained from 150mm grout cubes but 75mm cubes should give reliable results in practice. Care must be exercised, however, when attempting to correlate 75mm and 150mm cubes strengths. On demoulding, the larger cubes are invariably warmer, even when efficiently cured. In addition, the curing water takes longer to influence the centre of the larger cubes. Both these phenomena act to increase the early strength (1-7

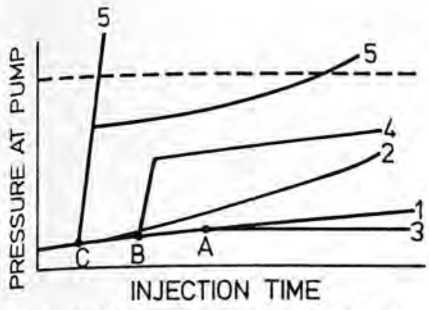


Fig. 7. Idealised representation of various grout injection time-pressure characteristics
(after Longbottom & Mallett, 1973)

days) but tend to depress the later strength of the larger cube sizes.

The recommended controls, including bleed measurements where corrosion protection by grout is vital, can be readily exercised during the actual grouting operation, which ideally should always be carried out the same day the fixed anchor section of the hole is drilled.

To ensure that injection pressures do not cause undue disturbance of the ground, the pump should be fitted with an effective control against pressure build-up. Pressure and pump speed may be considered as one control: the balance between the two is dictated by actual conditions. In this connection pressure gauges fitted with diaphragms are recommended to avoid contact with the grout. Pumping over distances in excess of 150m is strongly discouraged, as this can change the grout properties.

Monitoring the grout pressure during injection can provide useful information about the quality of the grout being pumped, and the efficiency of the operation. Idealised curves (see Fig. 7) for grouting progress are described by Longbottom and Mallett [1973].

Curve 1—Good grout, normal stiffening—"standard" mix.
Curve 2—Gradient greater than standard, possibly indicating that the grout is stiffening too quickly.
Curve 3—Indicates a fracture in the system at time *A*; leaking of the grout indicated by constant pressure.
Curve 4—Indicates partial blockage at *B*.
Curve 5—Serious blockage at *C*, possibly with stiffening. If the maximum pressure is exceeded, grouting should be stopped, and the system flushed.

It is concluded that problems associated with the crucial grouting operation will be eased if the equipment is kept clean and in good repair, adequate supervision and skilled labour is provided, and unnecessary complications (e.g. small amounts of admixture) are avoided. Data relating to the operation should be carefully recorded—w/c, type of cement, and/or additives, type of mixing and pumping equipment, mixing and delivery time, grout fluidity and strength, source and chemistry of mixing water, length of grout line, pressure and quantity of grout injection, air temperature, and the names of the operating personnel. Such data will help to pinpoint reasons for anchor malfunction, should it subsequently occur.

It is strongly recommended that specific gravity checks as well as flow cone or flow meter testing should be used to supplement the results of conventional cube crushing programmes—a retrospective source of data.

CORROSION AND CORROSION PROTECTION

Mechanisms and causes of corrosion

The corrosion of prestressing steel is largely electrolytic and Longbottom and Mallett [1973] list the pre-requisites as (*i*) an electrolyte having interfaces with (*ii*) an anode and a cathode which also have (*iii*) direct metallic interconnection.

The electrolyte is usually aqueous, and a mere surface film is adequate. Reactions are initiated as a result of inhomogeneities or impurities in the steel or grout, or by the presence of chlorides or other salts in solution.

The cathode has a higher electrical potential relative to the electrolyte than the anode, which is normally lower in the electrochemical table. The more common elements are arranged as follows:

(stainless steel)
Na Al Zn Cd Fe Ni *Sn Pb* H_2 *Cu Ag* O_2
(passivated *Al*)
Anodic <————————> Cathodic

The general rule is that electrolyte action will be more severe between electrodes which are widely separated in the table than between those which are closer.

There are generally held to be three major mechanisms of corrosion:

(1) **Corrosion by pitting.** Under conditions of chemical and/or physical inhomogeneity in the steel or electrolyte, ionisation will occur at both anode and cathode, constituting a bimetallic cell (Fig. 8a).

(2) **Corrosion involving crack formation under tension** ("hydrogen embrittlement"). This is more a physical corrosion, mainly affecting highly stressed carbon steels. The best known cause of brittleness is nascent hydrogen (Fig. 8b). The cathode reaction:

$$2H^+ + 2e^- \rightarrow H_2$$

is favoured by acid environments, and the hydrogen so produced tends to disrupt the structure of the steel.

From a survey of reports on hydrogen embrittlement it appears that oil quenched and tempered steels are far more susceptible to hydrogen embrittlement than drawn types. There is however no unanimous opinion about the susceptibility of prestressing steel to hydrogen embrittlement in highly alkaline grout.

(3) **Corrosion involving oxygen.** Local concentations of oxygen at a *cathode* act to accelerate corrosion:

$$O_2 + 2H_2O + 4e^- \rightarrow 4OH^-$$

The reaction is favoured by alkaline conditions (see Fig. 8c) and oxygen concentrations at an *anode* lead to the formation of a protective, passivating layer of rust:

$$2Fe^{3+} + \tfrac{1}{2}O_2 + 4OH^- + (n-2)\,H_2O \rightarrow Fe_2O_3\,nH_2O$$

In the alkaline environment provided by a good dense grout, steel is passivated in this way. As Portier [1974] noted, however, rust so formed is easily removed by the circulation or infiltration of water, thus leading to progressive dissolution of the steel.

There are two main chemical controls on these reactions—water, and electrochemical potentials.

(*i*) *Water.* Regardless of the type of corrosion, it can only occur in an ionic medium, and, under natural conditions, water is the most widespread bearer. The renewal of water increases the risk, while humidity is an even more dangerous parameter. The factors are closely interdependent: the supply of oxygen; the intensification of the microcell effect by the formation of a cathode at the water/air interface; and the action of hydrogen embrittlement.

(*ii*) *Electrochemical potentials.* With respect to Fig. 9, in Region I there is formation of ferrous ions, and generalised dissolution. Hence it would appear that to avoid corrosion, it suffices to remain within pH 8.5—13.5, i.e. in the range created by grouts. However

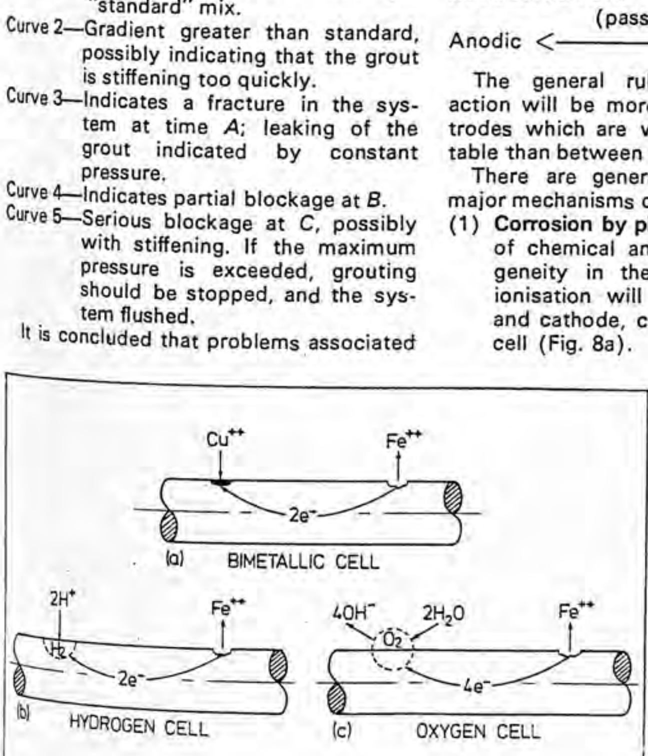
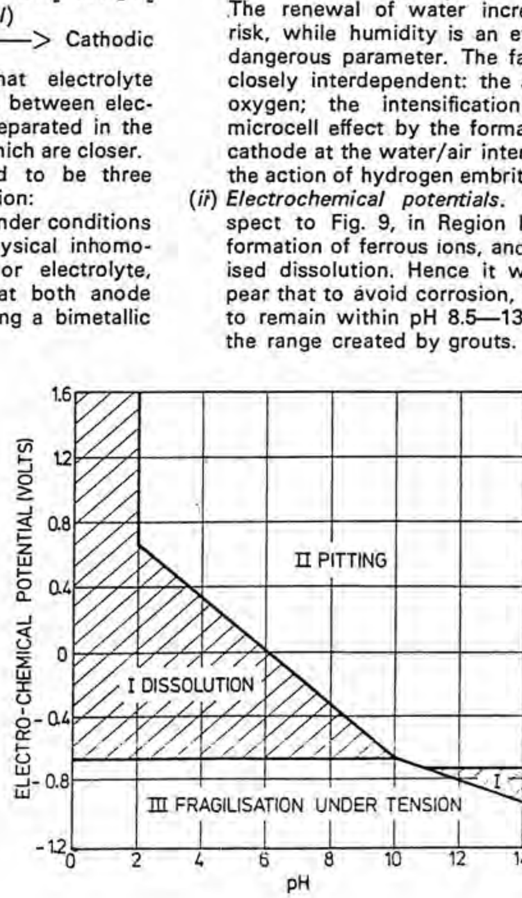


Fig. 8. (above). Idealised representation of three major modes of steel corrosion
(after Longbottom & Mallett, 1973)

Fig. 9. (right). Relation of types of corrosion to pH and electrochemical potential
(after Caron, 1972)



this protection is very inadequate since it is known that (Region II), despite the passivating action of Fe_2O_3 formation, there may be corrosion by pitting under the influence of ions such as Cl^- when present in the cement. Also, Region III, corrosion with crack formation may occur.

Thus, although no domain is absolutely safe, the risks of corrosion can be simply reduced by:

(a) creating a pH environment of 9-12 in the grout. Chloride, sulphide, sulphate and carbonate ions all tend to lower the pH of the grout, enhancing electrolytic action,

(b) avoiding the possibility of harmful ions, e.g. chlorides, sulphides, sulphites, contacting the steel surface,

(c) selecting steels with low susceptibility to corrosion under tension, and eliminating from grouts anions which favour the passage of hydrogen e.g. SH^-, NO_3^-, CN^-, and

(d) preventing, as far as possible, the circulation of water.

Corrosion is thus aided by porous grout or concrete, and Rehm [1968] has found that in certain cases a cover of 25mm is insufficient. Therefore in anchors the porosity of the grout, and not simply its thickness of cover, should be stipulated.

Prestressing the steel may accelerate the rate of intensity of corrosion, although the elastic and strength properties of non-stressed steel are similarly affected. Quenched and tempered steels are far more susceptible to stress corrosion than cold drawn carbon steels of the type used in the UK for strand. Stress corrosion is more acute than ordinary corrosion for three main reasons—

(1) Stressing and releasing, if repeated, constantly destroys any protective oxide film,

(2) Stressing facilitates the development of micro-fissures, and

(3) Prestressing steel is, *a priori*, more susceptible than ordinary steel.

There is an increasing realisation that the failure of highly stressed materials under the influence of corrosion may be complex, and as yet it is impossible to be specific as to the conditions which will or will not give rise to stress corrosion. The only safe principle to follow is that if conditions could be dangerous—as in permanent ground anchors—then the whole design of the system should be orientated towards ensuring complete protection of the prestressing steel.

Whilst many of the problems of corrosion protection in prestressed systems in general are not present in ground anchor works Portier [1974] has pointed out that there are a number of corrosion problems specific to ground anchors, namely—

(i) Risks due to uplift pressure. Anchors may serve to stabilise foundation rafts, generally located underneath the water table and hence liable to uplift pressure. The slightest orifice serves as a drain cock and water may then flow along the tendon. This is particularly serious for strand anchors, although Soletanche Co Ltd., now use an epoxy pitch which is claimed to penetrate the tendon core and ensure absolute imperviousness, and British strand manufacturers appear confident about the penetration of corrosion resistant greases used at present with polypropylene sheathing.

(ii) Sealing. There are two contrary trends—either the risks are considered great and attempts made to protect the steel (as described below) or the risks are thought minimal and the tieback is immersed in the cement grout.

The latter method is older, and about 90 per cent of existing permanent anchors appear to have been so constructed, and whilst no failures have been observed no systematic records of corrosion have been taken.

(iii) The free length. This usually consists of a steel sleeve, or more often a plastic sleeve, which may easily be rendered impervious at the join. The tendon which passes inside is already protected by this sleeve, and also has additional protection from the cement filling the space between the sleeve and borehole wall. A problem is to prevent the formation of longitudinal paths (along which water can flow) along the axis of the sleeve. Various substances have been used for filling as cement grout does have certain disadvantages, and recent trends are towards synthetic substances which can impregnate the core of the tendon, while being at the same time flexible.

(iv) The head. While often being the most susceptible zone, it invariably receives least attention. It is vulnerable for many reasons: grout settling affects it, leaks emerge through it, mechanical and heat stresses create electric couples out of proportion with those of the sealing, and, it is in contact with the potentially corrosive atmosphere. One possibility is to ensure that on completion of the final grouting operation the top anchorage is completely encased in concrete. This however pre-empts the possibility of restressing the anchor at a future date. An alternative is to enclose the top anchorage in a steel or rigid plastic cover filled with grease or bitumen, again after final stressing. The PCI Recommendations [1974] advocate the "asphaltic painting" of all top anchorage hardware.

Classification of groundwater aggressiveness

It has been demonstrated that certain ions, both in the grout and in the groundwater initiate and sustain corrosion. Quantitative limits on aggressivity of environments have been drawn up by Bureau Securitas [1972] and FIP [1973]. Ground and mixing waters classed as aggressive are:

(1) Very pure water. It is termed aggressive if the concentration of CaO is less than 300mg per litre. Such waters dissolve the free lime and hydrolyse the silicates and aluminates in the cement.

(2) Acid waters. If pH is less than 6.5, they are considered aggressive as they may attack the lime of the cement. They are normally industrial waters, water with dissolved carbon dioxide, or water containing humic acids.

(3) Waters with a high sulphate content. These react with the tricalcium aluminate of the cement to form salts which disarrange the cement by swelling. Among these are (*a*) selenious water, with a high content of dissolved sodium sulphate, and (*b*) magnesian water, with a high content of dissolved magnesium sulphate. Waters with these salts are classed as very aggressive when the concentration of the salts exceeds 0.5g/litre for selenious water and 0.25g/litre for magnesian water. It is noteworthy that these values refer to stagnant water, and for flowing water the concentrations are 40 per cent of the above values.

Recommendations also refer to the aggressivity of the grout towards the steel of the tendon. In order to avoid "stress corrosion" of the tendon, the cement must not have a chlorine content, from chlorides, which exceeds 0.02 per cent by weight, and sulphur from sulphides, which exceeds 0.10 per cent by weight. These are provisional values only.

Any admixtures used must likewise contain no elements aggressive towards the steel or cement, and so the use of calcium chloride is forbidden.

Degree of protection recommended in practice

Methods used to protect rock anchor tendons reflect the following factors; the intended working life, the aggressiveness of the environment and the consequences of failure due to corrosion. Systems should be capable of effective protection against mechanical damage, as well as chemical, and should not therefore be impaired by the operations of fabrication, installation or stressing.

Three different situations can be delineated for the purposes of discussion, but in practice their distinction is often difficult.

(a). Temporary anchors in a non-aggressive environment. It is normally safe to assume that the cement grout will protect the fixed anchor length and the specified

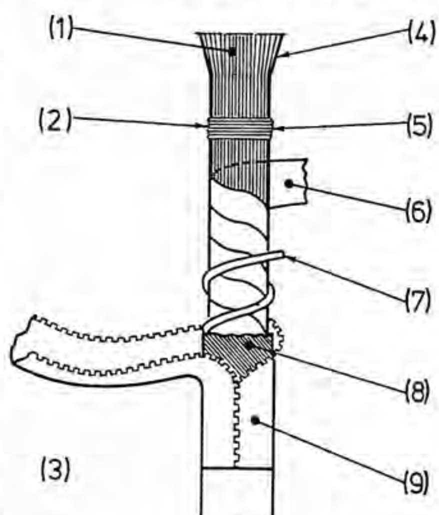


Fig. 10. Corrosion protection of tendons at Cheurfas Dam (after Cambefort, 1966)

(1) 630 5mm galvanised steel wires
(2) Average diameter of bound cable: 15cm
(3) Average diameter of finished cable: 20cm
(4) "Flint-kot" coating
(5) Bindings every 50cm
(6) "Flint-kot" coated tarpaulin
(7) Aloe rope
(8) Plastic mattress (mixture of grease and bitumen)
(9) Tarpaulin sheath with zip fastener
(10) Cement stopper sealing wires and tarpaulins
(11) Scraped wires
(12) White metal point
(13) Sealing tube

minimum cover for this type of anchor is not normally very large. Figures quoted by various engineers indicate a range of values for this type of anchor from 5 to 20mm. The need for some form of protection over the free length is not now disputed although it is not always enforced. White [1973] states that in the United States often no protection is provided even for a temporary anchor with a working life up to three years. Generally, however this is not so, and a combination of grease and tape is common practice. FIP [1973] recommend a grout cover of at least 5mm.

(b) Temporary anchors in an aggressive environment. The fixed anchor zone can still be satisfactorily protected with a good quality grout cover. However, the minimum cover now becomes more important and Matt [1973] has recommended that a minimum value of 30mm should be guaranteed. Greater importance is also placed on the assurance that this grout is not cracked: if this possibility cannot be excluded some additional protection system should be included. Protection of the free anchor length is still only a single protective system in most cases, plastic sheathing or greased tapes being the usual solution although grout or other protective coatings are also possible. The risk of failure due to corrosion of the tendon is greatly reduced if components of diameter in excess of 7mm are used. A minimum grout cover of 5mm is again recommended by FIP at present.

(c). Permanent anchors. These should always have protective systems designed assuming an aggressive environment: environmental changes during the life of the anchor cannot be anticipated and the possibility that the anchor will be exposed to an aggressive environment cannot therefore be excluded. It is now widely held that permanent anchors should be provided with a double corrosion protection system. It is recommended that, as far as possible, the protection should be made and checked under workshop or equivalent conditions. The chosen protection system should not adversely affect the handling of the tendon or the behaviour of the bond.

Over the fixed length, there is always grout cover, but it is common to provide an additional coating. The coating may be a high strength epoxy or polyester resin but any suitable material which has a proven resistance to the existing aggressivity and does not adversely affect the bond may be used. Sometimes it is considered sufficient to pregrout the anchor zone and inspect it before homing the tendon. The cover recommended by FIP is 5-10mm minimum.

The free anchor length is similarly doubly-protected. Grease packed plastic sheaths fitted under factory conditions are becoming a popular method. Various other elastic substances can also be used within a plastic tube; for example, bitumastic compounds like buto rubber, or greased tapes used within the sheath. The annular space outside the plastic tube is normally cement grouted but in some cases bitumen enriched grout is used.

Corrosion protection systems employed in practice

Numerous systems of protection against corrosion have been used—and in some cases abandoned—for rock anchors. Fundamentally, a distinction is drawn between systems for pre-protection and post-protection. The former are employed prior to homing, whereas the latter are effected after tendon installation.

With respect to systems of pre-protection sheathing is currently the most common method. PVC sleeving, or water resistant or greased tape is now almost standard protection for rock anchors. Greased tape in particular is easy to handle and apply with a 50 per cent overlap, and although the risk of damage during tendon installation is high, it does form an extremely efficient barrier to chemical attack. The grease should be supple to allow subsequent tendon extension during stressing without causing large friction losses, or being destroyed, and should thoroughly penetrate the tendon. With reference to sleeving, PVC or polypropylene sheathing may now be delivered to site already on the individual steel wires or strands, or it can be introduced in a separate process on site (Littlejohn & Truman-Davies, 1974).

Other pre-protection systems, which are described in an excellent article by Portier [1974] include:

(*a*) coatings providing cathodic protection,
(*b*) cathodic protection by electric current,
(*c*) synthetic, semi-rigid films,
(*d*) rigid synthetic anchor plugs, and
(*e*) metal casings under compression or tension.

Systems of post-protection are also numerous and consist basically of filling in-situ a sleeve over the free length, after tensioning. The substances used range from fluids, such as oils or water containing lime, to bitumens and cement, and the various materials have been described

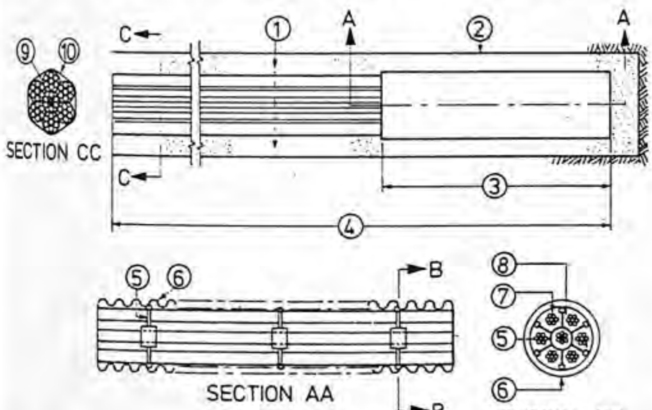


Fig. 11. Cementation Long Life anchor (after Golder Brawner, 1973)
1, Grout 2, Borehole wall, 3, Potted length, 4, Overall length, 5, Central spacers, 6, Corrugated sheathing, 7, Strands with polypropylene sheath removed and degreased, 8, Polyester resin, 9, Strands, each sheathed in polypropylene, 10, PVC adhesive tape binding

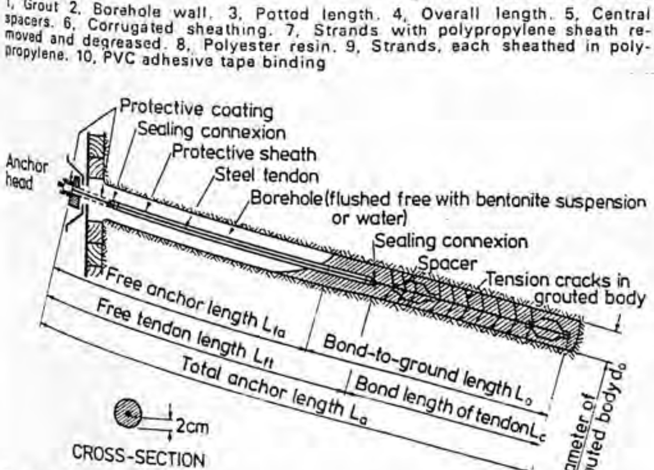


Fig. 12. Temporary anchor construction of Type A (after Ostermayer, 1974)

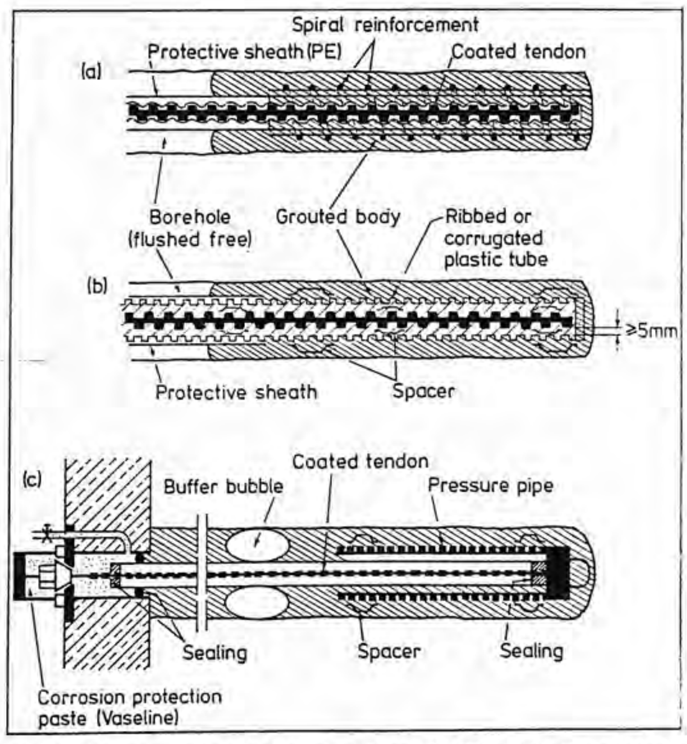


Fig. 13. Permanent anchor constructions; (a) Type A with coated tendon; (b) Type A with ribbed plastic tube; (c) Type B with pressure pipe (after Ostermayer, 1974)

and classified in some detail by Bureau Securitas [1972] according to duration of protection and aggressivity of the surrounding medium.

An interesting illustration of the development of corrosion protection systems is provided by comparing the anchors used at Cheurfas, in 1934 with a modern counterpart, As shown in Fig. 10 in the former, bitumen was liberally employed, and the wires were galvanised (except in the fixed length). A claim (Khaova *et al*, 1969) that about 11 per cent of the total tendon cross-sectional area has been lost due to corrosion in just over 30 years has been discredited recently by Portier [1974].

A sophisticated modern type is the Cementation Long Life Anchor (Fig. 11) in which the polyester resin not only ensures complete protection of the fixed anchor tendon length, but contributes a "deadman" effect to the whole anchorage system. The free length of the tendon consists of strands individually coated in grease and covered by polypropylene sheathing.

Ostermeyer [1974] discusses the classification of the sophisticated bar anchors most commonly used in Germany. For temporary anchors, Type A (Fig. 12) is generally used with only one stage of protection (sheath on the free length and at least 20mm of grout over the fixed length). Ostermeyer emphasises the importance of head protection and recommends that at least one coat of paint be applied to the head and the tendon above the sheathing.

Anchors of Type A (fixed under tension, Figs. 13a and b) and Type B (fixed anchor under compression, Fig. 13c) are used in permanent works. Such anchors have double protection—against both mechanical damage and chemical corrosion.

In Type B, the corrosion protection can be applied and tested under factory controlled conditions without difficulty. The protection on the whole length is examined electrically and then covered with a sheath. As the protection does not transmit load, a relatively elastic material can be used and paste or grease pressed into the annular space between sheath and tendon may be considered adequate.

In Type A, the application of a protection which remains undamaged during construction and stressing is difficult. For the anchors shown in Fig. 13a a synthetic coating is desirable which not only has an excellent bond with steel but, in addition, must also be flexible and strong enough to carry high bond stresses over a long period. When the coating is thick, the danger exists that the fixed anchor will be subjected to high bursting stresses. A spiral reinforcement is therefore provided to resist these stresses.

For the Type A anchor in Fig. 13b, the tendon is inside a ribbed plastic sheath. The annular space is filled with cement. Although this cement will crack, as in all type A anchors, the criterion of corrosion protection is considered to be fulfilled when the cement in the annular space is at least 5mm thick, and the sheath at least 1mm thick. When a ribbed plastic sheath is used, the danger of material fatigue is less than in cases where a protective coating has been directly applied to the tendon (Fig. 13a). The requirements of double corrosion protection are also met at the anchor head.

It is generally concluded that whilst the protection of rock anchors is a serious problem, it does not appear to be a crucial one at present and responsible engineers are clearly corrosion conscious. Nevertheless there is a growing need to establish standards of corrosion protection which will be accepted and used widely by consulting and contracting engineers.

References

ACI Committee 212 (1971): "Guide for Use of Admixtures in Concrete" J. Amer. Conc. Inst. (Sept.) pp. 646-676.
Anon (1969): "Ground Anchors for the World's Largest Ship Dock" *Contract Journal*, (June 19) pp. 850-851.
Associated Tunnelling Co. Ltd. (1973): "Report on Ground Anchor Tests—Thames Flood Prevention". Associated Tunnelling Co. Ltd., Lowton St. Mary's, near Warrington, England (Unpublished).
Barron, K., Coates, D. F. and Gyenge, M. (1971): "Artificial Support of Rock Slopes", Dept. of Energy, Mines & Resources, Mines Branch, Ottawa, Research Report R 228 (Revised), 145 pp.
Bomhard, H. and Sperber, J. (1973) "Der felsverankerte Spannleichbeton—Kragarm der Skiflugshanze Oberstdorf und Betrachtungen zur Anwendbarkeit und Beans pruchbarkeit von Konstruktionsleichtbeton". Beton-und Stahlbetonbau, 5 pp. 107-117.
British Standards Institution (1972): "The Structural Use of Concrete". CP 110, Part 1, BSI, 2 Park St., London.
Broms, B. B. (1968): "Swedish Tieback Systems for Sheet Pile Walls". Proc. 3rd European Conf. Soil Mech. & Found. Eng., Budapest, pp. 391-403.
Brunner, H. (1970): "Praktische Erfahrungen bei den Hangsicherungen In Neuenhof", Strasse und Verkehr, 9, pp. 500-505.
Bureau Securitas (1972): "Recommendations Regarding the Design, Calculation, Installation and Inspection of Ground Anchors". Editions Eyrolles, 61 Boulevard Saint-Germain, Paris-Ve (Ref. TA 72).
Buro, M. (1972): "Rock Anchbring at Libby Dam", Western Construction (March), pp. 42, 48 and 66.
Cambefort, H. (1966): "The Ground Anchoring of Structures", Travaux, *46*, (April-May) 15 pp.
Caron, C. (1972): "Corrosion et Protection des Ancrages Définitifs", Construction, (Feb), pp. 52-56.
Cementation Co. Ltd., (1962): "Anchor Stressing". Report No. 10, Cementation Co. Ltd., Mitcham Road, Croydon, Surrey. (Unpublished).
Cementation Co. Ltd., (1969): "Report on Tests Carried out on Ground Anchors in Keuper Marl". Cementation Co. Ltd., Mitcham Road, Croydon, Surrey. (Unpublished).
Comte, C. (1971): "Technologie des Tirants". Institute Research Foundation Kollbrunner/ Rodio, Zurich 119pp.
Conti, N. (1972): Reply to FIP Questionnaire (Unpublished).
Dixon, J. C. and Clarke, K. B. (1975): "Field Investigation Techniques", Chapter 3, p. 48 Site Investigations in Areas of Mining Subsidence Ed. F. G. Bell, Newnes & Butterworths, London.
Eberhardt, A. and Veltrop, J. A. (1965): "Prestressed Anchorage for Large Trainter Gate". Proc. A.S.C.E., 90, (ST6) pp. 123-148.
FIP (1972): "Draft of the Recommendations and Replies to FIP Questionnaire (1971)". FIP Subcommittee on Prestressed Ground Anchors.
FIP (1973): "Final Draft of Recommendations". FIP Subcommittee on Prestressed Ground Anchors.
Geddes, J. D. and Soroka, I. (1964): "The Effect of Grout Properties on Transmission Length in Grout Bonded Post-Tensioned Beams". Mag. Conc. Res., 16, No. 47, pp. 93-98.
Golder Brawner Associates (1973): "Government Pit Slopes Project III: Use of Artificial Support for Rock Slope Stabilisation". Parts 3.1—3.6, Golder Brawner Assocs. Vancouver, Canada. (Unpublished).
Gosschalk, E. M. and Taylor, R. W. (1970): "Strengthening of Muda Dam Foundation using Cable Anchors". Proc. 2nd Cong. Int. Soc. Rock Mech. Belgrade, 3 pp. 205-210.
Ground Anchors Ltd. (1974): "The Ground Anchor System", Ground Anchors Ltd., Reigate, Surrey., (28 pp.).
Hennequin, M. and Cambefort H. (1966) "Consolidation du remblai de Malherbe". Revue Generale des Chemins de Fer (Feb.).
Hilf, J. W. (1973): Reply to Aberdeen Questionnaire (1972) (Unpublished).
Holz, P. (1963): "Prestressed support at Bancroft", Canadian Mining Journal, Nov. pp. 61-62.
Irwin, R. (1971): Reply to FIP Questionnaire (Unpublished).
Khaova, M. Montel, B. Civard, A. and Lauga R. (1969) "Cheurfas Dam Anchorage: 30 Years of Controls and recent Reinforcement". Proc. 7th Int. Conf., Soil Mech. and Found. Eng., Paper 15-12.
Koch, J, (1972): Reply to FIP Questionnaire (Unpublished).
Leech, T. and Pender, E. (1961): "Experience in grouting rock bolts" Proc. 5th Int. Conf. Soil Mech. and Found. Eng., 2, pp. 445-452.
Littlejohn, G. S. (1972): "Some Empirical Design Methods Employed in Britain", Part of Questionnaire on Rock Anchor Design. Geotechnics Research Group, Department of Engineering, University of Aberdeen (Unpublished Technical Note).
Littlejohn, G. S. and Truman-Davies, C. (1974): "Ground Anchors at Devonport Nuclear Complex" *Ground Engineering*, 7 (6) pp. 19-24.
Littlejohn, G. S. (1975): "Acceptable Water Flows for Rock Anchor Grouting", *Ground Engineering*, 8, (2), pp. 46-48.
Littlejohn G. S. and Bruce, D. A. (1975): "Rock Anchors—State-of-the-Art" Part I, Design. *Ground Engineering*, 8, (3) pp. 25-32; 8 (4) pp. 41-48.
Longbottom, K. W. and Mallett, G. P. (1973): "Prestressing Steels", The Structural Engineer, 51, (12) pp. 455-471.
McGregor, K. (1967): "The Drilling of Rock"; 1st Ed. C. R. Books Ltd., London (306 pp.).
Maddox, J., Kinstler, F. and Mather, R. (1967): "Foundation Studies for Meadowbank Dam", 9th Int. Conf. on Large Dams, Istanbul, 1, pp. 123-141.
Mascardi, C. (1973): "Reply to Aberdeen Questionnaire (1972)" (Unpublished).
Mawdsley, J. (1970): "Choice of Drilling Methods for Anchorage Installation". Supplement on Ground Anchors, *The Consulting Engineer*, (May, 1970) pp. 5-6.
Mitchell, J. M. (1974): "Some Experiences with Ground Anchors in London", ICE Conference on Diaphragm Walls and Anchorages, London (Sept.) pp. 129-133.
Moschler, E. and Matt, P. (1972): "Felsanker und Kraftnessanlage in der Kaverne Waldeck II", Scheweizerische Bauzeitung, 90 (31), pp. 737-740.
Moy, D. (1973): Private Communication.
Mullett, L. J. (1970): "Deep Fixings for Anchors". Supplement on Ground Anchors, *The Consulting Engineer*, (May, 1970) pp. 25-26.
Neville, A. M. (1963): "Properties of Concrete", Pitman, London.
Ontario Hydro (1970): "Provisional Specification M388 and Standard Specification M285", Toronto, Canada.
van Ormer, H. P. (1974): "Determining Rock Drillability", *Rock Products* (February issue) pp. 50-51.
Ostermayer, H. (1974): "Construction, Carrying Behaviour and Creep Characteristics of Ground Anchors". ICE Conference on Diaphragm Walls and Anchorages, London, (Sept.) pp: 141-151.
Paone, J., Unger, H. F. and Tandanand, S. (1968): "Rock Drillability for Military Applications", Final Contract Report AD 671671 for Army Research Office, U.S. Dept. of the Interior, Bureau of Mines, Twin Cities Mining Res. Centre Minneapolis, Minnesota.
Paone, J., Madson, R. and Bruce, W. E. (1969): "Drillability Studies. Laboratory Percussive Drilling". Bureau of Mines Report of Investigations 7300 US Dept. of the Interior, Washington.
Parker, P. I. (1958): "The Raising of Dams with Particular Reference to the Use of Stressed Cables". Proc. 6th Cong. on Large Dams, New York Question 20, 22 pp.
PCI Post-Tensioning Committee (1974): "Tentative Recommendations for Prestressed Rock and Soil Anchors", PCI, Chicago, U.S.A., 32 pp.
Pender, E., Hosking, A. and Mattner, B. (1963): "Grouted Rock Bolts for Permanent Support of Major Underground Works", J. Inst. Eng., Aust., *35*, pp. 129-150.
Portier, J. L., (1974): "Protection of Tiebacks Against Corrosion", Proc. Tech. Session on Prestressed Concrete Foundations and Ground Anchors (pp. 39-53) 7th FIP Congress, New York.
Powers, T. C. (1968): "The properties of fresh concrete", J. Wiley & Sons Inc., New York.
Protydyakonov, M. M. (1962): "Mechanical Properties and Drillability of Rocks", Proc. 5th Symposium on Rock Mechanics, University of Minnesota, Minneapolis, U.S.A. (May, 1962) pp. 103-118.
Rehm, G. (1968): "Corrosion of Prestressing Steel", Proc. Sym on Prestressing, Madrid, (June).
Ryd, E. and Holdo, J. (1956): "Percussive Rock Drills—Their Contribution and Method of Operaion". Manual of Rock Blasting (Chapter 12:01-1-35) Atlas Copco, Stockholm.
Schmidt, R. L. (1972): Drillability Studies—Percussive Drilling in the Field". Bureau of Mines Report of Investigations 7684. US Dept. of the Interior, Washington.
Soletanche Co. Ltd., (1970): "Other Types of Anchor". Supplement on Ground Anchors, *The Consulting Engineer*, (May, 1970) p. 13 and 15.
South African Code of Practice (1972): "Lateral Support in Surface Excavation", The South African Institution of Civil Engineers, Johannesburg.
Standards Association of Australia (1973): "Prestressed Concrete Code CA35—1973 Section 5—Ground Anchorages" pp. 50-53.
Stocker, M. F. (1973): Reply to Aberdeen Questionnaire (1972) (Unpublished).
Thompson, F. (1970): "The Strengthening of John Hollis Bankhead Dam", *Civil Engineering* (NY), 39 (12), pp. 75-78.
Unger, H. F. and Fumanti, R. R. (1972): "Percussive Drilling with Independent Rotation", Bureau of Mines, Report of Investigations 7692. US Dept. of the Interior, Washington.
Universal Anchorage Co. Ltd., (1972): "Report on Rock Anchor Tests for Frigate Complex H. M. Dockyard, Devonport". Report No. 189, Universal Anchorage Co. Ltd., Egerton Street, Farnworth, Bolton, England (Unpublished).
US Army (1964): "Scientific and Technical Applications Forecast, 1964, Excavation". Office of the Chief of Research and Development. Department of the Army, Washington, D.C. (250 pp.).
White C. G. (1965): "A Rock Drillability Index", DSc Thesis, Colorado School of Mines, Golden, Colorado. 156 pp.
White, R. E. (1973): Reply to Aberdeen Questionnaire (1972) (unpublished).

Part 3: Stressing and testing

INTRODUCTION

PRESTRESSING AN ANCHOR automatically tests the installation, confirms to a certain degree design safety factors, and ensures satisfactory service performance. This is equally true for prestressed anchors and those subsequently intended to act as "passive" untensioned members, and in both cases an initial stress history often enhances subsequent behaviour.

In addition, acceptance criteria based on standardised tests gauge the suitability and effectiveness of the installed anchor with respect to the intended application. Possible errors made in either the design or construction stages will be pinpointed immediately and potentially dangerous and expensive consequences avoided.

Incorporating these important precepts, this third part of the rock anchor review§ describes anchor stressing techniques, the monitoring and presentation of data, and provides guidance on the interpretation of stressing results. This basic information is intrinsic to anchor testing.

The authors believe that a standard approach to the testing and analysis of anchor behaviour should be established, relating to both short and long-term behaviour. Accordingly, the following basic types of test and quality control are recommended for consideration, and are described in detail:

1, precontract component testing,
2, acceptance testing of production anchors,
3, long term monitoring of selected production anchors,
4, special test anchors, and
5, monitoring of the overall anchor/rock/structure system.

A final section deals with aspects of long-term service performance, and reviews the relatively small number of case studies published to date. These highlight various parameters and phenomena which influence anchor behaviour in the long term.

STRESSING

Mode of stressing

There are basically two methods of applying stress to an anchor tendon:

(*i*) torque, applied via a torque wrench to some form of anchoring nut threaded on to a rigid bar tendon (Fig. 1a), and

(*ii*) direct pull, which may be applied to the tendon by a jack seated for example on a stressing stool or chair (Fig. 1b).

Torquing is normally restricted to small capacity (150kN max.) single bar tendons i.e. rock bolts of various types. In practice care must be taken to ensure that torsional stresses are not incidentally applied to the tendon, since they may combine with the tensile stresses and reduce the effective strength of the bar. This disadvantage can be alleviated by introducing a friction reducing material e.g. a molybdenum disulphide based lubricant, beneath the lock-nut prior to stressing.

The required torque to produce a specified load is usually expressed empirically in the form

Tensile load (kN) $= C \times$ torque (kN.m)

but whilst C may be defined within narrow limits under controlled laboratory conditions, experience suggests that variations of $\pm$ 25% can be expected for the value of C under field conditions. In addition the installed load is subject to variations due to a number of conditions related to control of alignment, friction between mating parts and size of bar tendon. Bearing in mind also that torquing is usually accomplished with the aid of an air driven impact wrench, the output of which is subject to variation in airline pressures, it is not surprising that the equipment needs frequent calibration and that good maintenance is vital. For reliable results therefore it is recommended that a calibrated hand wrench be used as a check in all cases. Nevertheless, the equipment is light, compact, easy to handle, and the stressing procedure is simple, and cheap. As a result the torquing method of stressing rock bolts is very popular in practice, and for the interested reader more detailed information can be found in the ISRM draft publication "Suggested methods for rockbolt testing" (1974).

By far the most common and indeed for the vast majority of anchors the only suitable method is stressing by direct pull. Strand is now much more commonly used than wire, and as a result multistrand and monostrand direct pull jacks are the most common systems used today in prestressing. Monojacking relates to single strand stressing and the individual tendon units are tensioned in turn (Fig. 2a). Multistrand jacks permit all the strands of the tendon to be stressed simultaneously. These jacks may be of solid or hollow ram design (Figs. 2b & c).

Practical aspects of stressing

In order to introduce the reader to some basic procedures and concepts, as well as the stressing jargon, the following description deals with practical aspects relating to anchor stressing in the field.

Top anchor movements should be kept ideally to a minimum. Therefore the bearing plate may be placed directly on to strong competent rock, or alternatively embedded in a mass concrete block to spread the anchor forces in the case of weak rock. For anchors with design loads in excess of 150kN it is important, prior to start of

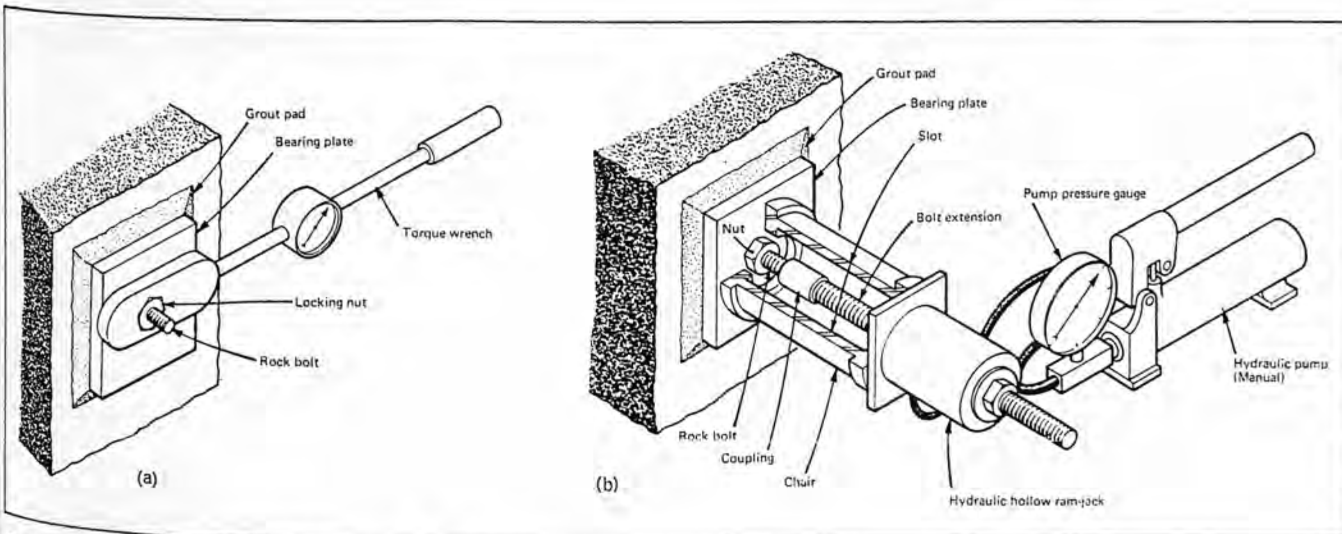


Fig. 1. Stressing by (a) torque wrench, and (b) direct pull

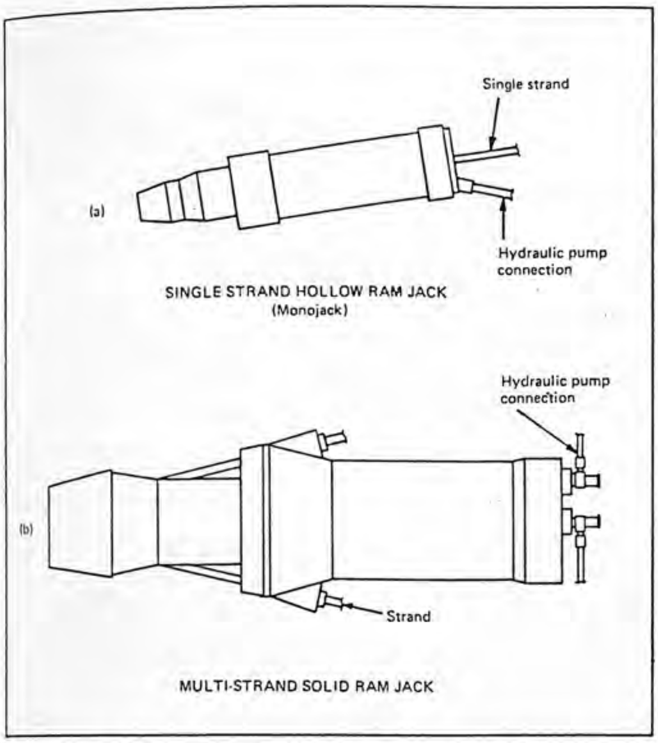


Fig. 2. Typical jacks for tensioning rock anchors; (c) (below) depicts a hollow ram multi-strand stressing jack
(photo, courtesy, Ground Anchors Ltd.)

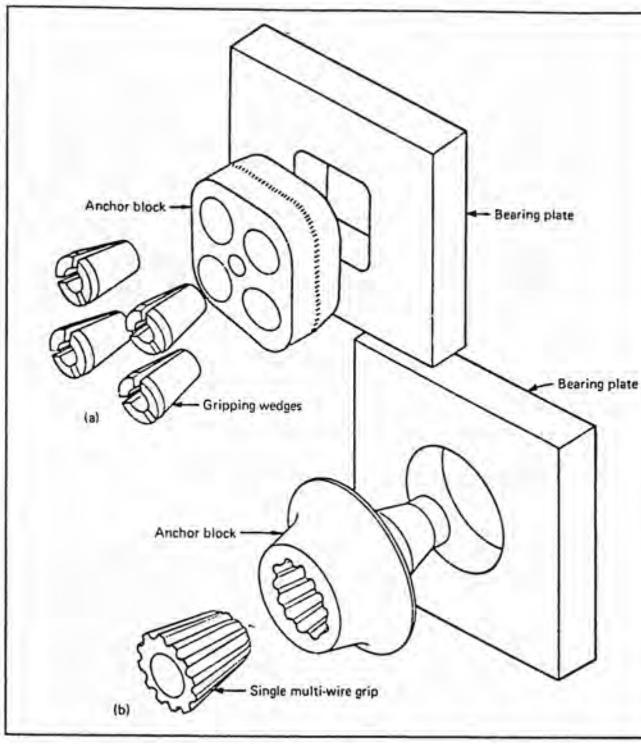
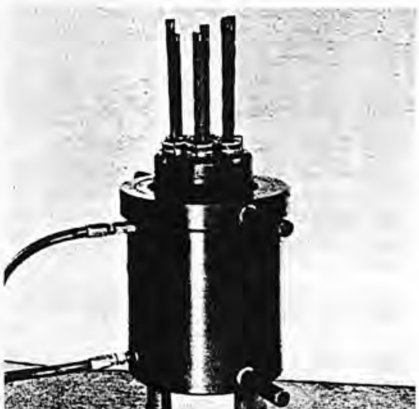


Fig. 3. Typical anchor heads for a strand or wire tendon



stressing, to check that the steel bearing plate has been correctly bedded centrally and normal to the tendon. This check can eliminate chaffing of the perimeter tendon components in the case of multiwire or strand tendons which splay outwards in the zone of the top anchorage or jack assembly (Fig. 3). This problem does not appear to be recorded for the case of parallel rigid bar groups.

Once the anchor grout has reached a specified strength, stressing may proceed. The authors recommend a crushing strength of 28N/mm² (see Part 2: Construction).

For solid bar or single unit tendons, the tensioning assembly may be fitted on to the tendon as soon as it has been thoroughly cleaned. For multi-unit tendons however it is important to verify that the wires or strands are not crossed within the free-length before fitting the anchor block and jack assembly (Fig. 4a). The correct alignment of strands is best accomplished by providing a form of comb grillage or fork (Fig. 4b), and the use of guide cords with caps is particularly beneficial on high capacity multi-unit tendons.

To simplify the description the remaining practical comments will relate primarily to multistrand stressing using a hollow jack.

If the tendon is to be subjected initially to a special test overload to prove its design capability, then the permanent grips are normally omitted from the anchor block at this stage of the work. The jack is now fitted over the strands and the temporary stressing units (Fig. 4a) are then assembled. The jack chair or stool which provides a support for the jack is placed centrally over the tendon and the side opening should be in a convenient position to allow the operator to inspect the anchor head during the tensioning operation (Fig. 2c).

The jack is now fitted manually or by a mechanical lifting device. Mechanical lifting and handling equipment is recommended for jacks weighing in excess of 80kg, and a guide relating the approximate weight of hollow ram steel jacks to their maximum rated capacity is given in Table I.

It is important, prior to stressing, to verify that the elongation at the top anchor will be be in excess of 30mm for the maximum load to be applied, otherwise the re-usable grips (wedges) in the temporary loading head (Fig. 4c) cannot be freed on destressing. Where extensions of 30mm or less are envisaged the jack piston should be advanced to 30mm before placing the temporary loading head. The re-usable grips must be lightly lubricated with high pressure grease prior to their fitting. These grips are finally homed to give a tight fit, by a gentle tapping with a special ring or U-shaped hammer. Stressing may now proceed.

TABLE I. APPROXIMATE WEIGHT OF HOLLOW RAM STEEL JACKS

Maximum-rated capacity (kN)	Approximate weight (kg)
200	20
500	40
1 000	80
2 000	150
3 000	200
4 000	300

It should be emphasised immediately that the space in front of the jack, and in line with the tendon axis must be kept free of personnel during the prestressing operation. Alternatively, a properly designed small aperture steel mesh cage should be provided for protection of the operators or passers-by.

A hand pump is the simplest means of pressurising the jack system to advance the ram but where many tendons need to be stressed and a high output is required a motor-driven pump is advantageous.

Bearing in mind that the stressing system may have been designed to operate at high pressures (quoted test pressures of 600 atmospheres by manufacturers are not uncommon), it is not always practical to monitor pump pressures below 40 to 50 atmospheres. The initial position of the jack piston is therefore noted at this pressure which is also considered sufficient to take up any slack in the tendon. The actual zero reading for the piston can be found by extrapolation when the ram extensions at subsequent higher pressures are noted (Fig. 4c). Throughout the stressing operation, both extensions and gauge pressures are recorded but this aspect is discussed in more detail below.

Where a test load has to be held for a period of time a slight fall in gauge pressure will be noted even though the extension of the piston remains constant. This

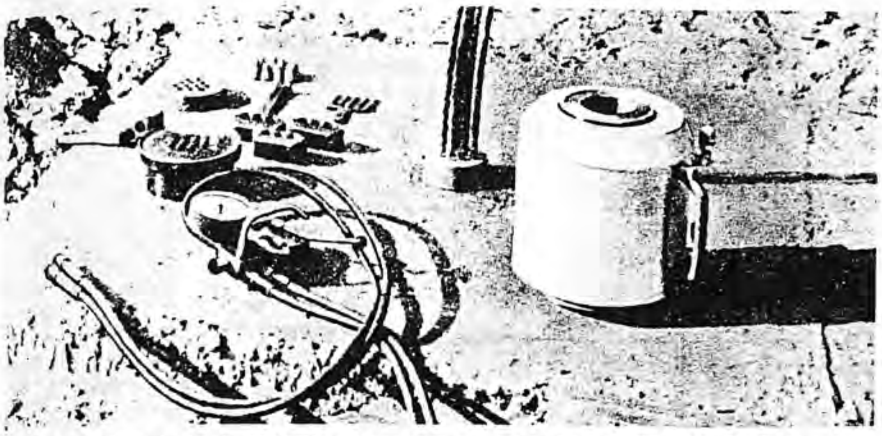
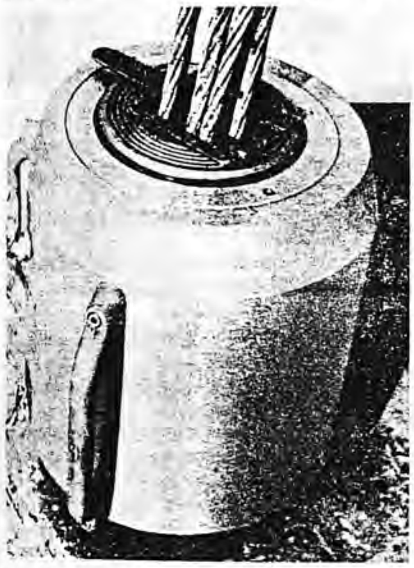
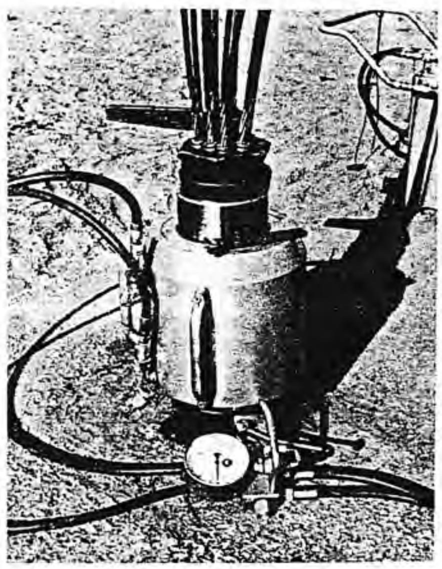


Fig. 4. (a) (above), Anchor block and components of jack assembly; (b) (below left), Fork for alignment of strands; and (c) (below right), Stressing through the temporary load bearing plate (photos, courtesy, Cementation Ground Engineering Ltd.)





loss is internal to the system and a gentle application of pressure to the original reading will in most cases produce the same extension as initially recorded. For long-term stressing a lock valve at the jack is recommended.

On completion of the initial stressing operation, the double-acting ram retracts and leaves the temporary loading head in position to allow its removal. Gentle tapping with a wooden or rubber mallet is usually sufficient to release the grips which should now be re-greased and kept in a clean condition ready for the next stressing operation.

In order that load can be finally locked into the tendon, permanent grips must be inserted into the permanent anchor block. This should be possible without the total removal of the jack and chair from the tendon, although the temporary grips will have to be removed until the permanent grips are fitted.

During stressing the chair provides a reaction head (Fig. 5) restricting the upward movement of the permanent gripping wedges. When the desired pump gauge reading is attained, the jack ram is retracted and immediately the wedges are drawn or pulled in around the tendon as it tries to retract, and so load is locked off. It is noteworthy that when this final load is considered insufficient (for reasons described below) the anchor may be restressed, and if necessary steel spacers or shims of various thicknesses inserted beneath the anchor block to raise the load at lock-off by increasing the tendon extension (Fig. 6).

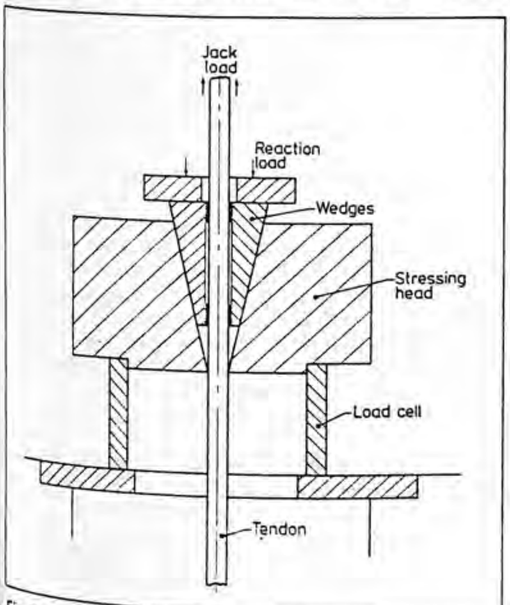


Fig. 5. Basic stressing mechanism at the top anchorage

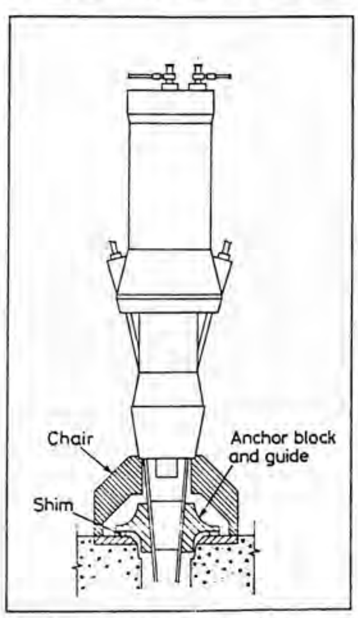


Fig. 6. Jack arrangement for shimming

Choice of stressing system

Multistrand stressing is swift and simple in operation once the jack has been correctly located, and requires relatively little data recording and back analysis in most cases. Nevertheless, multistrand stressing cannot provide a high degree of control over the behaviour of individual tendon units, or, at final lock-off, a guarantee of equal load in each unit. This is particularly important in anchors of free length less than 10m, where extensions are relatively small and so variations in the amount of wedge pull-in, for example, will represent proportionately larger load discrepancies than in a longer tendon.

Conversely, with respect to anchor block lift-off checks — detailed later — the multistrand jacking system alone can show the total load on the anchor in one stressing operation. Furthermore, for cyclic loading and unloading programmes, this system is easier and quicker to employ and gives more control, especially during the destressing stage. Some engineers also consider that a multistrand jacking system alone is capable of economically supplying prestressing loads in excess of 3 000kN. This view is based on the larger number of individual time-consuming stressing operations, and the larger spacing required to separate the strands in the anchor block if a monojack is employed.

On the other hand monostrand stressing is a relatively popular method for tensioning tendons of up to six strands, and close control over the force in each individual strand can be achieved. Since the development of high speed front gripping jacks, and bearing in mind the limited number of strands, the method is not unduly time consuming. In addition, most single strand stressing jacks are light and easy to handle, which is a major advantage on most sites.

There are however important points concerning monojack stressing operations which are widely recognised but remain largely unexplained. For example, when Mitchell (1974) monitored with strain gauges the load fluctuations in two adjacent strands of an anchor tendon, he observed that the load in the first tensioned strand decreased steadily during the stressing of the adjacent strand (Fig. 7). This effect was in fact exaggerated because in this experiment the load was not incrementally applied to each strand in sequence as recommended in practice. Nevertheless the results clearly justified Mitchell's subsequent advice that after application of a nominal seating load to each strand, the remaining load should be applied in four or five equal increments to each strand in turn, in a specific sequence to ensure a uniform distribution of load

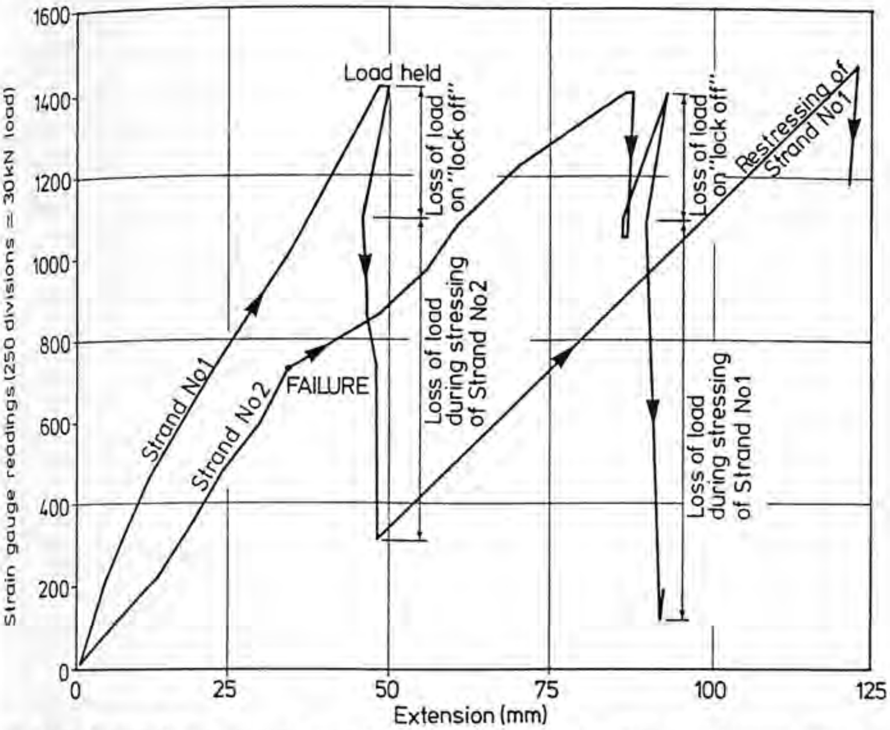


Fig. 7. Interference between two strands during monojack stressing (after Mitchell, 1974)

across the tendon. Mitchell also found that in a six strand tendon, at the completion of each stage of incremental loading, the greatest and least load losses monitored always occurred on the first and last strands loaded, respectively. This phenomenon has also been personally observed by Barley (1974) and the authors. In practice, after the final increment of one stressing sequence, the uneven distribution of the loading can be minimised by conducting a final stressing round to bring all strands up to the required load.

In general, it is wrong to recommend one stressing system over the other. Realistic comparisons, made to effect a choice, should only be attempted when the stressing and testing specification, and the environmental considerations e.g. accessibility, are known.

Whichever system is used, it is important in many cases to verify that the applied prestress is actually being resisted by the grouted fixed anchor zone, and further that the method of applying the tensile load is relevant to the particular application. For example the load may be applied remotely through a simply-supported beam, or by prestressing through a plate or pad bearing directly on the rock overlying the fixed anchor being tested. In the latter case, the tensioning procedure may simply prestress the rock and/or grout column between the fixed anchor and pad. This may have serious consequences if the test is supposed to check the stability of the pad against uplift, if it performs in service as the footing of a transmission tower for example. No work has been published on this phenomenon in rock, to the authors' knowledge, but current research being conducted by the Universal Anchorage Co. Ltd., and the Geotechnics Research Group suggests that, for shallow anchors installed in horizontally bedded flaggy sandstone, the load is resisted locally by the rock mass in the grouted fixed anchor zone, where the slenderness ratio (depth to top of fixed anchor/hole diameter) exceeds 15.

Monitoring procedures

The prestressing of any anchor, either production or special test, presupposes the graphical plotting of anchor load against tendon extension. Such a plot facilitates judgement as to the anchor's competence and efficiency. Therefore, it is most important to be familiar with the parameters to be investigated, and methods of their measurement, presentation, and interpretation.

The parameters

The two basic parameters are, obviously *load* and *extension*. The former is self evident, being the actual amount of prestress locked into the tendon at any one time. The tendon extension, however, involves other measurements, not always recognised as being significant in load — extension analyses. An extension, as measured *before* lock-off may be regarded as the "gross extension". At lock-off in the case of a wedge grip type top anchorage (Fig. 3a), pull-in of the wedges (and strand) will occur until the system is "tight". *After* lock-off, there may be movements due to bedding-in of the top anchor block and bearing plate, deflection of the structure, and/or permanent displacement of the fixed anchorage, in addition to the elastic extension of the tendon under load.

Long term monitoring may necessitate the recording of ground or air temperature, as variations in *temperature* will affect tendon prestress, and instrumentation such as vibrating wire gauges.

With regard to the recording of load-extension data Mitchell (1974) has recommended in practice that the details should be noted over four or five equal increments during loading or unloading cycles. However, Hanna (1969) considers that for a load extension diagram to be of "engineering use" it is essential that the load increments are small e.g. 10-20% of the working load (T_w). In this connection the Nicholson Anchorage Co. (1973) describe the stressing of test anchors at Greenwich, Connecticut, in six equal increments, after an initial seating load.

In general, it would appear that in any one stressing stage, at least five load increments should be monitored in routine production anchor tests. In special tests however, where a more basic analysis is being attempted, extensions should be monitored at load increments equivalent to 10% or less of the maximum load for each stage in the stressing investigation.

The various levels of measurement sophistication understandably reflect the time, money and personnel available. For *load* measurement, load cells have been installed on occasions to monitor anchor performance in both long and short term experimental programmes. Such cells are expensive, relatively fragile, and require regular care and maintenance if reliable performance is to be guaranteed.

Hanna (1973) discusses load measurement in considerable detail and this reference is strongly recommended to the interested reader, since many load cells are described which are applicable to anchor situations. By way of introduction Hanna indicates that the choice of load cell is usually controlled by three factors:

(*i*) cost,
(*ii*) environment e.g. access, temperature, humidity, susceptibility to damage, and
(*iii*) nature of load and accuracy required.

In summary, the major types of cell applicable to anchors are

(*a*) mechanical — based on
proving ring systems (up to 2 000kN)
force measuring blocks (up to 10 000kN), and
cup springs (greater than 4 500kN),
(*b*) strain gauged elements (up to 5 000kN), and
(*c*) vibrating wire systems (up to 10 000kN).

Other methods involving photoelasticity, hydraulics and springs have also been used in practice. In all cases at least 1% accuracy is preferable and, regardless of the cell type, eccentric loading of the cell should be either assessed or prevented. The upsetting effects of eccentricity on load cell readings in the field are well illustrated by McLeod & Hoadley (1974) referred to later. It is also imperative that load cells are calibrated prior to and after use in the field.

An alternative and cheaper method for measuring anchor load is to use the prestressing equipment available, together with a destressing stool or chair. The method is applicable to both individual strands or the tendon as a whole. In both cases the principle is the same: a feeler gauge of specified thickness (0.1mm) is inserted under the anchor block or individual grip unit upon stressing through the stool to a certain load. The jack pump pressure at the earliest moment of insertion is recorded, and the minimum load at "lift-off" is thus evaluated. This initial residual load is commonly referred to as the "lock-off" load. The method is very common in practice, if somewhat crude, but an accuracy of $\pm$ 2% can be obtained by a careful operator. In the case of a single unit tendon the accuracy can be improved since the access to the tendon often permits the moment of "lift-off" to be registered by a dial gauge reading to 0.01mm (Fig. 8). In this connection it is noteworthy that the Czech draft code (1974) suggests that the jack calibration accuracy should be $\pm$ 1% as measured from two gauges. In the case of torquing, the lift-off load is related to the reading on the hand torque wrench when the locking nut is just in motion.

In a similar way to load measurement there are a number of levels of sophistication in measuring the tendon *extension*. The

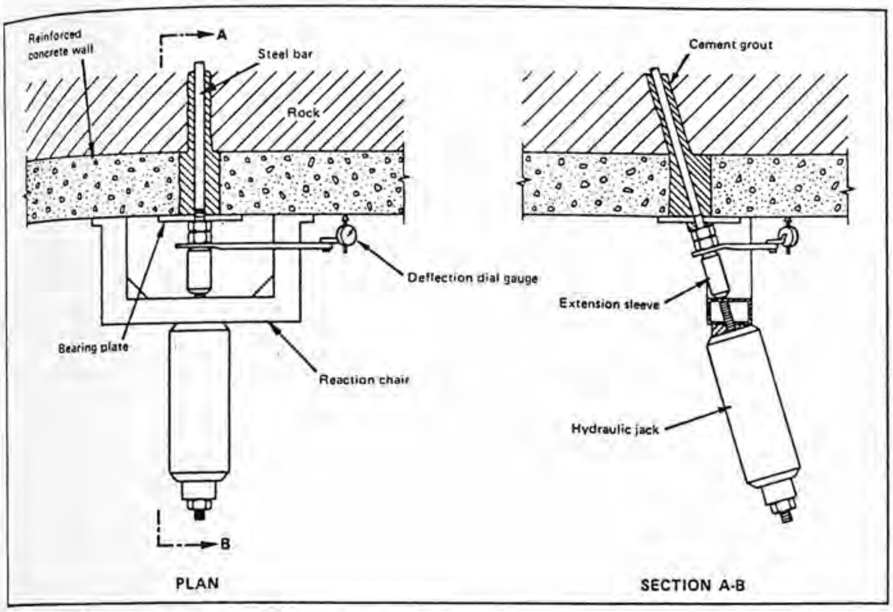


Fig. 8. Jack arrangement for mono-unit stressing and measurement of residual load (after da Costa Nunes, 1966)

simplest, and least accurate, method is to measure the jack ram extension. Even if the correct null extension point is noted — when the jack has fully gripped the tendon or strand — there is no guarantee that the jack extension thereafter is the same as the strand extension. This is particularly the case where slip of the strand relative to the temporary grip wedges on or in the jack occurs. Usually, therefore, the true tendon extension is overestimated by this method.

A preferable method of measurement is the one whereby a piece of adhesive tape or some other means is used to mark all or a representative number of strands at some distance above the permanent load bearing plate. The difference between this distance in the unloaded condition and that measured at successive load increments provides the basic data for a load-extension graph. For single strand stressing, this distance is measured after lock-off at each load increment when the jack has been removed. Where a solid ram multistrand jack is used no lock-off or jack removal is required at intermediate load increments. In the particular case of hollow ram multistrand stressing it may be more convenient to measure the distance between the strand mark and the temporary load bearing plate. This approach permits an accurate measurement of gross extension without removal of the jack, provided that the distance between the temporary and permanent bearing plates is recorded. These distances are usually measured with a stiff steel rule, and an accuracy of ± 1mm can be attained. In this connection the Czech draft code stipulates an accuracy of ± 0.1mm.

More refined methods, often associated with special test anchors, include the use of dial gauges attached to a simply supported datum beam, in order to monitor movement of the temporary bearing plate. In very special cases, strain gauges of either mechanical or electrical types are installed.

Remote survey is the method of accurately determining the *movement of the permanent load bearing plate* and should be considered whenever possible. Knowing these movements, gross extensions can be corrected to give extension data dependent solely on tendon elasticity and fixed anchor movements. The Czech draft code stipulates that precise observations be made of vertical and horizontal movements of the structure and those of the rock. Also, the supports for all measuring instruments should be such that they are independent of the structure and not influenced by deformations produced by the prestressing operations. Usually for anchors in competent rock, and prestressed against a properly designed bearing plate system, top anchorage movements provide a very small proportion of the total tendon elongation. PCI (1974) recommends that bearing plate movements greater than 13mm should be taken into consideration. There is no disagreement with this statement but the authors believe that the significance of the actual value of movement can only be appraised when the free length of the anchor is known. For example, a plate movement of only 5mm would be sufficient to lose 20-25% of the initial prestress in the case of a free length of some 4m. In general however where the top anchorage movement represents less than 5% of the tendon extension it is usually ignored in the routine stressing of production anchors.

A direct, as opposed to interpretive, method of measuring the amount of *fixed anchor movement* involves the embedment of a wire in the fixed anchor. The wire is decoupled over the free length and extends out of the top anchorage assembly. With the wire loaded in tension, simply to keep it taut, the wire movement indicates fixed anchor movement (Fig. 9). Alternatively a redundant tendon unit may be used in place of the wire. This method has been used successfully by Liu & Dugan (1972).

Another parameter involving measurement on the tendon is the *strand wedge pull-in* at lock off. It should be emphasised however that this parameter is solely monitored as an indirect means of establishing the amount of lock-off loss and the resulting residual load at that time.

By careful measurement, the amount of strand wedge pull-in can be estimated to at least ± 1mm accuracy. With a multistrand stressing system the difference between extensions immediately before and just after lock-off is the amount of pull-in. With monostrand stressing, this amount can be readily judged by close observation of the strand near the jack nose during the lock-off operation.

If accurate monitoring is required it is considered advisable to measure in the field the amount of wedge pull-in and express it as a distance in mm, rather than as contributing a certain prestress loss, since the magnitude of this loss is directly proportional to the free length of the tendon in question.

This point can be illustrated by reference to details of two test anchors reported by Barron *et al* (1971) and shown in Table II.

Recent research conducted jointly with the Universal Anchorage Co. Ltd. has led the authors to conclude that the amount of wedge pull-in increases linearly with load in the strand, after a comparatively large initial pull-in at loads up to 30kN/strand. At 200kN/strand for 15.2mm Dyform, the amount may be as high as 6mm but mostly averages between 2-4mm in fair agreement with Fenoux and Portier (1972) who estimated 2-3mm.

It has also been found that the amount of wedge pull-in is less in monostrand compared with multistrand stressing. This is due to the practice of tapping home the individual grip wedges immediately prior to lock-off, in the monojacking operation.

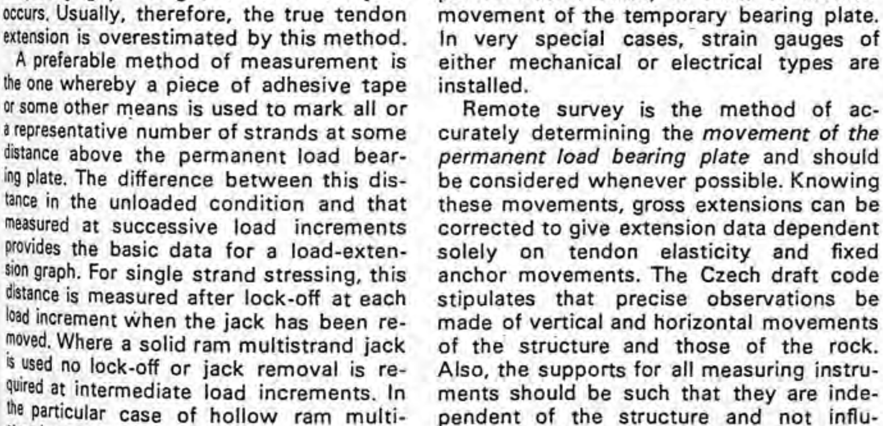


Fig. 9. Direct method of measuring fixed anchor movement

Presentation

All data relating to the stressing operation should be collected and carefully preserved. The list of items given in Table III is recommended for inclusion in a full stressing record. The data describe the rock anchor, jacking equipment and personnel, in addition to the load/movement readings which should be recorded during stressing, as already described.

There is limited published data on the stressing records recommended for torquing but a brief list of requirements is suggested in the ISRM draft document

TABLE II. LOCK-OFF LOSSES (after Barron *et al*, 1971)

Anchor	Free length	Applied load	Load after lock-off	Lock-off loss
1	3.96m	1 352kN	937kN	30.7%
2	10.67m	1 427kN	1 256kN	12.0%

"Suggested methods for rockbolt testing" (1974).

Although the final graph of load against extension will be based on corrected data, the original monitored data should also be presented on the stressing record since this information will not only provide historical data and facilitate back-analysis, but it will permit interpretation by other analysts.

When plotting the load against extension, another variable to define clearly is the point of origin of the graph. In most cases, the "zero" extension is recorded after the application of a certain seating load to the tendon, and not actually at zero load. The seating load is supposed to take up the slack in the tendon and jack, and compensate for friction and other losses in the jack/pump assembly thereby giving a more accurate measure of load-extension data.

For instance Larsson *et al* (1972) begin extension readings at 12% T_w — "to take up slack" — but also assume a zero extension of 2.5mm. On the other hand Longbottom and Mallet (1973) simply recommend starting at 10-20% T_w and N.A.C. Ltd. (1973) commonly begin reading from 10% T_t. The biggest seating load published to date is 25% T_w on anchors at the Frigate Complex, Devonport (Short 1975).

Most anchor codes e.g. Czechoslovakia and Germany advise reading from 10% T_w although P.C.I. (1974) recommends a start from 10% T_t. In the authors' view it would appear more realistic to try and gauge the actual seating load required for any particular anchor/jack assembly in order to optimise the measurement of residual displacements, e.g. due to fixed anchor movement at zero load. Nevertheless, the above recommendations are simple and although zero readings are extrapolated the method is probably adequate for routine short term testing.

The final presentation of load-extension should indicate the maximum possible measurement error in each parameter. Thus, when the line corresponding to the extension of the theoretical tendon length is drawn from the relationship

$$\text{extension} = \frac{\text{length} \times \text{load}}{E_{steel} \times \text{cross-section area}}$$

a meaningful and sensible comparison between actual and theoretical extension characteristics is permitted.

Similarly, a graph of load against time should have superimposed the theoretical relaxation curve for the tendon in question, as computed from the manufacturer's data. In this connection it is noteworthy that elevated temperatures occurring naturally or artificially e.g. adjacent to concrete nuclear reactor vessels, considerably increase the rate of loss. It is not generally appreciated that for wire and strand at 40°C the relaxation losses are at least 50% greater than at 20°C.

Interpretation

The fundamental property of the load-extension curve to be adjudged is its elastic behaviour, whether linear or non-linear. Due to limits on the accuracy of the monitored data collected, it is rare to obtain a perfectly linear plot, even for the most efficient anchor. However, if the deviation from linearity is both marked and consistent in trend, it is most likely that this is due to one or both of two factors:

(*i*) debonding in the fixed anchor at the grout/tendon interface, and

(*ii*) fixed anchor movement.

The latter phenomenon is unusual in all but the weakest rock strata, but unless some form of direct measurement (Fig. 9) has been incorporated, it can only be confidently dismissed by cyclically loading the anchor at least once to ensure that the load-extension characteristics of the anchor are reproducible.

Assuming allowance has been made for the top anchorage and fixed anchor movements, an interpretation can be made with respect to the amount of partial or total debonding within the fixed anchor zone, by calculating the effective free length to produce the true elongation of the tendon actually monitored at different loads. In practice, this analysis is facilitated by drawing construction lines, equivalent to the extension of different free lengths, on the load-extension graph (Fig. 10). During the initial loading of an anchor the characteristic trend of the measured load-extension curve is to approximate to lines of short free length initially, but to progressively intersect lines of longer free length with increasing load.

Cyclic loading not only highlights fixed anchorage movement, but generally facilitates back analysis, and confirms the degree of reproducibility of the elastic load-extension characteristics. It should be noted that when drawing straight, theoretical extension lines on such diagrams involving cyclic loading, a family of these lines should be drawn through each new zero load point, following the last loading cycle thereby eliminating the permanent set produced in the anchor by previous stressing.

A refined cyclic method is described by Fenoux and Portier (1972), which they consider to be systematic, easily conducted, and economic. The principle is that by careful destressing and restressing, without real change in tendon elongation, a value of load equivalent to twice the total frictional effects in the anchor can be deduced.

The method and interpretation is shown in Fig. 10. Assuming section X-Pm and X-Pb are sensibly parallel, the line X'-Y' represents the true values of loads corresponding to measured extensions since losses due to friction have been compensated. The point R, defined by X' and Y' and $\Delta l'$ gives the true final load sustained by the anchor. The method also permits lock-off losses to be readily determined.

Different failure modes within the anchor may be recognised during stressing and from close analysis of load-extension data. For example, a continuous cumulative permanent displacement indicated either by rapid load loss or from a cyclic loading plot usually indicates interface failure in the fixed anchor. Whether this is rock-grout or grout-tendon failure may be verified by loading each tendon unit with a monojack and comparing load-displacement characteristics.

Discrepancies between the theoretical and actual extensions are more often the rule than otherwise. Commonly, the amount of discrepancy permitted on any one site reflects the allowable anchor movements bearing in mind proximity of adjacent structures, the load safety factors, acceptable errors in measurement, and the consequences if failure occurs.

P.C.I. (1974) states however, as a general rule, that, where the measured and theoretical elongations have more than a 10% difference, "investigation shall be made to determine the source of the discrepancy".

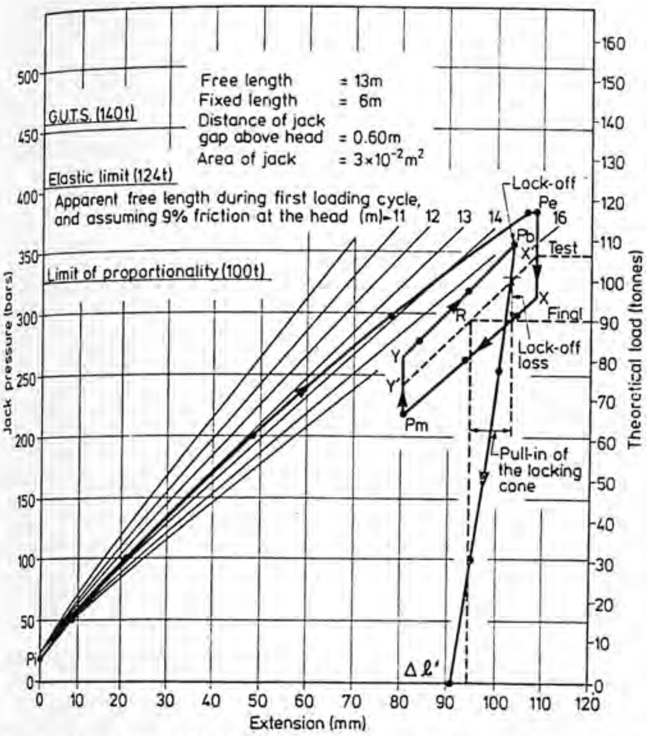
Numerous potential sources of error can be listed. For instances, as noted in Part I — Design, the *E* values given by the manufacturer for his prestressing steel, and based on short lengths may be in error.

Furthermore, Janische (1968) found that in extension measurements on long lengths of strand (100m) the extension for any particular applied load varied considerably, yielding *E* values in the range 180 000 — 220 000N/mm², averaging 196 000 ± 9 000 N/mm². Variations of this order were noted in strands for prestressing the Wylfa nuclear reactor, but even more relevant was the observation that the elongations of tendons were comparatively much greater than their constituent strands,

$$E_{strand} = 183\,000 - 195\,000\text{N/mm}^2$$
$$E_{tendon} = 171\,000 - 179\,000\text{N/mm}^2$$

TABLE III. RECOMMENDED ITEMS FOR INCLUSION IN STRESSING RECORD

General classification data			
Project	Contractor	Engineer	Inspector
Date	Time started	Time completed	Stressing personnel
Anchor No.	Free length	Fixed anchor length	Rock type
Tendon type	*E* value of steel	Working load (T_w)	Test load (T_t)
Jack type	Area of piston	Maximum rated capacity	Date of last calibration
Pump type	Pressure gauge range	Pressure gauge accuracy	Date of last calibration
Type of top anchorage assembly	Lock-off mechanism	Initial seating pressure	Strand pull-in
Data monitored during stressing			
Permanent bearing plate movement	Tendon extension	Jack pressure	Tendon pull-in at lock-off



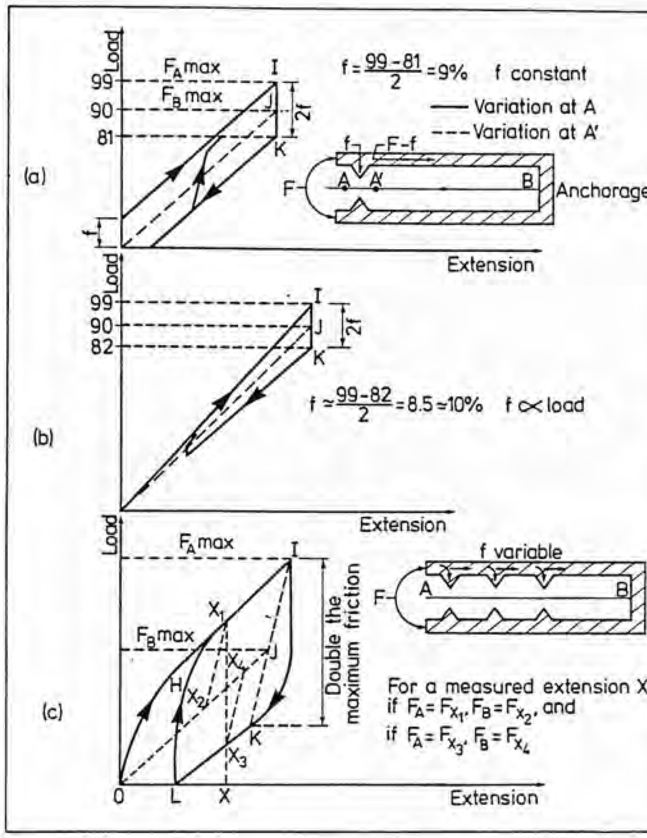
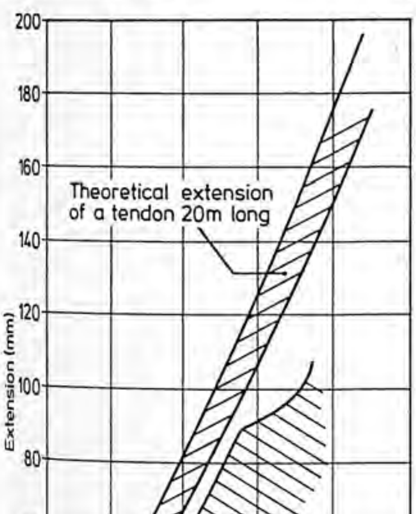


Fig. 10 (above) "La Méthode du Cycle" (after Fenoux & Portier, 1972)
Fig. 11 (below) Extension-load diagram illustrating the influence of friction (after Hennequin & Cambefort, 1966)
Fig. 12 (right) Influence of type of friction on form of load-extension graph (after Fenoux & Portier, 1972)



Janische attributed this to the possibility that with the stressing of multistrand tendons taking a longer period in the field than the testing of individual strands a plastic deformation occurs in the steel in the former application giving it a larger extension and so, apparently, a smaller *E* value.

Further information is supplied by Leeming (1974) who felt that instead of a possible maximum variation of ± 5% (three standard deviations from the mean), quoted by manufacturers, the total variation is more probably ± 7½%. He also highlighted the difference between valuation of *E* when testing long and short specimens, by noting that the value for a 137m specimen was some 9% less than that for a short test piece of the same strand.

On a less sophisticated level, overdrilling or underdrilling of the hole will alter the free length in practice, and the accuracy and reliability of the recording — as distinct from the accuracy of the instrumentation — should always be considered.

Friction is another major source of error. Even if allowance for friction losses in the jack is made — some manufacturers quote a figure of 1% over the whole loading range — friction still occurs along the free length, particularly in long sheathed tendons surrounded by a protective grout surcharge column, and around the grip assembly of the top anchorage.

Such friction will act to reduce the measured extension simply by dissipating a proportion of the applied load which can act over the total tendon length. This results in an extension corresponding to a free length apparently less than is actually present. For example, Hennequin & Cambefort (1966) describe stressing details from a contract near Paris. They noted that the measured extensions were markedly lower than those estimated theoretically, and concluded that on average, only about 70% the total applied prestress was transmitted over the whole tendon length (Fig. 11). Such frictional losses can often be overcome simply by overloading by an amount particular to each anchor type.

Fenoux & Portier (1972) have also discussed friction in anchor systems and detail three types:

(*i*) constant value,
(*ii*) proportional to load, and
(*iii*) variable.

Each type acts on the load extension graph form as shown schematically in Fig. 12. Friction around the top anchorage is thought to have two distinct sources:

(*a*) between tendon and grout due to the bending of the tendon units under the bearing plate; this is of the order of 3-6% but can be alleviated by efficient lubrication; and
(*b*) between tendon and bearing plate, which may be up to 50% if the bearing plate and anchor block are badly positioned.

Commonly however, up to 10% total frictional losses in the top anchorage assembly may be expected.

Data on errors in prestressing measurements have been supplied by Longbottom & Mallett (1973). This information suggests that the difference between the observed and the theoretical force may be as much as 15% when dealing with rock anchors (Table IV).

TABLE IV. ESTIMATED ERRORS ASSOCIATED WITH THE PRESTRESSING OF ROCK ANCHORS

Source	Variation
Different type of manometer	± 1%
Typical manometer error	± 2%
Internal jack friction	± 2%
Error in reading extension	± 1%
Stress-strain & production tolerance of tendon	± 6%
Calculation error	± 3%

However since these values will rarely act together, a more likely error is estimated to be ± 5%.

Remarks

It is encouraging to observe the increasing use of prestressing to load anchors, thereby subjecting the overall system to a stress history and so improving subsequent performance in service.

Measurement of anchor load in the field is generally regarded as a simple operation, although more regular calibration of jack and pressure gauge equipment would undoubtedly lead to a higher degree or precision.

Accurate monitoring of extensions is the exception rather than the rule, because these measurements in the field are often considered to be awkward or time-consuming, and in any case, less important than the ultimate attainment of anchor loads.

Insufficient attention is paid to the interpretation and consideration of the monitored load-extension data. As a result there has been little progress in the understanding of basic anchor behaviour with particular regard to component movements of the overall anchor system.

In spite of the background technology available in the field of prestressed concrete, there is currently a lack of awareness concerning the sources of discrepancies between the theoretical and field results for rock anchors.

During the stressing operation safety standards would be considerably improved by the use of protective barriers and warning signs.

TESTING

Precontract component testing

Prior to use on site, manufactured components such as the tendon and top anchor assembly units should be tested in an independent testing establishment to guarantee component safety factors and ensure efficient performance. Alternatively, it may be acceptable on occasions when employing a standard form of component, to obtain test certificates from the manufacturers in order to facilitate or substantiate the choice of appropriate components.

With regard to the testing of the tendon steel, manufacturers should be requested to supply load-extension characteristics for each reel or batch of material delivered. In the UK, testing and the supply of test certificates and stress/strain diagrams should be carried out in accordance with BS 2691 "Steel wire for prestressed concrete" and BS 3617 "Seven-wire steel strand for prestressed concrete". Useful guidance will also be found in FIP "Recommendations for approval, supply and acceptance of steels for prestressing tendons".

To confirm that the specified minimum stress/strain values have been met, the permanent extension method is used by manufacturers in routine testing. In the case of steel the non-proportional elongation, quoted in the definition of proof stress*, is equal to the permanent elongation which remains after the proof load has been removed. Provided the permanent elongation is less than that defining the proof stress (e.g. less than 1.0%), then the specification has been met.

The normal test procedures is as follows:

(1) An initial tensioning stress of 10% of the specified minimum tensile strength is applied to the test piece (gauge length = 0.6m)
(2) The extensometer is set at zero,
(3) The load is increased to the specified proof stress, and held for 10 seconds,
(4) The total extension is noted,
(5) The load is reduced to just below initial stress, and then increased to the initial stress,
(6) The permanent extension is noted, and
(7) By plotting the results, the modulus of elasticity can be calculated making use of the proportional stress/strain relationship.

Very little has been published on the effect of low temperatures on the ultimate strength of steel tendons. For 1 570/1 720 N/mm² steel wire a slight increase in strength occurs as the temperature falls. Sub zero temperatures (Farenheit scale) would, however, be necessary to produce a 5% increase in tensile strength, without the elongation being affected.

Apart from any question of the effect of temperature change on mechanical properties, it is useful to remember that a change in temperature of 1°C will produce a change in stress in a fixed wire of the order of 1.9 to 2.2N/mm². For applications where a significant range in temperature may be recorded in the anchorage zone, it is clear that provision of a coefficient of thermal conductivity will facilitate the analysis of test results.

Data on fatigue resistance of prestressing steels is also limited, and the manufacturers do not supply endurance diagrams for their products as a routine procedure. As Longbottom (1974) has stated, the provision of such data requires the investigation of a series of stress ranges each about a series of mean stresses (see for example FIP "Recommendations for approval, supply and acceptance of steels for prestressing tendon").

In practice ground anchors are seldom subjected to pulsations of stress of any magnitude relative to the prestress, but if in a particular case significant alternations of stress are predicted, these can be accommodated in the design of the tendon and top anchor components, and by prestressing to the service load plus the fluctuating stress. The successful application of prestressed concrete and steel in railway and highway bridges in resisting impact and fatigue (Lee, 1973) is ample evidence that satisfactory solutions can be produced. Eastwood (1957), Baus & Brenneisen (1968) and Edwards & Picard (1972) have described the fatigue strength of rolled threaded bar anchorages, prestressing strand and some types of wedge grip top anchorages.

With reference to the top anchorage system, which may be regarded as a combination of the tendon, grips, anchor block and load bearing plate or waling acting together, both the grip components which secure the bar, wire or strand within the top anchorage and the complete top anchorage assembly should be tested in accordance with BS 4447 "The performance of prestressing anchorages for post-tensioned construction". Useful guidance is also given in FIP "Recommendations for acceptance and application of post-tensioning systems".

The British Standard describes three methods of testing prestressing anchorages for prestressing applications.

(*i*) Test of load efficiency of the anchored tendon, consisting of a short term static tensile test on the proposed anchorage attached to the tendon. The load efficiency

$$\left(\frac{\text{test failure load}}{\text{average UTS of tendon}}\right)$$

must not be lower than 92%, where the average UTS of the tendon is determined in accordance with BS 18 "Methods of testing metals" and BS 4545 "Methods for mechanical testing of steel wire", as appropriate.

The characteristic strength of the anchored tendon is calculated as the characteristic strength of the tendon times the actual efficiency. In this test limits of percentage elongation are also stipulated.

(*ii*) Test of dynamic behaviour of the anchored tendon where a fluctuating force between 0.60 and 0.65 fpu at a frequency not exceeding 10Hz is applied for a minimum of 2×10^6 cycles. Loss of initial cross-sectional area of the tendon due to fatigue must not exceed 5%. It is considered that this dynamic test is only relevant where the anchor application involves fluctuating stresses which are transmitted to the tendon.

(*iii*) Test of force transfer to the load bearing block, consisting usually of a short term static compressive test on the complete top anchorage assembly to ensure that the load bearing block can continuously support a minimum force of 1.1 fpu.

It is suggested that the test of force transfer to the load bearing block of the form described in BS 4447: 1973 should be applied to all types of top anchorage assembly so that bearing plates, walings, and the additional reinforcment placed in a concrete diaphragm wall are subject to the same design and performance checks that are currently applied to reinforced concrete load-bearing blocks in prestressed concrete. The design of load-bearing blocks is currently covered by the recommendations of CP 115 "The structural use of prestressed concrete".

Bearing in mind the application of rock anchors in excavation engineering it is noteworthy that the German DIN 4125: 1972 stipulates that the anchor head should be in a position to bear secondary stresses imposed by unforeseen flexure with adequate safety e.g. by deformation of the excavation structure or by angle deviation from the planned axial direction of the tendon.

With reference to jacking equipment the authors are unaware of any codes which specify test procedures. In the light of discussions with jack and pump manufacturers it is recommended that all jacks and ancillary equipment should be tested in the factory to a proof loading or pressure equivalent to at least 1.25 times the rated capacity. Overloading above the maximum rated capacity must not be permitted in the field and the choice of jack should be such that the rated capacity can accommodate 85% of the characteristic strength of the largest tendon (largest tendon unit for a monojack) in the group of anchors being considered.

When new equipment is delivered certificates concerning proof testing, internal losses and load-pressure conversion charts or factors should be supplied by the manufacturer.

To ensure that the monitored data is accurate, pressure gauges, like the equipment, must be well maintained and calibrated regularly. It is recommended that the gauges should be calibrated for the start of every contract, and then checked on site against a control gauge at monthly intervals or every thirty production anchors depending on usage. Independent calibration of jack equipment is recommended every three months.

*The proof stress is defined as a stress which is just sufficient to produce under load, a non-proportional elongation equal to a specified percentage of the gauge length. The 0.1% proof stress is therefore obtained from the graph by marking off parallel to the straight line (or line of proportionality) a second line at a distance equal to 0.1% extension. The point of intersection of this offset line with the curve gives the proof stress.

Acceptance testing of production anchors

SHORT TERM ACCEPTANCE tests on all production anchors highlight potential difficulties pertaining to service behaviour and provide measured safety factors related to the design working load. These tests are associated with the initial stressing operations and normally include quality control observations over a period of up to 24 hours.

As a first priority, the testing procedure must yield a measured safety factor as determined by overloading for a short period. Such overloads, however, must be compatible with the allowable stresses and safety factors permitted in the country concerned. The relevant details are discussed in Part 1—Design (Table XV)§, and these suggest an encouraging trend towards standard safety factors throughout the world at the present time.

To check the measured performance against that predicted by calculation, it is essential that a load-extension graph be plotted for each anchor, in the manner discussed in Part 3—Stressing§.

In addition, an attempt should be made on either preliminary test anchors or on early production anchors to obtain an indication of fixed anchor movement, since this information allows the analyst to assess a component of permanent displacement which in turn permits a reasonable estimate of the degree of debonding, if any.

Finally, it is necessary to ensure that the service load locked-off after stressing is stable. The alternative methods employed in practice are monitoring loss of prestress with time, and monitoring creep displacement of the anchor with time.

Acceptance testing of temporary anchors in Germany is covered by DIN 4125 (1972). This standard concentrates solely on soil anchors but it is considered relevant to describe the recommendations in this review since the tests are rigorous and have been carefully devised. In addition, important principles are introduced which may well be stipulated for rock anchor testing in the future, particularly in the case of highly weathered materials, or fractured rock masses with interstitial clay.

Each production anchor is subjected to an initial load T_o equivalent to 0.1 T_y (T_y = yield strength of the tendon, assumed to be the 0.1% proof load which is equivalent to 83.5% fpu) after which it is stressed in one operation to 1.2 T_w (T_w = specified working load) and held for at least 5 minutes in non-cohesive soils, and 15 minutes in cohesive soils, whilst tendon extensions are monitored at the top anchorage (Type I test).

Where the spacing between grouted fixed anchor zones is less than one metre, a check on interaction may be necessary. This will involve several adjacent anchors being loaded and observed simultaneously.

For the first ten anchors, and thereafter one in ten of all subsequent anchors, a slightly more rigorous approach is taken and the extensions must be monitored from a fixed datum, at load increments equivalent to 0.4 T_w, 0.8 T_w, 1.0 T_w and 1.2 T_w, due account being taken of strand slippage (Type II test). At the maximum test load the observation times are as stated for the Type I test, and on destressing to the initial load (T_o), an indication of the permanent extension is provided. In the case of prestressed anchors, the working load is subsequently applied and locked-off.

For the Type II test the results are plotted as shown in Figs. 13 a & b and at 1.2 T_w (Point X) where unloading is first carried out, the elastic component (Δ_{ze}) and permanent component (Δ_{zp}) of the total displacement Δ_z can be distinguished. The curve, T_o X_e in Fig. 13b is taken as an approximate path for the elastic displacement.

It is further specified that at least 5% of the anchors must be tested up to 1.5 T_w, bearing in mind that the maximum test load cannot exceed 0.9 T_y (Type III test). At the maximum test load the observation times are as stated for the Type I test.

In general, the acceptance regulations are met for Type I tests, when at a load of 1.2 T_w the displacements stabilise within the observation time, and when the elastic extension curve lies between two boundary lines plotted on the load-extension graph.

The upper boundary line (*a*) corresponds to the tendon extension equivalent to the free length plus 50% of the fixed anchor length, or 110% of the free length in the case of a fully decoupled tendon with an end plate or nut. The lower boundary line (*b*) corresponds to 80% of the free length of the tendon. It is important to emphasise that account should be taken of sources of error as already described in Part 3—Stressing, and generally it is merely recommended that the observed load-extension line should be compared with the calculated theoretical extension due to the elastic extension of the free length of the tendon.

The permanent displacement, calculated with the aid of the approximate elastic extension line T_o X_e should conform closely with the results of the basic test but the permanent displacement (Δ_p) must not be greater than that observed for the basic test over the load range T_w to 1.2 T_w (see "Special test anchors").

For Type II and III tests, the acceptance conditions are met when at maximum test load the creep displacement stabilises within the observation time, and when the free length of the tendon and permanent displacement have been proved in a similar way to the Type I test, through back-analysis of the observed extensions.

In the case of permanent anchors, generally regarded as having a service life in excess of two years, current thinking in Germany is illustrated in the Draft DIN 4125 (1974) which has been published for comment. In this document, it is suggested that each anchor should be tensioned from the initial load T_o to 1.5 T_w, with a preliminary reading at T_w. The anchor is then unloaded to T_o, the permanent elongation is measured, after which the anchor is retensioned to T_w.

For the first ten anchors, and thereafter one in every ten, the test load is to be applied at stages, 0.4 T_w, 0.8 T_w, 1.0 T_w, 1.2 T_w and 1.5 T_w. Unloading then occurs in the same stages to T_o, before T_w is re-applied.

The displacements occurring at 1.5 T_w should be measured 1, 2, 3, 5, 10, and 15 minutes after lock-off. The specified observation period of 15 minutes should be extended if displacements occurring between 5 and 15 minutes are greater than 0.5mm, and monitoring should be continued until a clear estimate of the creep rate is possible. An observation period of 5 minutes is considered sufficient in frictional soils, provided that the displacements are smaller than 0.2mm.

The results of these measurements compare favourably with test anchor results, and a comparison of elastic extensions and the creep rates is usually sufficient. The acceptance test is considered to be satisfactory if the elastic extensions fall between the two boundary lines (*a*) and (*b*) previously described. Further, the creep should be less than 2mm at a load of 1.5 T_w (see "Special test anchors").

With regard to acceptance testing in France, Bureau Securitas (1972) states that overloads of 1.2 T_w and 1.3 T_w should be applied to temporary and permanent production anchors, respectively. In the case of permanent works, where anchors are in service for more than 18 months, it is further suggested that 5% of all anchors could be tested to 1.5 T_w. No maximum permissible stress is specified for the steel tendon, but the Bureau warns that great viligance is required when the elastic limit is exceeded (83.5% fpu), and normally the test would be stopped if the extension reached 150% of the extension at the 0.1% proof stress.

Accurate estimation of load losses e.g. through friction, is emphasised when plotting load-extension data, and an accuracy of not less than 3% is stipulated for manometers. Tensioning by stages starts at 0.15-0.20 T_t and at least five stages are recommended in order to draw accurately the load-extension diagram. In frictional soils the test load is held for 1-2 min. During this time the displacement should not exceed 1mm and the observed free length of the tendon, based on back analyses of the load-extension diagram, should lie between the theoretical free length and the theoretical free length plus 50% of the fixed anchor length. For anchors with a working life less than nine months, an observed free length equivalent to 90% theoretical free length is accepted. If these tests are satisfactory the service load is locked off plus an allowance for losses.

In cohesive soils, the test load is held for five minutes, and the curve of displacement with respect to time should compare closely with the performance of anchors subjected to creep tests (see "Special test anchors"), in addition to complying with the extension criteria described above.

In Czechoslovakia the draft standard for prestressed rock anchors (Klein, 1974) stipulates the test loading of all temporary anchors to 1.2 T_w in cycles as shown in Fig. 14: (a higher test loading for permanent anchors is expected but yet to be specified). The maximum permissible stress in the steel tendon is the 0.2% proof stress which is equivalent to 87% fpu. The observed displacements are separated into elastic and permanent portions and the observed elastic displacement at 1.2 T_w should lie between the boundary lines (*a*) and (*b*) as specified in DIN 4125 (1972). The permanent displacement due to the increase in load from T_w to 1.2 T_w should not exceed by more than 10% the permanent displacement obtained in the basic anchor test over the same load range (see "Special test anchors"). With regard to creep under a constant service load, it is stipulated that the displacement should not exceed 0.135mm/m of free tendon for every tenfold increase in time. To simplify measurements on production anchors, the draft Code suggests that constant time intervals should be chosen for the observations, and that changes in displacement must not increase in these time intervals. For the specific time intervals in Fig. 15, the displacements must be less than 0.02mm/m of free tendon, and for acceptance testing, the total period of observation must be at least ten minutes. Finally, the creep displacement is compared with the results

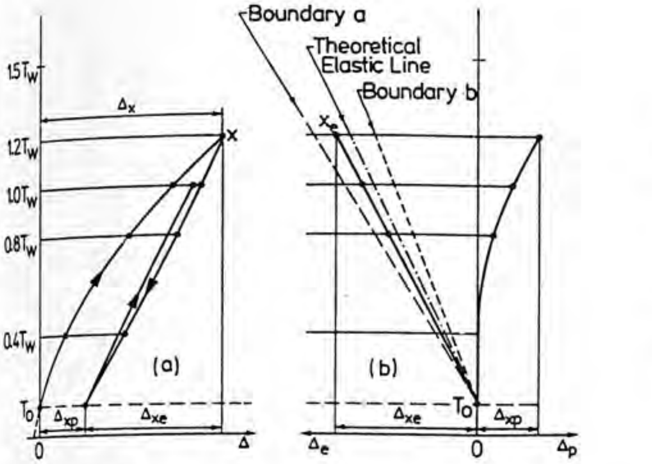


Fig. 13 (above). Stressing programme for acceptance tests
(after DIN:4125-1972)

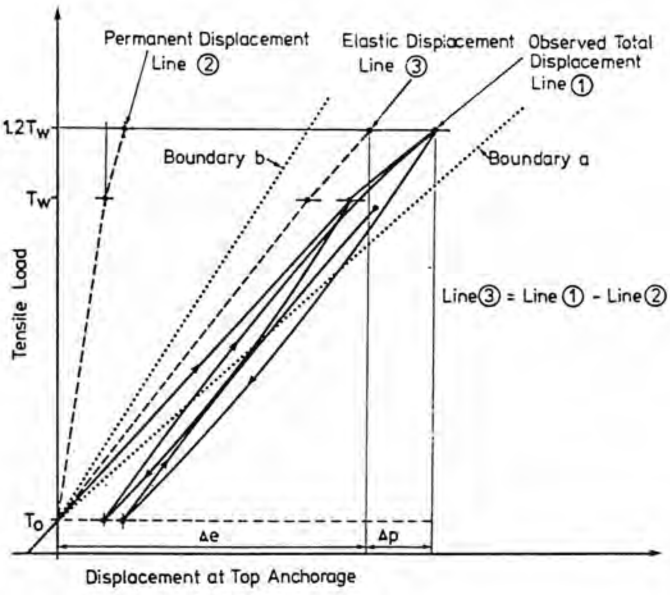


Fig. 14 (right). Working diagram for acceptance tests
(after Czech Draft Code, 1974)

from basic tests.

On every site, it is specified that the first three production anchors and 5% of the remainder should be subjected to a more rigorous test loading to 1.4 T_w and 1.5 T_w for temporary and permanent works, respectively. A service life of less than two years is considered temporary.

The FIP final draft (1973) suggests that the tensile stress in the tendon must never exceed 0.9 T_y (75% fpu, assuming T_y is equivalent to the 0.1% proof stress) and all production anchors should be tested to 1.2 T_w and 1.3 T_w for temporary and permanent works, respectively. A service life of less than two years is considered temporary.

Details of the acceptance test are shown in Fig. 16 and extensions are monitored at load increments equivalent to 0.15-0.20 T_f. For soils and rock not susceptible to creep the test load is held for 2-5 min., and the anchor is accepted if:

(*i*) no noticeable displacement (approx. 1mm) is observed during the period of observation, and

(*ii*) the measured total displacement at the top anchorage is in reasonable agreement with the results of the "extended acceptance" test (see below).

For soils and rocks susceptible to creep, the observation period at constant test load must be long enough to enable the relationship between creep displacement and time to be ascertained, and a minimum period of five minutes is specified. The anchor is locked off at the required service load if the measured total extensions and creep displacements conform closely to those of the "extended acceptance" test.

At the beginning of a contract, it is recommended that between three and ten production anchors should undergo an "extended acceptance" test. The stressing programme is shown in Fig. 17 and this test is applied to approximately 10% of the production anchors constructed thereafter.

In this test the anchor is accepted if:

(*a*) the displacement of the anchor under test load has stabilised within the observation period, and

(*b*) the measured elastic tendon extension corresponds to the calculated elastic extension.

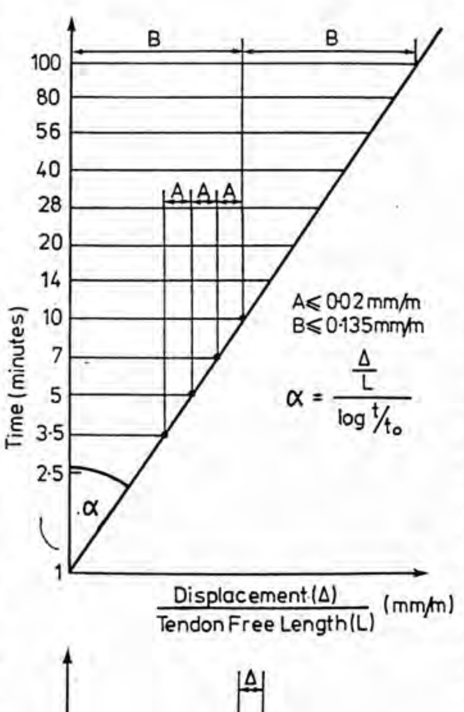


Fig. 15. Working diagram for acceptance criteria for creep displacement
(after Czech Draft Code, 1974)

In connection with (*b*), the calculated free tendon lengths based on the observed elastic extension of the tendon must not exceed the free tendon length plus 50% of the fixed anchor length or 110% of the free length, or be less than 90% of the free tendon length.

Current practice in Italy has been revealed by Arcangeli and Tomiolo (1975) of Rodio. From an initial seating of 0.10 Rak (Rak = characteristic tensile rupture stress of steel), extensions are recorded at 0.15 Rak intervals up to 0.85 Rak. This load is applied usually for 10-15 minutes until creep losses in the steel are negligible (less than 0.1mm in 5 min.). Thereafter following destressing down to 0.3 Rak in 0.15 Rak increments, the anchor is restressed to 0.85 Rak before locking off at the required load.

All anchors are tested in this way to provide a measured safety factor of 1.3 and to compensate for frictional effects and lock-off losses the procedure of Fenoux and Portier (1972) is used. In general the results from each site or geotechnically distinct anchor area are analysed and compared statistically to verify the service conditions of the installations.

In the United States, PCI (1974) suggests the test loading of every anchor to at least 1.15 T_w. During the test loading the prestressing load in the tendon should not exceed 80% fpu. The maximum test load is usually applied for up to 15 minutes, and extensions should not diverge by more than 10% from the calculated values, otherwise an investigation is required. For temporary anchors in rock (up to three years where there is no apparent danger of corrosive attack) it would appear that extension measurement is not usually required. With reference to losses of prestress during service, PCI states that meaningful lift-off

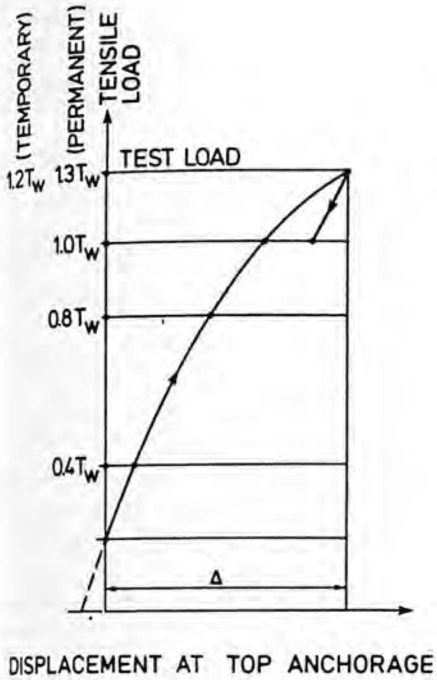


Fig. 16. Stressing programme for acceptance tests (after FIP Draft, 1973)

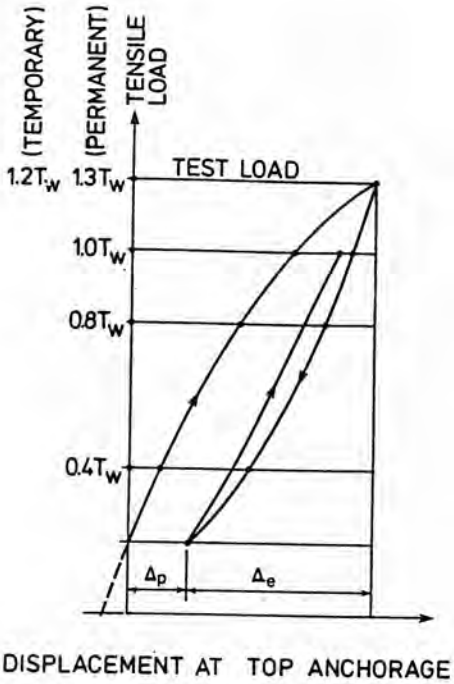


Fig. 17. Stressing programme for extended acceptance tests (after FIP Draft, 1973)

checks can be carried out after 24 hours and that in most cases of rock anchors the primary time dependent loss is steel relaxation.

In Britain, CP 110 (1972) permits tensile testing to 80% of the characteristic tensile strength (fpu) of the steel tendon and the authors' recommendations on safety factors related to acceptance tests are reaffirmed in Table V.

The above recommendations are gradually being adopted in Britain, but for temporary and permanent anchors the most common method in current practice consists of test loading in increments up to 1.25 T_w with a minimum observation period of five minutes at this maximum test load. The anchor load is then reduced to zero before restressing in increments up to a lock-off load of 1.10 T_w (Littlejohn, 1970). Tendon extensions are monitored but since the movement at the top anchorage during the initial loading stage may comprise fixed anchor displacement, tendon extension, wedge pull-in, bearing plate and structural movement, the interpretation and analysis of the data are usually restricted to the load-extension graph obtained during the second loading cycle.

The observed extension should compare closely with the value estimated from the free length of the tendon and the permissible discrepancy on any site varies, the value often being directly related to the accuracy of the measurements and parameters used in the calculation.

In order to give some insight into service behaviour of anchors in Britain, emphasis to date has been placed on monitoring loss of prestress with time which is a simple alternative to the German and French practice of measuring creep displacement. A lift-off check is carried out immediately after lock-off to measure the actual residual load in the anchor. This residual load, which is usually 1.10 T_w, is then checked after 24 hours. Bearing in mind the errors in measurement referred to in Part 3 "Stressing", a loss of up to 5% is acceptable in practice. If the load is less than 0.95 T_w, the anchor should be replaced or otherwise dealt with as agreed with the Engineer.

Where the anchor load lies between 0.95 T_w and 1.05 T_w, the tendon should be retensioned to 1.1 T_w, and retested after a further period of 24 hours. If the anchor, after three such tests, still fails to retain a load of 1.05 T_w, the anchor should be derated or replaced, as agreed with the Engineer. In the former case it is recommended that the load be reduced until no prestress losses are observed, over a period of at least one week, based on daily readings. A safe working load may then be established equal to 62.5% and 50% of this reduced stable load for temporary and permanent applications, respectively (see Table V).

If a component of a multi-unit tendon fails during the stressing stage, a reduced anchor capacity, in proportion to the number of components left, may be agreed with the Engineer, unless the individual components have stresses in service which are below the limits specified. In this situation it may be possible to upgrade the load in each component to compensate to some extent the loss of the redundant component. For example, a tendon consisting of 10 No. 15.2mm Dyform strands might be required in Britain for a permanent anchor with a working load of 1 400kN. In this case each strand would be resisting only 140kN (46.7% fpu) and could therefore be upgraded to 50% fpu (150kN) to give a safe working load on the anchor of 1 350kN if one strand failed. The same approach may be applied if gripping wedge failure occurs and fresh wedges cannot be fitted.

In South Africa the Code of Practice "Lateral Support in Surface Excavations" (1972) stipulates a test load of 1.25 T_w for every prestressed anchor. This load is maintained for a period of not less than ten minutes to test the anchorage, and is then reduced to a load of 1.1 T_w.

Between 24 and 48 hours after lock-off, the tendon is retensioned until the anchor block just lifts off the permanent load bearing plate, and the residual load at this point is recorded. If this residual load is greater than 0.80 T_w but less than 1.05 T_w the tendon should be retensioned to 1.10 T_w and then retested 24 hours later. If after three such retests at 24 hour intervals, the tendon still fails to maintain a load above the working load it should be condemned and replaced, or derated as approved by the Engineer.

In the case of tendons which are to be permanently protected against corrosion by grouting, they may be grouted after the 24-48 hour test but not later than seven days after this test. Such fully bonded tendons are not subjected thereafter to further tests.

In this connection Parry-Davies (1968) emphasises the advantages of leaving the tendon ungrouted over a period of, say, 12 months in order to facilitate tests.

He further reasons that since the working strain in the tendon is only a small fraction of the ultimate strain, a generous safety factor against "catastrophic collapse" is provided. Since a small extension of the tendon supporting a basement excavation, for example, will relieve excess forces

TABLE V. RECOMMENDED SAFETY FACTORS AND TEST FORCES IN BRITAIN

Item	Anchor category	
	Temporary (Life < 2 years)	Permanent
Design or working force (T_w)	62.5% fpu	50% fpu
Test force (T_t)	78% fpu	75% fpu
Measured safety factor	1.25	1.5

which may build up, the safety of the system lies not so much, therefore, in the ratio of actual stress in the tendon to ultimate stress, but in the ratio actual strain to ultimate strain.

Remarks

The value of overloading an anchor to give a measured load safety factor and to impose a stress history which can improve subsequent behaviour, is widely appreciated.

With reference to the interpretation of load-extension data, however, important differences in acceptance criteria are apparent, and clearly use of boundary lines which reflect permissible discrepancies must not be employed inflexibly or without a basic understanding. For example, an anchor with negligible fixed anchor movement but which has apparently debonded along half the fixed anchor might be judged acceptable. An anchor in which only 80% of the applied load has seemingly been transferred to the fixed anchor zone might also be considered satisfactory. These two extremes illustrate that ill-considered use of the extension criteria could be misleading and potentially dangerous expecially when considering the corrosion risk (debonding at tendon interface) or the overall stability (inadequate load beyond potential failure plane).

It is suggested that while load-extension boundary lines are favoured in practice, care and attention is required in interpretation. To alleviate problems of interpretation, the authors recommend that all production anchors should be subjected to at least one stage of cyclic preloading. The analyst should then concentrate on the load-extension plots of the second and any subsequent load cycles, from which most of the initial non-recoverable movements have been removed e.g. plate "bedding-in". In this way, closer correspondence between theoretical and observed extensions should be apparent, easing the analysis of anchor performance.

If discrepancies are still considered significant, on-site discussions are necessary to decide the appropriate action, which may lead to acceptance, derating or replacement of the anchor, depending on the circumstances and the consequences of failure.

Long-term monitoring of selected production anchors

Long-term monitoring over periods in excess of 24 hours checks service behaviour and acts as a control to verify that anchor performance is satisfactory. Furthermore, the collection of data relating loss of prestress or creep displacement to time, type of rock, and anchor load and geometry, will improve understanding of the service behaviour of anchors and could well lead to future refinements in design. In the short term, such data establish if overload allowances applied to the working load at initial load-off, are adequate and realistic.

Long-term losses within the anchor are due to a combination of steel relaxation and anchor creep (see "Service behaviour of production anchors"). The relaxation characteristics of prestressing steel are well known and readily available from manufacturers. Less is known about creep in rock anchor systems largely because basic information regarding the magnitude and distribution of stresses in the fixed anchor zone is not available. Nevertheless, in weathered rock or fractured rock with clay infill, creep losses may be significant and an estimation of the amount to be expected should be gauged from test anchors installed well in advance of full-scale production.

Where test anchor results are not available and the rock is of poor or variable quality, it has been recommended in Britain that periodic checks of anchor stress should be carried out on production anchors as follows:

(*i*) The load in all anchors should be checked 24 hours after stressing to provide an early warning of load loss, if any. This check applies to temporary and permanent anchors.

(*ii*) On a large contract where the consequences of failure are severe, the first ten anchors should be checked weekly for one month, then monthly for the next three months.

(*iii*) Subject to satisfactory results after four months, 5% of all production anchors should be checked at six months, and again at 12 months.

The permissible variation in anchor load is usually $\pm$ 0.1 T_w and restressing is only carried out after careful consideration. For example, in the case of a retaining wall tied back by several rows of anchors installed in a weak shale, loss of prestress due to consolidation of the shale in the fixed anchor zone may be observed without accompanying movements of the retaining wall. In these circumstances remedial measures may not be required.

Bureau Securitas (1972) considers that although the ground anchor tie-back system is now a safe and thoroughly tried and tested method, it is absolutely necessary to plan a monitoring or control procedure which will detect possible failures in time. As a result, periodic monitoring of permanent anchors for a period of at least ten years is compulsory in France.

During the first year, monitoring takes place at intervals of three months, at six month intervals in the second year, and thereafter at yearly intervals. As already indicated, the Bureau classifies anchors according to basic geometry and type of ground at the fixed anchor. In each category, the minimum number of anchors to be monitored is:

10% of production anchors (total installed, 1-50)

7% of production anchors (total installed, 51-500)

5% of production anchors (total installed, over 501)

The Bureau further states that the control apparatus must be reliable, simple, and have an adjustable sensitivity; it need not be a measuring device, and a limit device capable of detecting load losses of between 15 and 25% is adequate. In this connection the authors would add that the control apparatus should also be capable of monitoring prestress gains, particularly in the case of anchors for retaining walls.

In selecting the production anchors to be observed, the FIP Draft Recommendations (1973) indicate that for "extended acceptance" tests, an initial number of 3-10 anchors should be monitored, followed by a percentage of all others—usually 10%.

It would seem that the South African Code "Lateral support in surface excavation" (1972) recommends the most rigorous approach at the present time, namely that each anchor should be tested at the following intervals after stressing unless it is to be permanently protected against corrosion by grouting:

(i) Not less than 24 hours and not more than 48 hours.

(*ii*) Seven days if the 24/48 hour test is satisfactory.

(*iii*) One month if the 7 day test is satisfactory.

(*iv*) Monthly intervals for the first six months and thereafter at three monthly intervals if the first monthly test is satisfactory.

After 12 months, all tendons remaining in service should be tested at intervals laid down by the Engineer; in no case should such intervals exceed six months.

As an alternative to the measurement of loss of prestress, creep displacement may be monitored since test results in Germany and France have indicated that, under constant load, the stabilisation of displacements of the tendon, the fixed anchor, and the ground in the vicinity of the fixed anchor proceeds linearly, when displacement increments Δ are plotted against the logarithm of time. The displacement increments increase with increase of load and when the stresses at the fixed anchor/ground interface approach the ultimate strength of the ground the displacements accelerate in relation to time on a semi-logarithmic scale.

On the basis of these observations certain authorities clearly consider that the displacements may be considered stabilised when, for a constant applied load, the displacements are successively smaller, or that they do not increase more than linearly when plotted on a semi-logarithmic scale against time (see "Special test anchors" in Germany).

In current practice where an attempt is made to gauge the long-term performance, this commonly consists of one lift-off check but the time of observation varies considerably e.g. at 24 hours (Buro, 1972, Mitchell, 1974), 72 hours (Australian Standard, 1973), 7 days (Gosschalk and Taylor, 1970, Chen and McMullan, 1974) or 28 days (Morris and Garrett, 1956). Certainly few production anchor checks are as thorough as those executed by McLeod and Hoadley (1974), all anchors being checked at 3, 7 and 21 days, and 100 out of 1 800 by load cell each day for six months.

Remarks

For economic as well as operational reasons the time involved for the stressing and control of anchors on a construction site should be minimised. The question remains whether it is realistic, or indeed possible, to judge the long-term load holding capacity of the anchor on the basis of a short-term test. Although prestress losses due to lock-off, friction and steel relaxation are predictable, the creep behaviour of different types of rock due to anchor loading is largely unknown. Field experience indicates that such losses may be significant in heavily weathered rock, or fractured rock with clay infill.

A prestress loss of up to 5% in 24 hours or a creep displacement of up to 4mm in 72 hours has been used as an upper threshold of acceptability in practice, but these figures are rather arbitrary and should be regarded as provisional.

Only when creep losses are monitored over long periods for a variety of anchor loads and geometries, and for a wide range of classified rock types, will an accurate predictive capacity be available. In the meantime, therefore, it is recommended that periodic checks of anchor stress or creep displacement should be carried out on anchors whenever possible, and every effort should be made to publish the field data obtained in the form of case histories.

Special test anchors

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

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

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

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

In Germany, the basic suitability of any ground anchor system is ascertained from *basic tests* on at least three anchors in recognised types of ground (DIN 4125: 1972). The construction, testing and subsequent excavation of the anchors must be monitored by a recognised professional institution which also classifies the ground.

Approximately one week after grouting, stressing is carried out and top anchor displacements are measured from a remote datum for different loads above the initial seating load ($T_o \not> 0.1\ T_y$). Proceeding from this initial value T_o, load increments equivalent to 0.15 T_y are applied until failure, or until the yield stress of the tendon is reached (Fig. 18a). After the load increment equal to 0.3 T_y and thereafter at each successive higher load increment, the tendon is unloaded to T_o to provide data on permanent displacements, and to enable calculation of the effective free length of the tendon. The top anchorage displacements occurring at loads below T_o are not measured.

Before each unloading operation displacements are observed under constant load in non-cohesive soils until the movements stop, but for at least five minutes. At 0.6 T_y the load is held for 15 minutes and the associated displacement Δ_1 is noted (Fig. 18b). At 0.9 T_y the observation time is increased to at least one hour (associated displacement = Δ_2). In cohesive soils the observations at 0.6 T_y and 0.9 T_y are continued until the displacement during the last two hours is less than 0.2mm. If the working load (T_w) is less than 0.6 T_y the maximum applied test load should be at least 1.5 T_w (observation time at least 1 hour), and the working load (T_w) should be applied for at least 15 minutes.

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

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

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

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

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

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

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

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

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

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

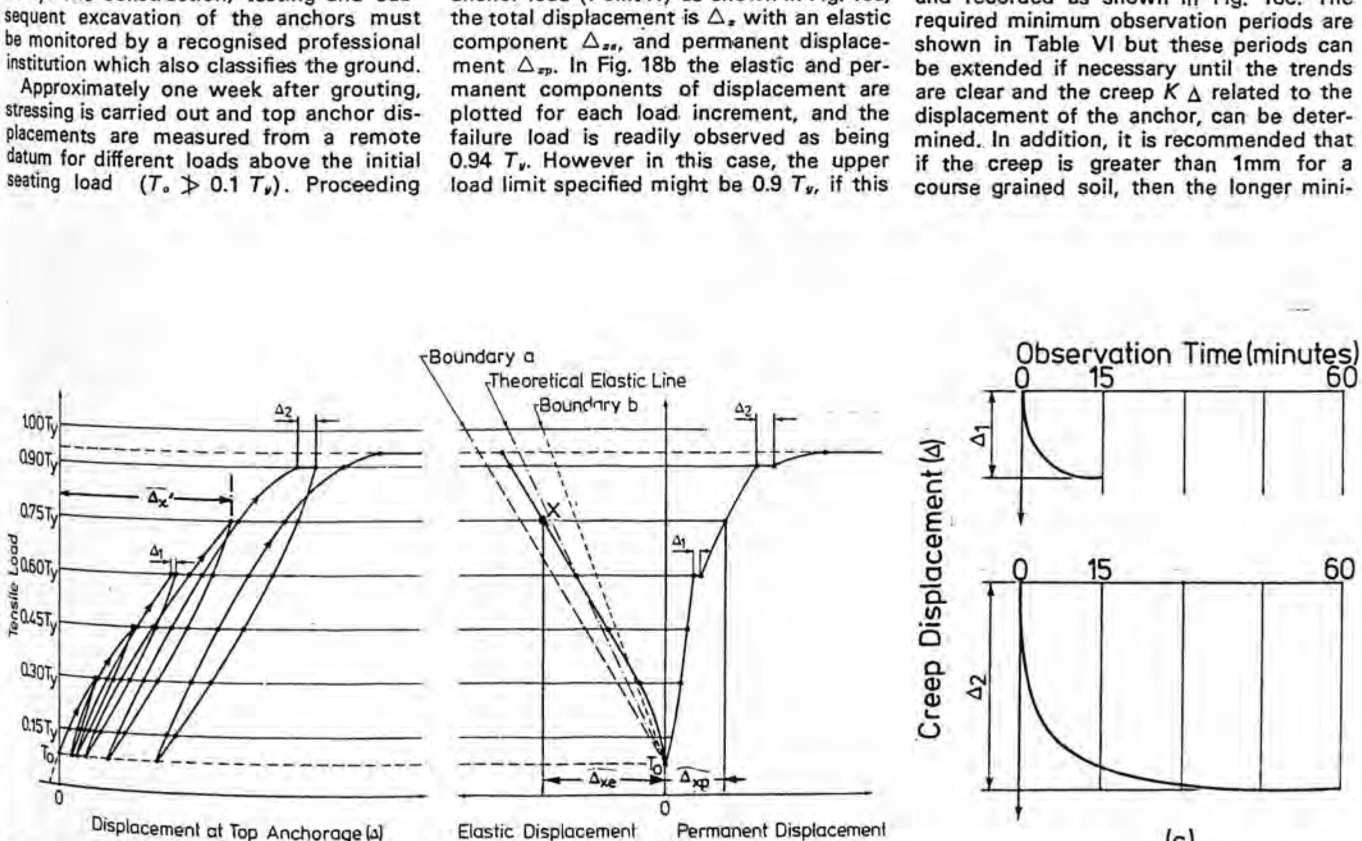


Fig. 18, Stressing programme for basic or suitability tests

(after DIN 4125-1972)

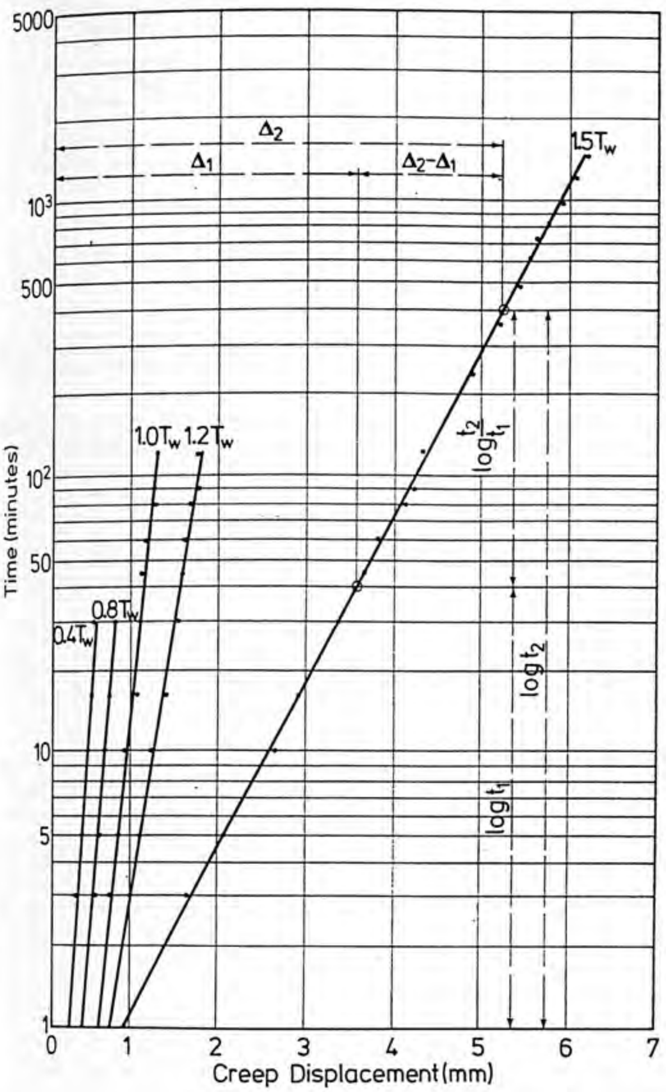


Fig. 19 (left). Method for the determination of $K\Delta$ (after Draft DIN 4125-1974)

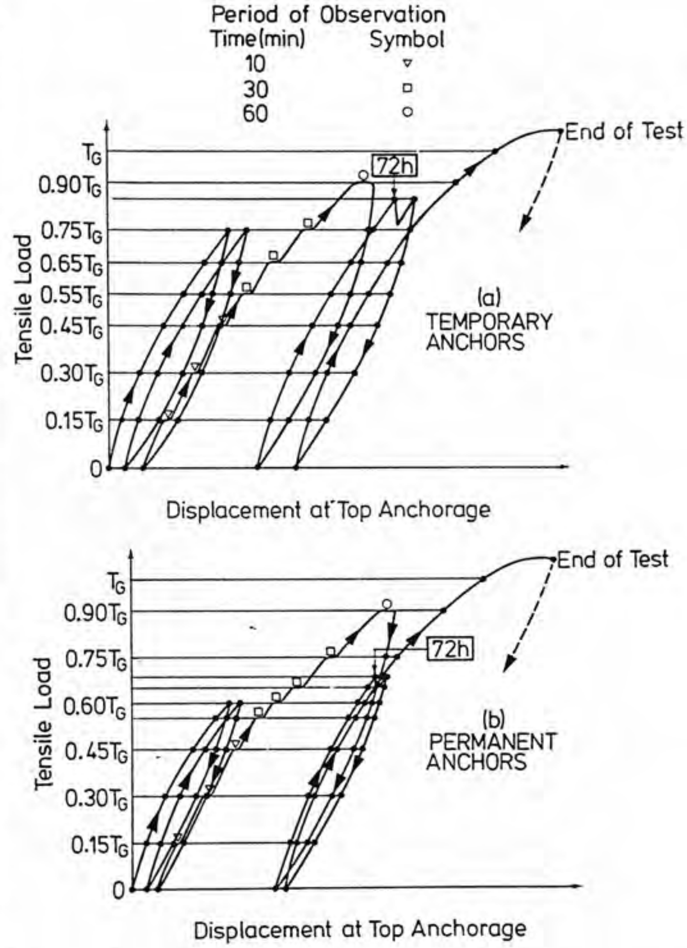


Fig. 21 (above). Stressing programmes in soils where anchor behaviour is known (after Bureau Securitas, 1972)

mum observation periods for fine grained soils should be adopted.

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

$$K\Delta = \frac{\Delta_2 - \Delta_1}{\log t_2/t_1} \qquad \ldots (1)$$

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

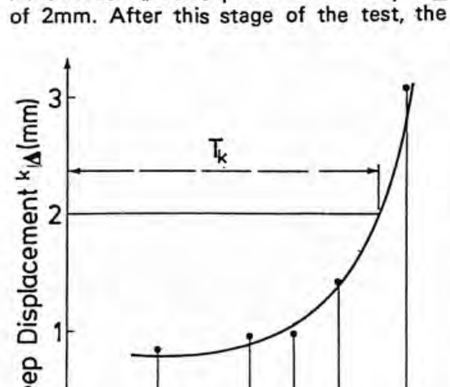


Fig. 20. Method for the determination of limit force T_k (after Draft DIN 4125-1974)

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

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

(*i*) $T_y/1.75$ (T_y = guaranteed yield strength of the tendon),
(*ii*) $T_f/1.75$ (T_f = failure of the bonded fixed anchor), and
(*iii*) $T_k/1.50$ (T_k = limit force for creep $\not>$ 2mm according to equation (*1*) above.

In France, basic test anchors as detailed by Bureau Securitas (1972) are categorised by geometry and ground type, and the minimum number of test anchors is related to the number of production anchors in one category, as shown in Table VII.

TABLE VI. LOAD STAGES AND OBSERVATION PERIODS FOR BASIC AND CONSTRUCTION SITE SUITABILITY TESTS (after Draft DIN 4125: 1974)

Stage of loading		Minimum period of observation	
Basic test $T_o \not> 0.1\,T_y$	Suitability tests* $T_o \not> 0.2\,T_w$	Coarse grained soils	Fine grained soils
$0.30\,T_y$	$0.40\,T_w$	15 min	30 min
$0.45\,T_y$	$0.80\,T_w$	15 min	30 min
$0.60\,T_y$	$1.00\,T_w$	1 hour	2 hours
$0.75\,T_y$	$1.20\,T_w$	1 hour	3 hours
$0.90\,T_y$	$1.50\,T_w$	2 hours	24 hours

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

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

No. of test anchors	No. of production anchors
2	1— 200
3	201— 500
4	501—1 000
5	1 001—2 000
6	2 001—4 000
7	4 001—8 000

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

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

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

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

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

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

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

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

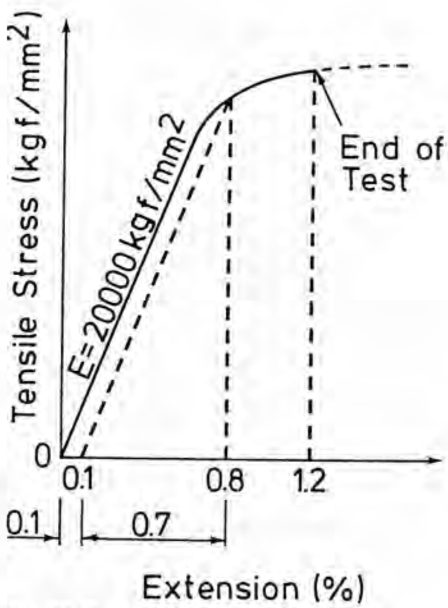


Fig. 22. Typical stress-strain curve for prestressing steel (after Bureau Securitas, 1972)

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

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

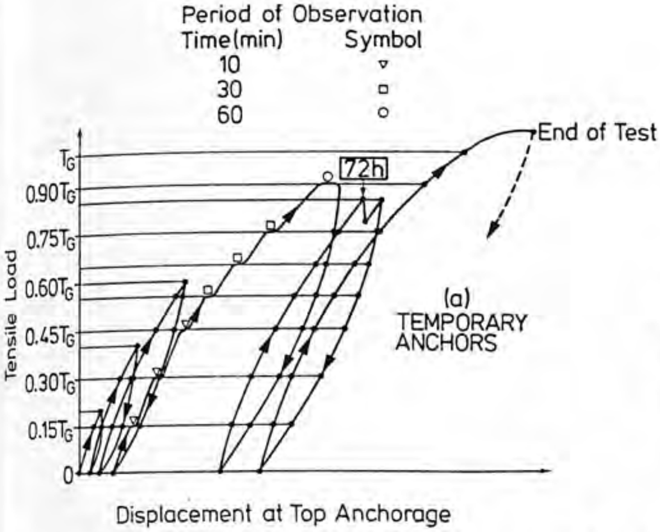
The maximum working load is specified equivalent to 0.67 T_{min} and 0.50 T_{min} for temporary and permanent anchors, respectively (T_{min} = minimum ultimate load for test anchors). If none of the test anchors fails, the maximum working load must not exceed 0.75 T_g and 0.60 T_g for temporary and permanent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

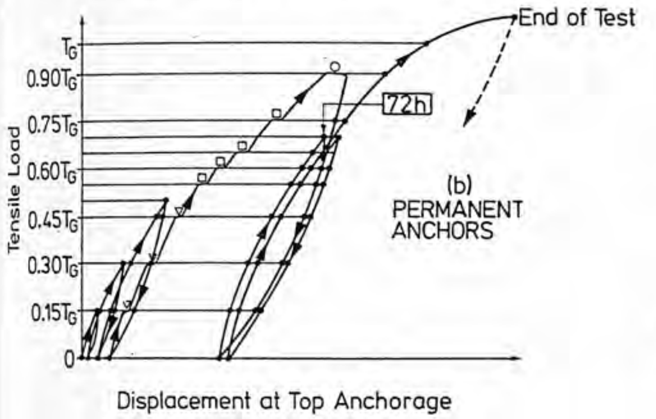
The Czech Draft Code (1974) relates to

TABLE VIII. RECOMMENDED LOAD INCREMENTS AND PERIODS OF OBSERVATION FOR BASIC TEST ANCHORS (after Bureau Securitas, 1972)

Temporary anchors			Permanent anchors		
Load increment		Period of observation (minutes)	Load increment		Period of observation (minutes)
Initial two load cycles*	Third load cycle		Initial two load cycles*	Third load cycle	
$0.15\,T_g$	$0.15\,T_g$	10	$0.15\,T_g$	$0.15\,T_g$	10
$0.30\,T_g$	$0.30\,T_g$	10	$0.30\,T_g$	$0.30\,T_g$	10
$0.45\,T_g$	$0.45\,T_g$	10	$0.45\,T_g$	$0.45\,T_g$	10
$0.55\,T_g$	$0.55\,T_g$	30	$0.55\,T_g$	$0.55\,T_g$	30
$0.65\,T_g$	$0.65\,T_g$	30	$0.60\,T_g$	$0.60\,T_g$	30
$0.75\,T_g$	$0.75\,T_g$	30		$0.65\,T_g$	30
	$0.90\,T_g$	60		$0.75\,T_g$	30
				$0.90\,T_g$	60

*For these load cycles, there is no pause other than that necessary for the recording of extension data





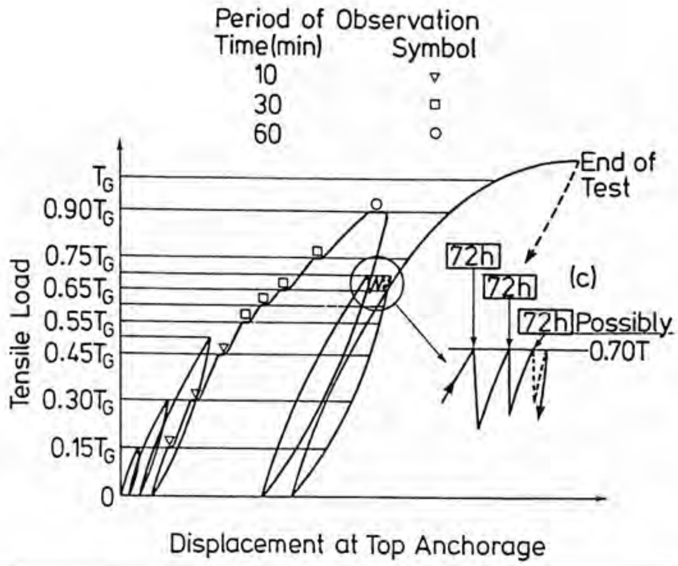


Fig. 23 (a & b), (above). Stressing programmes in soils where anchor behaviour is not known. (c), (above right), Stressing programme where creep replacement is excessive
(after Bureau Securitas, 1972)

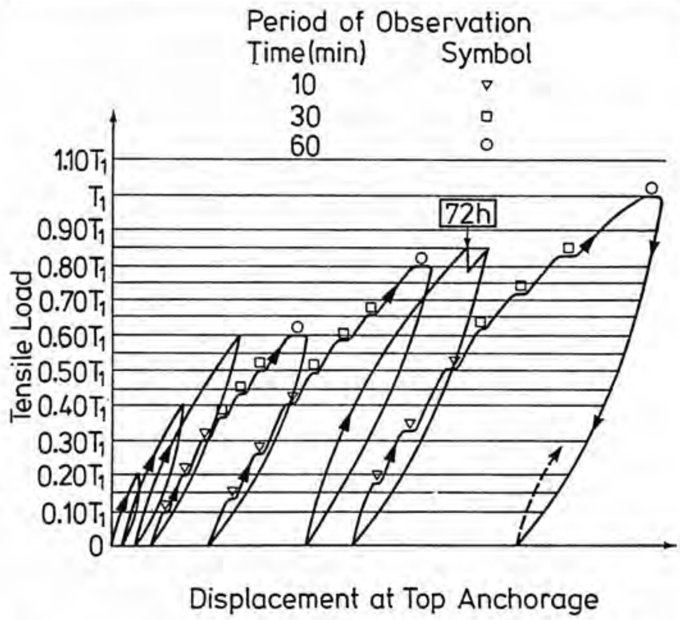


Fig. 24 (right). Temporary anchors: stressing programme for the second test anchor after failure of the first at load T_1
(after Bureau Securitas, 1972)

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

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

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

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

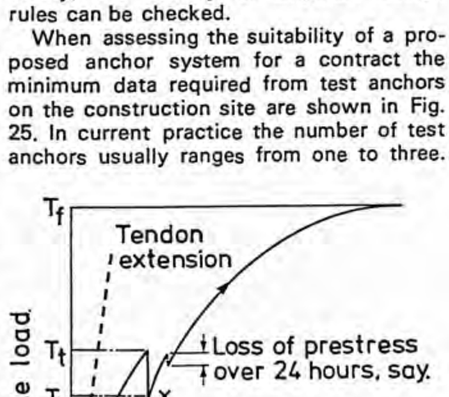


Fig. 25. Minimum stressing programme for test anchors (after Littlejohn, 1970)

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

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

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

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

Remarks

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

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

(*ii*) a check on the suitability of the proposed anchor system for the construction site,

(*iii*) advance warning of construction difficulties, and

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

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

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

Monitoring of the overall anchor rock structure system

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

This type of monitoring is particularly important in excavation engineering e.g. stabilisation of opencast pit slopes, where it is advantageous to observe overall behaviour of the anchored slope as excavation proceeds.

Clearly, monitoring of overall behaviour is expensive and time consuming and in practice may be restricted to major mining operations or prestigious civil engineering projects. Nevertheless, only by such studies in the field can important concepts relating to overall stability and group effects be verified.

SERVICE BEHAVIOUR OF PRODUCTION ANCHORS

Introduction

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

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

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

Time-dependent behaviour of steel tendons

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

Relaxation

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

Details of tendon relaxation have already been presented in Part 1—Design, of this review. However, it is relevant at this point to consider the major conclusions reached by Antill (1965), Bannister (1959), and Mihajlov (1968):

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

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

(*iii*) The rate of load relaxation increases rapidly with temperatures above 20°C.

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

In fact for initial stresses greater than 0.55 fy the relationship is

$$\frac{f_s}{f_i} = 1 - \frac{\log t}{10}\left(\frac{f_i}{f_y} - 0.55\right)$$

where f_s = residual stress after time t
f_i = initial stress.
f_y = 0.1% proof stress at working conditions and temperatures, and,
t = time in hours after application of initial stress

(*v*) With initial stresses of 70% GUTS, restressing at 1 000 hours reduces the amount of ultimate relaxation to almost one-quarter of its normal value and for initial stresses of 80% GUTS the reduction is about one half. Insufficient information is available at present to permit firm conclusions with respect to the effect of restressing at 100 hours.

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

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

(*viii*) A feature of importance in the field is the effect of the design of strand jacks upon the relaxation behaviour of the prestressed strand. The tendency of strand to "unwind" under load has been discussed by Bannister (1959): it arises from the presence of a torsional component approximating to 10% of the load applied to the tendon. The presence of this component would appear to have a marked effect upon relaxation losses and in tests on 12.7mm strand (Duckfield, 1964), the relaxation at 1 000 hours was found to be of the order of about 5% and 8% with and without torsional restraint, respectively. Hence, for practical purposes, those jacks designed with a key way or other device to prevent rotation during stressing may be preferred.

Creep

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

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

(a) The creep rate $\alpha(F)$ increases over the range 0-30% GUTS, is constant to the limit of proportionality (68% GUTS in the case studied), and then increases rapidly at higher loads.

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

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

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

To illustrate the importance of creep for a test stress near the limit of proportionality, Fenoux & Portier have stated that the creep in 2 minutes is 0.2mm/m of free length.

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

$\beta(F) = E \times \alpha(F)$

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

Field observations

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

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

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

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

For new dams, Zienkiewicz and Gerstner (1961) have estimated that load loss is primarily due to creep in the concrete of the dam and only secondarily to tendon losses. They computed that an ultimate prestress loss of 9% was possible—compared to an allowance of 10% at the Alltna-Lairige Dam, where the anchors were installed in fissured granite.

Eberhardt & Veltrop (1965) conclude the the 24 hour load check is much too soon to check "one significant possible source of stress loss; namely shrinkage and creep of the concrete". They estimate ultimate load losses to be of the order

Concrete-creep	2.0%
Concrete shrinkage	3.6%
Steel creep	1.0%

but overload by 10% to cover the worst possible case.

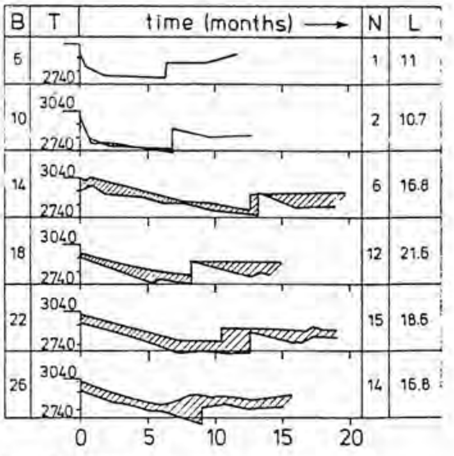
Thompson (1969) describes six test BBRV anchors (fixed anchor length = 9m, diameter = 152mm) as detailed in Table IX, at the John Hollis Bankhead Dam, Alabama.

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

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

A claim by Khaova *et al* (1969) that the long-term load loss was due principally to corrosion of the tendons has proved unfounded (Portier, 1974).

Gosschalk & Taylor (1970) describe various aspects of 2 740kN anchors (fixed



B - Buttress No.
T - Tendon load (kN)
N - No. of tendons in buttress and included in envelope.
L - Length of tendon from top to centre of fixed anchor (m)

Fig. 26. Envelopes of tendon load variations (after Gosschalk and Taylor, 1970)

TABLE X. RECORD OF PRESTRESS LOSS FOR CHEURFAS DAM

Years after stressing	Amount of loss (kN)	% Loss
3	408	4
6	449	4.4
9	459	4.5
18	561	5.5

anchor length 5-6.5m, diameter = 140mm) installed in quartzite at Muda Dam, Malaysia. The stressing procedure involved stressing to 3 030kN, followed by two complete load—unload cycles. The residual load was measured at seven days, and was found to have dropped by up to 450kN. Restressing to 3 030kN resulted in all loads being above 2 887kN three days later. Subsequently 25% of the anchors were monitored, and were found to have "remained fairly steady" as shown in Fig. 26. Measured settlements of the anchorage blocks at service were considered negligible.

The long-term performance of anchors designed for service in other applications has also been briefly recorded.

Comte (1965) describes 1 250kN BBRV anchors in very variable fissured argillaceous schist in the Nendaz Cavern and recorded losses of 4-8%—notably less than the 10% margin allowed. The greater part of this loss was found to occur in the very early stages of a five year period of observation.

In the course of stressing two test anchors (fixed anchor length 6m, diameter = 99mm), Barron *et al* (1971) subjected one to three loading cycles prior to lock-off, whereas the other was loaded directly to the lock-off load. Both were installed in jointed granite, the elastic modulus of which was 40-50 times less for the mass ($0.15 \pm 0.04 \times 10^{+4}$ N/mm²) than for the material $6.3 \times 10^{+4}$ N/mm²).

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

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

Anchor No.	Free Length (m)	Initial load (kN)	Total extension	Residual load (kN)	Time elapsed	Load loss (kN)	% Load loss
1	35	3336	206mm	3336	16 hrs	0	0
2	35	3363	205mm	3278	18 hrs	85	2.5
3	35	3278	206mm	3220	19 hrs	58	1.8
4	35	3336	214mm	3278	31 hrs	58	1.7
5	35	3363	210mm	3336	5 & 10 days	27	0.8
6	29	3363	217mm	3163	5 & 10 days	200	6.0

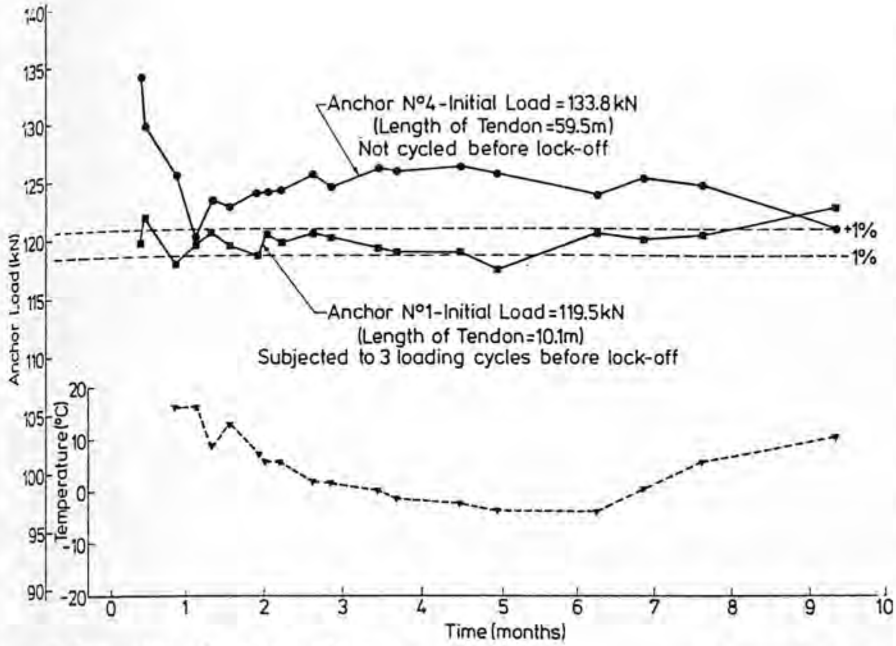
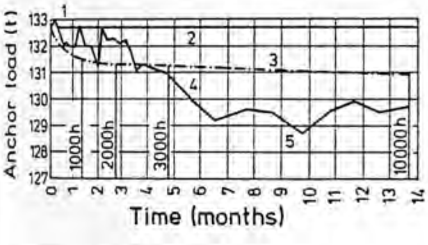


Fig. 27. Comparison of anchor performance with time (after Barron et al, 1971)



1. Initial reading.
2. Designed load (132.7 t)
3. Theoretical tendon relaxation curve.
4. Actual anchor performance.
5. Lowest load recorded. [Loss of 4t = 3%]

Fig. 28. Performance of one monitored anchor (after Moschler & Matt, 1972)

to its maximum level for several cycles, in order to minimise load loss after lock-off. There would appear to be a temperature effect on the apparent load—but this could be due to the susceptibility of the load cells to temperature variation.

Möschler and Matt (1972) presented data on the performance of a 1 330kN VSL anchor (fixed anchor length 4.50m) after test loading to 1 725kN in fractured calcareous schist in the Waldeck Cavern. This is shown in Fig. 28, in which the theoretical steel relaxation curve is also plotted.

As noted previously, one of the largest scale anchor performance programmes described (McLeod & Hoadley, 1974) involved the placement of load cells under 100 anchors (diameter = 76mm) installed in Silurian mudstone in Melbourne. The maximum working load was about 900kN with most locked-off at 250-300kN, following a test load of $1.4T_w$.

Of the results considered satisfactory, the average load loss after 3-6 months was 9%, but 80% of the anchors had an average loss of only 5%. The rather higher apparent losses in the other anchors may have been due to instrument malfunction. On a second site where more care was taken with the load cells, the average loss after one month was only 1%, with no large losses recorded in that time. The authors concluded that in general load loss can be expected, normally 5-10%, but occasionally up to 20%.

One of the most informative case histories has been published by Hutchinson (1970). Six rows of anchors were installed into Upper Chalk on the Isle of Thanet to stabilise a cliff face (Fig. 29a). With a factor of safety on the ultimate chalk-grout bond (0.5N/mm²) of 3.75, the fixed anchor lengths ranged from 5-8m (hole diameter = 102mm) to provide working loads from 167 to 265kN.

The anchors were initially locked-off at $1.25\ T_w$ and checked ten days later when they were restored to the designed initial values. The maximum recorded loss in this time was 14% in one of the upper rows of anchors (in the poorest quality chalk). Load restoration was repeated three times on all anchors, after which all but one in each row were finally grouted up and locked off.

The remaining six anchors were monitored over 1.1 years, and the results after that time are shown in Fig. 29b. A maximum loss of 16% was recorded in the uppermost anchor.

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

Remarks

The quality and accuracy of information published on the time-dependent behaviour of steel tendons would appear to be wholly suitable for application to rock anchor systems.

On the other hand, the authors find that too few long-term records of actual field behaviour provide sufficient data about anchor load and geometry, and rock classification. One important consequence is that optimum overload allowances cannot be determined to accommodate long-term losses.

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

GENERAL CONCLUSIONS

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

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

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

ACKNOWLEDGEMENTS

In the preparation of this series of articles the authors have collated data on a world-wide basis. They are pleased to take this opportunity to gratefully acknowledge the advice and information given by the following engineers in particular: R. Berthier, P. Habib (France); H. Ostermayer, M. F. Stocker (Germany); G. Berardi, C.

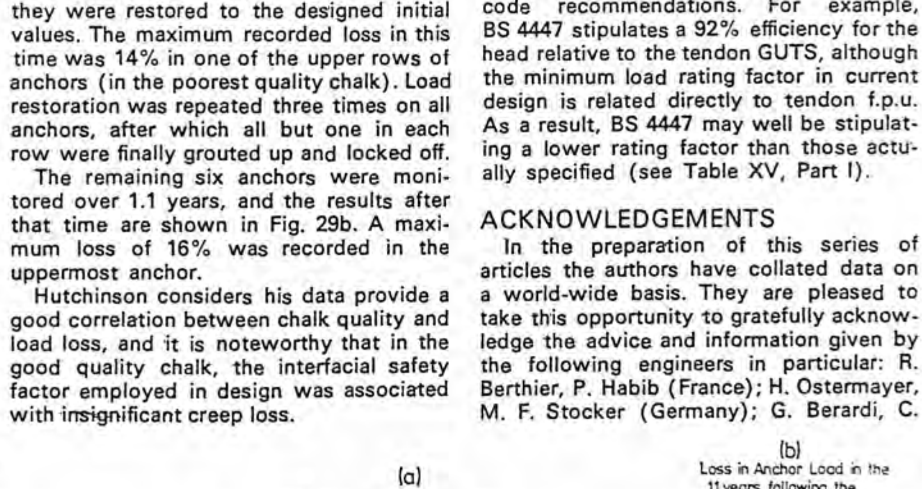


Fig. 29. Anchor performance related to chalk quality (after Hutchinson, 1970)

Mascardi (Italy); P. Matt (Switzerland); L. Hobst, K. Klein (Czechoslovakia); J. I. Adams, D. F. Coates, J. A. Franklin, G. C. Meyerhof, R. J. Van Ryswyk (Canada); J. W. Hilf, R. Miller, R. L. Schmidt, R. E. White, P. T. Wycliffe-Jones (USA); A. J. da Costa Nunes (Brazil); P. I. Mahaffey (Australia); M. Matsuo (Japan); R. Parry-Davies (South Africa); A. D. Barley, J. Harvey, K. W. Longbottom, J. Mitchell, D. Moy, D. G. Price, W. Deppner, and C. Truman-Davis (UK).

References

Arcangeli, E. & Tomiolo, A. (1975): "Modalita' di controllo delle caratteristiche dei tiranti ed elaborazione statistica dei risultati". XII Convegno Nazionale Di Geotecnica, Associazione Geotecnica Italiano—Consenza, 18-21 Sept. 1975, pp 3-13.

Antill, J. M. (1965): "Relaxation characteristics of prestressing tendons", Civ. Eng. Trans. Inst. Eng. Aust., 7 (2) pp. 151-159.

Bannister, J. L. (1959): "Characteristics of strand prestressing tendons" Structural Engineer, 37 (March), pp 79-86.

Barley, A. D. (1974): Private communication.

Barron, K., Coates, D. F. & Gyenge, M. (1971): "Artificial support of rock slopes". Dept. of Energy, Mines and Resources, Mines Branch, Ottawa, Research Report R 228 (Revised), 145 pp.

Baus, R. & Brenneisen, A. (1968): "The fatigue strength of prestressing steel". Proc. FIP Symp. on Steel for Prestressing, Madrid (6-7 June) pp. 95-103.

British Standards Institution: 2 Park Street, London, W.1.

- BS 18 (1962) Methods for tensile testing of metals.
- BS 3617 (1963) Stress relieved seven wire steel strand for prestressed concrete.
- BS 2691 (1969) Steel wire for prestressed concrete.
- BS 4545 (1970) Methods for mechanical testing of steel wire.
- BS 4447 (1973) The performance of prestressing anchorages for post tensioned construction.
- CP 115 (1969) The structural use of prestressed concrete in buildings. (Part 2, metric units).
- CP 110 (1972) The structural use of concrete (part 1).

Bureau Securitas (1972): "Recommendations regarding the design, calculation, installation and inspection of ground anchors". Editions Eyrolles 61 Boulevard Saint-Germain, Paris-Ve (Ref. TA 72).

Buro, M. (1972): "Rock anchoring at Libby Dam". Western Construction (March), pp, 42, 48 & 66.

Chen, S. C. & McMullan, J. G. (1974): "Similkameen Pipeline Suspension Bridge" ASCE, Transportation Eng. Jour., 100 T.E.I. (Feb.) pp. 207-219.

Comte C. (1965): "L'utilisation des ancrages en Rocher et en Terrain Meuble". Bull. Tech. de la Suisse Romande, 22 (Oct), pp. 325-338.

Da Costa Nunes, A. J. (1966): "Slope stabilisation—Improvements in the techniques of prestressed anchorages in rocks and soils". Proc. 1st Cong. Int. Soc. Rock Mech., Lisbon, 2, pp. 141-146.

Deutsche Industrie Norm (1972): "Soil and rock anchors; bonded anchors for temporary uses in loose stone; dimensioning, structural design and testing". DIN 4125, Sheet 1, June 1972.

Deutsche Industrie Norm (1974): "Soil and rock anchors; bonded anchors for permanent anchorages in loose stone; dimensioning, structural design and testing". DIN 4125, Sheet 2, 1974 (Draft).

Duckfield, B. J. (1964): Private communications. United Power Co Ltd., England (see Antill, J. M. (1965, op. cit.)

Eastwood, W. (1957): "Fatigue tests on prestressed concrete beams". Civ. Eng. Publ. Wks. Review, 52, (July), pp. 786-787.

Eberhardt, A. & Veltrop, J. A. (1965): "Prestressed anchorage for Large Trainter Gate". Proc. ASCE, J. Struct. Div. 90(ST6) pp. 123-148.

Edwards, A. D. & Picard A. (1972): "Fatique characteristics of prestressing strand". Proc. ICE, 53, Part 2, (7534), pp. 323-336.

Fenoux, G. Y. & Portier, J. L. (1972): "La mise en Precontrainte des Tirants". Travaux, 54 (449-450), pp. 33-43

FIP (1973): "Final draft of recommendations". FIP Subcommittee on Prestressed Ground Anchors.

FIP (1974): "Recommendations for approval, supply and acceptance of steels for prestressing tendons." Cement and Concrete Assoc., 52 Grosvenor Gardens, London. (Ref. 15.321).

Geffriaud, J. P. & Rouget, M. (1972): "Contrôle de la tension des tirants précontraints. Tirants définitifs de la paroi moulée du Musée Archaeologique de Fourvière à Lyon. Travaux, 54, pp. 44-54.

Gosschalk, E. M. & Taylor, R. W. (1970): "Strengthening of Muda Dam foundations using cable anchors". Proc. 2nd Cong. Int. Soc. Rock Mech., Belgrade, 3, pp. 205-210.

Hanna, T. H. (1969): "Tunnel approaches, subways and underpasses excavation support by anchorage systems". Tunnels & Tunnelling, 1 (1), pp. 27-33.

Hanna, T. H. (1974): Foundation instrumentation. 1st Ed., Trans. Tech. Publications, 372 pp.

Hennequin, M. & Cambeiort, H. (1966): "Consolidation du remblai de Malherbe". Revue Generale des Chemins de Fer (Feb.).

Hutchinson, J. N. (1970): Contribution to Discussion on Soil Anchors. In: Proc. Conf. Ground Engineering, Institution of Civil Engineers, London (16 June), pp. 85-86.

Khavoa, M., Montel, B., Civard, A. & Lauga, R. (1969): "Cheurfas Dam Anchorage; 30 years of controls and recent reinforcement". Proc. 7th Int. Conf. Soil Mech. and Found. Eng., Paper 15-12.

Littlejohn, G. S. (1970): Discussion on Paper "Soil anchors". In proc. Conf. Ground Engineering. Inst. of Civ. Engrs., London, June 16, pp. 33-44, discussion pp. 115-120.

Littlejohn, G. S. & Macfarlane, I. M.; "A case history study of multi-tied diaphragm walls". ICE conf. on Diaphragm Walls & Anchors, London, Sept., pp 113-121.

Longbottom K. W. & Mallett, G. P. (1973): "Prestressing steels". The Structural Engineer, 51 (12), pp. 455-471.

Longbottom, K. W. (1974): Discussion on paper "Prestressing steels" by Longbottom & Maliett (op. cit). The Structural Engineer, 52, (9), pp. 357-362.

MacLeod, J. & Hoadley P. J. (1974): "Experience with the use of ground anchors". Proc. Tech. Session on Prestressed Concrete Foundations and Ground Anchors, pp. 83-85, 7th FIP Congress, New York.

Mihajlov, K. V. (1968): "Stress relaxation of high tensile steel". Proc. FIP Symp. on Steel for Prestressing, Madrid, (6-7 June) pp. 57-78.

Mitchell, J. M. (1974): "Some experiences with ground anchors in London". ICE Conference on Diaphragm Walls and Anchors, London (Sept.) Paper No. 17.

Morris S. S. & Garrett, W. S. (1956): "The raising and strengthening of the Steenbras Dam", (and discussion). Proc. ICE Pt. 1, Vol. 5, No. 1 pp. 23-55.

Möschler, E. & Matt, P. (1972): "Felsanker und Kraftnessanlage in der Kaverne Waldeck II". Schweizerische Bauzeitung, 90 (31), pp. 737-740.

Nicholson Anchorage Co Ltd. (1973): "Rock anchor load tests; for sheet pile bulkhead for the General Reinsurance Corp. Project, Steamboat Road, Greenwich, Conn., USA. Unpublished Report (8 pp.).

Ostermayer, H. (1974): "Construction, carrying behaviour and creep characteristics of ground anchors". ICE Conference on Diaphragm Walls and Anchorages, London (Sept.) pp. 141-151.

Parry-Davies D. (1968): "The use of rock anchors in deep basements". Ground Engineering Ltd., Johannesburgh, South Africa, unpublished paper.

PCI Post-Tensioning Committee (1974): "Tentative recommendations for prestressed rock and soil anchors". PCI Chicago, USA, 33 pp.

Portier, J. L. (1974): "Protection of tiebacks against corrosion". Proc. Tech. Session on Prestressed Concrete Foundations and Ground Anchors, pp. 39-53 7th FIP Congress, New York.

Saxena, S. K. (1974): "Measured performance of a rigid concrete wall at the World Trade Centre". ICE Conference on Diaphragm Walls and Anchors, London (Sept.), Paper No. 14.

Schwarz, H. (1973). "Permanent Verankerung Einer 30m Hohen Stutzwand in Stuttgarter Tonmergel durch Korrosions—Geschützte Injektionsanker System Duplex". Die Bautechnic, 9 pp. 305-312.

Short, A. (1975): "DoE studies creep and stress behaviour at Navy Complex". Construction News, Sept. 4, pp. 28-29.

South African Code of Practice (1972): "Lateral support in surface excavations". The South African Institution of Civil Engineers, Johannesburg.

Standards Association of Australia (1973): "Prestressed concrete Code CA35—1973, Section 5—Ground anchorages" pp. 50-53.

Stefanko, R. & de la Cruz (1964): "Mechanism of load loss in roof bolts". Proc. 6 Symp. on Rock Mechanics, Missouri, (Oct.), pp. 293-309.

Thompson, F. (1970): "The strengthening of John Hollis Bankhead Dam". Civil Engineering (NY), 39 (12) pp. 75-78.

Walther, R. (1959): "Vorgespannte Felsanker". Schweizerische Bauzeitung, 77 (47) pp. 773-7.

Zienkiewicz, O. C. & Gerstner, R. W. (1961): "Stress analysis of prestressed dams". ~~Proc.~~ ASCE Journal of the Power Division, 87 (POL), Pt. 1, pp. 7-43.

A study of rock slope reinforcement at Westfield open pit and the effect of blasting on prestressed anchors

G.S. Littlejohn, B.Sc.(Eng), Ph.D., C.Eng., M.I.C.E., M.I. Struct.E., F.G.S.
P.J. Norton, B.Sc.
M.J. Turner, B.Sc., M.Sc., C.Eng., M.I.C.E., F.G.S.

A study of rock slope reinforcement at Westfield open pit and the effect of blasting on prestressed anchors

G.S. Littlejohn, B.Sc.(Eng), Ph.D., C.Eng., M.I.C.E., M.I. Struct.E., F.G.S.
Technical Director, Colcrete Ltd., Rochester, Kent.

P.J. Norton, B.Sc.
Geotechnical Engineer, Scottish Region, N.C.B. Opencast Executive.

M.J. Turner, B.Sc., M.Sc., C.Eng., M.I.C.E., F.G.S.
Contracts Engineer, Ground Anchors Ltd., Reigate, Surrey.

Summary

The paper describes a test anchor project, initiated by the N.C.B. Opencast Executive as part of a feasibility study concerned with the use of high capacity prestressed anchors to stabilise rock slopes in open-pit mines.

The construction and testing of trial anchors with various grouted fixed anchor lengths are described, which indicate the load holding capacity of the interbedded volcanic and sedimentary strata at Westfield. Maximum interfacial bond values are presented, together with typical load-extension data.

The service behaviour of 1500 kN capacity anchors is monitored with particular regard to prestress fluctuations during blasting. Instantaneous and residual load increases are presented for a major blast in the immediate vicinity of the anchors.

The overall results of the project indicate that anchored rock slopes are technically and economically attractive compared with restoration of slopes to a lower angle, and that prestressed anchors are capable of withstanding close proximity blasting.

General

Westfield Opencast Coal Site, situated in Fife, Scotland, is currently the largest open pit in Britain (Fig 1), with original reserves of 25 million tonnes of recoverable coal and producing a million tonnes of washed coal per annum. The site covers an area of 372 hectares, of which the pit itself occupies 140 hectares with a proposed depth of 200 metres. Below this level a further 1.5 million tonnes of coal is sterilised but which could be tapped with the aid of rock reinforcement of final slopes, or by a complete re-design of slopes to a lower angle.

In order to assess the relative merits of these alternatives and the technical feasibility of rock reinforcement, the N.C.B. Opencast Executive commissioned, as part of their overall study, a full-scale test anchor programme at Westfield to observe method and rate of anchor installation, load holding capacity, and service behaviour when subjected to routine mining operations such as close proximity bulk blasting.

Geology

Coal is being extracted from the Productive Coal Measures and Passage Group strata of Carboniferous Age, the majority of coals lying within the Passage Group which rests unconformably upon interbedded volcanic and sedimentary strata of the Upper Limestone Group. Structurally the site occupies a syncline at the northern end of the Bowhill Basin, with a north

Fig 1. Aerial view of Westfield open pit looking south, showing forward reduction in Phase III to the north, and backfill area in Phase II to the south (courtesy of Costain Mining Ltd – Summer 1975).

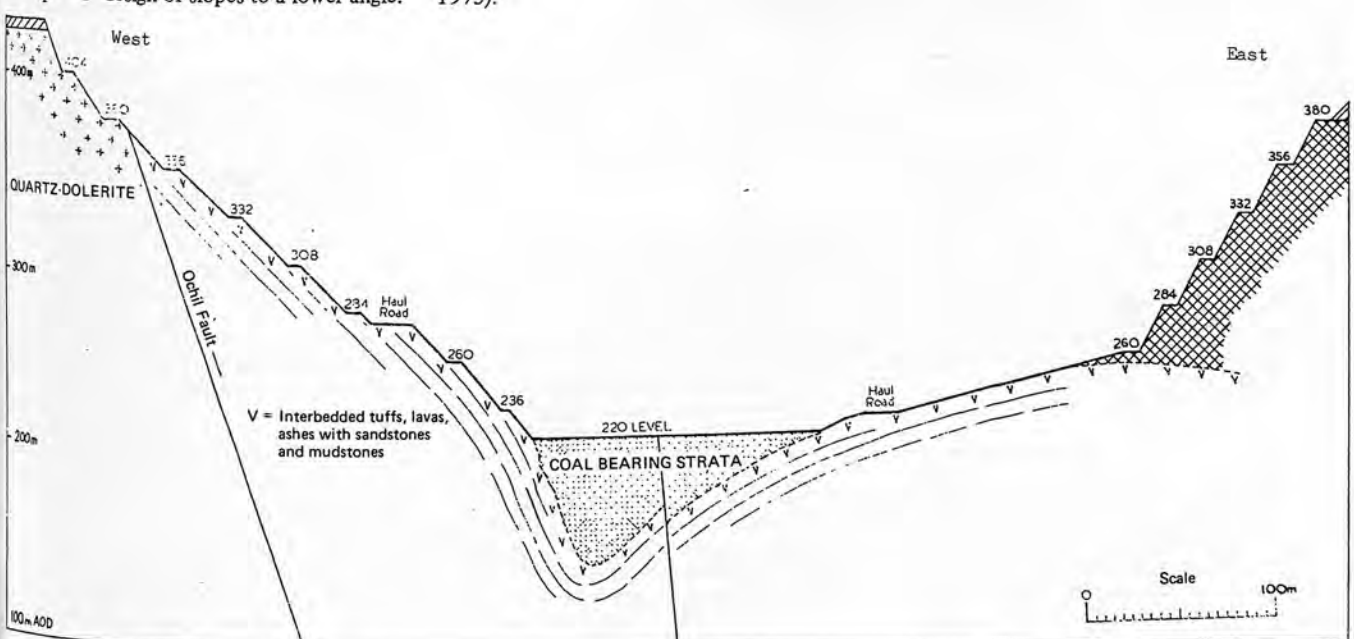


Fig 2. Section of Opencast Pit with Proposed Batters.

east-south west axis plunging towards the north east and lying to the south of the Ochil Fault, beyond which is a large quartz-dolerite sill. The structure is further complicated by faulting and small folds superimposed on the limbs of the main syncline, with resultant dips varying from 70° to horizontal. The steepest dips are generally on the north west limb (west wall) up against the Ochil Fault; to the east of the pit the strata gently falls away to form smaller and more gently dipping structures. The coals thicken up considerably towards the axis of the syncline where the maximum depth from surface to the base coal ('P' Pavement) is 308 metres at the northern end of the structure.

Ground Conditions

The coal-bearing strata comprise the normal sequence of sandstones, siltstones, mudstones and seatearths, with occasional fragmental clay rocks and clay mylonite bands, the latter usually associated with the coals and having low shear strength properties (C residual = 0; ϕ residual = 10° – 15°). Once the coals have been won from this stratum the final batters are predominantly composed of the Upper Limestone Group (Fig 2). This particular zone of strata is extremely variable in both lateral and vertical extent, being composed of rapidly alternating tuffs, lavas, tuffaceous mudstones, agglomerates, with some mudstones and sandstones (Fig 3).

Some of the volcanic tuffs have low strengths (C residual = 0; ϕ residual = 25°) and on weathering undergo a form of slaking. It is also thought that many of the volcanic rocks have suffered penecontemporaneous sub-aerial weathering, resulting in many of the tuffaceous strata being highly fractured and altered. The presence of swelling clays in the mineralogy of the argillaceous rocks is also being investigated. Fortunately, interbedded with these rocks are a number of competent olivine-basalts and some tuffs, which have proved to be suitable for the grouted anchor section in any potential artificial support system.

Water is a continual problem as the site, situated in low-lying ground, provides a focal point for both surface and ground water. The highly stratified nature of the geology gives rise to a complicated system of aquifers, and ground water pressure is a significant factor in stability calculations. The ground water on site is also aggressive, demanding serious consideration of corrosion protection measures for any proposed anchorage system.

Excavation

The site is divided into three working areas; Phase I and part of Phase II are now backfilled and occupy the southern end of the syncline (Fig 1). The forward reduction and main excavation areas are now in operation in Phase III, which is the deepest and also the largest part of the contract operated by Costain Mining Ltd on behalf of the N.C.B. Opencast Executive. The method of extraction is by face shovel and truck in 8 metre horizontal layers or 'lifts', which have been previously blasted to obtain the required fragmentation. Both down-the-hole and rotary rigs are used for blast-hole drilling in patterns which vary from 3m x 3m to 6m x 6m with a depth of about 6m.

Due to the predominantly wet conditions encountered, explosives of the slurry type are used along with the normal AN/FO type (Fig 4).

Eleven electric face shovels owned by the Executive are employed to load overburden into a fleet of 40 – 50 dump trucks which haul rock waste to the backfill areas in Phase II and coal to the washery bunkers. In general the excavation lifts are compounded into units of three to give a final benching system 24m apart, each bench being roughly 10m wide (Fig 2).

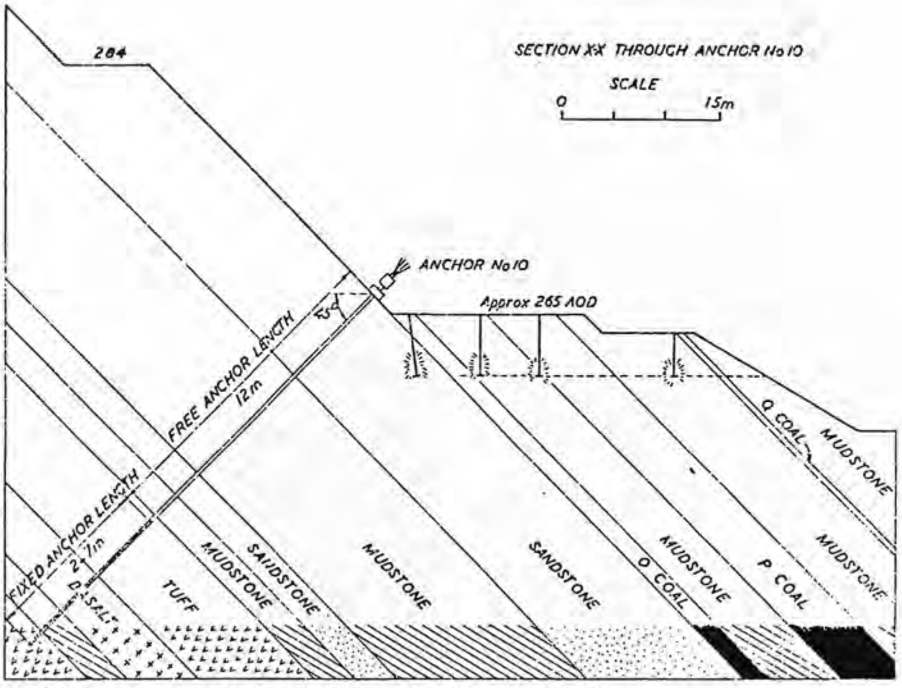


Fig 3. Typical section X-X through Rock Anchor Zone (see Fig 10).

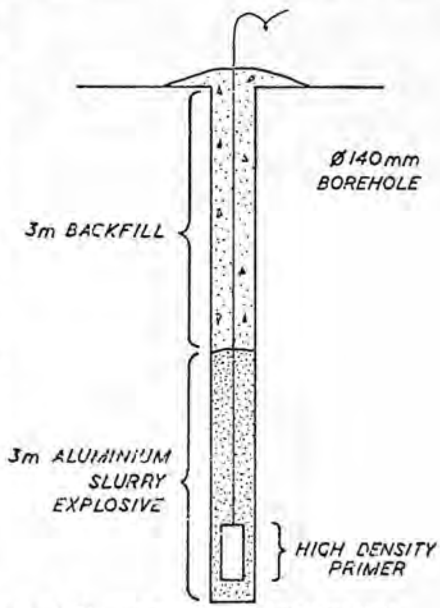


Fig 4. Diagrammatic Section of Blast Hole.

The final product is then washed and screened, and despatched by Merry-Go-Round Trains to a large Power Station.

Slope Design

The overall slope angle on the West Wall varies, but on average is about 40°. This area has been recently re-designed since a number of failures occurred during the winter of 1974/75, and the wall is currently being re-excavated. This re-design allows for the possibility of winning coal from the deeper part of the basin, and also for any unforeseen changes in the structure of 'P' Pavement which may be present at depth. A programme of forward drilling has been set up to give as much advance information on this aspect as possible, since even very small changes in structure at depth in a pit of this size have a compound effect on the batters above. Since it is not often possible to re-excavate batters, then any significant change, especially steepening of the dips, results in lost coal.

In January 1974 a large failure took place in Phase II of the pit involving about 100,000 tonnes of material. Opinion at the time of the failure favoured a double slab and buckle slide caused by the geometry of the slope which was convex upwards at the base. This part failed first with a subsequent slab slide of the then unsupported material above. Only a thin slab of material about 3 – 4m thick lying on top of a clay mylonite band was involved. A similar geometry exists in many of the future slopes on the site, especially where the strata roll over into the central part of the syncline. The revised batters will ensure coal recovery to 220 level. However, without undergoing further costly re-excavation, some form of artificial slope support will be needed if coal recovery below the present proposed working depth is to be attempted. With this in mind the Opencast Executive investigated background technology concerning artificial support of rock slopes[1-4], and the feasibility of using prestressed anchors for this purpose in their larger opencast mines[5]. As a result of this study, the use of prestressed anchors was recommended, but, before any actual design was contemplated, it was thought prudent to carry out a field investigation to prove that anchors could perform in an opencast coal environment, and in particular that the ground conditions at Westfield could be accommodated.

Test Anchors

The anchoring programme consisted of two stages. In the first stage ten anchors of various lengths were installed and stressed to investigate the load-carrying capacity of the bedrock. The second stage involved four "production" anchors with a working capacity of 1500 kN using appropriate grouted lengths as determined from the first stage results. The service behaviour of these "production" anchors was monitored with the aid of load cells while normal pit blasting operations took place.

Anchor Construction

Anchor holes (140mm dia.) were drilled at an inclination of 45° to the horizontal in the open pit in order to encompass as many rock types as possible (Fig 3). A Hands-England crawler-mounted rotary hydraulic drill rig with rock roller bits was used throughout, and penetration rates for the rig averaged around 2.5m/hour, falling to as low as 0.5m/hour in the basaltic rocks.

Eight Stage I anchors were constructed initially, all with a 12m free length, and two each with fixed anchor lengths of 2, 3, 5 and 7m. The fixed anchor length is the grouted zone where load is transferred from the tendon through the cement grout to the surrounding rock. The shorter fixed anchor lengths were installed first, and, as drilling and installation continued, it became clear that many of the fixed anchor lengths were being installed in a competent unweathered oliving basalt, in which failure of the longer anchors would be unlikely. It was decided, therefore, to install a further two anchors of 3m fixed length with shorter free lengths so that they would be anchored in a weaker rock type.

Fig 5. Stressing of Stage 1 Anchor.

The anchor tendons were composed of 13 No. 15.2mm diameter Dyform high tensile steel strands enabling a maximum tensile force of 3120 kN to be applied. Over its free length each tendon was greased and sheathed on site in high density polythene to decouple the tendon from the surrounding rock. Over the fixed anchor length, the steel strands were left unsheathed and were bound into the anchor configuration using specially designed spacers to ensure eventual grout cover around the tendon and between individual strands.

Prior to tendon installation each anchor hole was water-tested by a simple falling head method. Where the acceptance of water was too high, i.e. in excess of 3 litres/min./atmosphere, thus indicating likely cement grout loss, the holes were pre-grouted using a neat 0.4 water/cement ratio grout[6]. Such a pre-grouted hole was then re-drilled the following day and again water-tested. Five of the ten Stage 1 anchors had to be pre-grouted, and were waterproofed in this way.

After a successful water test the anchor tendon, which weighed up to 0.3t, was installed complete with tremie tube, using a purpose-made transporter. Once the tendon had been placed, the water from the water test was blown out with compressed air and the fixed anchor length was tremie grouted. Rapid Hardening Portland cement was used without admixtures for the 0.4 W/C grout. In order to control the quantity of grout placed, a grout level indicator was attached to each tendon at the top of its fixed anchor length.

After the grout had set, the top level of the grouted fixed anchor zone was checked by rodding, and recorded.

During the grouting stage of each anchor, 150mm grout test cubes were taken to monitor the gain in strength of the cement grout, since a minimum cube strength of 28 N/mm^2 was specified before stressing could take place. This strength was generally achieved within 4 days, although stressing was normally carried out after 7 days.

The construction sequence for Stage 2 anchors was generally similar to those of Stage 1, except that tendons consisted of 10 No. Dyform strands, and both the fixed and free anchor lengths were fully grouted in a single operation. Shortly after grouting the grout at the top of each hole was flushed out to give clearance below the anchorage block required for stressing, thereby avoiding a strutting effect between the grout column and the block.

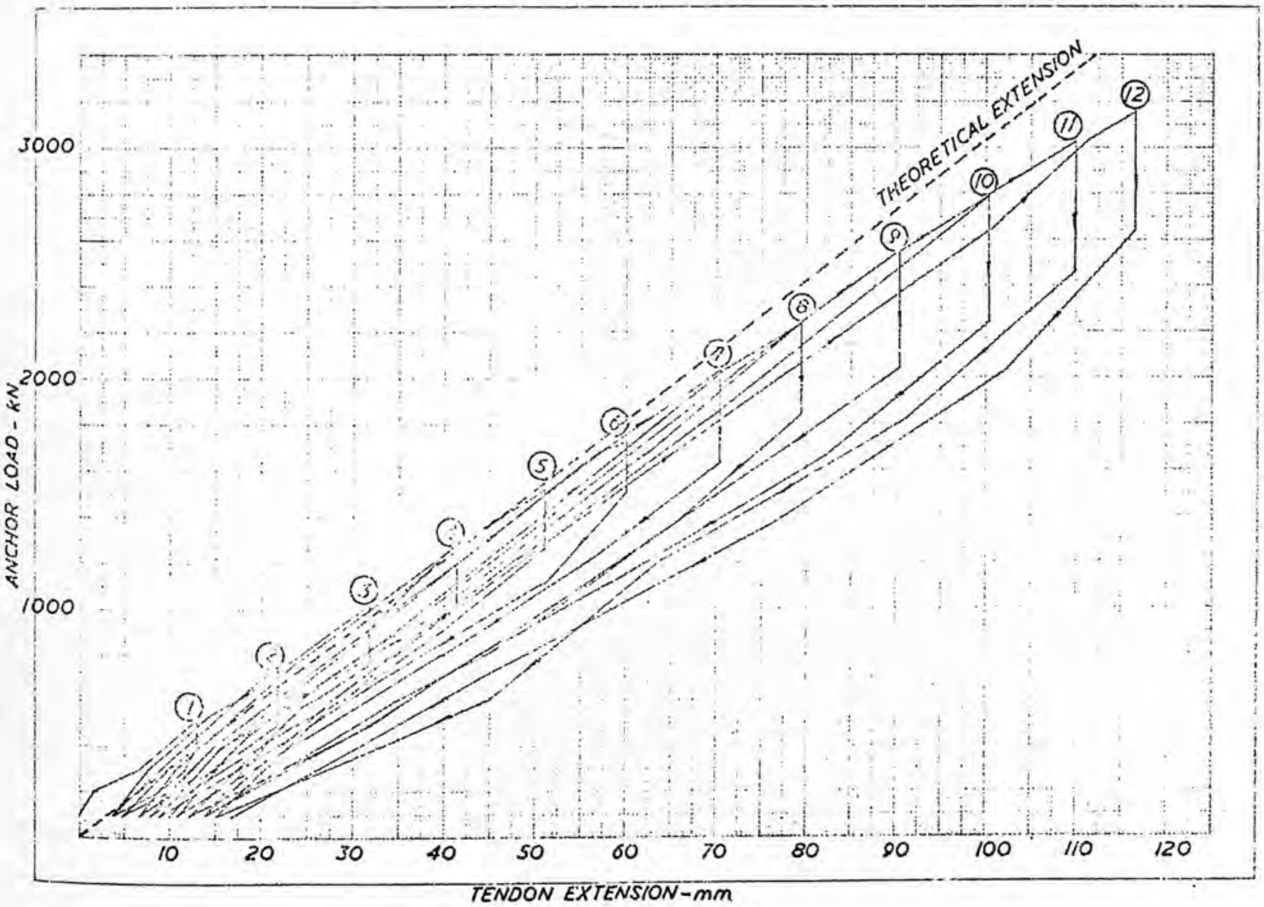


Fig 6. Typical Load-Extension for Stage 1 Anchor.

Stage 1 – Testing

The anchor loads were applied using a P.S.C. multi-strand hydraulic jack, which enables all the strands of a tendon to be tensioned simultaneously, and also facilitates load cycling of the anchor.

The jacking and anchor loads were transferred onto the rock surface by purpose-made precast concrete anchorage blocks up to one metre square (Fig 5). These were bedded on cement grout or sand to even out surface irregularities of the rock slope.

The Stage 1 anchors were loaded cyclically up to failure or a maximum test load of 3120 kN (equivalent to 80% of the characteristic strength of the tendon). In general each loading cycle was 250 kN higher than the preceding cycle, and gave a total of twelve loading cycles up to the maximum test load. (Fig 6).

During any loading or unloading cycle the load increment was only maintained for sufficient time to take and plot the reading, which was considered sufficient to judge the reproducibility of the anchor's load-extension characteristics.

The movement of the precast concrete anchorage blocks was also monitored using dial gauges. (Fig 7).

To facilitate analysis, the load-extension graph of Fig 6 was then divided into its permanent and elastic portions as shown in Fig 8.

This graph shows a close correspondence between actual and theoretical elastic extensions, indicating that the free length is decoupled and that the load is being successfully transferred into the rock over the fixed anchor length. This data can therefore be used in practice to check that the anchor load is being resisted in stable ground beyond the potential sliding mass.

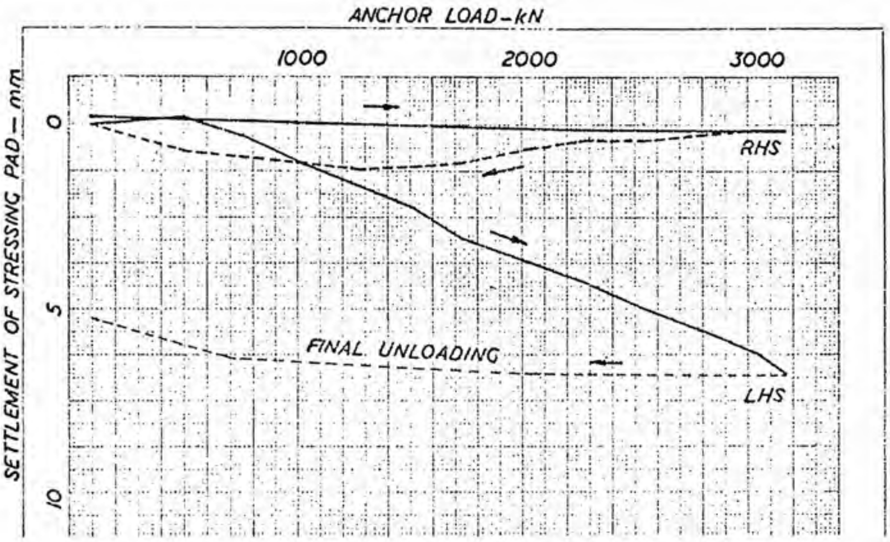


Fig 7. Typical Settlement of Stressing Pad during Loading Cycle.

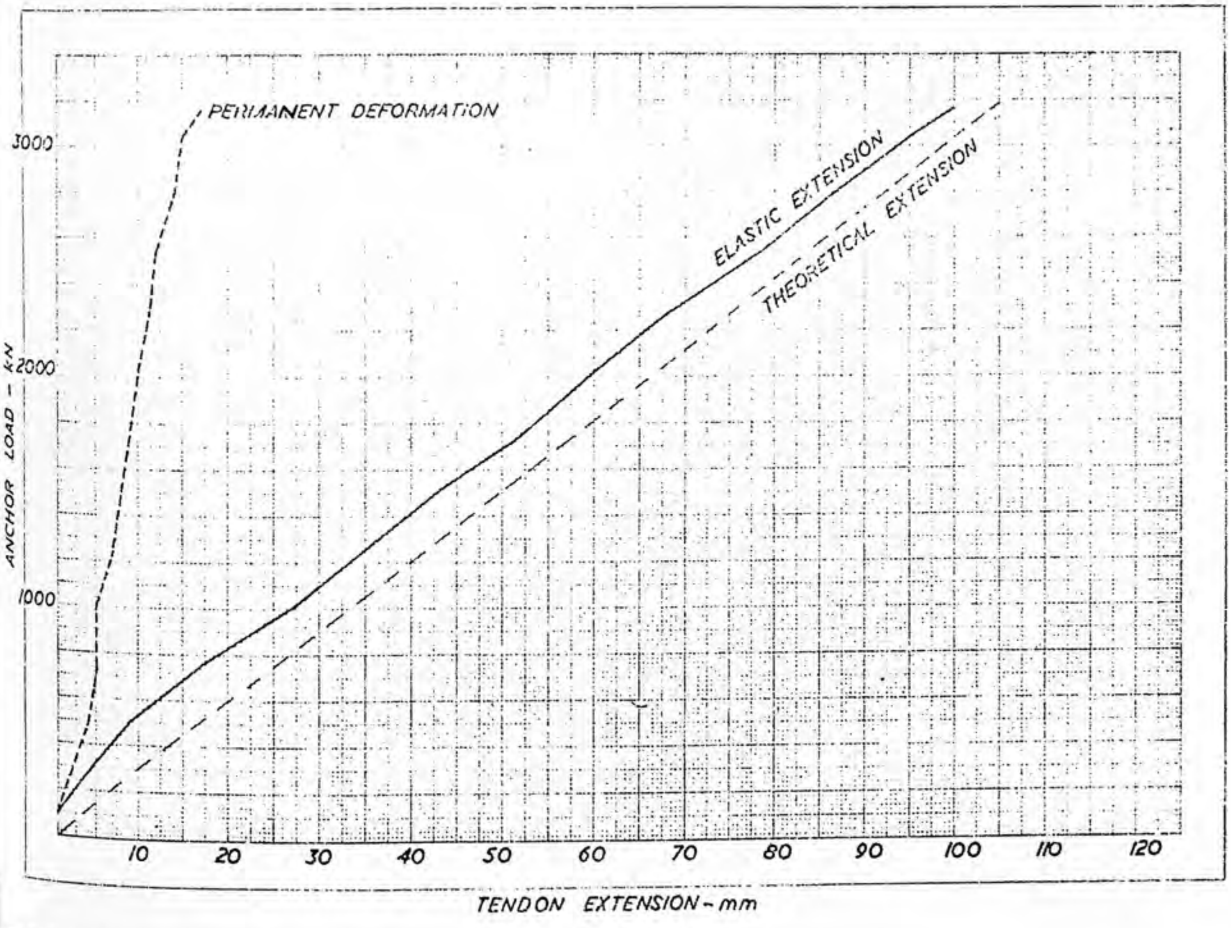


Fig 8. Typical Elastic and Permanent Deformations for Stage 1 Anchor.

Stage 1 – Test Results

None of the eight original anchors, installed in interbedded basalt and volcanic tuffs with fixed anchor lengths varying between 2 and 7m, could be failed at a load of 2.1 times the production anchor working load (Table 1). This is equivalent to an ultimate rock/grout bond in excess of 3.5 N/mm^2 for the shortest anchor length in that formation.

One of the anchors of shorter free length, (No. 13) with a 3m fixed anchor installed in interbedded sandstone, basalt and tuff, began to fail at a load of 2250 kN (1.5 times working load), equivalent to an ultimate rock/grout bond stress of 1.7 N/mm^2, and finally failed at a maximum load of 3000 kN (2.3 N/mm^2).

For the variable and interbedded rock strata at Westfield a safety factor (S_f) of not less than 3 is recommended against failure. These limited data therefore suggest a fixed anchor length of 6m for production anchors. As a result, of the four Stage 2 anchors installed, two had 6m fixed lengths ($S_f \geqslant 3$) and the second pair had 4m fixed lengths ($S_f \geqslant 2$).

Stage 2 – Testing

These anchors are cyclically loaded, in a similar fashion to the Stage 1 anchors, up to a maximum load of 2400 kN, equivalent to 1.6 times their working load of 1500 kN. (Table 2).

After a successful proof load each anchor was then locked off at working load plus 10% to allow for prestress losses such as wedge pull-in and plate bedding-in.

Immediately after locking-off, the anchor was check-lifted to establish the actual load locked into the tendon, and a second series of load cycling was carried out up to 1.5 times working load to establish the load-extension behaviour of the anchor at this stage. It was hoped that subsequent lift checks and load cycling would establish whether any changes

Anchor No.	Free Anchor Length (m)	Fixed Anchor Length (m)	Maximum Test Load (kN)	Equivalent Uniform Bond Stress (N/mm^2)
1	12	3	3120	2.4
2	12	3	3120	2.4
3	12	5	2895*	1.3
4	12	7	3000*	0.9
5	12	7	3120	1.0
6	12	2	3120	3.5
7	12	2	3120	3.5
8	12	5	3120	1.4
13	9	3	3000f	2.3
14	9	3	3120	2.4

* – strand broke during stressing. f – failure load

Table 1. Stage 1 – Maximum test loads

Anchor No.	Free Anchor Length (m)	Fixed Anchor Length (m)	Maximum Test Load (kN)	Initial Service Load (kN)	Remarks
9	12	4	2400	1655	Load cell installed
10	12	6	2165	1600	Two strands broke during stressing. Load cell installed
11	12	4	2400	1640	–
12	12	6	2400	1605	–

Table 2. Stage 2 – Maximum test loads

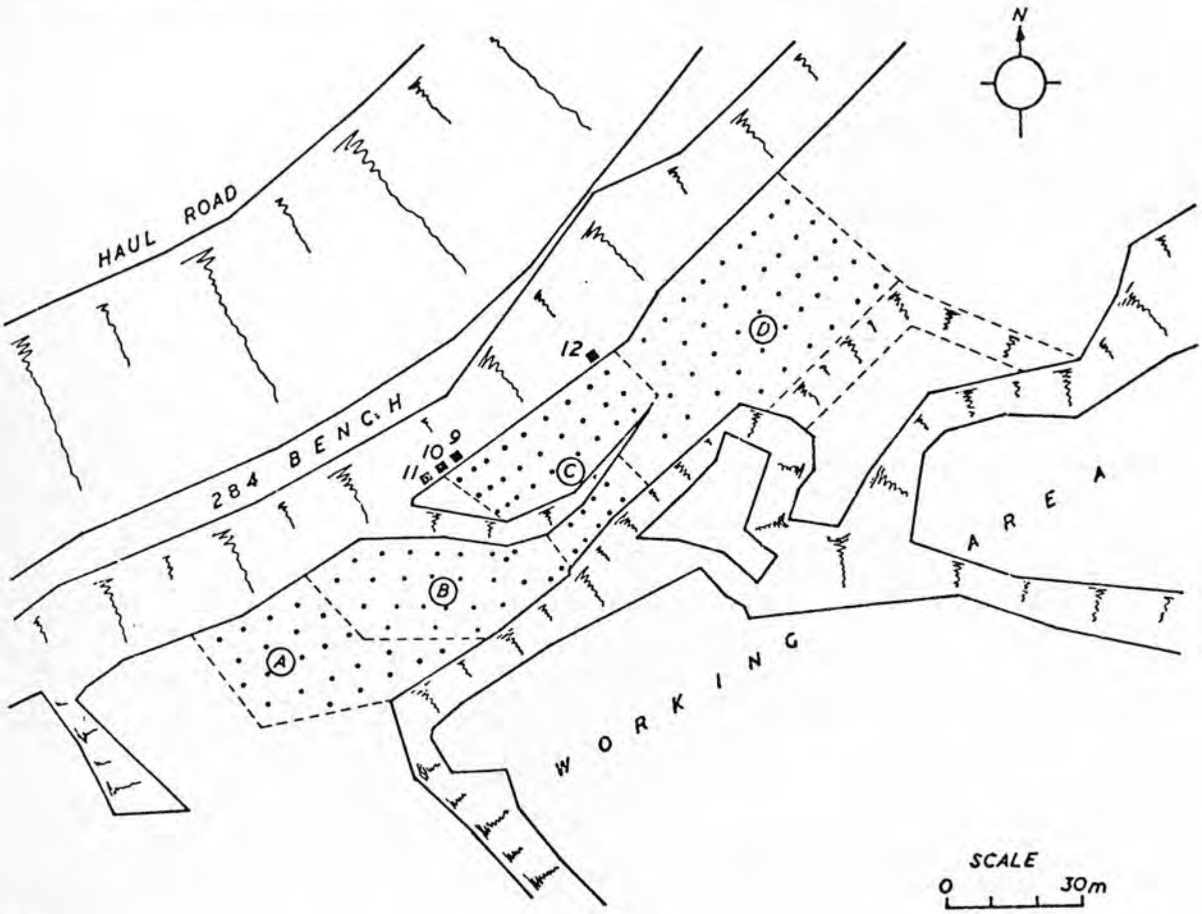


Fig 9. Pattern of main blasts.

Period of Observation (days)	Anchor Service Load (kN)		Remarks
	No. 9	No. 10	
0 4	1655 1610	1600 1580	Initial service load after check lift.
5	1610) 1610)	1575) 1575)	Blast A (730 Kg aluminium slurry). Average charge = 31.7 Kg/hole. No change in load.
6 7	1610 1610	1570 1570	
8	1600) 1600)	1575) 1575)	Blast B (603 Kg aluminium slurry). Average charge = 23.2 Kg/hole. No change in load.
11 12 13 14	1600 1610 1610 1610	1570 1570 1570 1570	
15	1620) 1660) 1650 (1 min) 1645 (5 min) 1645 (15 min)	1565) 1625) 1620 (1 min) 1620 (5 min) 1620 (15 min)	Blast C (1205 Kg aluminium slurry). Average charge = 30 Kg/hole. Increase in load observed = 2.5% (No. 9), 3.8% (No. 10).
18 19	1640 1625	1630 1630	
22	1625) 1625)	1600) 1630)	Blast D (1518 Kg AN/FO). Average charge = 35.3 Kg/hole. Momentary load increase of 1.9% observed in No. 10 but within a few seconds reading restored.
27 32 36	1625 1610 1610	1600 1585 1585	

Table 3. Record of prestress fluctuations during period of close proximity blasting.

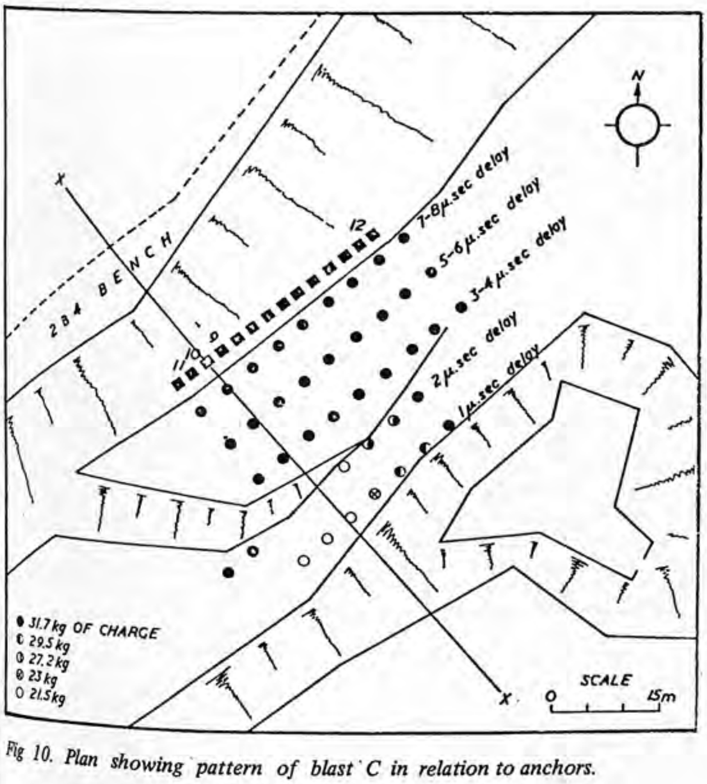


Fig 10. Plan showing pattern of blast C in relation to anchors.

in either the load carrying capacity or the load-extension behaviour of the anchor could be discerned, before and after blasting. Due to the advancement of the pit excavation however it was not economically feasible to gain access to the anchor heads to carry out these tests subsequently.

Stage 2 – Service Behaviour During Blasting
Anchorages Nos. 9 and 10 were monitored using 200t capacity load cells in order to highlight any serious losses of prestress due to creep in the ground, say, but more particularly to register prestress fluctuations when the anchors were subjected to the close proximity blasting shown in Fig 9.

Table 3 highlights the observations made during this period of blasting, and these results confirm that the prestressed anchors behaved remarkably well under the circumstances, the most significant increases occurring during blast C, where the first line of charge holes was positioned only five metres from the anchor heads.

Fig 10 shows full details of blast C which increased the load in Anchorages Nos. 9 and 10 by 40 and 60 kN, respectively. These increases, observed visually from remote readout meters, represent only 2.5 to 4 per cent of the service loading, and within a few minutes the prestress values had stabilised.

To gauge the instantaneous effects of blast C, a portable tape recorder was coupled to Anchor No. 10 and the results are shown in Fig 11. The effect of the blast is more dramatic, and an increase of 110 kN (7% of service load) was recorded within one second of detonation, the vibration dampening to show a fairly stable increase of 64 kN (4.1%) after 10 seconds. In general, it is considered that overall increases in prestress were probably caused by an opening of the joints in the rock, which in turn extended the tendons.

Bearing in mind that permanent anchors are designed to accommodate without distress load increases of up to 60%, the resilient anchor performances at Westfield are very encouraging, and confirm that high capacity prestressed anchors can operate reliably in an open pit mining environment. In this respect the authors believe that the instantaneous and residual prestress fluctuations monitored during blasting operations are the first of their kind to be published in relation to ground anchorages.

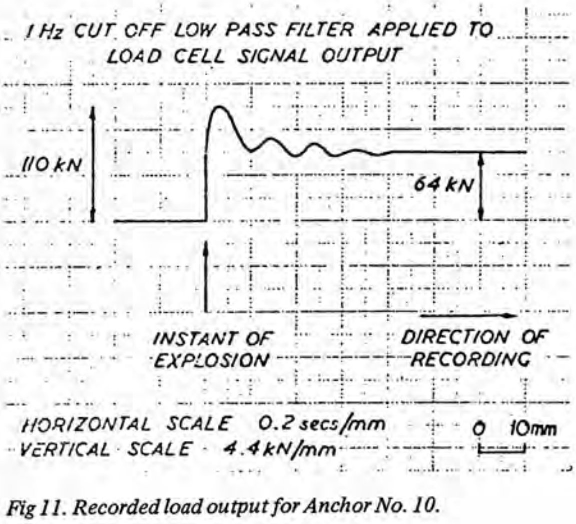


Fig 11. Recorded load output for Anchor No. 10.

Conclusions

Employing the 'P' Pavement failure as a hypothetical test case in conjunction with the test anchor results, the N.C.B. Opencast Executive has found that it is economical to use anchors compared with re-excavation of potentially unstable slopes. Further, since the cost of open pit mining is strongly influenced by the angle of the batters, anchors can play an important part in saving costly excavation, especially in the deeper parts of the mine. In these respects the use of anchors can now be assessed for other pits owned by the Executive where large slopes (in the order of hundreds of metres) are planned in areas of adversely dipping strata (say 1:5) and low strength materials are encountered.

From a practical as well as economical point of view it is evident that anchors are most effective if designed into the site at the planning stage. In this way savings in excavation and anchor design can be sensibly brought together to maximise production and cost effectiveness. In already existing slopes they can still be used (although less cost effective) as remedial measures on slopes which are found to be potentially unstable, and in areas where re-excavation is impracticable.

At Westfield, maximum test loads of up to 3120 kN have been mobilised in the interbedded volcanic and sedimentary strata of the Upper Limestone Group, indicating interfacial bond values in the range 2.3 to 3.5 N/mm^2. During service safe working loads of 1500 kN can be sensibly maintained by prestressed anchors, even when subjected to blasting in the immediate vicinity.

Acknowledgements

The authors gratefully acknowledge the assistance and support given by the N.C.B. Opencast Executive and their operating Contractor at Westfield Site, Messrs. Costain Mining Ltd. in the preparation of this paper.

References

(1) **Barron, K. Coates, D.F. Gyenge, M.** (1970) "Artificial Support of Rock Slopes", Canadian Dept. of Energy, Mines & Resources, R.228.

(2) **Coates, D.F. and Sage, R.** (1973) "Rock Anchors in Mining", Canadian Dept. of Energy, Mines & Resources, Mines Branch T.B.181.

(3) **Golder Brawner Assocs.** (1973) "U.S. Government Pit Slopes Project Part III: Use of Artificial Support for Rock Slope Stabilization" (Not published).

(4) **Seegmiller, B.L.** (1975) "Cable bolts stabilize pit slopes, steepen walls to strip less waste". World Mining (July) 1975.

(5) **Norton, P.J.** (1976) "The Use of Rock Anchors as a means of attaining slope stability in opencast coal mining", N.C.B. report (Not published).

(6) **Littlejohn, G.S.** (1975) "Acceptable water flows for rock anchor grouting". Ground Engineering 8, (2), 46-48.

The views expressed in this paper are those of the authors and do not necessarily represent the views of the National Coal Board.

Reprinted from The Proceedings of a Conference on Rock Engineering at the University of Newcastle Upon Tyne, England – 7 April 1977.

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/m³)	2.45 — 2.60	2.50
Ultimate Pulse Velocity (km/sec)	2.00 — 4.50	3.50
Diametral Point Load Strength (N/mm²)	0.50 — 6.00	3.80
Elastic Modulus - material (N/mm²)	$(1.3 - 2.8) \times 10^{+4}$	$2.0 \times 10^{+4}$
Elastic Modulus - mass (N/mm²)	$(0.5 - 1.6) \times 10^{+4}$	$1.0 \times 10^{+4}$

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.

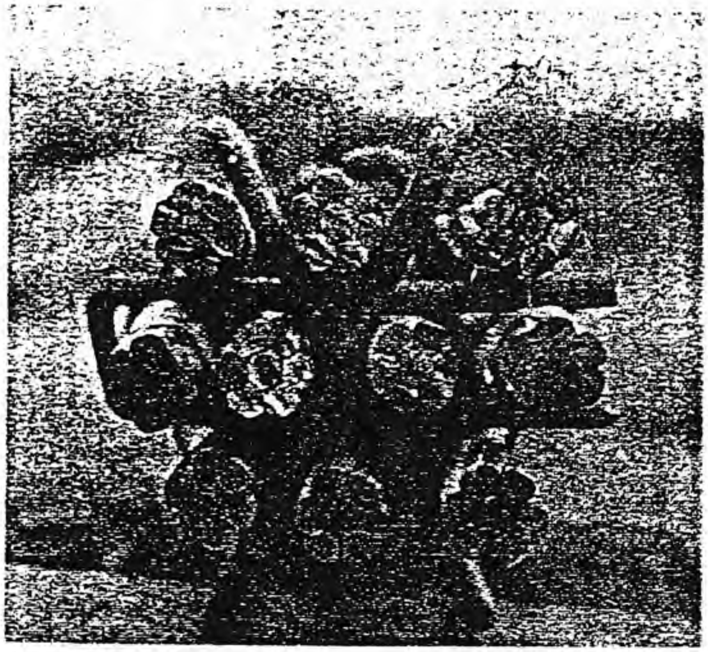


Fig. 1. — **Arrangement for standard ten strand tendon.**

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/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 250 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 0.024 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 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.

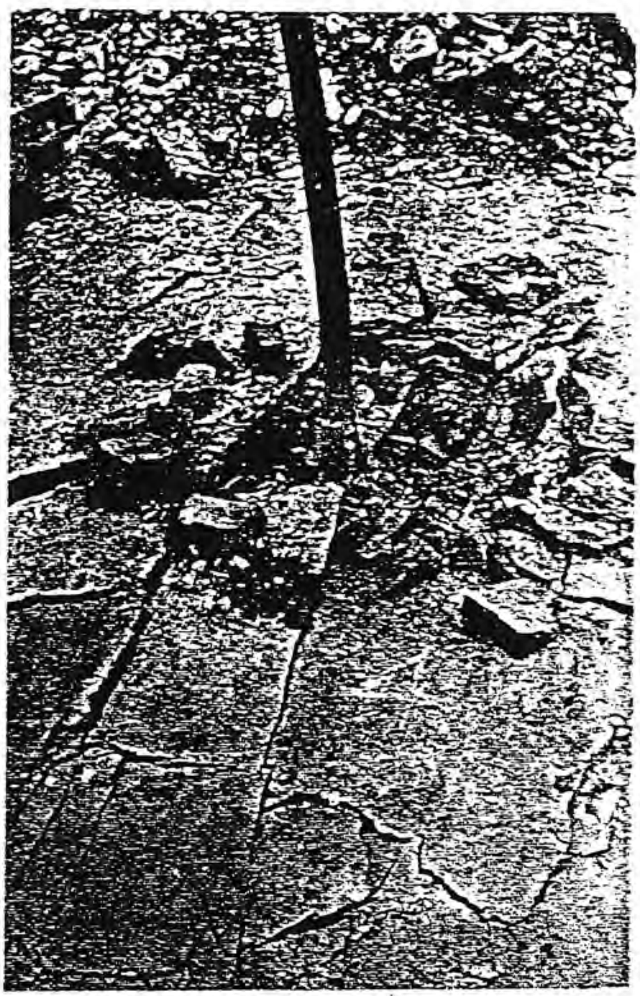


Fig. 2. — Failure volume induced at anchor 4, showing the control of the dominant NW-SE joints.

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	47

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 1 (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.

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	1 920	64	48	1.79	1.34 F	Grout-tendon failure
21	strands	2 093	70	50	1.95	1.46 F	Grout-tendon failure
31	general noding of	2 481	83	48	2.31	1.73	load held - large extensions
32	tendon	2 248	75	49	2.09	1.38 F	Initial yield at 1 978 kN
33	local noding	2 411	80	50	2.24	1.68	Load held
34	of strands	2 411	80	48	2.24	1.68	Load held

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

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).

Long-term performance of high capacity rock anchors at Devonport

by G. S. LITTLEJOHN*, BSc(Eng), PhD, CEng, MICE, MIStructE, FGS, and D. A. BRUCE§, BSc, PhD, FGS

Introduction

WITH THE STEADY development of ground anchor technology over the years, there has been an increasing awareness of the need to obtain and provide information concerning long-term performance of post tensioned anchors.

The present authors (1977) have written of the benefits to be gained from such data; the engineer being able to "feed back" performance observations into future designs and thereby optimise such parameters as overload allowances and safety factors; the prospective client being accurately and confidently informed of how anchors installed at his expense will perform after installation. Furthermore, such data collection permits all parties to judge at the earliest possible stage whether anchors being monitored are, in fact, acting satisfactorily. On a more general front, this form of monitoring may permit correlation of anchor load and structural movement, which will lead to a better understanding of anchor/ground/structure interaction.

*Technical Director, Colcrete Ltd.
§Contracts Engineer, Colcrete Ltd.
At the planning and instrumentation stage of this study, both authors were members of the Geotechnics Research Group, Department of Engineering, University of Aberdeen

Site description

The first stage in the construction of the Submarine Refit Complex at HM Dockyard, Devonport, featured one of the largest and most interesting anchoring contracts undertaken in the UK.

Twin dry docks were constructed in an existing basin approximately 140m square, originally formed as part of the major dockyard expansion between 1896 and 1907, and surrounded on three sides by mass concrete retaining walls founded directly on bedrock. The minimum depth of the walls is 18m but around the north-west corner a depth of 30m is reached due to the areal dip of the rockhead.

Initially, the project featured the production of a dredged and dewatered basin some 18m deep necessitating the construction of a cellular steel sheet pile cofferdam across the south of the basin, and the stabilisation of the existing basin walls against overturning (Fig. 1). In addition part of the dock floor was prestressed, by installing and post-tensioning anchors beneath 15m of water using specially trained divers†.

†The horizontal thrust slab illustrated in Fig. 2 was designed to give additional support to the basin walls at the north west corner. The thrust slab does form a foundation for one section of dock floor but the anchoring of the dock floors was carried out under the main civil engineering contract

Fig. 1. General view of dewatered basin prior of construction of dry docks at HM Dockyard, Davenport

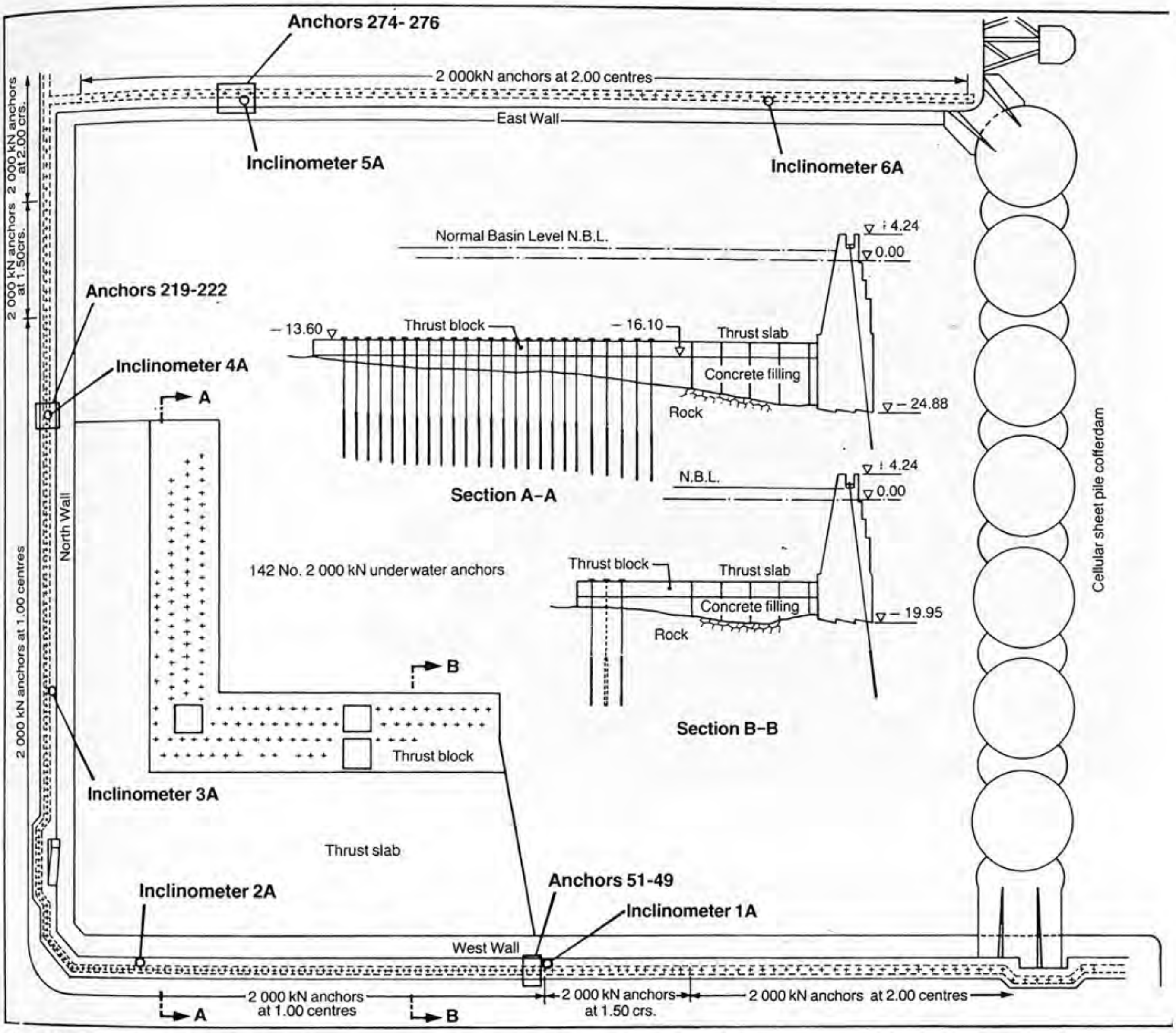


Fig. 2. Layout of anchors for the Devonport Submarine Refit Complex, Stage 1, highlighting the position of the anchors and inclinometers monitored during service

The method of ensuring wall stability was to install 330 No. 2 000kN anchors in holes angled as near to the heel of the wall as possible, and founded in the underlying bedrock (Fig. 2 — Sections *AA* and *BB*). The design required that these anchors, inclined 7-15° from the vertical, were not more than 2.5m apart, and in places, as close as 1.0m. An existing redundant services trench, 1.2m wide and 2.0m deep, running around the top of the walls provided the location for a heavily reinforced anchor beam, into which the load distribution plates and guide tubes were cast prior to drilling the holes, and installing the anchors, (Fig. 3). The design, construction and stressing of the anchors have been described by Littlejohn & Truman-Davies (1974).

Monitoring programme

At an early stage in the anchoring contract, permission was granted for the authors to monitor the time-related performance of selected production anchors. The study had two principal aims:

(1) to investigate the actual anchor loads during the crucial basin dewatering and subsequent construction stages and thereby judge the performance of the anchor/wall system, and,

(2) to provide a case history of the long-term behaviour of permanent high capacity rock anchors.

The authors trust that this report will illustrate the relative ease and simplicity with which such a programme may be inaugurated, and the value of the results: it is hoped that it will thus act as a spur to the conduct and publicising of similar projects.

Site geology

The site is underlain by a series of geosynclinal Upper Devonian sediments, mainly in the form of hard grey, purple, and dark blue slates, known locally as "shillet". Numerous thin quartzitic greywacke beds, and less frequent igneous intrusions are found in nearby exposures, but none appear to have been intersected by any boreholes drilled in the vicinity of the anchors.

The rock surface dips at an average of 3.5° from north-east to south-west across the site, and the uppermost 1.5m or so is commonly recorded as very weathered and fissile, with frequent softer shale or clay bands. Generally the rock is tightly and strongly folded, due to its participation in the Amorican orogeny, and the cleavage dip varies from 60-80°.

Hand specimens show frequent quartz and calcite veins both along and across the fissility, whilst iron staining is also common along virtually every planar surface.

Very little geotechnical data were actually made available upon which to base design — core recoveries of 80-100%, and a submerged density of 1.28Mg/m³. Some core samples were later obtained which enabled diametral point load tests to be conducted. The actual specimens were not of ideal shape, due to the small angle between core axis and rock cleavage, and the very close separation of the cleavage planes. However, twelve tests gave values of I_s in the range 0.45 – 0.97N/mm², and an average of 0.67N/mm² (moderately weak to moderately strong). According to Walker (1975) this average value would relate to estimates of uniaxial compressive strength, elastic modulus and uniaxial tensile strength of 12.0, 3.1 × 10³, and 1.0N/mm² respectively.

The anistropy index ranged from 8 to 18 with a mean of 11.

Anchor and instrumentation details

General

The salient features of the anchors monitored may be summarised as follows:

(i) The fixed anchor length was 8.0m, with a nominal diameter of 140mm as drilled by DTH hammers, giving an average rock-grout bond at service load of approximately 0.6N/mm². A factor of safety in excess of 3 against failure of the rock-grout bond was verified by one test anchor.

(ii) The tendons consisted mostly of twelve Dyform 15.2mm strands, with a working stress of 55% fpu and a steel section/borehole area ratio of 14.2%. Over the free length the strands were individually protected from corrosion, and debonded from the surcharge grout, by 1.5mm wall thickness plastic sheath with grease infilling.

(iii) Special spacer-centraliser units were located at 2m centres in the fixed length and the tendons were noded at intermediate distances.

(iv) The tendons were homed mechanically into the holes, and then fully tremie grouted in one operation, with neat 0.45 w/c Rapid Hardening Portland cement grout.

Anchors under observation

Ten anchors were selected for monitoring as detailed in Table I, and their location in plan is shown in Fig. 2.

All those anchors except Nos. 49 and 51 had been previously stressed by multistrand jack, and therefore were destressed prior to installation of the instrumentation.

Each group of anchors also straddled an inclinometer station (Fig. 2) so that any wall movement could be analysed and correlated with anchor performance and vice versa.

Installation of load cells

Vibrating wire load cells, as supplied by Cementation Research Ltd., were chosen in the belief that cells of this type were most suited to the demands of long-term monitoring programmes.

Following removal of the original anchor head plate where necessary, the surface of the load distribution plate was thoroughly cleaned with a wire brush.

A carefully machined bright steel bearing ring (tolerance = 0.1mm) was then fixed to the plate with Devcon, a rapid hardening "liquid steel". The annular load cell was located in the recess formed in this bearing ring, and finally a new top anchor block, with a specially machined locating recess on the underside, was fitted. The use of Devcon and the closely matched surfaces of the bearing ring, load cell, and anchor block were designed to promote axial loading conditions for the cell.

After stressing, each anchor head assembly, including projecting strands, was enclosed by a protective sheet metal cylinder. The wires from the cell led to a socket fitted to this unit and into which the recording unit could be connected.

Each load cell contained three sensing elements arranged at 120° intervals and sampled by means of a portable battery powered meter, manufactured by Gage Technique Ltd. The average of the three vibratory frequency readings was related to a frequency-load correlation chart.

Anchor stressing

Due to the increased height of the anchor head above the thrust pad once the load cell had been installed, a multistrand jack could not be easily employed. A Titan 30 monojack was therefore used throughout, and stressing proceeded in four equal increments per strand, in a sequence designed to ensure uniform loading of the cell. A final fifth operation was applied on occasions to mobilise the appropriate total anchor load, or to attain an equal load distribution between strands. Typical load-extension data, in this case for Anchor 221, are shown in Fig. 4: extensions were measured by stiff rule to an accuracy of 1mm, after each stressing stage.

In general the load-extension curves of individual strands were parallel, as should be expected, although at lock-off the total extensions deviated by up to 15% from the mean. As the strands for each tendon were cut from the same reel, and therefore had approximately the same E value, it is most likely that these deviations were caused by frictional and lock-off losses in the main.

The stressing records for Anchors 220 and 276 indicated that one strand from each tendon had experienced grout-steel failure. It is noteworthy that both "failed" strands were located on that part of the tendon circumference which, due to the inclination of the boreholes, would have most intimate contact with the borehole wall during homing, and therefore the highest chance of contamination.

A simple analysis and comparison of the original multijack and subsequent monojack stressing records appear in Fig. 5. Both records show that the recorded extensions for the complete tendon were less than those calculated theoretically, although field results were generally within the limiting boundaries A and B, first proposed in DIN 4125 (1972). The authors consider that these limits offer realistic acceptance criteria in practice pro-

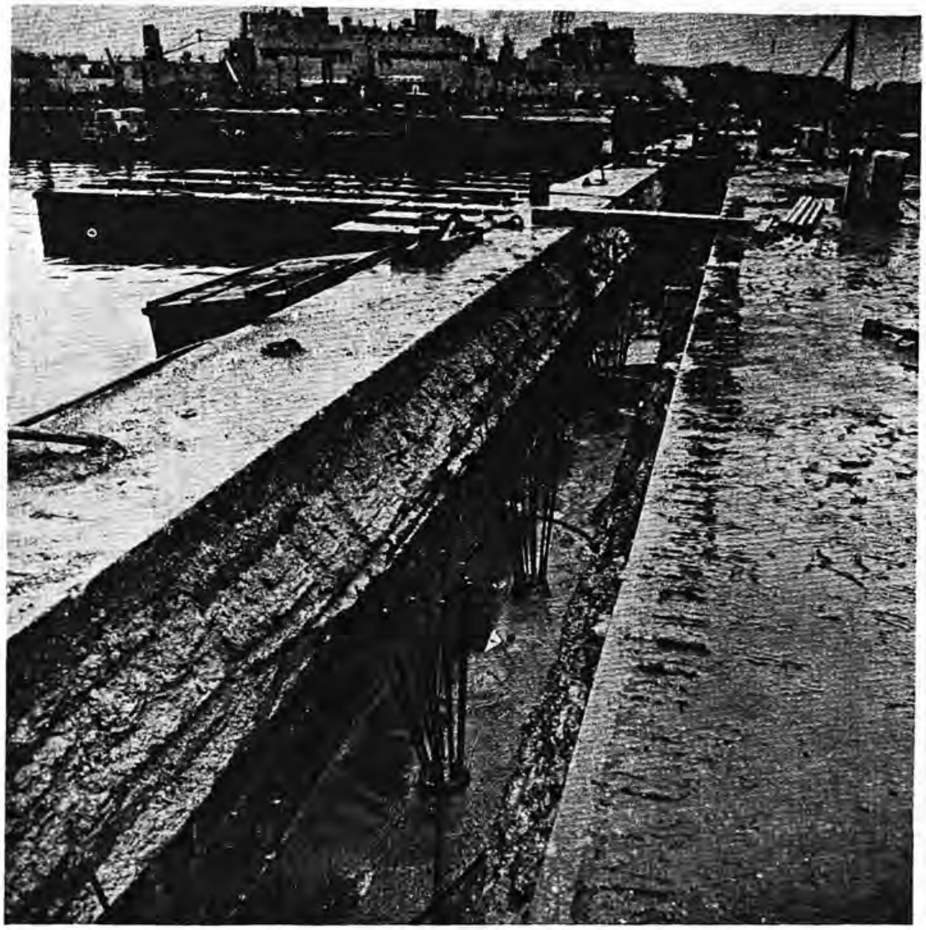


Fig. 3. Anchor heads located in redundant services trench

TABLE I. REFERENCE NUMBER, POSITION AND LENGTH OF MONITORED ANCHORS

Anchor No.	Load cell No.	Wall	Anchor head spacing, m	Anchor free length, m
49	1	West	1.2	26.33
50	2	West		29.16
51	3	West	1.0	27.16
219	4	North	1.0	27.21
220	5	North	1.0	29.21
221	6	North	1.0	27.21
222	7	North		29.21
274	8	East	2.0	22.69
275	9	East	2.0	22.69
276	10	East		22.69

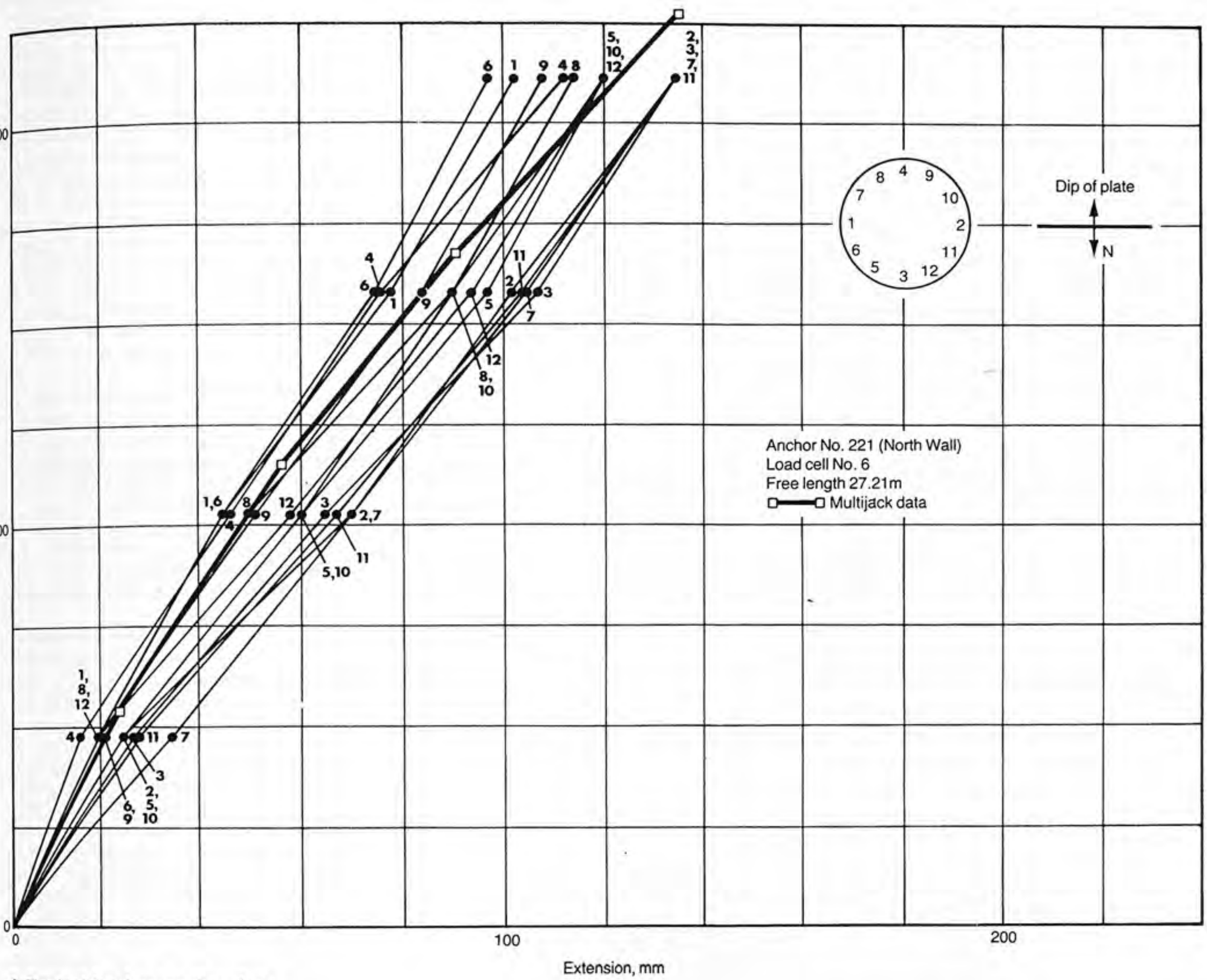


Fig. 4. Typical load-extension data

vided that they are applied wisely in that accuracy of monitoring and probable variations in E value are taken into account during interpretation of the results.

Table II provides a summary of the initial loads for each anchor. The period of installation, instrumentation, and stressing of the ten anchors was four days.

Service behaviour of monitored anchors

(*i*) Up to 24 hours

The load on each anchor was recorded at approximately hourly intervals in the first day after proof stressing. Seven of the anchors gave records similar to that shown by Anchor 50 (Fig. 6).

Rather anomalous patterns were noted in the case of Anchors 49, 51 and 274, where the load actually increased by 160kN (7.3%), 14kN (0.6%) and 67kN (2.9%) respectively.

(*ii*) Up to 4 500 hours

This period extended for approximately 27 weeks after final stressing and included the crucial basin dewatering operation.

Fig. 7 shows typical load-time records in this case for Anchors 50 (previously stressed multijack) and 51. The beneficial influence of prestressing history (2 000kN for 3 100 hrs) on the subsequent performance of Anchor 50 is clearly illustrated. Air temperatures were recorded throughout as tests had shown that apparent load fluctuations of up to 4kN might reasonably be ascribed to temperature variations affecting the load monitoring equipment. Fig. 7 also illustrates the frequency of readings, which was tailored to provide maximum information at certain stages: daily for the first week, weekly for the next 6 months, but twice daily during dewatering.

As noted in the earlier section on "Anchors under observation", wall movements were measured by inclinometers, and during dewatering, when the support

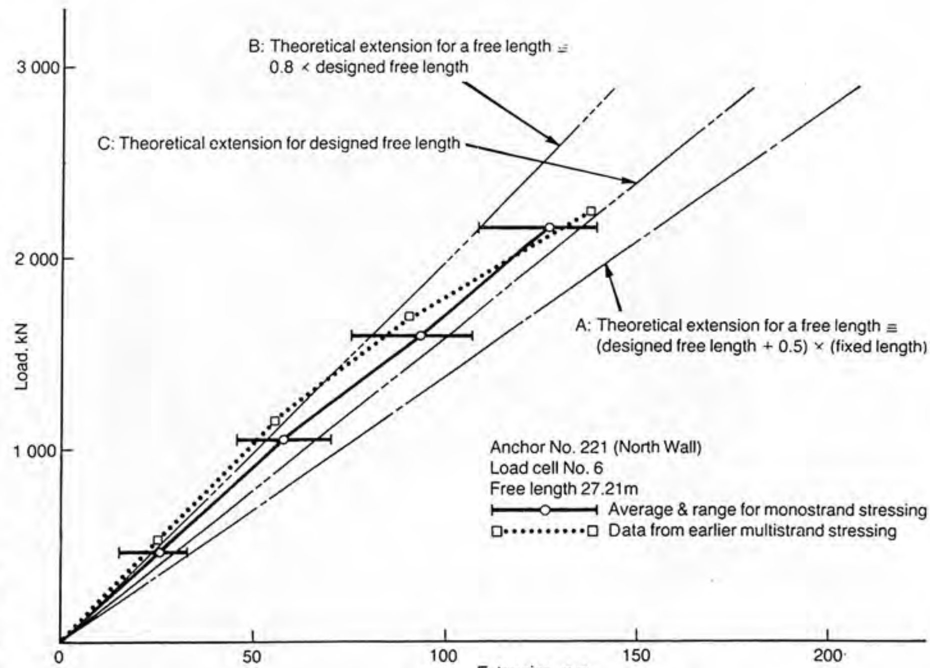


Fig. 5. Comparison of original multijack and subsequent monojack stressing data

of 13m of water was removed, regular monitoring over a period of one month indicated that the walls inclined inwards from the base to give maximum lateral displacements in the range 5 to 50mm at crest level over wall depths of 18 to 30m.

(iii) **Up to 33 000 hours**

Due to the contractor's site activities, it proved both inconvenient and impractical to take regular readings throughout the period. Following dewatering, each cell was read at monthly intervals until 10 000 hours after stressing. Thereafter, readings were taken at approximately four monthly intervals up to about 18 000 hours, followed by a final set at 33 000 hours (196 weeks).

The records obtained after 10 000 hours were further influenced by the progress of the works as a whole. For example the North Wall anchors were destressed after 13 600 hours (Anchor 219) or 14 600 hours (Anchors 220-222), and other cells were damaged during construction activities, as summarised in Table III.

The long-term records for each group of anchors are shown in Figs. 8-10.

Discussion of results

(i) **Behaviour to 24 hours**

Seven of the ten anchors illustrated patterns which are both predictable and widely recorded — a relatively rapid initial load loss, quickly and progressively reducing in rate. Such load losses are ascribable partly to the relaxation characteristics of the tendons themselves, but also to other discrete sources such as movements associated with the bedding-in of the anchor head assembly. The maximum loss recorded on site was 44kN (2%) in Anchor 50, 90% of this occurring in the first four hours. However, the immediate performance of the other three anchors was both surprising and anomalous in that load increases were recorded, after final lock-off, and the complete removal of the jack.

The interpretation of this phenomenon, which has also been observed although not officially recorded on a number of occasions by field operatives known to the authors, is outside the scope of this study, and the particular fields of knowledge of the authors. What is clear is that some external source of energy acted upon the three anchors in question after final lock-off; what is not clear, is the source of this energy.

By way of speculation, the reader is referred to the results of laboratory and field experiments conducted by Nichols & Abel (1975). They highlighted that residual energy locked into igneous and metamorphic rock masses may be released by engineering activities. This energy release is usually manifested by rock bursts, small scale deformations, and larger scale deformations along geological discontinuities and equivalent to earthquakes up to Richter magnitude 3.

An excellent review by Lee, Nichols & Savage (1976) also concluded that "appreciable recoverable internal energy exists in most rocks at shallow depths" and cite a multiplicity of recorded examples.

It seems possible, therefore, that the "engineering activities" of drilling a borehole and subsequently exerting over 2 000kN of prestress on the rock mass may just have triggered off such a release of captive energy, which in three cases resolved to effect an increase in tendon prestress.

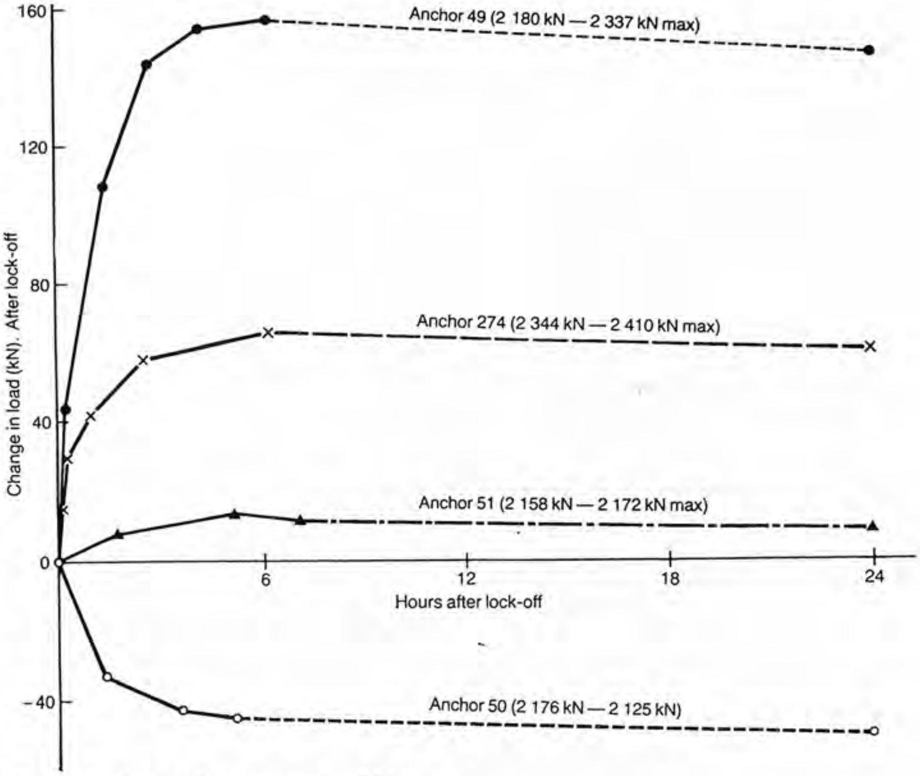


Fig. 6. Anchor performance up to 24 hours

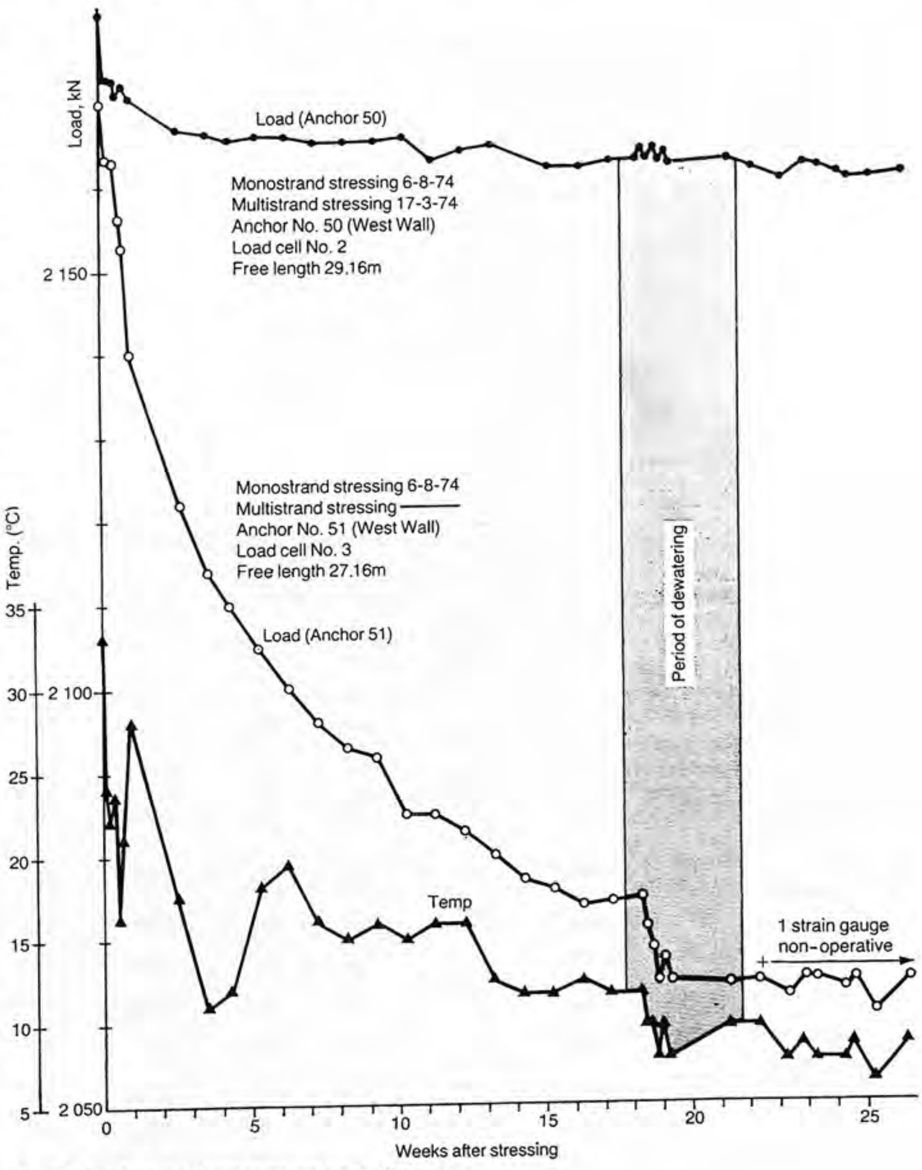


Fig. 7. Anchor performance up to 4 500 hours

(ii) **Behaviour to 4 500 hours**

With regard to the crucial dewatering phase, the records showed no major fluctuations in load; those recorded were scarcely above the order of variation to be expected from the influence of ambient air temperature on the load sensing equipment.

On the basis of the ten anchors monitored it is apparent that the major change in external loading in the basin upon dewatering achieved minimal load change in the wall anchors.

(iii) **Behaviour to 33 000 hours**

As explained in the earlier section "Service behaviour of monitored anchors": (iii) up to 33 000 hours", several occurrences restricted the number and accuracy of readings after 10 000 hours (60 weeks) although good records for the East and West Wall anchors were maintained beyond 17 500 hours (104 weeks).

It is evident from Figs. 8-10 that the long-term load time curve consists of two distinct phases — a rapid loss phase (I), followed by a slower and more uniform reduction in prestress (Phase II). This is illustrated in Table IV. In six cases the amount of load lost in this initial phase (of up to 18 weeks i.e. 3 000 hours duration) was in excess of 85% of that measured at around the two-year stage, and of the other four, the lowest figure was 57%.

Conclusions

(i) Monitoring of the service performance of ten anchors on this site indicate two distinct phases in terms of rate of prestress loss. Phase I is reflected by a stabilising, but fairly rapid loss with time, occurring within a period of 3 000 hours. Thereafter, a slower and more uniform rate of prestress loss is observed (Phase II). Based on these limited results, it is recommended that where service performance is being studied the duration of the study should be at least 5 000 hours. This period should cover completion of Phase I, and hopefully provide sufficient results at say monthly intervals to indicate a clear trend for Phase II and thereby permit an extrapolation of the results to cover the service life of the anchors.

(ii) The final set of readings which could be taken revealed residual loads in Anchors 49, 51 and 275 of 2 275kN, 2 020 kN, and 2 190kN respectively, after 33 000 hours. In all three cases, this confirms the very gradual rate of load loss of Phase II, the total losses being 62kN (2.7%), 152kN (7.0%) and 60kN (2.7%) respectively. The maximum prestress loss recorded was 4.7% at 3 000 hours when the rapid loss Phase I was complete and 7% after 33 000 hours. These values are reassuring bearing in mind the 10% overload allowance commonly stipulated in anchor practice.

(iii) It is known that restressing tendons after a certain period reduces the subsequent prestress losses due to relaxation (Littlejohn & Bruce, 1977). Bearing in mind that eight of the tendons in question had undergone two phases of stressing (multijack and monojack), the generally low prestress losses recorded are consistent with this view. This is particularly well illustrated by the West Wall anchors.

(iv) It may be generally concluded that the anchors have functioned satisfactorily in terms of load-holding capacity during a crucial construction phase, and for the monitoring period of almost four years after stressing.

TABLE II. INITIAL SERVICE LOADS AFTER PROOF STRESSING

Anchor No.	Initial load, kN	Initial tendon stress, % fpu	Remarks
49	2337	65	Not previously stressed (multijack)
50	2180	61	Previously stressed (multijack) to 2 000kN for 3 100 hrs.
51	2172	60	Not previously stressed (multijack)
219	2094	58	
220	2085	63	11 strands effective
221	2110	59	All North Wall anchors previously stressed (multijack) to 2 280kN for 1 150 hrs.
222	2384	65	
274	2410	73	11 strands effective
275	2250	63	All East Wall anchors previously stressed (multijack) to 2 280kN for 1 200 hrs.
276	2114	59	

TABLE III. LENGTH OF RECORDS, AND CAUSES OF TERMINATION

Anchor No.	Length of record (hours)	Notes
49	33 000	2 out of 3 channels stable and reliable.
50	10 000	Cell destroyed due to construction activity.
51	33 000	2 out of 3 channels stable and reliable.
219	10 000	Destressed after 13 600 hours.
220	14 000	Destressed after 14 600 hours.
221	14 000	Destressed after 14 600 hours.
222	14 000	Destressed after 14 600 hours.
274	18 500	Cell destroyed due to construction activity.
275	33 000	Only 1 out of 3 channels stable and reliable.
276	18 500	Cell destroyed due to construction activity.

TABLE IV. SUMMARY AND ANALYSIS OF LOAD LOSSES

Anchor	Max. initial load (kN)	Rapid loss Phase I (kN) (A)	(weeks)	Final load loss monitored (kN) (B)	(weeks)	(A)/(B) %	Remarks
49	2337	39	(16)	45	(110)	87	Not previously stressed
50	2180	13	(3)	20*	(110)	65	
51	2172	102	(18)	114	(110)	89	Not previously stressed
219	2094	31	(8)	34*	(81)	91	11 strands effective
220	2085	85	(5)	89*	(87)	96	
221	2110	52	(16)	56*	(87)	93	
222	2348	7	(2)	9*	(87)	78	
274	2410	20	(4)	35	(110)	57	11 strands effective
275	2250	32	(16)	42	(110)	76	
276	2114	13	(1)	15	(110)	87	

*Extrapolated

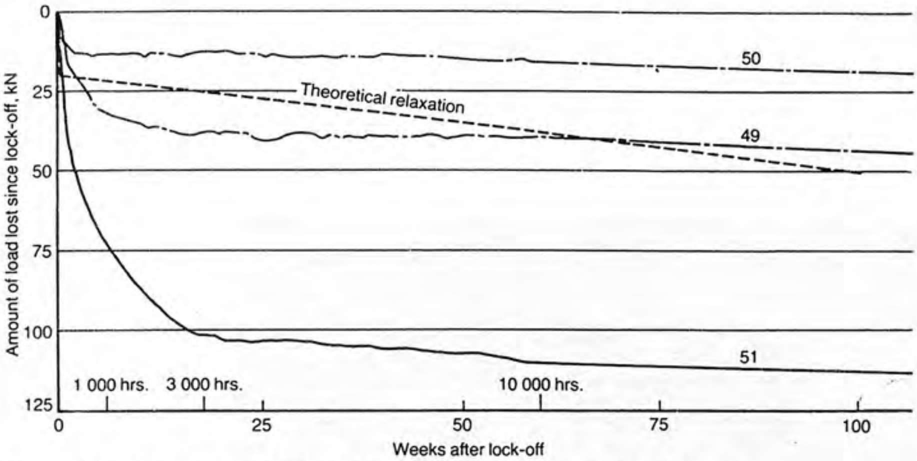


Fig. 8. Long-term performance of West Wall anchors up to 18 500 hours

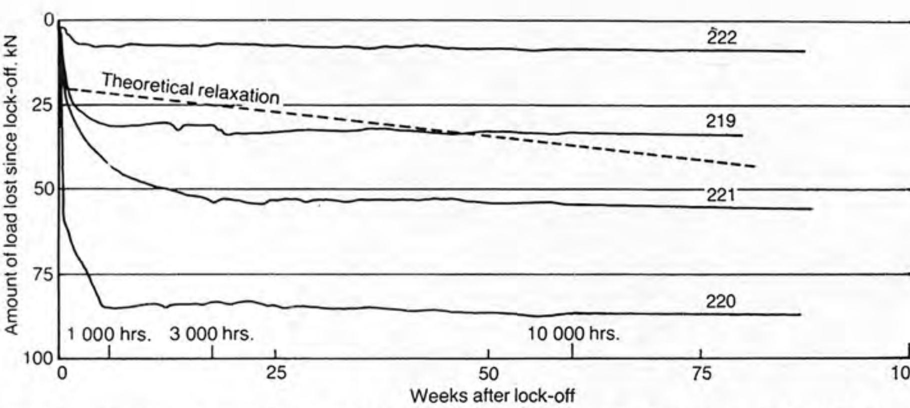


Fig. 9. Long-term performance of North Wall anchors up to destressing at 14 600 hours

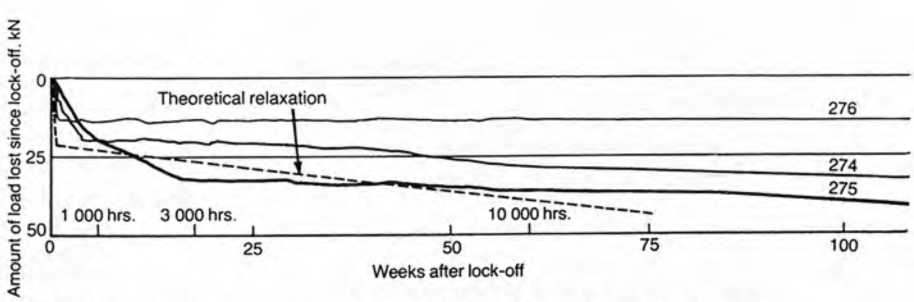


Fig. 10. Long-term performance of East Wall anchors up to 18 500 hours

Acknowledgements

The authors wish to thank Sir Alexander Gibb & Partners, Costain Civil Engineering Ltd., and French-Kier Construction Ltd., for their co-operation and assistance throughout the project. Financial support for the work was provided by the Property Services Agency, Department of the Environment, and technician support was provided by Ground Anchors Ltd., and the Engineering Department, Aberdeen University, with the aid of a Science Research Council grant.

References

Littlejohn, G. S. & Bruce, D. A. (1977): "Rock Anchors: State of the Art" Foundation Publications Ltd., 7 Ongar Road, Brentwood, Essex CM15 9AU, England.

Littlejohn, G. S. & Truman-Davies, C. (1974): "Ground anchors at Devonport Nuclear Complex". Ground Engineering, **7** (6), pp. 19-24.

DIN 4125 (1972): "Soil and rock anchors; bonded anchors for temporary uses in loose stone; dimensioning, structural design and testing". Sheet 1.

Nichols, T. C. & Abel, J. F. (1975): "Mobilised residual energy — a factor in rock deformation". Bull. Assoc. Eng. Geol., **12** (3), pp. 213-225.

Lee, S. T., Nichols, T. C. & Savage, J. F. (1976): "The relation of geology to stress changes caused by underground excavation in crystalline rocks at Idaho Springs, Colorado". U.S. Geological Survey professional paper 965. 51 pp.

Walker, R. (1975): "A comparative study of two point load strength indices". BSc dissertation. Department of Engineering, Aberdeen University.

Printed by Edward Wells & Sons Ltd. (TU), 143/145 Camberwell New Road, SE5

Ground anchors : state-of-the-art

by Dr G S Littlejohn,
Technical Director, Colcrete Ltd,
Strood, Kent, England

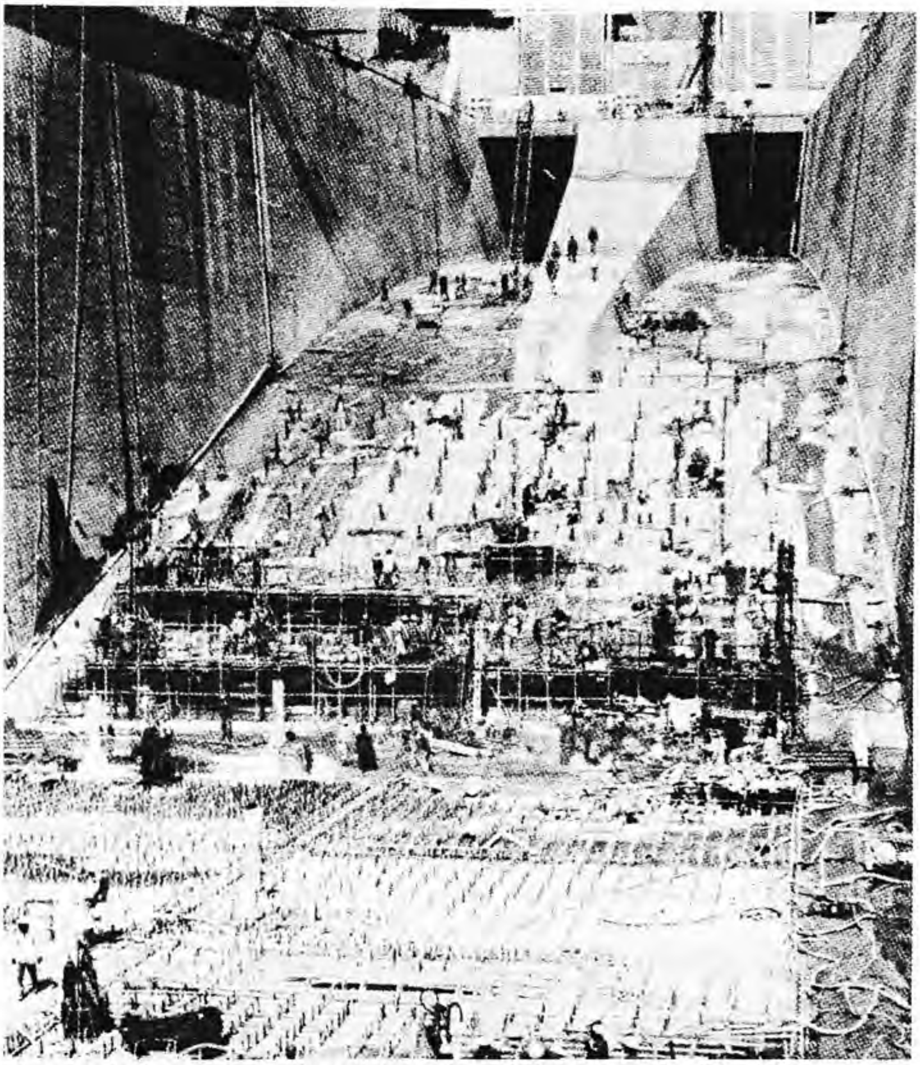


Figure 1 Resistance to static and dynamic uplift forces provided by 576 permanent anchors installed in rock to depths of 17 to 38 m, and prestressed to working loads up to 2 490 kN, at stilling basin No. 3, Tarbela Dam, Pakistan.

Synopsis

Following an introduction to anchor applications, techniques and recently-produced codes of practice, the paper discusses the major site investigation requirements pertaining to ground anchors, and present day short-comings, including an assessment of ground aggressivity.

In anchor design, topics include stability, ground/grout interface, grout/tendon interface, grout mix, tendon, anchor head, safety factors, corrosion, corrosion protection and stressing equipment.

The text provides an insight into the state-of-the-art on cement grouted anchors but also discusses recommendations to improve practice, and highlights areas where further knowledge is required. In anchor construction, current practice and relevant quality controls are discussed for drilling, water testing, tendon fabrication and homing, grouting, anchor head installation and stressing.

For the final testing and acceptance stage, five classes of test are highlighted, special attention being directed to proof loading, load-extension data and service behaviour.

Finally, the importance of records and quality controls are emphasised including proof loading of every anchor to comply with specified acceptance criteria. More research and field monitoring are recommended to investigate service performance of anchors, debonding and corrosion.

Sinopsis

Na 'n inleiding wat oor ankeraanwendings, -tegnieke en onlangs-daargestelde gebruikskodes handel, bespreek die referaat die belangrikste terreinvereistes in verband met grondankers en hedendaagse tekortkominge, insluitende 'n raming van grondaggressiwiteit.

Ankerontwerp sluit in onderwerpe soos stabiliteit, grond/bry-tussenvlak, bry/spankabel-tussenvlak, brymengsel, spankabel, ankerkop, veiligheidsfaktore, korrosie, korrosiebeskerming en spantoerusting.

Die teks verskaf nie slegs insig in die huidige stand van kennis wat sementbryvulling van ankers betref nie, maar bespreek ook aanbevelings om praktyke te verbeter en werp lig op gebiede waar verdere kennis benodig word. In ankerkonstruksie word huidige praktyke en toepaslike gehaltebeheer vir boorwerk, watertoetse, spankabelvervaardiging en russtandherstelling, bryvulling, ankerkopinstallering en spanning, bespreek.

Vir die finale toets- en aanvaardingstadium word vyf soorte toetse uitgelig, met spesiale klem op proefbelasting, lasverlengingsgegewens en diensgedrag.

Ten slotte word die belangrikheid van rekords en gehaltebeheer, insluitende proefbelasting van elke anker om te verseker dat dit voorgeskrewe aanvaardingsvereistes nakom, beklemtoon. Meer navorsing en veldkontroles word aanbeveel om die diensgedrag van ankers, bindverlies en korrosie te ondersoek.

This paper was presented at a CSSA symposium on prestressed ground anchors in October 1979.

1. Introduction

There has been a dramatic increase in the use of ground anchors during the past ten years. Not only has the number of anchor installations increased but the range of applications has widened considerably and today anchors may be associated with retaining walls, dry docks, cofferdams, water tanks, concrete dams and spillways (Figure 1), tall buildings, suspension or arch bridges, tension roofs, pile and plate loading tests, lighthouses (Figure 2), towers, masts, ski jumps, cliff stabilisation, open mine pits, shafts, tunnels, underground caverns (Figure 3), pipelines, and oil platforms (Figure 4).

For these applications anchors can be employed to solve problems involving direct tension, sliding, overturning, dynamic loading and ground prestressing.

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

Although ground anchor technology is still in an active stage of development the use of anchors is widespread for both temporary and permanent works in soils and rocks, and there is clear evidence that a variety of design and testing concepts exists in current practice. It is considered that if reliable performances are to be maintained in the future a more detailed technical appraisal of anchor systems is required by the practising engineer in addition to the routine comparisons on the basis of cost and duration of contract. In this regard several countries [(1)-(8)] and organisations[(9)-(11)] have introduced standards or recommendations in recent years. These documents bring together the results of practical experience and scientific investigation, and define certain aspects of anchor technology which are generally accepted. In attempting to standardise specifications and improve practice however these documents should not be used to restrict the advancement of techniques that are currently in the course of evolution.

Figure 2 Resistance to sliding and overturning provided by 6 permanent anchors installed in 17 m of alluvium and prestressed to 850 kN working load, at the lighthouse at Kullagrund on the Baltic coast of Sweden.

2. Site investigation

2.1. General

The planning and general requirements of site investigations are well understood and the purpose of the following note is to highlight particular features which are relevant to ground anchors. Of prime importance is a detailed knowledge of the ground. Whilst there may be adequate data to indicate both the feasibility and advantages of an anchor system, it is usual to find that there is insufficient detailed information to permit its economic design or construction. In this regard the data required for the safe design of temporary anchors is often similar to that necessary for permanent works. Since ground anchors are installed horizontally as commonly as they are vertically, lateral variations in ground properties must be investigated as thoroughly as the more easily investigated vertical variations. For structures such as an anchored retaining wall it is recommended that the maximum centres of investigation locations should not exceed 20 m, unless a well known "solid" geological formation is encountered. In addition the plan dimensions of the site need to be carefully defined so as to

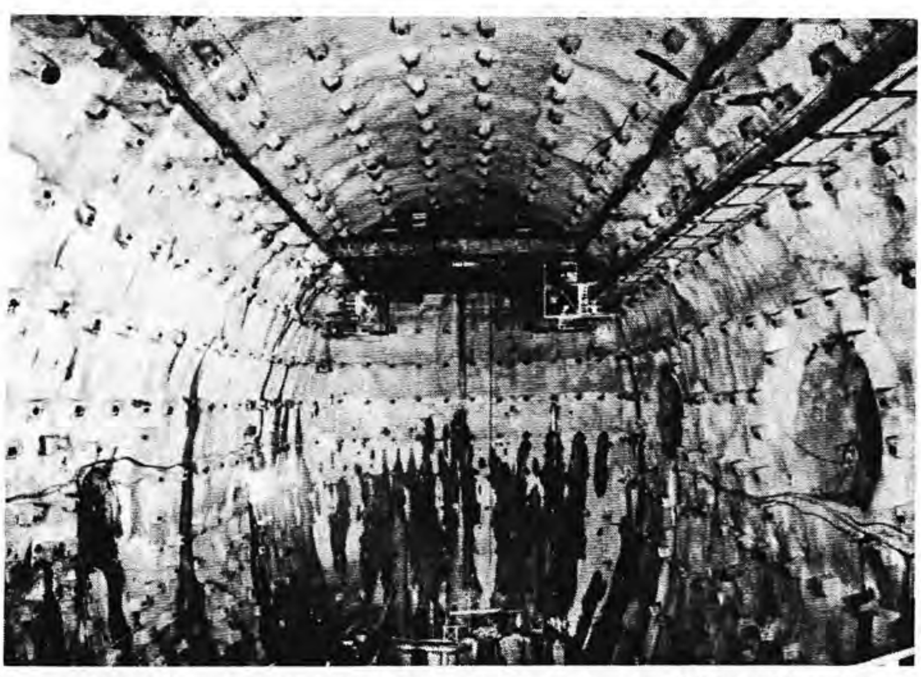


Figure 3 Self supporting cavern 106 m long, 54 m deep and 33,5 m wide using 784 permanent rock anchors installed to lengths of 20 to 28 m and prestressed to working loads of 940 to 1 300 kN, at Waldeck II Pumped Storage Scheme in West Germany.

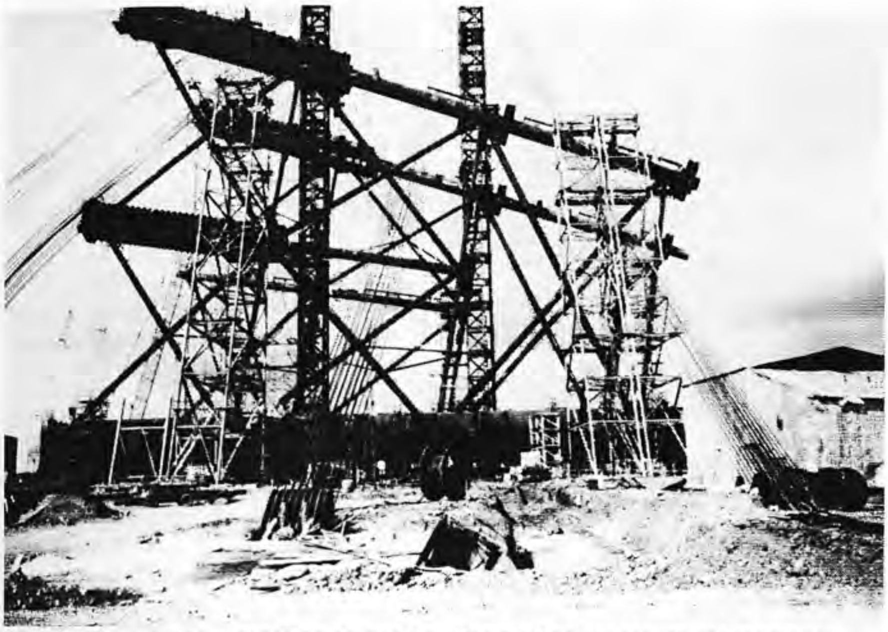


Figure 4 To provide a reaction to enable lifting of heavy structural members into place on the Brent 'A' jacket, four reinforced concrete blocks were prestressed into the ground with a force of 5000kN each.

include the probable fixed anchor zone (Figure 5). Too often in practice, particularly for deep excavations, anchors are installed beyond the site perimeter where there is a dearth of ground data.

2.2 Main field investigation

The geometry of an anchor system and its mode of operation requires, in particular, knowledge of ground conditions local to the grouted fixed anchor zone. Minor variations in ground conditions therefore assume a greater degree of importance than, for example, in foundation design. For soil anchors the recording of the structure or fabric of the soil is recommended since the presence of thin partings of silt or sand within a clay can have a marked effect on the behaviour of the soil in shear and on the softening action of drilling water (Figure 6).

This in turn can severely limit the load which can be placed on clay anchors, particularly the under-reamed type. In the case of rock anchors, discontinuity frequency and orientation data together with joint continuity and roughness can be vital in determining the size and shape of a rock mass liable to fail in service (Figure 7) and, therefore, are critical in any overall stability analysis. On the practical side these data obtained from rock exposures, borehole inferface observations and parameters, such as Rock Quality Designation, can be invaluable when back analysing water test data to determine the need for pregrouting. Careful fabric or structure observations such as outlined above are rarely made in routine investigations.

In regard to sampling, the available techniques are well documented and in soils, samples for examination and laboratory tests should be taken from each stratum and at maximum intervals of 1,5m in thick strata. Intermediate disturbed samples, suitable for simple classification tests, should also be obtained, so providing a specimen of the ground at a maximum of 0,75m intervals of depth. In variable strata, continuous undisturbed sampling may be necessary in the probable vicinity of the fixed anchor zone. Where granular soils are encountered, investigation of density by Dutch Cone Penetrometer or Standard Penetration Test (SPT) may well be justified. These results in turn indicate the insitu shear characteristics when combined with grading and particle shape assessments. In the probable zone of the fixed anchor length, tests should be made at, at least, 1m intervals of depth in each borehole. In rocks, emphasis must be placed on obtaining maximum continuous core recovery, which generally implies core diameters of not less than 75mm. In addition, the use of double- or triple-

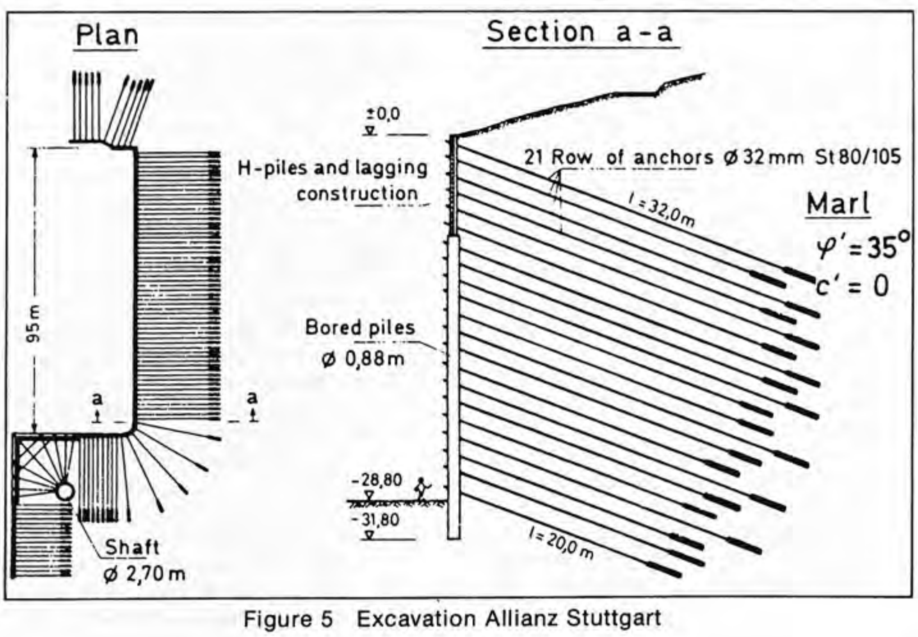


Figure 5 Excavation Allianz Stuttgart

tube core barrels is recommended. For weak rocks which are difficult to core, the SPT has been exploited to give a relative measure of insitu quality. Stress/strain characteristics, eg, E-values, are important in design since they influence bond distribution, and may dictate the failure mechanism in the fixed anchor (see section 3.2). Radial stress/strain characteristics of the ground mass can be obtained in granular and cohesive soils, as well as in soft rocks, by a pressuremeter test within a borehole. For strong rocks the Goodman Jack is appropriate although, if results are difficult to determine, deformability measurements from cores should be seriously considered. These tests are rarely carried out in practice for ground anchors.

Determination of the groundwater conditions on the site will almost certainly be essential for the overall design of the project as well as the anchor system, particularly where excavations are proposed. This area of investigation, and the recording of long term groundwater conditions by properly sealed piezometers, although well understood, are too often given scant attention. Allied to this subject, the permeability of the ground mass can be assessed from pumping tests in order to assess groutability. In hard rock where low permeability is confirmed it is noteworthy that the environment is sometimes regarded as virtually non-aggressive on the basis of low groundwater percolation rates.

2.3 Laboratory Investigation
For general classification of soils the grading of granular soils and the Liquid and Plastic Limits of cohesive soils should be determined for every stratum encountered in the investigation. Grading can give an emperical guide to permeability and

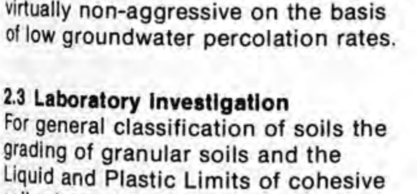


Figure 6 Influence of sandfilled fissures on underream configuration.

Figure 7 Rock failure strongly controlled by incipient mass structure.

this value in turn influences the radius of grout travel by permeation. The determination of low Plasticity Indices in a cohesive soil, even in very localised zones, can influence both the type of anchor and the method of drilling eg, under-reaming may be precluded.

In granular soils of mixed grading, peak shear strengths of samples may be obtained from direct shear tests for a series of densities from loose to dense, at the relevant stress level. For cohesive soils the shear strengths should be obtained by triaxial compression tests on representative samples. The type of test to be employed, namely undrained or drained, will depend on the design method, the mass permeability of the soil and the probable rate of stressing of the anchor. Where the ground under investigation possesses a high mass permeability eg, silts, clays possessing a permeable fabric, chalk or marl, both undrained and effective shear parameters should be determined to permit a study of the influence of the relatively high permeability under different loading conditions of the anchor. In current practice, anchor design in clay is based primarily on undrained parameters. For systems which will apply a high average stress to a clay lying between the fixed anchor length and the structure, the compressibility characteristics C_v and M_v should be determined, since such data can provide guidance as to possible loss of prestress particularly through case history comparisons. This aspect is important in view of the current lack of predictive capacity related to long term performance.

In rocks, index tests such as Point Load Strength are attractive since they are cheap, easily undertaken and correlate approximately with other parameters such as uniaxial strength, although currently it is more common to determine directly the uniaxial compressive strength or occasionally the tensile strength on specially prepared test cylinders. Alternatively, a large shear box test may be used to assess the shear strength of intact material or an existing discontinuity. The shear strength of joints may also be estimated by a detailed study of the joint geometry and materials characteristics. All these tests are used by the designer to estimate rock mass stability and the bond or skin friction in the fixed anchor zone. In regard to bond distribution however, rock deformability is the key parameter, and stress/strain relationships should be obtained from uniaxial compression tests in the laboratory, or preferably from insitu pressuremeter tests.

The susceptibility of rock to weathering can be assessed by the Slake Durability Test apparatus, augmented by a microscopic examination of the nature of the minerals. With this information the sensitivity of the rock to flushing water, and the possibility of mineral reaction with grout or groundwater can be investigated. In this regard swelling tests are also pertinent. In general, there is a need to concentrate more on ground parameters which affect or may be affected by the drilling process. For example, quartz content combined with strength are useful figures when assessing drillability.

2.4 Chemical analyses
Sulphate and chloride contents are established as a routine and dictate choice of cement, but the overall corrosion hazard is seldom quantified. As a guide, Table 1 illustrates some aggressivity limits with respect to cement, whilst Table 2 proposes limits for two key parameters with respect to metals[12].

2.5 Investigation during construction

Where the initial investigation shows that ground conditions are liable to random variations eg, glacial drift, then all ground data obtained during anchor drilling should be recorded and subjected to daily analysis. Such a system will act as an "early warning" device should variation in strata levels or ground type require changes in design or installation method. The behaviour of test anchors should also be considered as part of the ground investigation and carefully compared with all field and laboratory data. In particular, the behaviour of an anchor in a semi-permeable soil will be dependent on the ratio between the rate of dissipation of porewater pressure and the rate of loading. Thus it is possible that the available anchor load may well be increased if stressing can be applied slowly, eg, in a series of stage increments over a period of time. Similarly the behaviour of a clay soil between the fixed anchor length and the anchor head may well depend on the stressing procedure. These aspects have received little study to date, yet they influence proof loading, and, more important, subsequent service behaviour, with particular reference to prestress loss with time.

3. Design

3.1 Stability

Individual anchors must be installed at a depth sufficient to resist safely the applied working load without failure occurring in the ground mass. For rock, calculations on uplift capacity are based on crude cone or wedge mechanisms (Figures 8a and b)[13] which are invariably conservative since they are primarily based on weight, no tensile or shear strength being apportioned to the rock[14]. In many cases rock mass heterogeneity or the presence of discontinuities restrict application of these simple methods and necessitate modifications by the experienced rock engineer using his engineering judgement (See Figures 8c, d and e)[13].

For soils, an expanding conic plug increasing in diameter from the top of the fixed anchor might be assumed at failure (Figure 9)[15]. In this case however shear resisting forces are considered to act at the failure surfaces in addition to soil weight. In current practice, anchors are rarely installed at such shallow depths where a general shear failure of the ground mass occurs, and it is more common to find fixed anchors founded at a depth where the top of fixed anchor is not less than 5 metres below ground surface or, alternatively, with a depth/diameter ratio exceeding 15. In these circumstances, field experience indicates that failure is localised and does not generate to the ground surface.

Overall, there is little experimental or practical evidence to substantiate the methods currently used to calculate the ultimate resistance to pull-out of individual, or groups of anchors, although present practice appears to be conservative. It is important to observe however the trend towards higher anchor loadings combined with a growing exploitation of weaker ground. In the case of a rock mass, careful classification with particular reference to fracture geometry would facilitate an optimum design. Where large groups of anchors are located in the same rock horizon, and the rock mass is horizontally bedded, the possibility of laminar failure should be considered[16]. This subject requires more research, although in unfavourable conditions it is common practice to stagger anchor lengths to reduce the intensity of stress on any plane[3],[14].

Table 1 Aggressivity of groundwater with respect to cement

Groundwater environment	Remarks on aggressivity
Very pure water (CaO< 300 mg/litre)	Such waters dissolve the free lime and hydrolyse the silicates and aluminates in the cement
pH< 6,5	Acid waters attack the lime in the cement, but pH values of 9-12 are passivating
Selenious water (SO_3)>0,5 g/litre (stagnant) >0,2 g/litre (flowing) Magnesium water (SO_3)>0,25 g/litre (stagnant) >0,1 g/litre (flowing)	These sulphates react with the tricalcium aluminate to form salts which disarrange the cement by swelling

Table 2 Aggressivity of soils with respect to metals

Ground Environment		Remarks on aggressivity
Resistivity (ohm cm)	Redox potential (corrected to pH = 7) Normal hydrogen electrode (mV)	
<700	<100	Very corrosive
700-2 000	100-200	Corrosive
2 000-5 000	200-400	Moderately corrosive
>5 000	>400 >430 if clay soil	Mildly or non-corrosive

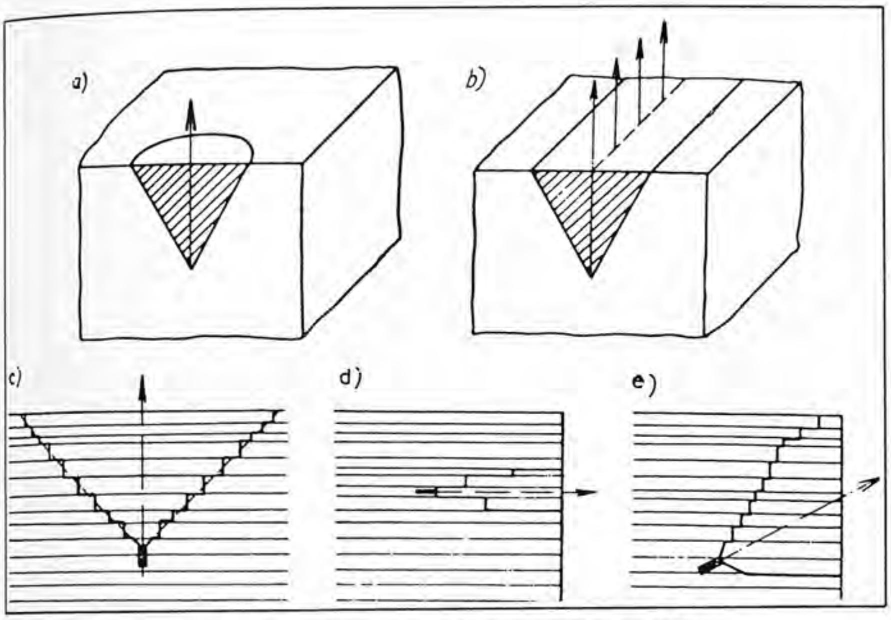


Figure 8 Geometry of rock mass assumed to be mobilised at failure (a) individual anchor in isotropic medium, (b) line of anchors in isotropic medium, (c) perpendicular to planes of discontinuity, (d) parallel to planes of discontinuity, (e) at an acute angle to planes of discontinuity.

3.2 Ground grout interface

A straight shafted, cement grouted anchor in rock relies mainly on the development of bond or shear in the region of the rock/grout inferface. A uniform distribution of bond along the fixed anchor is assumed, and the bond value is commonly based on field experience.[14] In the absence of this experience or field test data, the ultimate bond stress is often taken as one-tenth of the uniaxial compressive strength of massive rocks up to a maximum value of 4,2 N/mm².
In fact, the distribution of bond is unlikely to be uniform unless the rock is "soft", and non-uniformity applies to most rocks where E_{grout}/E_{rock} is less than 10. Although stress transfer from anchor to rock is imperfectly understood, the rock mass can accommodate our ignorance in most cases and rock reinforcement systems can be remarkably effective in spite of a lack of precision in design. In soils the load transfer mechanism is chiefly one of surface adhesion in clays, and skin friction in sands and gravels. For straight-shafted or under-reamed anchors current designs are analogous to those employed for bored, cast insitu piles.[17] A major exception is the post-grouted anchor where the ground and primary grout are hydrofractured. Since the magnitude and extent of fracturing are virtually impossible to quantify, all hope of calculating pull-out capacities is abandoned, and the results of many field tests are used to draw up statistics and correlations. As an example Figure 10 illustrates for sands and gravels the relationships between uniformity coefficient, fixed anchor length and ultimate load holding capacity.[18]

In practice, fixed anchor lengths are seldom less than 3 m. Where load is transferred primarily by bond or shear an upper limiting length exists beyond which the extra length is redundant unless the proximal end of the fixed anchor yields. A knowledge of the tensile stress/strain relationship of the tendon reinforced grout, and the ground restraint/displacement curve for the fixed anchor is required to calculate the maximum fixed anchor length for any particular situation. In practice, fixed anchor lengths seldom exceed 10 m, even in weak soils, and the above calculation is not usually contemplated. For those anchors where the tendon load is transferred through a plate or prefabricated capsule, axial and radial compressive stresses are mobilised which must be resisted by the lateral restraint of the surrounding ground (Figure 11). To investigate this possibility it is necessary to obtain E-values for rocks and stiff clays, and insitu stress levels together with elastic and dilatancy properties for sands and gravels. These field characteristics, although rarely available, are required if the behaviour of instrumented anchors is to be properly understood, with particular reference to stress/strain behaviour around the fixed anchor. Further research on this topic will also facilitate prediction of long term service performance in due course, and permit a more detailed analysis of failure mechanisms.

3.3 Grout/tendon interface

Recommendations pertaining to grout/tendon bond values commonly take no account of the length and type of tendon, tendon geometry, or grout strength, and for these reasons it is still advisable to measure experimentally the embedment length for known field conditions. Three mechanisms of bond, namely adhesion, friction and mechanical interlock are widely recognised (Figure 12) and our understanding of these mechanisms, including bond distribution, is still indebted to American research in the 1940's.[19]

As a guide, bearing in mind the compressive strength 30 N/mm² often required for cement based grouts prior to stressing, the permissible average bond stress under proof loading conditions should not exceed

(i) 1,0 N/mm² for clean plain wire or plain bar tendon
(ii) 1,5 N/mm² for clean crimped wire tendon
(iii) 2,0 N/mm² for clean strand or deformed bar.

The above values also apply in parallel multi-unit tendons provided the clear spacing is not less than 5 mm. For

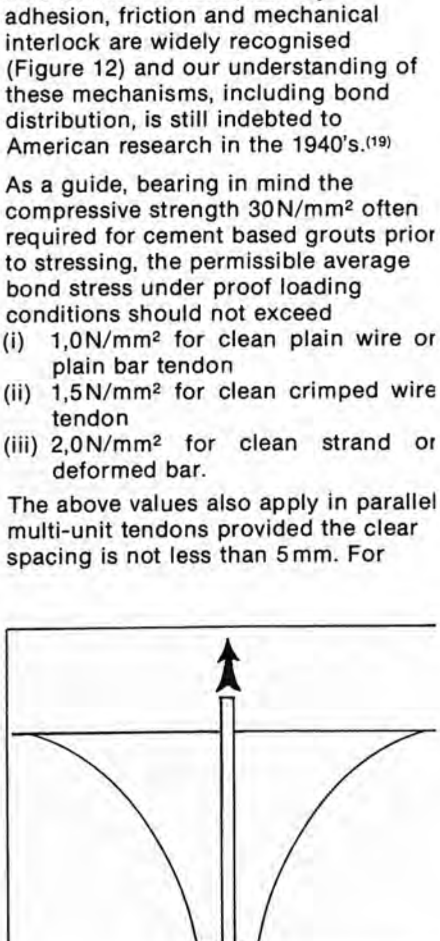


Figure 9 Geometry of soil mass assumed to be mobilised at failure.

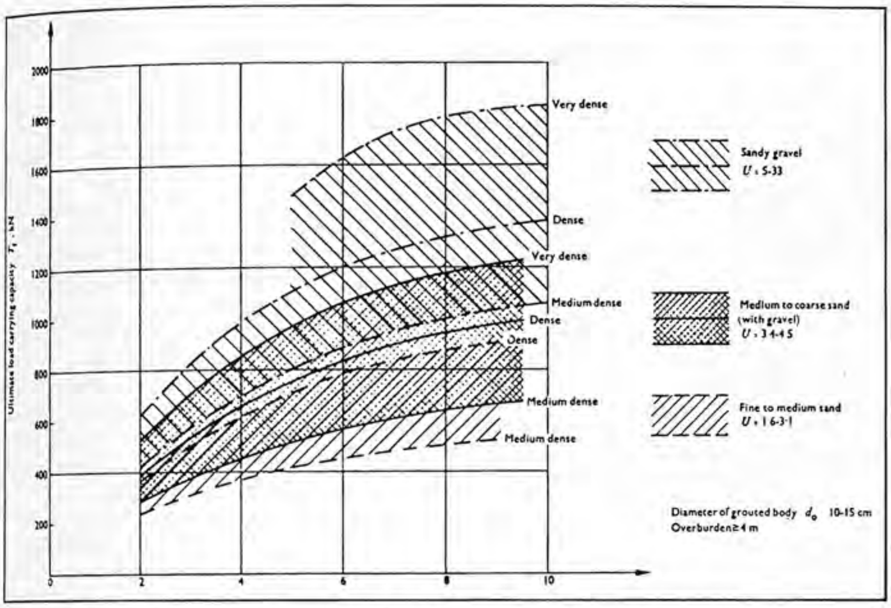


Figure 10 Load carrying capacity of anchors in frictional soils showing influence of soil type (uniformity coefficient), density and fixed anchor length.

noded tendons which can mobilise mechanical interlock or the shear strength of the grout, the minimum spacing criterion does not apply. For cement grouted anchors minimum tendon embedment lengths of 3 and 2 metres, are also applied for tendons bonded insitu, and under factory controlled conditions, respectively. For shorter bond lengths or where doubt exists concerning the factor of safety, the proposed design should be checked by full scale tests. Slightly rusted tendons are acceptable since this surface condition does not impair bond properties. Deep, flaky or loose rust however must be removed from the tendon prior to homing, and steel with surface pitting should be rejected. In current practice some enigneers have a fetish about tendon cleanliness, and it should be emphasised that certain protective waxes and films from degreasing

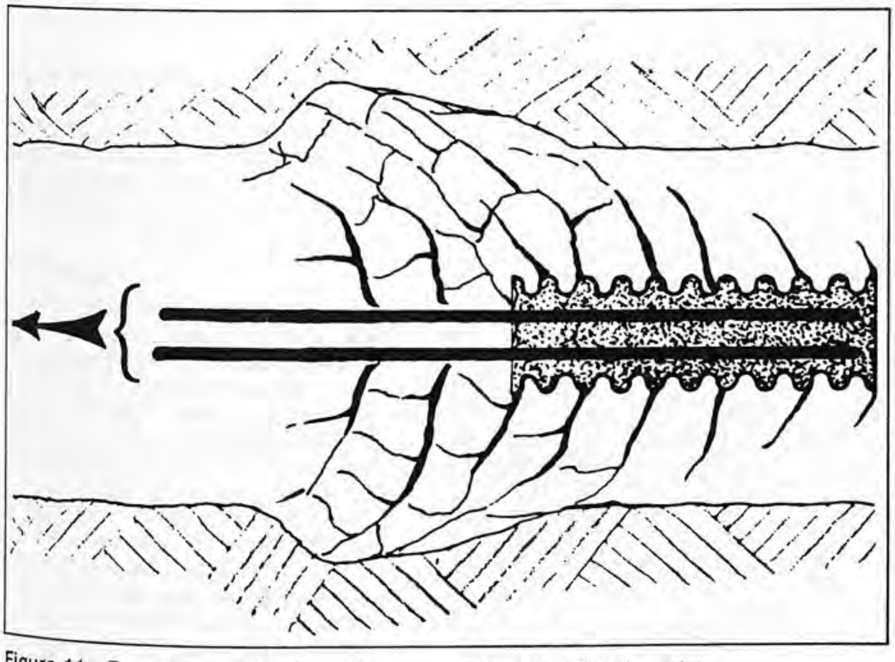


Figure 11 Bursting mechanism of compression type fixed anchor.

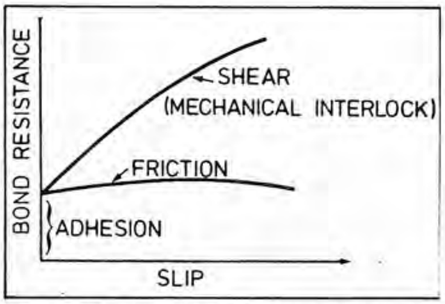


Figure 12 Idealised representation of major components of bond.

agents can have a deleterious effect on bond. A light uniform surface rusting may in fact provide a useful visual quality control but there is a need to classify corrosion, so that acceptance criteria can be established.

A topic that demands more detailed study, particularly in high capacity anchors (>200 tonnes) is debonding, which occurs as the ductile tendon transfers stress to the brittle cement grout (Figure 13).[14]

Micro cracking of the grout at the grout/tendon interface is inevitable, with a probable loss of adhesion and friction over a critical length of the tendon embedment. Over this length tendon extension occurs, the value of which is important when interpreting load/extension data at the anchor head during stressing, particularly when acceptance criteria are related to extension limits (See section 5.3). Equally important is the effect of grout cracking on the long term performance of anchors installed in aggressive environments (See section 3.8). Tendon density appears to be related to magnitude of debonding, and a current recommendation is that the steel tendon should not exceed 15 per cent of the borehole area.[20]

3.4 Grout Mix

All conventional hydraulic cements namely ordinary, rapid hardening, sulphate resisting, and low heat varieties are acceptable. In order to avoid "stress corrosion" of the steel tendon, however, the cement must not have a chlorine content from chlorides which exceeds 0,02% by weight, and sulphur from sulphides which exceeds 0,10% by weight. Use of High-alumina Cement is now more restricted worldwide, and may be confined to test anchors, and temporary anchors with a service life not exceeding six months, in view of the high heat of hydration and problems of reversion.

Admixtures should only be used if tests have shown that their use improves the properties of the grout, eg, by increasing workability, reducing bleed, or expanding the grout slightly. Admixtures must be free from any product liable to damage the steel or the grout itself. For example, no admixture should be used which contains in total more than 0,1 % of chlorides, sulphates or nitrates. Use of calcium chloride is forbidden. In general, use of admixtures is restricted to clay, where water reduction is important, and protective grouts in permanent anchors where low bleed is desirable. Bearing in mind the many benefits attributed to admixtures by manufacturers, care is required when making a technical assessment of the admixture prior to its adoption eg, effect of dosage on long term performance, or permanence in the anchor environment. As a guide to the data required for the mix before grouting is approved, key characteristics are listed below:

- Water/cement ratio
- Admixture concentration
- Flow reading
- Expansion, shrinkage or bleed
- Setting time
- Strength development with time

To ensure that the cement grout has good bond and shear strength the mix should be designed to attain a uniaxial compressive strength of 40 N/mm² minimum at 28 days. Further, the bleed of the tendon bonding grout should ideally not exceed 1 % by volume.

Figure 14 illustrates the extent to which these properties are related to the w/c ratio of an OPC grout. W/c ratios generally lie in the range 0,35 to 0,50, although higher values are still used in sandy alluvium.

3.5 Tendon

Tendons may consist of bar, strand or wire used either singly or in groups. In practice popular sizes are non-alloy steel wire (7 mm dia), non-alloy 7-wire strand (12,5 to 18 mm dia.), and low alloy steel bar (20 to 40 mm dia.). Stainless steel wire (7 mm dia.) and bar (10 to 32 mm dia.) are now available, but there is limited data currently available on relaxation characteristics. In this regard the maximum prestress loss allowed for in design is normally taken as the relaxation after 1 000 hours for the jacking force imposed at transfer. Centralisers

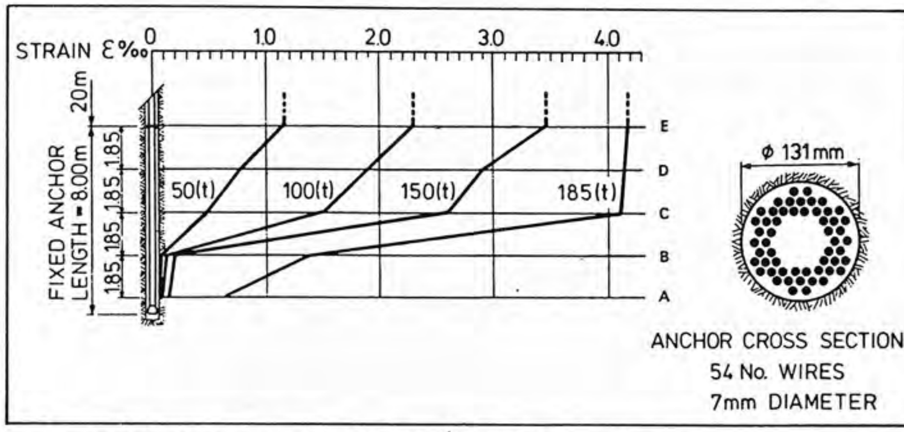


Figure 13 Strain distribution along tendon in fixed anchor zone of a 220 tonne capacity anchor.

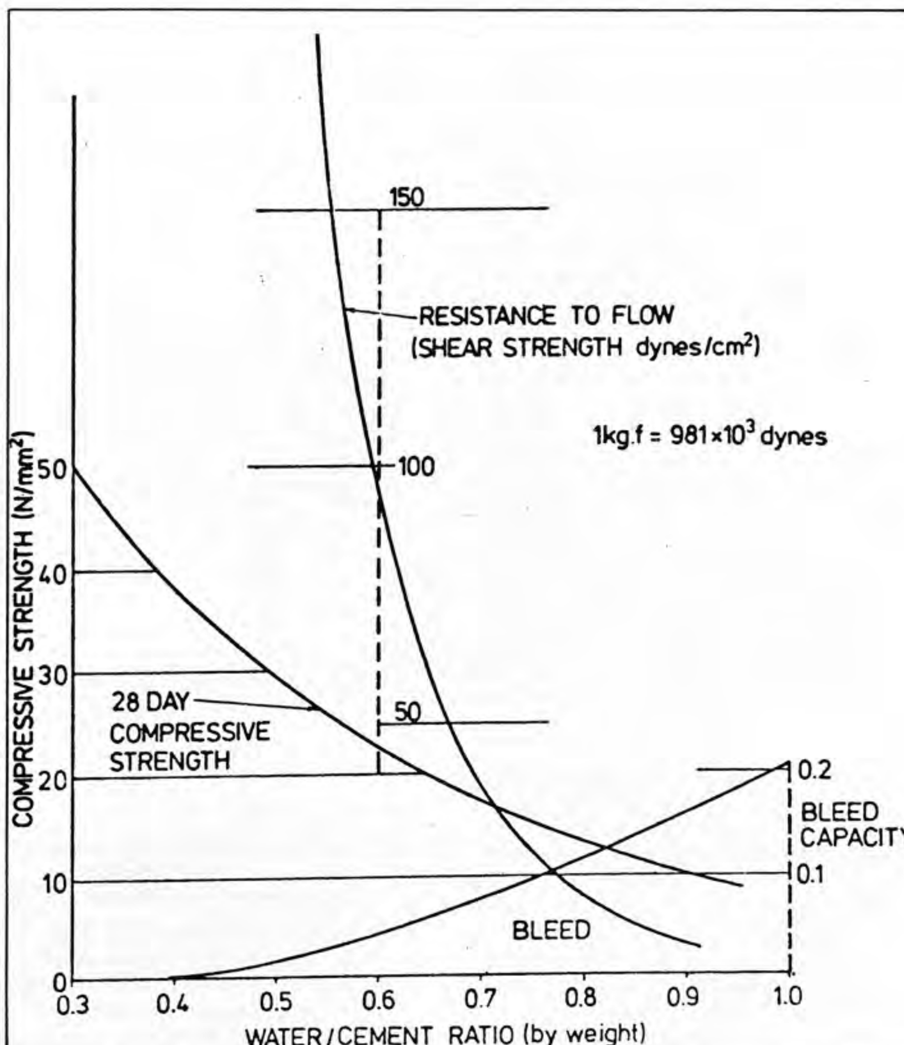


Figure 14 Effect of water content on OPC grout properties.

should be provided on all tendons to ensure that the tendon is a) free of borehole contamination, and b) centred in the grout column and thus protected against corrosion. Good centralisers ensure a minimum grout cover of 10mm to the tendon, and spacing is dictated by anchor inclination and tendon sag between points of support. In current practice tendon centralisation does not receive sufficient attention, and grout/tendon bond failures due to contamination are recorded occasionally at the proof loading stage. Spacers are required in the fixed anchor length of all multi-unit tendons to ensure separation between the individual components of the tendon and thus effective penetration of grout to provide adequate bond. A minimum of three should be provided in each fixed anchor length, and field tests to date indicate that for parallel strands no reduction in bond is apparent for clear spacings down to 5mm.[20]

3.6 Anchor head

The anchor head normally consists of a stressing head in which the tendon is anchored and a bearing plate by which the tendon force is transferred to the structure. Secondary distribution systems in the form of concrete end blocks or steel walings then transfer the force to the main structure. Whilst the design of such systems is well covered by structural engineering practice it is noteworthy that the head should be fitted to a tolerance of ± 5mm concentrically with the tendon which, in turn, should not suffer an angular deviation in excess of ± 3° from the axial position (Figure 15). Excess deviation reduces load transfer efficiency, and creates difficulties in wedge pull-in. In this regard no problems should be anticipated provided wedges are homed within a 5mm depth band (Figure 15).

3.7 Safety factors

The safety factor of an anchor is the ratio of the ultimate load holding capacity to the working load. In design potential failure mechanisms should be investigated for all major constituent materials and component interfaces of the anchor system, for example:

(a) within the ground mass
(b) at the ground/grout, or ground/mechanical anchor interface

Table 3 Suggested safety factors for anchor design

Anchor category	Minimum safety factor
Temporary anchors where the service life is less than 6 months and failure would have few serious consequences and would not endanger public safety eg, short term pile test.	1,3
Temporary anchors with a service life of up to 2 years, where although the consequences of failure are quite serious, there is no danger to public safety without adequate warning eg, retaining wall tie backs.	1,6
All permanent anchors. Temporary anchors in a highly aggressive environment, or where the consequences of failure are serious eg, temporary anchors for main cables of a suspension bridge or as a reaction for lifting heavy structural members.	2,0

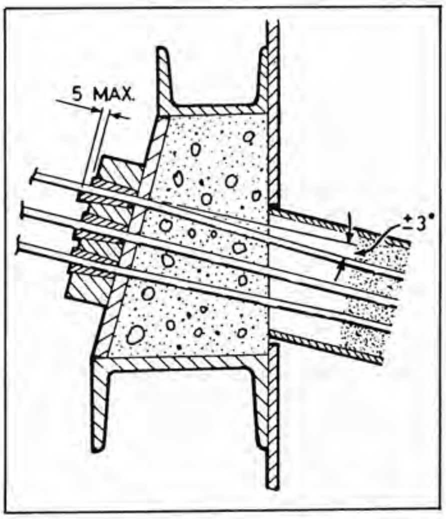


Figure 15 Recommended tolerances at anchor head.

(c) at the grout/tendon, and/or grout/capsule interface
(d) the tendon and anchor head.

For each potential failure mechanism a safety factor must be chosen having regard to how accurately the relevant characteristics are known, whether the system is temporary or permanent, ie, service life, and the consequences if failure occurs ie, danger to public safety and cost of structural damage. Suggested minimum safety factors for different anchor categories are listed in Table 3.

Since the minimum safety factor is applied to those anchor components known with the greatest degree of accuracy, the values listed invariably apply to the tendon or anchor head.

In regard to the major interfaces where failure may also occur design safety factors range generally from 2 to 4. Since the quantity and quality of data pertaining to the properties of the materials at the interface are extremely varied, it is not considered advisable to stipulate specific values and the designer must judge on the basis of the data presented in each case, what safety factor is prudent for the circumstances.

3.8 Corrosion

Having established broad guidelines for aggressivity in the ground (See section 2.4) in relation to cement and metal, it would appear that engineers are currently most interested in tendon corrosion. In this regard the corrosion of steel bar in concrete and its relation to cracking, has been surveyed recently.[21] It is clear that corrosion is likely to start first where a bar intersects a crack and in the short term (say up to 2 years) there is a significant influence of crack width on the amount of corrosion found near a crack. In the long term (say 10 years or more) however, the influence of the width of cracks on the amount of corrosion is negligible. Figure 16 illustrates the nature of cracking around a ribbed deformed bar.[22] It will be seen that the force is dominantly transferred from steel to concrete by the mechanical action of the ribs, and adhesion is largely lost.

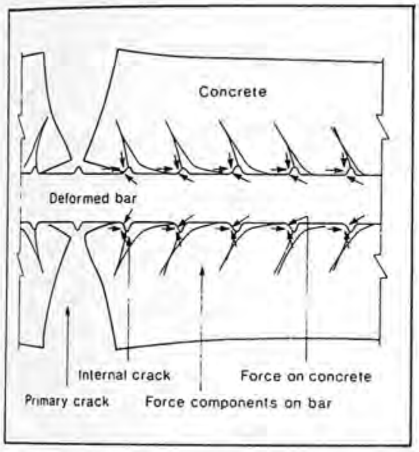


Figure 16 Schematic diagram of conditions close to a deformed reinforcing bar.

When steel rusts, the corrosion products generally occupy a volume of 2 to 3 times the volume of metal removed, and the ability of the surrounding grout to resist the internal stresses mobilised depends on the location of the tendon unit, the tensile capacity of the grout cover and, finally, the lateral restraint which may be available in the surrounding ground. In reinforced concrete structures the ratio of cover to bar diameter is a major parameter, and little significant corrosion damage, ie, spalling, has been observed when this ratio exceeds 3.

The object of design against corrosion is to ensure that, during the economic design life of the anchor, the probability of unacceptable corrosion occurring is acceptably small. It has been suggested[21] that a rational approach to design would be to satisfy the following inequality:

design life $= \leqslant t_0 + t_1$

where t_0 = time from construction to the initiation of corrosion

t_1 = time from initiation of corrosion to the occurrence of unacceptable corrosion.

The time t_0 is the time taken for a depassivating front to penetrate to the tendon steel; it will depend on whether or not the grout is cracked, the crack width, the cover and the nature of the environment. For cracked grout in a marine environment, $t_0 = 0$ would seem a reasonable assumption. In considering the variables that will control t_1, the first problem is to define unacceptable corrosion; for reinforced structures, the onset of spalling has been proposed; for anchors the monitoring of a loss of prestress in excess of 10% would cause concern, and assuming that the sole cause was corrosion, it is likely that loss of mechanical interlock would occur in advance of spalling. The time taken to cause loss of mechanical interlock clearly depends on rate of corrosion and currently no predictive capacity exists to establish values for t_0 and t_1 More fundamental research and field monitoring are urgently needed in this important field.

3.9 Corrosion protection

The problem is to decide whether the rate of corrosion merits the expense of protection. Exposure either to combinations of oxygen and chlorides or to anaerobic conditions in the presence of sulphates, or to severely fluctuating and high stress levels all enhance the rate of corrosion. However, since there is no certain way of identifying corrosive circumstances beforehand with sufficient precision to predict corrosion rates it is considered that all permanent anchors should be fully protected. Double protection is commonly specified, which means that at least two stages of protection are applied to the anchor, and it is preferred that both stages be susceptible to inspection prior to tendon homing.

Protective systems consist almost exclusively of those aiming to preclude a gaseous atmosphere around the metal by totally enclosing it within a covering or sheath. Their effectiveness depends on:

(a) the maintenance of continuity of coverings
(b) external fluid pressure gradients across coatings and joints in coverings
(c) content and cleanliness of atmosphere during application of coatings
(d) details of junctions in coverings especially at fixed anchors and anchor heads
(e) the electrochemical environment at the metal surface.

Commonly, individual tendons are provided with a protective covering such as plastic sheaths and grease infilling over their free length. For plastic, a minimum wall thickness of 1 mm is often specified, but control on greases is lax. Not all greases maintain their required properties for the life of the anchor and it is useful, for example, to establish the oxidation resistance for comparative purposes. For the fixed anchor or tendon bond section, the complete tendon is often encased inside a corrugated plastic or steel tube, using epoxy/polyester resin or cement grout. Figure 17 illustrates a fully protected permanent anchor.

3.10 Stressing equipment

Stressing equipment which is normally hydraulically operated for wire and strand tendons should preferably

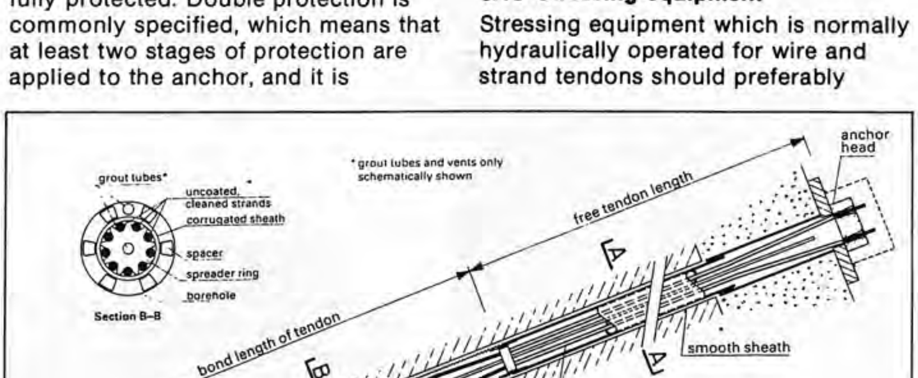


Figure 17 Permanent VSL anchor.

tension the whole of the tendon in one operation. This technique facilitates cyclic loading tests, but both single unit and multi unit operations are used in practice. Where the maximum force in a bar unit does not exceed 0,33 x characteristic strength, stressing by torque wrench is permitted. The design of the jack should permit the tendon elongation at every stage to be measured to an accuracy of ± 1mm. In this regard the hydraulic pump should be designed to permit precise control of the stressing jack, whether opening or closing, to an accuracy of nor more than ± 0,5mm of jack stroke. The circuit must permit the accurate reading of the pressure gauge without vibration and the pump unit should always be equipped with a site-regulated pressure overload relief valve to prevent tendon damage by over-tensioning. All flexible connections between pump and jack should have a burst pressure at least twice the maximum pump pressure rating and should be fitted with non-spill connectors to avoid contamination. Torque-wrenches should be capable of tensioning the tendor bar units to an accuracy of ± 5%. When provided with a calibration certificate the condition of calibration, the specification of the nut and washer surfaces, and the lubrication should be stated. For the measurement of force, pressure gauges are commonly used, but rarely supplied with a calibration certificate. Where load cells are provided they should have means of accurately centering them on the jack to ensure co-axiality with the tendon, and the surface tolerance on seating may have to be 0,1mm or better if accurate readings are required. Cells should be provided with calibration certificates which show the effects of sustained loading on the cell unit where this is appropriate. In general, stressing equipment is manufactured to a high mechanical standard but on site, regular calibration to ensure reliability and accuracy is relatively rare. It is recommended that pressure gauges should be calibrated either every fifty stressings or after every fourteen days use (whichever is the more frequent) against properly maintained class A gauges on site. Similarly, load cells should be calibrated every fifty stressings or after every 28 days use (whichever is the more frequent), unless complementary pressure gauges used simultaneously indicate no significant variation, in which case the interval between calibrations may be doubled.

4. Construction

4.1 Drilling

Drilling methods normally involve a rotary, percussive or rotary-percussive mechanism and, occasionally vibratory driving techniques. Diamond core drilling is rarely used for anchor holes because of high cost and the belief that the smoothness of the bore reduces the bond capacity. The drilling method should be chosen to give minimal disturbance to the surrounding ground. For example, air flushing should be used with caution in weak fine grained soils, and in the neighbourhood of buildings control of flushing pressure is vital to avoid hydrofracture of the surrounding ground (Figure 6). Hole stability is critical, and special care is required to ensure that the drilling or flushing method does not give rise to loss of ground, significantly above the volume of the specified drill hole. In this respect the volume of material removed during drilling should be checked. For specified drill holes, the entry point should be positioned within ± 75mm with an angle tolerance of ± 2° , and an overall hole deviation of 1 in 50 should be anticipated (Figure 18). Drilling of the fixed anchor length should be carried out on the same day as tendon homing and grouting. A delay between completion of drilling and grouting can have serious consequences due to deterioration of the ground, particularly in over consolidated, fissured cohesive materials and soft rocks.

4.2 Water testing

On completion of drilling in rock strata the hole should be tested for "watertightness" by measuring rate of water loss or gain in the drillhole. The purpose of this test is to provide an assessment of the likelihood of cement grout loss, when fractures have been encountered during drilling, or the rock formation is suspect. Grout loss from around the tendon in the fixed anchor zone is of prime importance in relation to efficient distribution of load and corrosion protection. Pregrouting is recommended if leakage or water loss in the hole exceeds 3 litres/minute/atmosphere, measured over a period of 10 minutes[24] (see also Figure 18). This flow threshold is based on experience in rock grouting which indicates that cement is not suitable, because of its particle size, for the treatment of fissures which are less than 160 microns thick. A single 160 micron fissure gives rise to a flow of 3,2 litres/minute/atmosphere. Where the water test indicates a connection to an adjacent unstressed anchor then stressing of that anchor should be carried out prior to, or 7 days after pregrouting. In unconsolidated deposits where large voids or cavities are suspected, eg, by a complete loss of flushing medium during drilling in the anchor zone, water or pregrouting tests should be carried out prior to homing the tendon to measure the insitu permeability or grout-take respectively. If grouting proceeds in the absence of these tests and then excessive grout-takes are recorded at nil back pressure, grouting should terminate when the volume injected exceeds three times the theoretical

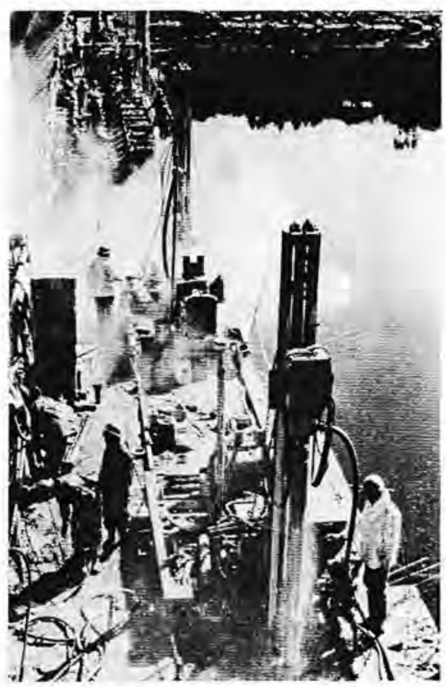


Figure 18 Drilling operations at Laing Dam, South Africa, where a tolerance on hole verticality of 1:150 was specified, together with a watertightness criteria of 0,05 litres/metre of hole/minute at 5 bars. 131 high capacity anchors were installed into rock, with overall lengths of 14 to 63m, and working loads in the range 4800 to 6000kN.

Table 4 Recommended items for stressing record

General classification data			
Project	Contractor	Engineer	Inspector
Date	Time started	Time completed	Stressing personnel
Anchor No.	Free length	Fixed anchor length	Ground type
Tendon type	E-value of steel	Working load (T_u)	Test load (T_t)
Jack type	Area of piston	Maximum rated capacity	Date of last calibration
Pump type	Pressure gauge range	Pressure gauge accuracy	Date of last calibration
Type of anchor head	Lock-off mechanism	Initial seating pressure	Strand pull-in
Data monitored during stressing			
Permanent bearing plate movement	Tendon extension	Jack pressure	Tendon pull-in at lock-off

volume calculated for the hole. In this regard, useful grout consumption figures may already be available from adjacent anchors. Water testing, although considered prudent by engineers, is not common practice.

4.3 Tendon

Anchor tendons should be fabricated in a workshop or in the field under a covered area, using trained personnel. Tendons must be free of detrimental rust or any deleterious substance and during fabrication and subsequent storage, tendons should be supported off the floor by trestles. In multi-unit tendons spacers and centralisers made of steel or plastic must be securely fixed, so that their positions are maintained during subsequent handling and homing operations. When homing the tendon a steady controlled rate should be maintained and for heavy tendons weighing in excess of 200kg, mechanical handling equipment should be employed as manual operations can be difficult and hazardous (Figure 19). The use of a funnel or radiused entry pipe at the top of a cased hole is recommended to avoid mechanical damage to the tendon. On occasion, particularly at the start of a contract, the tendon should be withdrawn after the homing operation, in order to judge the efficiency of the centraliser and spacer units, and also to observe damage, distortion or the presence of smear, eg, in weak, cohesive ground.

4.4 Grouting

The function of the grouting may be defined as follows:

(i) the formation of the fixed anchor in order that the applied load may be transferred from the tendon to the surrounding ground
(ii) to augment the protection of the tendon against corrosion, and
(iii) where pregrouting is deemed necessary, to fill voids and/or fissures in the ground prior to tendon installation.

Mixing should be carried out mechnically with a high shear action in order to obtain a homogeneous grout. On completion of the mixing, the grout should be kept in continuous movement eg, slow agitation in a storage tank. Each stage of injection should be performed in one continuous operation, and if, for any reason, grouting is interrupted or delayed beyond the setting period, the tendon should be removed from the borehole. The initial grout should then be removed by flushing or redrilling, and the tendon homing and grouting stages repeated. Where grouting is carried out under pressure, such pressure should be limited to avoid distress in the ground or on adjacent structures. A limiting pressure of 20 kN/m² per metre depth of ground is common in practice. Basic data to be recorded for the grouting operation are as follows:

Mix constituents
Type of mixing equipment
Mixing time
Grouting pressure
Quantity of grout injected
Injection time
Details of samples and tests.

Quality controls should include fluidity and specific gravity measurements during the fluid stage, setting time and

Figure 19 At Milton Dam, Ohio, USA due to difficulty of access, the anchor tendons were transported and homed by helicopter.

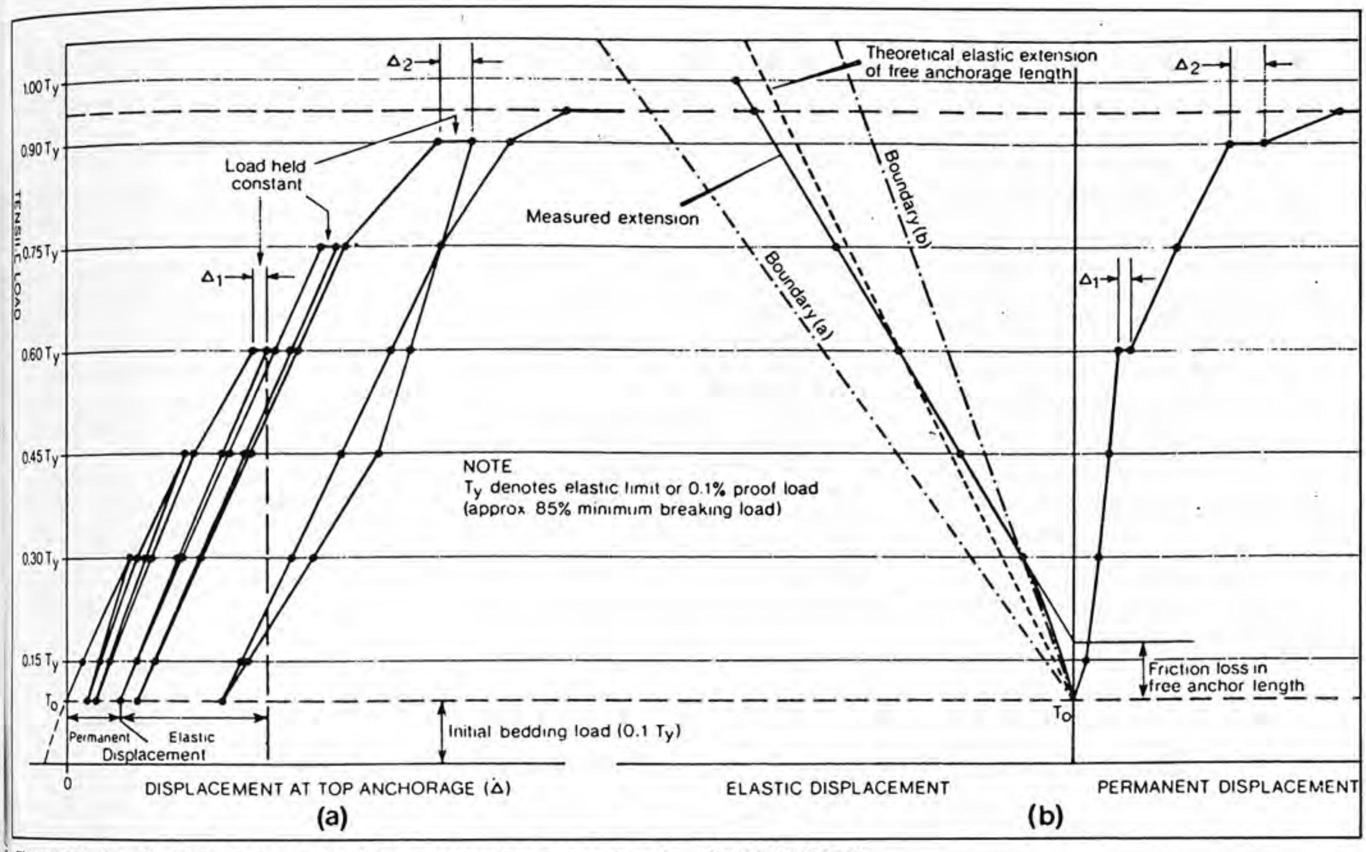


Figure 20 Load-displacement data from stressing procedure in DIN 4125 (1972).

bleed during the stiffening stage, and cube crushing strengths at, say, 7 and 28 days. The number and frequency of tests will vary according to site circumstances, but generally these tests should be carried out daily. As a principle in quality control, emphasis should be placed on those tests which permit the grout to be assessed prior to injection. The current amphasis is on cube tests.

4.5 Anchor head

In soil conditions where some creep or relaxation of the fixed anchor may continue after grouting of the free length of the tendon, grouting should be terminated below the anchor head and the void in the anchor head zone subsequently filled by a flexible corrosion protection material eg, bitumen. In this way fixed anchor displacements cannot impose excessive compressive stresses on the grout column in the free length. This technique also prevents falsely high loads being mobilised.

4.6 Stressing

Stressing is required to fulfill two functions and the equipment and procedures should be specified accordingly:

(a) to tension the tendon and to anchor it at the required force
(b) to ascertain and record the behaviour of the anchor so that it can be compared with the behaviour of control anchors.

Stressing and recording should be carried out by experienced personnel, but preferably not before the primary grout forming the fixed anchor has attained a crushing strength of at least 30 N/mm², as verified from tests on 100 mm cubes. No tendon which is to form part of any temporary or permanent works should be stressed at any time beyond either 80 % of the characteristic strength, or 90 % of the 0,2 % proof strength. Table 4 details items for inclusion in the stressing record for each anchor.

The cutting of the tendon after final grouting or stressing should preferably be done without heat, eg, disc-cutter. Where cutting torches are used the cut should not be closer than four tendon unit diameters from the face of the anchorage wedge or nut otherwise load transfer efficiency may be reduced. For disc cutters the minimum distance is two tendon unit diameters.

5. Testing

5.1 General

Five classes of test may be relevant for an anchor project, namely:

(i) Precontract component testing
(ii) Field test anchors
(iii) Routine acceptance tests
(iv) Monitoring of service behaviour of individual anchors
(v) Monitoring of the overall anchor/ground/structure system.

Classes (i) and (ii) are often combined in the form of precontract proving tests where the objective is to demonstrate in advance of any site use the quality and adequacy of the design and material, and the levels of safety which the design provides. Test anchors should, ideally, be installed at the same inclination and in the same strata as proposed for the service anchors, to ensure that the results are relevant and directly applicable.

Routine acceptance testing of each production anchor demonstrates the short term ability of the anchor to act satisfactorily, in addition to providing a measured load safety factor.

Although these tests are commonplace, acceptance criteria have not been standardised and some current procedures are described briefly in Sections 5.2, 5.3 and 5.4. In contrast, there is a dearth of data on long term monitoring both for individual anchors and complete systems. One important consequence for acceptance testing is that optimum overload allowances cannot be determined to accommodate long term losses. Field monitoring studies of anchorage systems are necessary if important concepts relating to overall stability and group effects are to be verified.

5.2 Proof loading

Proof loading an anchor automatically tests the installation and confirms to a certain degree design load safety factors. Significant errors made in either the design or construction stages will be pinpointed immediately and potentially dangerous and expensive consequences avoided. At the present time there is a clear trend towards proof loading each anchor to 1,25 and 1,5 times the working load for temporary and permanent works, respectively. Whilst proof loading is common practice however, accurate monitoring of extensions is the exception rather than the rule.

5.3 Load-extension data

Insufficient attention is paid to the interpretation and consideration of the monitored load-extension data. As a result there has been little progress in the understanding of basic anchor behaviour with particular regard to component movements of the overall anchor system. In this regard, the acceptability of anchors should also be judged from the load-extension curve when compared with the load-extension curves obtained from previous test anchors and from the similarity between the calculated free length and the specified free length. In some foreign codes[2],[6],[7] it is stipulated that the plotted results should lie between the lines corresponding to:

(a) the extension of a tendon of length equivalent to 80% free length, and

(b) the extension of a tendon of length equivalent to the free length plus 50% fixed length (Figure 20), or 110% of the free length in the case of a fully decoupled tendon with an end plate or nut.

5.4 Service behaviour

For creep or relaxation losses a load loss of up to 5%, or a creep displacement of 1 mm, measured after 24 hours, has been specified as acceptable on occasions, but no reliance should be placed on these arbitrary figures. Can, in fact, a 24-hour acceptance test guarantee satisfactory performance over a service period of 50 years? More results should be published where long-term monitoring over periods in excess of 24 hours has been carried out to check service behaviour and act as a control to verify that anchor performance is satisfactory.[25]

6. Anchor location records

Proper records are important for both temporary and permanent installations. Lack of knowledge of the location of temporary anchors may lead to damage of construction plant. It is usual for copies of records to be deposited at the local authority Building Regulations department. Further, for the benefit of future developers, plans showing the details and locations of the anchors should be retained with the Deeds of the property. Records may also be required by the owners of adjacent property.

7. Conclusion

Experience indicates that higher quality and more detailed ground investigations are required at the planning stage of many anchor projects in order to permit their economic design and construction. A proper design should consider static and dynamic loads, location of anchors, load transfer lengths, overall stability and service life. In this respect, account must be taken of loads and accompanying deformations under service conditions, as well as the deformation mechanisms developed at failure. Systematic full scale testing remains the finest source of information on the behaviour of anchors.

During anchor construction the quality of workmanship greatly influences subsequent performance. This workmanship factor limits the ability to predict anchor behaviour solely on the basis of empirical rules and ground investigation data. As a consequence quality controls and record keeping are strongly recommended during construction because precautionary measures save more time and money than remedial measures. Further, each anchor, once installed, should be subjected to an initial proof load greater than the required working load, followed by a check on the "lock-off" load after a short period of service. In this way the safety and satisfactory performance of each anchor should be ensured. Short term acceptance criteria have not however been correlated with long term behaviour to date.

In general, much more field research needs to be conducted on the service performance of anchors, the mechanisms of debonding and corrosion, all of which are still not properly understood.

Millions of anchors have already been installed, apparently successfully, and bearing in mind the absence of serious failures, there is a strong base upon which anchor specialists can build and expand their market with confidence. There is no room for complacency however; engineers must rigorously apply high standards and much field development remains to be tackled.

References

(1) Bureau Securitas (1972), "Recommendations regarding the design, calculation, installation and inspection of ground anchors". Editions Eyrolles 61 Boulevard Saint-Germain, Paris-VE (Ref TA72)

(2) Deutsche Industrie Norm (1972), "Soil and rock anchors; bonded anchors for temporary uses in loose stone; dimensioning, structural design and testing". DIN 4125 Part 1, 1972, see also Part 2, 1976 for permanent anchors.

(3) South African Code of Practice (1972), "Lateral support in surface excavations" The South African Institution of Civil Engineers, Johannesburg.

(4) Standards Association of Australia (1973), "Prestressed concrete code CA35" Section 5-Ground Anchorage pp50-53

(5) Swedish Building Code (1974) "Tie rods for temporary sheet pile walls". Swedish Planning Board Report 27. Stockholm.

(6) Klein, K, (1974), "Draft standard for prestressed rock anchors". Symposium on rock anchorage of hydraulic structures. Vir Dam. Czechoslovakia, pp 86-102.

(7) Austrian Standards Institute (1976), "Prestressed anchors for soil and rock" Onorm B4455. Osterreichisches Normungsinstitut, Wien.

(8) Schweizer Norm (1977), "Ground and rock anchors" SN 533 191 Schweizerischer Ingenieurund Architekten-Verein. Postfach, 8039 Zurich.

(9) International Society for Rock Mechanics (1974), "Suggested methods for rockbolt testing" Committee on Field Tests Document No. 2 (Final Draft) March 1974.

(10) Prestressed Concrete Institute (1974), "Tentative recommendations for prestressed rock and soil anchors". PCI Post-Tensioning Committee. 20 North Wacker Drive, Chicago, Illinois 60606.

(11) Federation of International Pre-stressing (1975), "Guides to good practice" Practical Construction Report (FIP/2/1 (September) pp 22-32.

(12) King, R A, (1977), "A review of soil corrosiveness with particular reference to reinforced earth". TRRL Supplementary Report 316 Transport and Road Research Laboratory, Crowthorne, England.

(13) Hobst, L (1977), & Zajic, J, "Anchoring in Rock" Elsevier Scientific Publishing Co. Amsterdam.

(14) Littlejohn, G S, (1977) & Bruce, D A, "Rock ancors — state of the art". Foundation Publications Ltd., Brentwood, Essex, England.

(15) Mariupol' skii L G, (1965), "The bearing capacity of anchor foundations". Osnovaniya, Fundamentary: Mekhanika Gruntou 1 (Jan/Feb), 14-18.

(16) Brown, D G, (1970), "Uplift Capacity of Grouted Rock Anchors" Ontario Hydro Research Quarterly 22(4) pp 18-24.

(17) Littlejohn, G S, (1970), "Soil Anchors". Ground Engineering Conference Institution of Civil Engineers, London pp 33-44.

(18) Ostermayer, H, (1974), "Construction, carrying behaviour and creep characteristics of ground anchors". Proc. Conf. on Diaphragm Walls and Anchorages, Institution of Civil Engineers, London. pp 141-151.

(19) Gilkey, H J, Chamberlin, S J, & Beal, R W, (1940), "Bond between concrete and steel". Reproduced in Eng. Rep. No. 26, Iowa Eng. Exp. Stn., Iowa State College. Ames (1956) pp 25-147.

(20) Littlejohn, G S, Bruce, D A & Deppner W, (1978), "Anchor field tests in carbonferous Strata". Revue Francaise de Géotechnique No. 3, pp 82-86.

(21) Beeby, A W, (1978), "Corrosion of reinforcing steel in concrete and its relation to cracking" The Structural Engineer 56A, 3, pp 77-81.

(22) Goto, Y, (1971), "Cracks formed in concrete around deformed tension bars" Journal of American Concrete Institute 68, 4, pp 244.

(23) Houston, Atimtay & Ferguson, P M, (1972), "Corrosion of reinforcing steel embedded structural concrete". Research report 112-1-F, Centre for Highway Research, University of Texas at Austin.

(24) Littlejohn, G S, (1975), "Acceptable water flows for rock anchor grouting". Ground Engineering 8, 2, pp 46-48.

(25) Littlejohn, G S, & Bruce, D A, (1979), "Long term performance of high capacity rock anchors at Devonport". Ground Engineering 12, (October).

Design estimation of the ultimate load-holding capacity of ground anchors

by G. S. LITTLEJOHN

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

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

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

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

Introduction

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

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

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

In this context general failure is defined as the full mobilisation of slip lines or the generation of significant deformations, extending to ground surface. Field experience indicates that general failure does not occur for slenderness ratios§ in excess of 15, and for the small diameters involved, the top of the fixed anchor is usually founded at depths in excess of 5m. In such circumstances the ultimate load-holding capacity of the anchor (T_f) is dependent on the following factors, although due to lack of knowledge item 5 is not generally isolated in design calculations:

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

§Slenderness ratio = depth to top of fixed anchor/effective diameter of fixed anchor

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

Anchor types

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

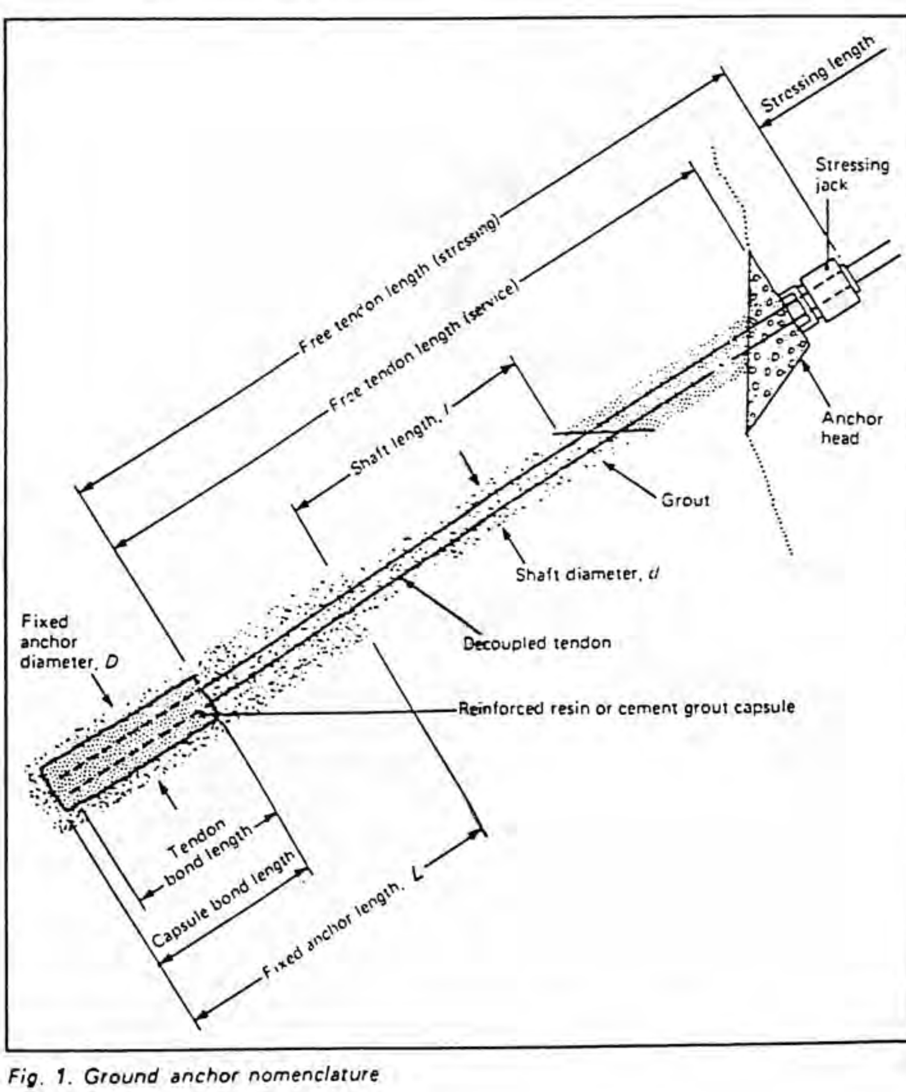


Fig. 1. Ground anchor nomenclature

technique where choice and method of flush are important. Accordingly, the types of anchor to which the design rules are applicable are now described. The four types are illustrated in Fig. 2. These comprise:

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

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

Type C: High-pressure grouted borehole via a lining tube or insitu packer, where the grouted fixed anchor is enlarged via hydrofracturing of the ground mass to give a grout root or fissure system beyond the core diameter of the borehole. Where stage grouting along the fixed anchor or regrouting are envisaged a tube-à-manchette system [1] can be incorporated as shown in Fig. 3. This anchor type is employed primarily in cohesionless soils although some success has also been achieved in stiff cohesive deposits. Design is based on the assumption of uniform shear along the fixed anchor.

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

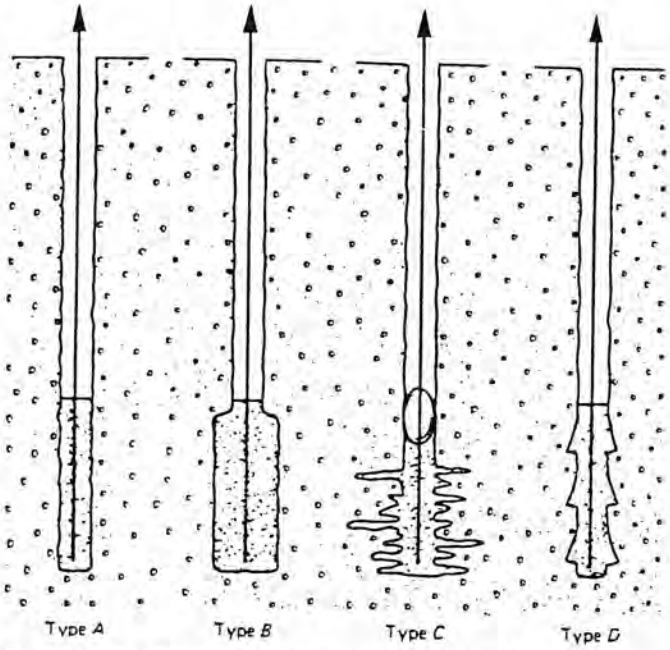


Fig. 2. Main types of cement grout injection anchor

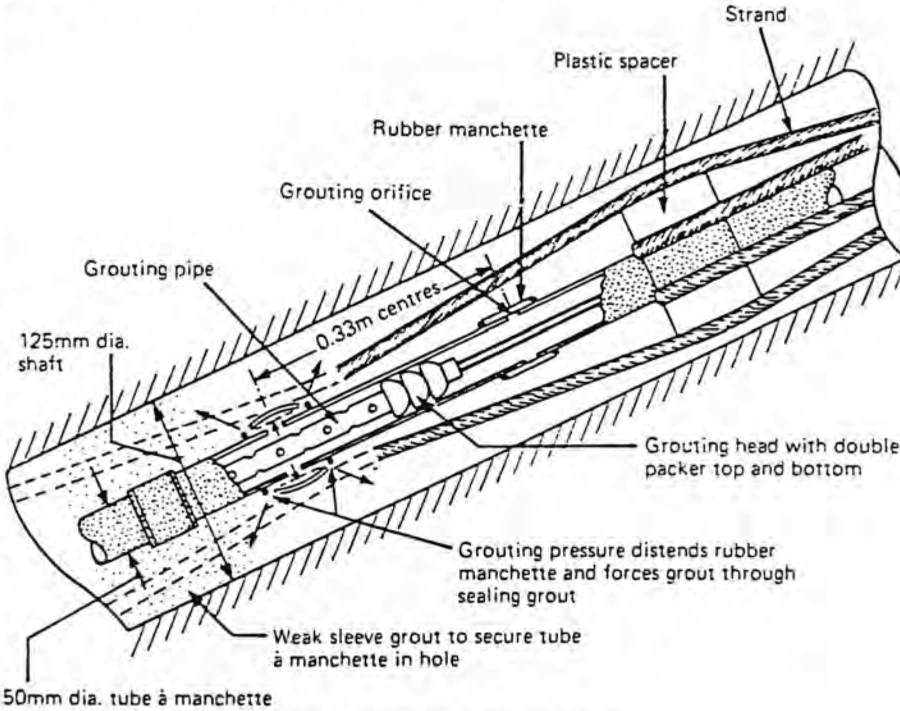


Fig. 3. Detail of tube à manchette for pressure grouting control

Rock

The earliest reports of anchoring bars into rock to secure a roof date from 1918 in the Mir Mine of Upper Silesia in Poland [2], and by 1926 faces of an inclined shaft, in Chustenice shales in Czechoslovakia, were secured against caving by grouted bars installed in a fan pattern[3]. In the field of civil engineering the history of rock anchors dates from 1934 when Coyne pioneered their use during the raising of Cheurfas Dam in Algeria[4]. On this project 37 anchors were constructed in sandstone, fixed with the aid of double under-reams, and then tensioned individually to 1 000tonnes.

Whilst all anchor types *A - D* are applicable to rock, the straight shaft tremie-grouted Type *A* is the more popular in current practice on the basis of cost and simplicity of construction. For such anchors designs are based on the assumption of uniform bond distribution [5]. Thus the pull-out capacity is estimated from Eqn. *1*.

$$T_f = \pi D L \, \tau_{ult} \qquad \dots (1)$$

where τ_{ult} = ultimate bond or skin friction at rock/grout interface.

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

Eqn. *1* is based on the following simple assumptions:

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

Where shear strength tests are carried out on representative samples of the rock mass, the maximum average working bond stress at the rock/grout interface should not exceed the minimum shear strength divided by the relevant safety factor (normally not less than 2). This approach applies primarily to soft rocks where the uniaxial compressive strength (UCS) is less than 7N/mm^2, and in which the holes have been drilled using a rotary-percussive technique. In the absence of shear strength data or field pull-out tests the ultimate bond stress is often taken as one-tenth of the uniaxial compressive strength of massive rocks (100 per cent core recovery) up to a maximum value τ_{ult} of 4.2N/mm^2. As confirmation τ_{ult} = 4.3N/mm^2 is indicated for design

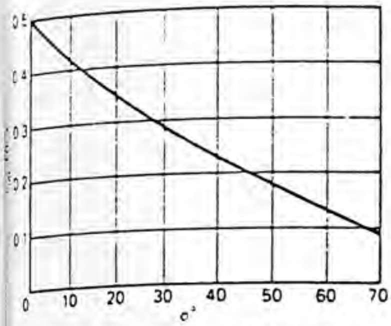


Fig. 4. Effect of ϕ on τ_{ult}/UCS ratio

in hard coarse grained sandstone by Canadian research[4].

In some rocks, particularly granular weathered varieties with a relatively low value, the assumption that τ_{ult} equals 10% UCS may lead to an artificially low estimate of shear strength (Fig. 4). In such cases, the assumption that τ_{ult} equals 0.35% UCS may be justified.

Bond values which have been recommended[3] for a wide range of igneous, metamorphic and sedimentary rocks, are presented in Table I. Where included, the factor of safety relates to the ultimate and working bond values, calculated assuming uniform bond distribution. It is common to find that the magnitude of bond is simply assessed by experienced engineers and the value adopted for working bond stress often lies in the range 0.35 - 1.4N/mm².

The Australian Code[7] states that whilst a value of 1.05N/mm² has been used in a wide range of igneous and sedimentary rocks, site testing has permitted bond values of up to 2.1N/mm² to be employed. In this connection the draft Czech Standard[8] concludes that since the estimation of bond magnitude and distribution is a complex problem, field anchor tests should always be conducted to confirm bond values in design, as there is no sufficient or reliable alternative. Certainly, a common procedure amongst anchor designers is to arrive at estimates of permissible working bond values by factoring the value of the average ultimate bond calculated from test anchors.

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

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

$$\tau_{ult} = 0.007N + 0.12 \text{ (N/mm}^2\text{)} \quad \ldots (2)$$

where N = number of blows per 0.3m

Similarly, eqn. 3 has been established for stiff/hard chalk[10]

$$\tau_{ult} = 0.01N \text{ (N/mm}^2\text{)} \quad \ldots (3)$$

Fixed anchor length

The recommendations made by various engineers with respect to length of fixed anchor[3] are presented in Table II. Under

TABLE I. ROCK/GROUT BOND VALUES WHICH HAVE BEEN RECOMMENDED FOR DESIGN

Rock type	Working bond (N/mm²)	Ultimate bond (N/mm²)	Factor of safety	Source
Igneous				
Medium hard basalt		5.73	3-4	India—Rao (1964)
Weathered granite		1.50-2.50		Japan—Suzuki et al (1972)
Basalt	1.21-1.38	3.86	2.8-3.2	Britain—Wycliffe-Jones (1974)
Granite	1.38-1.55	4.83	3.1-3.5	Britain—Wycliffe-Jones (1974)
Serpentine	0.45-0.59	1.55	2.6-3.5	Britain—Wycliffe-Jones (1974)
Granite & basalt		1.72-3.10	1.5-2.5	USA—PCI (1974)
Metamorphic				
Manhattan schist	0.70	2.80	4.0	USA—White (1973)
Slate & hard shale		0.83-1.38	1.5-2.5	USA—PCI (1974)
Calcareous sediments				
Limestone	1.00	2.83	2.8	Switzerland—Losinger (1966)
Chalk—Grades I-III (N=SPT in blows/0.3m)	0.005N	0.22-1.07 0.01N	2.0 (Temporary) 3.0-4.0 (Permanent)	Britain—Littlejohn (1970)
Tertiary limestone	0.83-0.97	2.76	2.9-3.3	Britain—Wycliffe-Jones (1974)
Chalk limestone	0.86-1.00	2.76	2.8-3.2	Britain—Wycliffe-Jones (1974)
Soft limestone		1.03-1.52	1.5-2.5	USA—PCI (1974)
Dolomitic limestone		1.38-2.07	1.5-2.5	USA—PCI (1974)
Arenaceous sediments				
Hard coarse-grained sandstone	2.45		1.75	Canada—Coates (1970)
Weathered sandstone		0.69-0.85	3.0	New Zealand—Irwin (1971)
Well-cemented mudstones		0.69	2.0-2.5	New Zealand—Irwin (1971)
Bunter sandstone	0.40		3.0	Britain—Littlejohn (1973)
Bunter sandstone (UCS>2.0N/mm²)	0.60		3.0	Britain—Littlejohn (1973)
Hard fine sandstone	0.69-0.83	2.24	2.7-3.3	Britain—Wycliffe-Jones (1974)
Sandstone		0.83-1.73	1.5-2.5	USA—PCI (1974)
Argillaceous sediments				
Keuper marl		0.17-0.25 (0.45 c_u)	3.0	Britain—Littlejohn (1970) c_u = undrained cohesion
Weak shale		0.35		Canada—Golder Brawner (1973)
Soft sandstone & shale	0.10-0.14	0.37	2.7-3.7	Britain—Wycliffe-Jones (1974)
Soft shale		0.21-0.83	1.5-2.5	USA—PCI (1974)
General				
Competent rock (where UCS>20N/mm²)	Uniaxial compressive strength—30 (up to a maximum value of 1.4N/mm²)	Uniaxial compressive strength—10 (up to a maximum value of (4.2N/mm²)	3	Britain—Littlejohn (1972)
Weak rock	0.35-0.70			Australia—Koch (1972)
Medium rock	0.70-1.05			
Strong rock	1.05-1.40			
Wide variety of igneous and metamorphic rocks	1.05		2	Australia—Standard CA35 (1973)
Wide variety of rocks	0.98			France—Fargeot (1972)
	0.50			Switzerland—Walther (1959)
	0.70			Switzerland—Comte (1965)
		1.20-2.50		Switzerland—Comte (1971)
	0.70		2-2.5 (Temporary) 3 (Permanent)	Italy—Mascardi (1973)
	0.69	2.76	4	Canada—Golder Brawner (1973)
	1.4	4.2	3	USA—White (1973)
		15-20 per cent of grout crushing strength	3	Australia—Longworth (1971)
Concrete		1.38-2.76	1.5-2.5	USA—PCI (1974)

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

Fixed anchor length (metres) Minimum	Range	Source
3.0		Sweden—Nordin (1966)
3.0		Italy—Berardi (1967)
	4.0- 6.5	Canada—Hanna & Seaton (1967)
3.0	3.0-10.0	Britain—Littlejohn (1972)
	3.0-10.0	France—Fenoux et al (1972)
	3.0- 8.0	Italy—Conti (1972)
4.0 (very hard rock)		South Africa—Code of Practice (1972)
6.0 (soft rock)		South Africa—Code of Practice (1972)
5.0		France—Bureau Securitas (1972)
5.0		USA—White (1973)
3.0	3.0- 6.0	Germany—Stocker (1973)
3.0		Italy—Mascardi (1973)
3.0		Britain—Universal Anchorage Co. Ltd. (1972)
3.0		Britain—Ground Anchors Ltd. (1974)
3.5 (chalk)		Britain—Associated Tunnelling Co. Ltd. (1973)

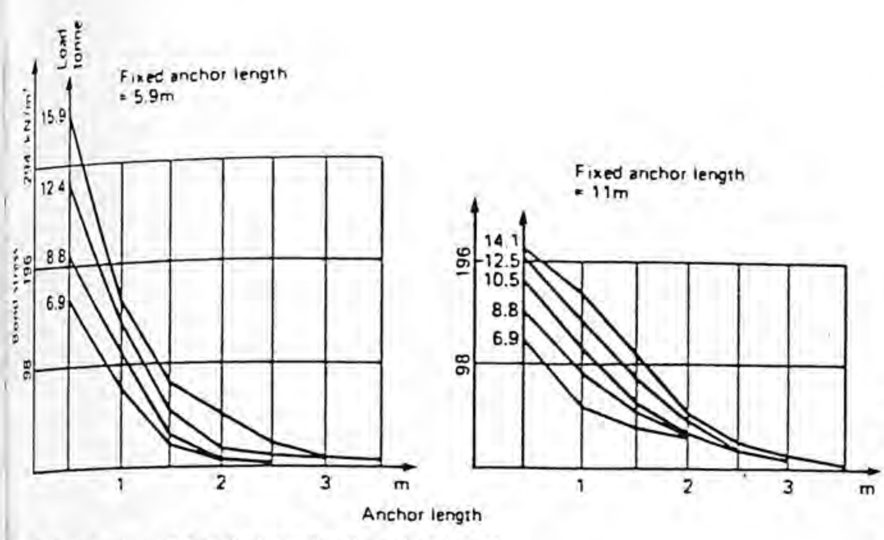


Fig. 5. Distribution of bond along fixed anchor length

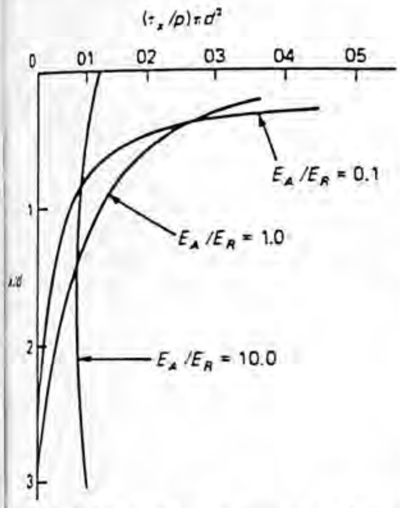


Fig. 6. Variation of shear stress with depth along the rock/grout interface of an anchor ($E_A \equiv E_{grout}$)

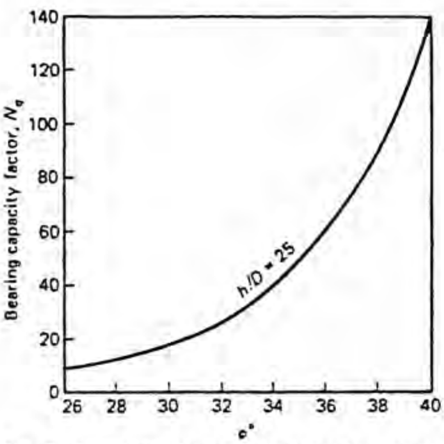


Fig. 7. Relationship between bearing capacity factor N_q and angle of internal friction φ

certain conditions it is recognised that much shorter lengths would suffice, even after the application of a generous factor of safety. However, for a very short anchor the effect of any sudden drop in rock quality along the fixed anchor zone, and/or constructional errors or inefficiencies could induce a serious decrease in that anchor's capacity. As a result a minimum length of 3m is often specified.

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

Fig. 5 shows typical diagrams[11] which illustrate the uneven bond distribution as calculated from strain gauge data. Both anchors were installed in 120mm diameter boreholes in marly limestone ($E = 3 \times 10^6$ kN/m²; UCS = 100N/mm² approx.). Other results show that the bond distributions are more uniform for high values of E_{grout}/E_{rock}, non-uniform for low values of this ratio, i.e. for rock of high elastic modulus. These findings have also been predicted by Canadian researchers[12] (Fig. 6).

Remarks

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

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

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

Cohesionless soils

It was in Germany in 1958 that Bauer[13] for the first time demonstrated that a bar could be anchored into gravels through a 150mm diameter borehole with the aid of cement grout injection under pressure. Since then the development of grouted anchors in frictional soils has steadily gained momentum, particularly in Europe, the Americas and South Africa.

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

$$T_f = L\, n \tan \phi \qquad \ldots (4)$$

where L = fixed anchor length (m)
φ = angle for internal friction
n = factor which apparently takes account of the drilling technique (rotary-percussive with water flush), depth of overburden and fixed anchor diameter, grouting pressure in the range 30 - 1 000kN/m², insitu stress field and dilation characteristics.

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

Eqn. *4* is simple but crude and is used mainly by specialist contractors familiar with their own particular anchorage system. The rule tends to be conservative in view of the limited use of information concerning anchor dimensions and ground parameters, and the underestimate can be significant if the rule is applied to dense "over-consolidated" alluvium where the n values were initially established in "normally consolidated" materials. In this regard the over-consolidation ratio (OCR) should be quantified in ground investigation reports, to permit more field studies into the effect of OCR and relative density on pull-out capacity. For more general use eqn. 5 is recommended since it relates anchor pull-out capacity to anchor dimensions and soil properties[10].

$$T_f = A\,\sigma'_v\,\pi D L \tan\phi + B\,\gamma\,h\,\frac{\pi}{4}(D^2 - d^2)$$

(side-shear) + (end-bearing) ... (5)

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

In practice the fixed anchor diameter (D) is rarely assessable with any accuracy, but approximate estimates can be made from grout takes in conjunction with ground porosity. For boreholes of 100 to 150mm, D values of 400 - 500mm can be attained in coarse sands and gravels, say 3 - 4d. Where grout permeation is not possible and only local compaction is achieved, D values for the above borehole diameters and an applied pressure up to 1 000kN/m², may range from 200 - 250mm for medium dense sand[10], say 1.5 - 2d. For very dense sand D values of 180 - 200mm have been attained[14], say 1.2 - 1.5d.

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

The value of A depends to a large extent on construction technique and for the Type B anchor relevant to eqn. 4, values of 1.7 and 1.4 have been recorded in compact sandy gravel (ϕ = 40°) and compact dune sand (ϕ = 35°) respectively[10].

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

$$T_f = K_1 \pi DL \sigma'_v \tan \phi \qquad \ldots (6)$$

where K_1, coefficient of earth pressure, varies from 1.4 to 2.3 with no grout injection pressure.

For fine sands and silts recommended values for K_1 are 1.0 and 0.5 for high and low relative densities, respectively[18], although it is recognised that K_1 is probably dependent on injection pressure[19]. For dense sands in Boston, Massachusetts[20], K_1 = 1.4 has been obtained for the Bauer anchor. Bearing in mind the difficulty in assessing the effective fixed anchor diameter (D), eqn. 7 using the shaft or borehole diameter (d) has been suggested for design in Sweden[21].

$$T_f = K_2 \pi dL \sigma'_v \tan \phi \qquad \ldots (7)$$

Based on tests in coarse silt and fine sand at Sundsvall, and sand and gravel at Uppsala, K_2 ranges from 4 to 9 with an average value of 6 for injection pressures of 300-600kN/m².

In 1970 it was estimated[10] for eqn. 5 that A lay in the range of 1-2, but that if the soil was not compacted or displaced during the casing installation and no residual grout pressure was left at the fixed anchor grout/soil interface on completion of the injection stage, A might reduce to a value approximating to K_o. In the light

TABLE III. APPROXIMATE RELATIONSHIP BETWEEN N_q AND SLENDERNESS RATIO

h/D	ϕ 26°	30°	34°	37°	40°
15	11	20	43	75	143
20	9	19	41	74	140
25	8	18	40	73	139

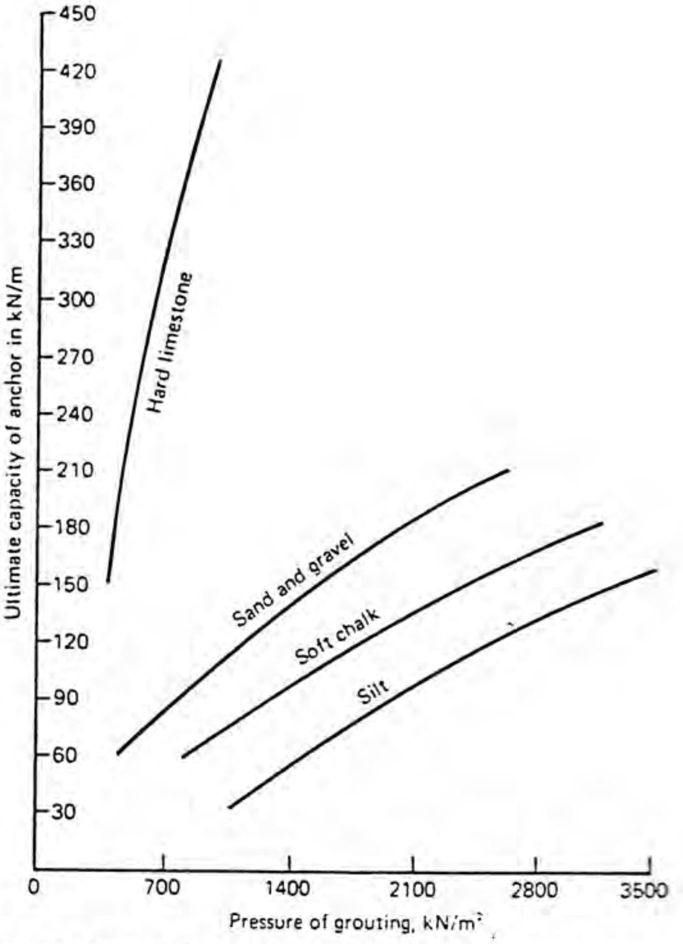


Fig. 8. Influence of grouting pressure on ultimate load-holding capacity

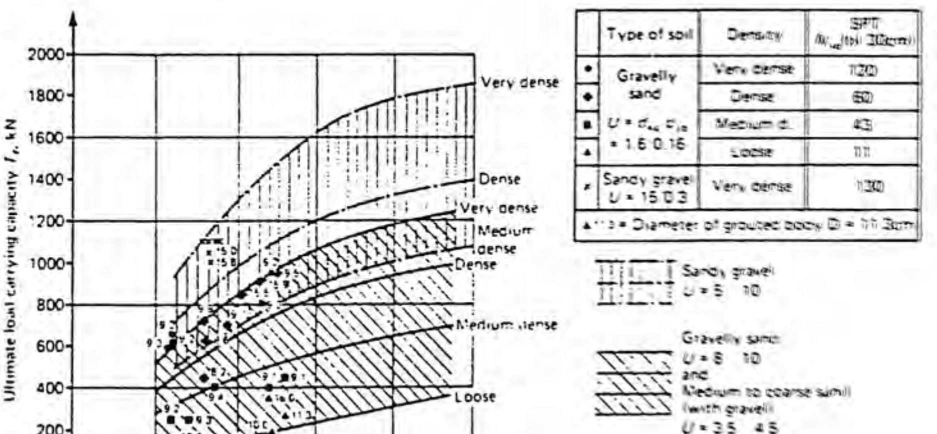


Fig. 9. Ultimate load-holding capacity of anchors in sandy gravel and gravelly sand showing influence of soil type, density and fixed anchor length

of experience this reduction is now considered unduly pessimistic since even with tremie grouting the full hydrostatic head of the grout is applied at the fixed anchor interface, which creates a contact pressure greater than $K_o \sigma'_v$ in normally consolidated ground.

As a consequence, even for the tremie grouting method it is difficult to envisage a value of A less than 1 for design purposes. In fine grained materials A depends greatly on the residual grout pressure at the fixed anchor/soil interface which is some function of the injection pressure since during injection the cement forms a filter cake at the interface through which only water travels. Thus, the injection pressure is transmitted to the soil, and when the grouting is complete there is sufficient shear strength in the grout placed coupled with ground restraint to enable a residual pressure to be locked into the system. In such circumstances eqn. 8 has been used by some contractors, particularly in Continental Europe.

$$T_f = P_i \pi DL \tan \phi \qquad \ldots (8)$$

where P_i = grout injection pressure.

This rule has been tested recently for injection pressures of 1 000-2 000kN/m² in dense fine uniform sand (ϕ = 40°) at Küçük Cekmece Lake in Turkey[22]. In such soil the rule is shown to overestimate

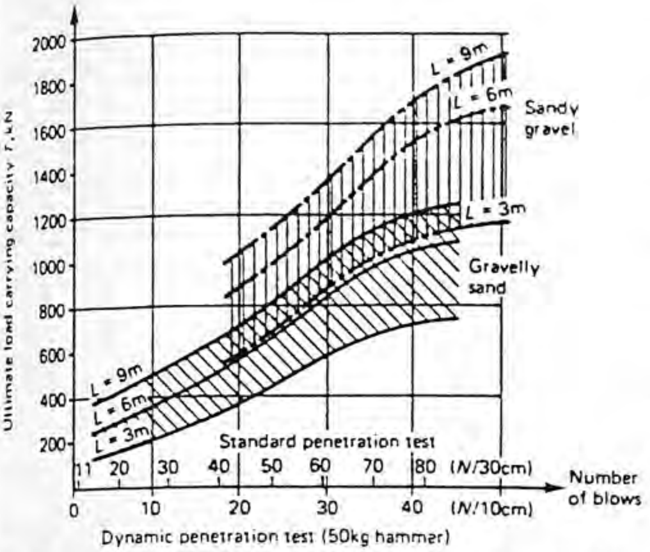


Fig. 10. Relationship between ultimate load-holding capacity, fixed anchor length and dynamic penetration resistance for two types of frictional soil

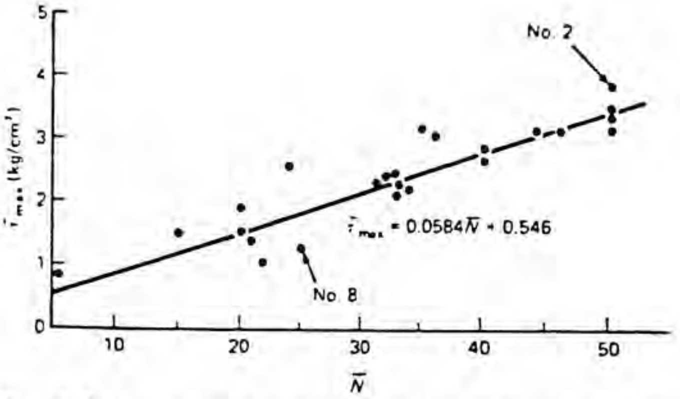


Fig. 11. Relationship between maximum skin friction and mean N value ($\bar{N}$)

pull-out capacity and a modified version (eqn. 9) is recommended.

$$T_f = 2/3\ P_i\ \pi DL \tan \varphi \qquad \ldots (9)$$

The overestimate of eqn. *8* has been further highlighted for the very dense shelly sand at Orford Ness, England[14] and injection pressures of 1 000 to 1 400 kN/m², where the residual pressure approximated to 1/3 P_i. It is considered that as the in situ permeability of the soil increases, filter cake formation becomes more difficult and hence more of the injection pressure is dissipated during the plastic stiffening stage, as the grout slowly permeates through the soil. In this regard the stiffening time and shrinkage of the grout, together with the load/deformation properties of the soil may also be influential. In spite of this apparent restriction design curves, based on the work of Jorge[23], have been published relating grouting pressure directly to ultimate load capacity per metre of fixed anchor for major classes of ground[24] (Fig. 8). These curves are used primarily for Type *C* anchors where the injection pressures usually exceed 1 000kN/m².

It is a feature of Type *C* anchors that calculations are based on design curves created from field experience in a range of soils rather than relying on a theoretical or empirical equation using the mechanical properties of a particular soil. In alluvium for example, test results[23] in medium sand in Brussels, alluvium at Marcoule, sands and gravels at St-Jean-de-Luz, and Seine alluvium at Bercy have indicated for 100-150mm diameter boreholes ultimate load-holding capacities of 90-130kN/m of fixed anchor at P_i of 1 000kN/m², and 190-240kN/m at P_i of 2 500kN/m².

In more recent years design curves for Type C anchors have been extended through basic tests in Germany[25, 26], and for sandy gravels and gravelly sands Fig. 9 shows[26] how the ultimate load increases with density and coefficient of uniformity. Compared with these two soil properties, increases in grouting pressure over the range 500-5 000kN/m², and fixed anchor diameter (100-150mm) are found to have little influence on pull-out capacity which contrasts with the French observations[23]. In this regard the particular use of the tube-à-manchette system in the French tests to provide a secondary stage of grouting at high pressure may explain the different emphasis on injection pressure.

For the German design curves average skin frictions can be as high as 500kN/m² for sand, and 1 000kN/m² for sandy gravel. Since these skin frictions are much higher than would normally be predicted by conventional soil mechanics theory, the values attained in ground anchors are explained by an interlocking or wedging effect due to dilation of the soil as the fixed anchor is withdrawn. The effect is an increase in radial or normal stress at the ground/grout interface, and values of 2-10 times the effective overburden pressure have been noted. For very dense fine to coarse gravelly sand at National Capital Bank in Washington DC[27], (P_i = 2 800-3 100kN/m²), radial stresses of approximately 20 × the overburden pressure have been deduced.

In practice density is commonly measured indirectly by in situ penetrometer tests, and Fig. 10 illustrates how penetration resistance can be used to provide a rough estimation of ultimate load holding capacity for 3m 6m and 9m fixed anchor lengths[26]. The authors emphasise, however, that certain fluctuations in test results are possible due to the soil inhomogeneity even when anchors have been properly installed. Japanese investigators[28] have also provided a relationship between maximum skin friction and mean N value (Fig. 11).

The most sophisticated attempt to calculate accurately load-holding capacity is provided by an evaluation of test anchors in Hannover, West Germany[29] using statistical methods, specifically a linear multiple regression analysis. For frictional soils eqn. *10* is recommended:

$$T_f = a_0 + a_1\,\tau DL + a_2 . D_5 + a_3 . D_6 + a_4 . D_7 + a_5 . D_8 + a_6 . k + a_7 \tau \qquad \ldots (10)$$

where $\tau = \dfrac{2 - \sin\varphi'}{2}\,\gamma\, h_m \,.\, \tan\varphi'$ (kN/m²)

- a_j = regression constants,
- D = effective fixed anchor diameter (cm),
- L = fixed anchor length (m),
- D_5 = % soil grains with diameters < 0.2mm,
- D_6 = % soil grains 0.2mm < dia. < 0.6mm,
- D_7 = % soil grains 0.6mm < dia. < 2.0mm,
- D_8 = % soil grains dia. > 2.0mm,
- k = coefficient of permeability (cm/sec),
- γ = unit weight (kN/m³), and
- h_m = depth of overburden to mid-point of fixed anchor (m)

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

a_0 =	− 2679.36	a_4 =	+ 20.63
a_1 =	+ 34.12	a_5 =	+ 31.92
a_2 =	+ 29.20	a_6 =	− 2051.48
a_3 =	+ 30.94	a_7 =	+ 9.73

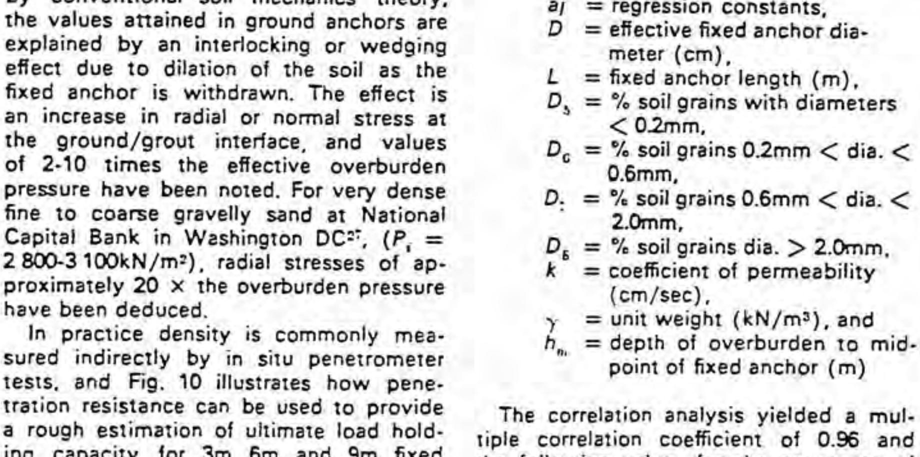


Fig. 12. Boundaries of the grain size distribution of the investigated frictional soil

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

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

$0.98\,m^2 \leqslant \pi DL \leqslant 3.61\,m^2$
$7.40\,cm \leqslant D \leqslant 11.50\,cm$
$4.10\,m \leqslant L \leqslant 15.00\,m$
$0\% \leqslant D_s \leqslant 86\%$
$10\% \leqslant D_n \leqslant 78\%$

$0\% \leqslant D_r \leqslant 17\%$
$0\% \leqslant D_b \leqslant 77\%$
$0.122\,10^{-2}\,cm/s \leqslant k \leqslant 25.2\,10^{-2}\,cm/s$
$31.7\,kN/m^2 \leqslant \tau \leqslant 95.6\,kN/m^2$

The importance of these limits and boundaries cannot be overemphasised as field experience[30] indicates that use of one parameter outside the stipulated range e.g. *k* which may then be incompatible with the grain size, can produce anomalous results.

Distribution of skin friction

Designs are normally based on the assumption of an equivalent uniform skin friction; actual field values[20, 26] are rare and even then are estimated from bond stresses at the grout/tendon interface. For the last loading step before failure is reached Fig. 13 shows for instrumented anchors the distribution of skin friction on fixed anchors ranging from 2 to 4.5m in length[26].

The decisive influence of soil density is clearly shown by the maximum τ_s values of 150, 300 and 800kN/m² for loose, medium dense and very dense gravelly sand, respectively. For the 4.5m long anchors in loose and medium dense gravelly sand, skin friction is more or less constant over the ground/grout interface. For dense and very dense sands the maximum values are effective along a relatively short length, and the location of this peak zone shifts distally as the test load increases. These observations for Type *C* anchors have been confirmed in similar very dense frictional soils in Washington DC[27], where it was also noted that fixed anchor displacements of only 2-3mm were required to mobilise high values of load transfer (150-370kN/m).

Assuming that the limit value or maximum τ_s is identical for different fixed anchor lengths, the mean values of τ_s for long anchors are smaller than for short anchors, a feature which is apparent in Fig. 9. Taken to the extreme there exists a critical limit to the effective fixed anchor length beyond which there is no evident increase in load-holding capacity. Fig. 14 for dense frictional soil ($N=50$) indicates[24] very small load increases for *L* greater than 6.7m, which supports Ostermayer[25] who concluded that 6-7m was optional from an economic point of view.

Remarks

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

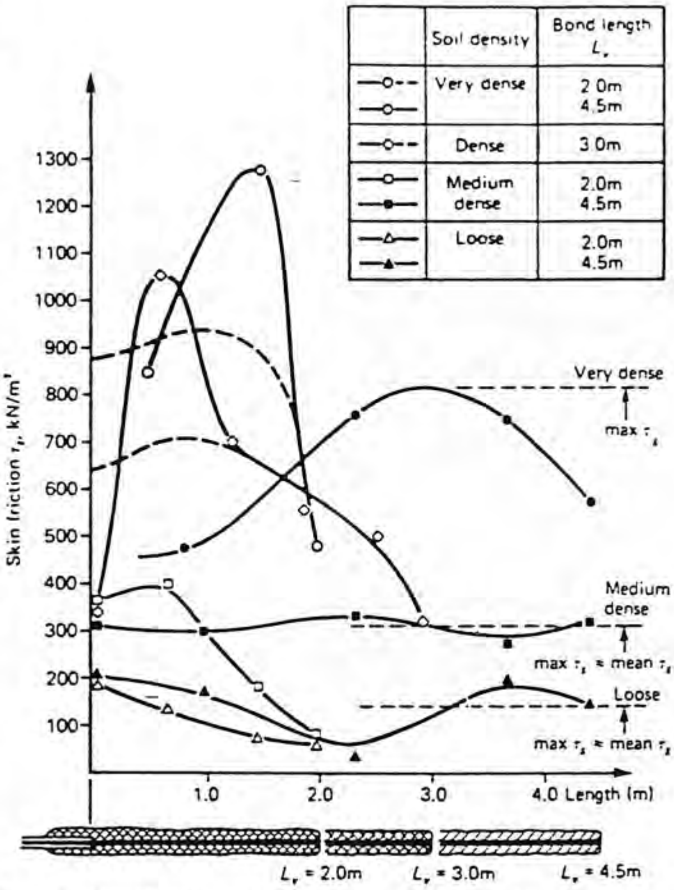


Fig. 13. Distribution of long-term friction τ_s at ultimate load in relation to tendon bond length and soil density (D = 91 – 126mm in gravelly sand)

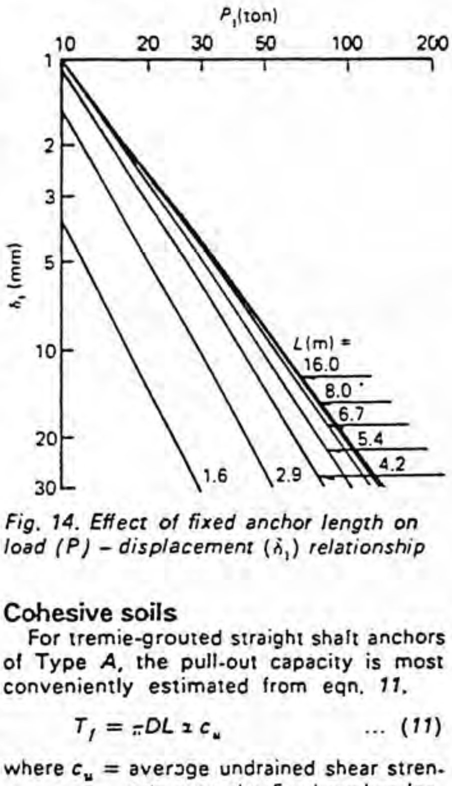


Fig. 14. Effect of fixed anchor length on load (P) – displacement (δ_1) relationship

Cohesive soils

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

$$T_f = \pi DL\,\alpha\,c_u \qquad \dots (11)$$

where c_u = average undrained shear strength over the fixed anchor length, and
α = adhesion factor.

In stiff London Clay ($c_u > 90\,kN/m^2$) α values of 0.3-0.35 are common[31], bearing in mind the dilute cement grout (w/c ≤0.40) usually employed. Type *A* anchors installed in stiff overconsolidated clay ($c_u = 270\,kN/m^2$) at Taranta, Southern Italy[32], have indicated similar values of α = 0.28-0.36. For stiff to very stiff

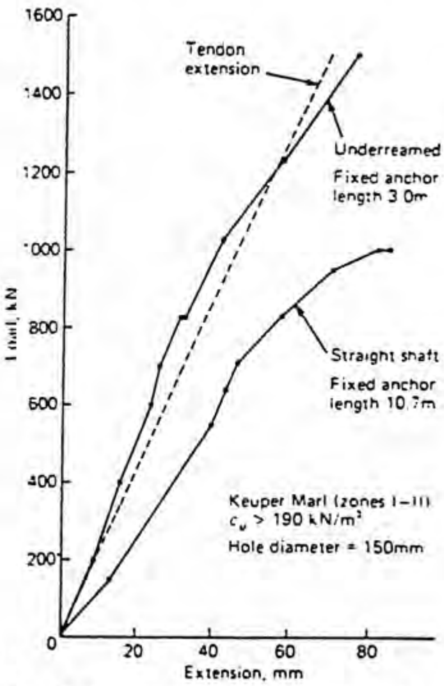


Fig. 15. Comparison of load-extension responses of an under-reamed anchor and a straight shaft anchor

marls ($c_u = 287\,kN/m^2$), at Leicester in England, values of 0.48-0.60 have been monitored, although $\alpha = 0.45$ is suggested for design[10]. A value of $\alpha = 0.45$ has also been confirmed for stiff clayey silt ($c_u = 95\,kN/m^2$) in Johannesburg[33]. Anchorages of Type *A* are generally of low capacity, and various construction methods have been attempted[10, 25], including the use of explosives in London Clay at Herne

Bay[31] as early as 1955, in order to increase resistance to withdrawal. The most successful method to date in terms of ultimate load-holding capacity is the multi under-reamed Type *D* anchor which was developed from the field of piling.

Under-reaming of pile bases was pioneered in locations such as Texas, USA, the Orange Free State in South Africa[34] and India[35, 36] where severe foundation problems in expansive soils were experienced. Of particular note is the development of single, double and multi under-ream piles which has taken place at the Central Building Research Institute at Roorkee dating from 1955. In design terms the result of this work[36] includes (*i*) development of equations for estimating ultimate bearing capacity, (*ii*) confirmation that under-reamed piles act similarly in tension or compression, and (*iii*) optimisation of the under-reamed spacing/diameter ratio at 1.25-1.50.

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

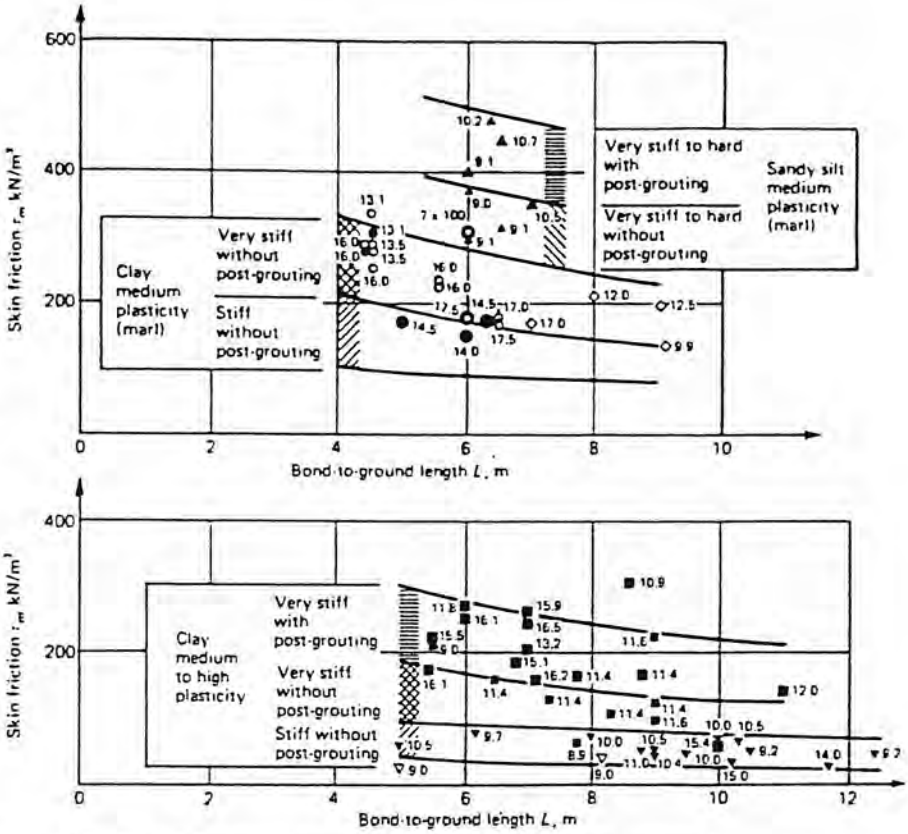
$$T_f = \pi D L c_u + \frac{\pi}{4}(D^2 - d^2) N_c c_u + \pi d l c_a \quad \ldots (12)$$

The rule was proved initially in London Clay at Lambeth (c_u = 134–168kN/m²), for the following dimensions, using a brush under-reamer

Diameter of under-ream	(D)	=	350 – 400mm (2.5 – 3d)
Diameter of shaft	(d)	=	130 – 150mm
Fixed anchor length	(L)	=	3.1 – 7.6m
Shaft length	(l)	=	1.5 – 3m
Shaft adhesion	(c_a)	=	(0.3–0.35) c_u based on shaft tests
Bearing capacity factor	(N_c)	=	9 (assumed from bored pile experience in London Clay)

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

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



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

Fig. 16. Skin friction in cohesive soils for various fixed anchor lengths, with and without post-grouting

With regard to spacing of under-reams (δl), eqn. *14* can be used to estimate the maximum allowable spacing to give failure along a cylindrical surface[10].

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

i.e.

$$\delta l < \frac{(D^2 - d^2)}{4D} N_c \quad \ldots (14)$$

For example, if D = 400mm, d = 150mm and N_c = 9, then δl = 0.77m. Quite independently, a similar design approach was developed in London Clay at Orford Ness[14] (c_u = 54 – 72kN/m²) where Type *D* anchors were constructed using a mechanically expanded double flight under-reamer (see eqn. *15*):

$$T_f = \pi D L f_u c_u + \frac{\pi}{4}(D^2 - d^2)(N_c c_{u\,end} + \sigma'_L) + \pi d l f_s c_{u\,shaft} \quad \ldots (15)$$

where f_u = 0.75 – 0.95
f_s = 0.3 – 0.6
N_c = 6.5 (range 6 – 13 or greater)
σ'_L = effective stress normal to proximal end.

The anchor dimensions on site were D (460 – 530mm), d (140mm) L (3m) and l (7.6m). In regard to under-ream spacing it is stipulated that $\delta l \ngtr$ (1.5 – 2) D and $d \ngtr$ (0.6 - 0.7) D in order to ensure cylindrical shear failure. For stiff to very stiff fissured silty clay (c_u = 130 – 290kN/m²) at Neasden Underpass, London, with a mean value of 175kN/m² assumed for design, test results[40] for a multi-flight mechanical under-reamer (D = 540mm, d = 175mm) have indicated an efficiency factor f_u = 0.75.

The success of multi under-reamed anchors over straight shafts can perhaps be illustrated best[41] by reference to Fig. 15. Based on the same augered hole diameter of 150mm, the straight shaft Type *A* anchor with a fixed anchor length of 10.7m failed at 1 000kN, whereas the under-reamed anchor of only 3m withstood, without any sign of failure, a load of

1 500kN. The advantages have also been quantified for London Clay[42] where measurements of brushed under-reams by borehole caliper indicate D (363mm) and d (140mm) i.e. an improvement of 2.59 and test anchor back-analysis gives an adhesion factor $\alpha = 0.78$ c.f. the straight shaft α of 0.35 i.e. an improvement of 2.23. Consequently, an overall improvement of more than five times is confirmed by both examples.

As a result of tests of this type and accumulated field experience of commercial anchors, safe working loads of 500 – 1 000kN can be obtained in stiff to hard clays using the multi under-reamed anchor Type D, compared with 300 – 400kN using straight Type A anchors. These figures are based on load safety factors of 2.5 – 3.5, which are considered necessary to minimise prestress losses due to consolidation of the clay.

In general, there is still a serious shortage of field performance data for anchors in cohesive soils, and little information is available on soil strength below which under-reaming is impracticable. In the writer's experience, under-reaming is ideally suited to clays of c_u greater than 90kN/m², but some difficulties in the form of local collapse, or breakdown of the neck portion between the under-reams should be expected where c_u values of 60 – 70kN/m² are recorded. Under-reaming is virtually impracticable below c_u of 50 kN/m².

In such circumstances, use of the high pressure Type C anchor, with and without post-grouting, is worthy of study. The results of a large number of fundamental tests[23] are shown in Fig. 16 which can be used as a design guide for borehole diameters of 80 – 160mm. Skin friction increases with increasing consistency and decreasing plasticity. In stiff clays (I_c = 0.8 – 1.0) with medium to high plasticity, skin frictions of 30-80 kN/m² are the lowest recorded, whilst the highest values ($\tau_M > 400$kN/m²) are obtained in sandy silts of medium plasticity and very stiff to hard consistency ($I_c = 1.25$). The technique of post-grouting is also shown to generally increase the skin friction of very stiff clays by some 25-50%, although greater improvements (from 120 up to about 300kN/m²) are claimed for stiff clay of medium to high plasticity. From Fig. 17 the influence of post grouting pressure on skin friction is quantified for clays of medium to high plasticity[25], showing a steady increase in τ_M with increase in p_i.

For Type C anchors in cohesive soil, the Hannover analysis[29] provides eqn. 16.

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

where
$$\tau_c = \gamma h_m \tan \phi^1 / \cos^2 \alpha + \sin^2 \alpha\,(1 + 2\tan^2 \phi^1) + 2 \sin \alpha \cos \alpha + c' \cos^2 \phi$$

α = angle of inclination of anchor
D_1 = % soil grain d < 0.006mm
D_2 = % soil grain 0.006mm < d < 0.02mm
D_3 = % soil grains 0.02mm < d < 0.06mm
D_4 = % soil grains d > 0.06mm
I_c = consistency index

$$= \frac{LL - m}{LL - PL}$$

The multiple correlation coefficient for this equation was 0.98 and the following values for the constants have been calculated:

$a_0 = +\ 721.51$	$a_4 = -\ 21.22$
$a_1 = +\ 71.84$	$a_5 = +\ 10.34$
$a_2 = -\ 9.81$	$a_6 = +\ 95.15$
$a_3 = -\ 1.99$	$a_7 = +\ 2.56$

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

$0.98\text{m}^2 \leqslant \pi DL \leqslant 6.48\text{m}^2$	$4\% \leqslant D_3 \leqslant 27\%$
$6.50\text{cm} \leqslant D \leqslant 16.80\text{cm}$	$2\% \leqslant D_4 \leqslant 34\%$
$4.10\text{m} \leqslant L \leqslant 15.00\text{m}$	$0.84 \leqslant I_c \leqslant 1.35$
$20\% \leqslant D_1 \leqslant 76\%$	$50.7\text{kN/m}^2 \leqslant \tau_c \leqslant 165.3\text{kN/m}^2$
$12\% \leqslant D_2 \leqslant 27\%$	

Distribution of shear stress

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

For stiff overconsolidated clay at Taranta[32] (c_u = 270kN/m² average), Fig. 19 illustrates the shear stress distribution at failure, where $E = 6.9 \times 10^4$kN/m² was deduced[44].

Bearing in mind that E values for grout can be in the range (1-2) × 10^7kN/m², and that for rocks a uniform stress distribution is anticipated[22] where E_{grout}/E_{rock} exceeds 10, it is interesting to observe non-uniformity in Fig. 19, where the elastic modular ratio is well in excess of 100.

Remarks

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

Factors of safety

When a grouted anchor fails, it must be by one of the following modes:
(*a*) Failure of the ground mass.
(*b*) Failure of the ground/grout bond.
(*c*) Failure of the grout/tendon bond, or
(*d*) Failure of the tendon or anchor head,
and in order to determine the mechanism of failure and actual safety factor for the anchor, consideration must be given to all of these aspects.

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

"Have you heard of the wonderful one-hoss shay,
That was built in such a logical way
It ran for a hundred years to a day,
And then, of a sudden it . . .
. . . went to pieces all at once, —
All at once, and NOTHING FIRST, —
Just as bubbles do when they burst."
The Deacon's Masterpiece, by
Dr. Oliver Wendell Holmes.

Thus for each potential failure mechanism a safety factor must be chosen having regard to how accurately the relevant characteristics are known, whether the system is temporary or permanent, i.e. service life, and the consequences if failure occurs i.e. danger to public safety and cost of structural damage. Since the minimum safety factor is applied to those anchor components known with the greatest degree of accuracy, the values[45] suggested in Table IV invariably apply to the

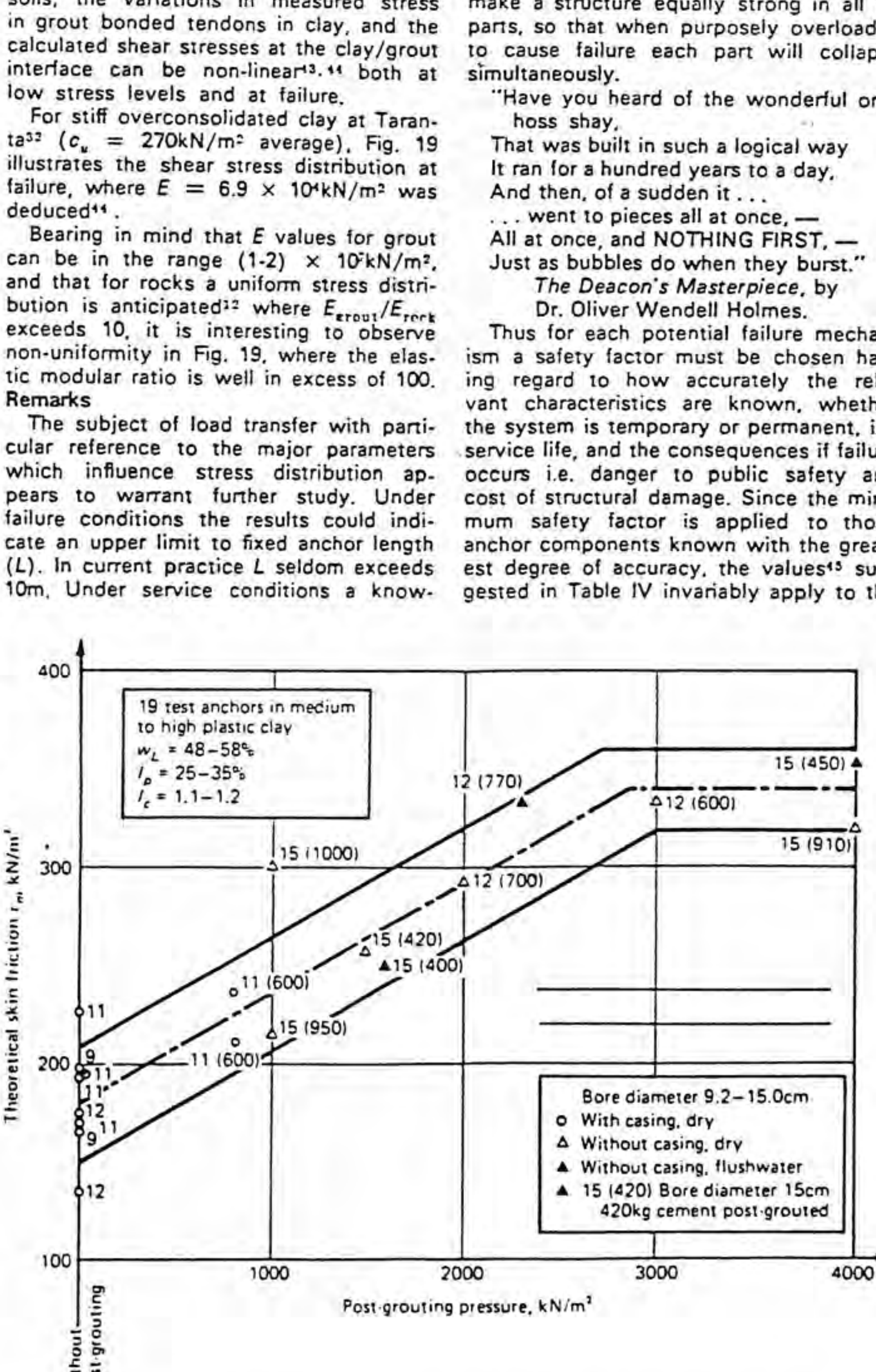


Fig. 17. Influence of post-grouting pressure on skin friction in a cohesive soil

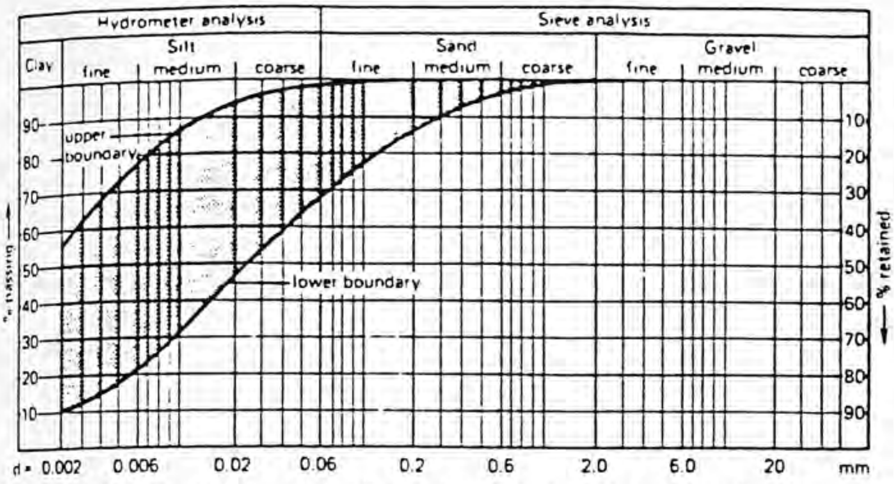


Fig. 18. Boundaries of the grain size distribution of the investigated cohesive soil

characteristic strength of the tendon or anchor head.

In regard to the ground/grout interface of the fixed anchor upon which this Paper has concentrated, overall design load safety factors (S_f) range from 2-4 generally, where S_f is applied to the ultimate load-holding capacity (T_f). T_f may be defined as the constant load at which the fixed anchor can be withdrawn at a steady rate e.g. creep in cohesive soils, or the maximum load attained prior to a distinct failure involving a sudden loss of load e.g. breakdown of bond in rock. As more poor quality ground has been exploited by anchors, so safety factors have increased in value to take account of (*i*) larger fixed anchor displacements for given load increments[46] (Fig. 20), or (*ii*) creep phenomena. In the case of (*ii*) for example, S_f values of 2-2.5 for temporary anchors in clay where the service period is less than 2 years, rise to 3-3.5 for permanent anchors, in order to keep prestress fluctuations within acceptable limits. In other words designers are quietly building in Serviceability Factors.

To avoid the growing situation where engineers simply specify the latest an largest safety factors irrespective of th ground, there is a need for a more thoroug investigation of load-displacement rela tionships for fixed anchors in differen ground conditions, since these relation ships influence choice of safety factor which should be related to permissible movement as well as ultimate load. Fo specific ground conditions it may be pos sible for example to establish a correla tion between a yield load giving unaccep table movement, and the ultimate load holding capacity. Thus, if an enginee wishes to specify a factor of safety (S_y) against a yield condition, it may be feasible then to apply a modifying factor tc S_y to provide an estimate of S_f for the ultimate load-holding capacity (T_f) or vice versa when T_f is estimated from an empirical equation or design envelope. This concept of safety factors may grow ir importance with the advent of Limit State Codes. In an effort to encourage the analysis of test anchor results there is perhaps a case for two levels of safety factor depending on whether actual test results or calculated ultimate loads are used for design purposes.

TABLE IV. SUGGESTED SAFETY FACTORS FOR ANCHOR DESIGN

Anchor category	Minimum safety factor	Proof load factor
Temporary anchors where the service life is less than 6 months and failure would have few serious consequences and would not endanger public safety e.g. short term pile test.	1.4	1.1
Temporary anchors with a service life of up to 2 years, where although the consequences of failure are quite serious, there is no danger to public safety without adequate warning e.g. retaining wall tie backs.	1.6	1.25
All permanent anchors. Temporary anchors in a highly aggressive environment, or where the consequences of failure are serious e.g. temporary anchors for main cables of a suspension bridge or as a reaction for lifting heavy structural members.	2.0	1.5

Conclusions

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

In 1969 at the Mexico Conference, Reporter Habib observed spectacular progress in anchoring in loose soils, but stated that it was rather odd to realise that the theories were still empirical in nature. Since empiricism in design is still prevalent today it might be argued that little progress has been made. In the author's view some sympathy must be expressed for the attitude that resists the creation and application of the more sophisticated theories, since they invariably demand accurate values of a multitude of ground and anchor parameters in order to attain the improved accuracy. In this regard, a good example is short, low capacity rock bolts where the cost of investigating the detailed variation in a heterogeneous rock mass far outweighs the cost of installation and proof-loading additional bolts, in the event of unsatisfactory performance.

In reality a period of technical consoli-

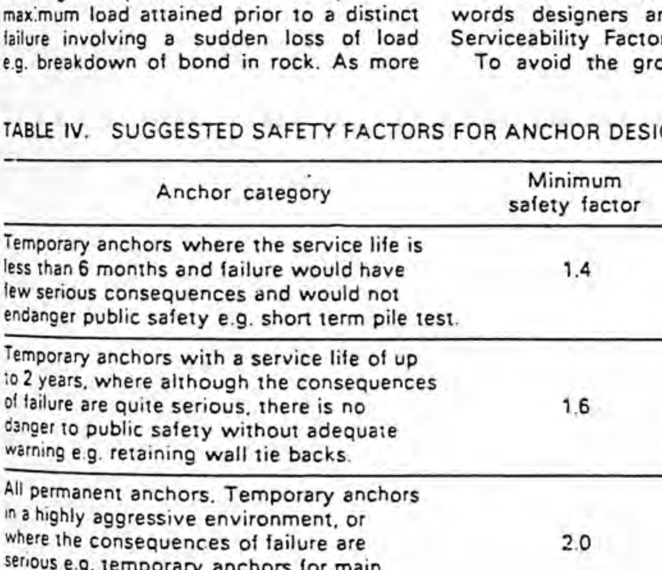


Fig. 19. Calculated shear stresses at failure

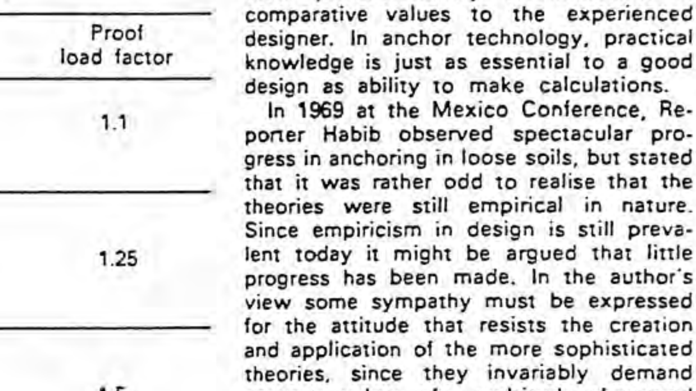


Fig. 20. Load/displacement relationship for compact fine/medium sand (ϕ = 35°)

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

References

1. *Mitchell, J. M.* (1974): "Ground anchors". DIC Dissertation — Dept. of Civil Engg., Imperial College of Science & Technology, Jan.
2. *Thomas, E.* (1962): "Stabilisation of rock by bolting". Reviews in Engineering Geology 1 & 11, New York.
3. *Host'ák, P.* (1960): "Závesná vystroj suorniková a lanová". Praha, UVR.
4. *Drouhin, M.* (1935): "Consolidation du barrage de Cheurfas par tirants métalliques mis en tension". Annales des Ponts et Chaussées (Août).
5. *Littlejohn, G. S.* & *Bruce, D. A.* (1977): "Rock Anchors: State-of-the-Art" Foundation Publications Ltd., Brentwood, Essex, England.
6. *Coates, D. F.* (1970): "Rock mechanics principles". Department of Energy, Mines and Resources Mines Monograph No. 874 — Ottawa.
7. Standards Association of Australia (1973): "Prestressed Concrete Code CA35". Section 5 — Ground Anchorages pp. 50-53.
8. *Klein, K.* (1974): "Draft standard for prestressed rock anchors". Symposium on Rock Anchoring of Hydraulic Structures, Vir Dam pp. 86-102.
9. *Suzuki, I., Hirakawa, T., Morii, K.,* & *Kanenko, K.* (1972): "Developpments Nouveaux Dans les Foundations de Pylons pour Lignes de Transport THT de Japon". Conf. Int. des Grande Réseaux Electriques à Haute Tension, Paper 21-01, 13 pp.
10. *Littlejohn, G. S.* (1970): "Soil Anchors". ICE Conference on Ground Engineering, London pp. 33-44, and discussion pp. 115-120.
11. *Berardi, G.* (1967): "Sul Comportamento Degli Ancoraggi Immersi in Terreni Diversi". Univ. of Genoa, Inst. Const. Sc. Series 111, No. 60, 18 pp.
12. *Coates, D. F.* & *Yu, Y. S.* (1970): "Three-dimensional stress distributions around a cylindrical hole and anchor". Proc. 2nd Int. Conf. on Rock Mechanics, Belgrade, **2**, 175-182.
13. *Bauer, K.* (1960): "Injektionszuganker in nicht bindigen Boden". Bau und Bauindustrie **16**, 520-522.
14. *Bassett, R. H.* (1970): Discussion to Paper on soil anchors, ICE Conference on Ground Engineering, London, pp. 89-94.
15. *Berezantzev, V. G., Khristoforov, V. S.* & *Golubkov, V. N.* (1961): "Load-bearing capacity and deformation of piled foundations". Proc. 5th Int. Conf. Soil Mech. & Found. Engng. **2**, 11-15, Paris.
16. *Trofimenkov, J. G.* & *Mariupolskii, L. G.* (1965): "Screw piles used for mast and tower foundations". Proc. 6th Int. Conf. Soil Mech. & Found. Engng. **2**, 328-332, Montreal.
17. *Robinson, K. E.* (1969): "Grouted rod and multi-helix anchors". Proc. 7th Int. Conf. on Soil Mech. & Found. Engng. Specialty Session No. 15, pp. 126-130, Mexico.
18. *Lundahl, B.* & *Adding L.* (1966): "Dragförankringer i flytbenägen mo under grundvattenytan". Byggmästaren, **44**, 145-152.
19. *Broms, B. B.* (1968): "Swedish tie-back systems for sheet pile walls". Proc. 3rd Budapest Conference on Soil Mech. and Found. Engng. 391-403.
20. *Oosterbaan, M. D.* & *Gifford, D. G.* (1972): "A case study of the Bauer Earth Anchor" Proceedings of the Specialty Conference on Performance of Earth and Earth Supported Structures. Purdue University, Lafayette, Indiana. ASCE **1**, Pt. 2 pp. 1391-1401.
21. *Moller, P.* & *Widing, S.* (1969): "Anchoring in soil, employing the Alvik, Lindo and J. B. drilling methods". 7th Int. Conf. for Soil Mech. & Found. Engng. Specialty Session No. 15, pp. 184-190, Mexico.
22. *Togrol, E.* & *Saglamer, A.* (1978): "Short-term capacity of ground anchors". Bulletin of the Technical University of Istanbul, **31**, No. (1 (13 p).
23. *Jorge, G. R.* (1969): "The re-groutable IRP anchorage for soft soils, low capacity or karstic rocks". Proc. 7th Int. Conf. on Soil Mech. & Found. Engng. Specialty Session No. 15, pp. 159-163, Mexico.
24. *Anon* (1970): "Ground Anchors". The Consulting Engineer (Special Supplement) May, p. 15.
25. *Ostermayer, H.* (1974): "Construction, carrying behaviour and creep characteristics of ground anchors". ICE Conference on Diaphraom Walls and Anchorages, London, pp 141-151.
26. *Ostermayer, H.* & *Scheele, F.* (1978): "Research on ground anchors in non-cohesive soils". Revue Française de Géotechnique No. 3, pp. 92-97.
27. *Shields, D. R., Schnabel, H.* & *Weatherby, D. E.* (1978): "Load transfer in pressure injected anchors". Proc. ASCE **104** (GT9), 1183-1196.
28. *Fujita, K. Ueda, K.* & *Kusabuka, M.* (1978): "A method to predict the load displacement relationship of ground anchors". Revue Française de Géotechnique No. 3, pp. 56-62.
29. *Kramer, H.* (1978): "Determination of the carrying capacity of ground anchors with the correlation and regression analysis" Revue Française de Géotechnique No. 3, pp. 76-81.
30. *Locher, H. G.* (1979): Private discussion, Losinger Ltd, Berne.
31. *Littlejohn, G. S.* (1968): "Recent developments in ground anchor construction". Ground Engineering **1** (3), 32-36 and 46.
32. *Sapio, G.* (1975): "Comportamento di tiranti de ancoraggio in formazioni de argille preconsolidate". Atti XII Convegno Nazionale de Geotecnica, Cosenza.
33. *Neely, W. J.* & *Montague-Jones, M.* (1974): "Pull-out capacity of straight-shafted and under-reamed ground anchors". Die Siviele Ingenieur in Suid-Africa Jaargang 16, NR 4, 131-134.
34. *Jennings, J. E.* & *Henkel, D. J.* (1949): "The use of under-reamed pile foundations in expansive soils in South Africa". CSIR Research Report No. 32, pp. 9-15, Pretoria.
35. *Mohan, D.* & *Jain, G. S.* (1958): "Pile loading and pull-out tests on Black Cotton Soil". The Journal of the Institution of Engineers (Jan) **38**, 409-421.
36. *Mohan, D., Murthy, V. N. S.* & *Jain, G. S.* (1969): "Design and construction of multi-under-reamed piles". Proc, 7th Int. Conf. Soil Mech. & Found Engng. pp. 183-186, Mexico.
37. *Jones, N. C.* & *Kerkhoff, G. O.* (1961): "Belled caissons anchor walls as Michigan remodels an expressway". Engineering News Record, May 11 issue, pp. 28-31.
38. *White, R. E.* (1970): "Anchorage practice in the United States". The Consulting Engineer. Ground Anchors Special Supplement (May) pp. 32-37.
39. *Parry-Davies, R.* (1966): Private correspondence Ref. 1523/66 — 6th September. Cementation (Africa Contracts) (Pty) Ltd., 50 Booysens Road, Selby, Johannesburg.
40. *Bastable, A. D.* (1974): "Multibell ground anchors in London Clay". 7th FIP Congress. Tech. Session on Prestressed Concrete Foundations and Ground Anchors. 33-37, New York.
41. *Wroth, C. P.* (1975): Report on discussion on Papers 18-21, page 166. ICE Conf. on Diaphragm Walls and Anchorages, London.
42. *Truman Davies, C.* (1977): Report on discussion to Session IV. A review of diaphragm walls, ICE London.
43. *Adams, J. I.* & *Klym, T. W.* (1972): "A study of anchorages for transmission tower foundations". Canadian Geotechnical Journal, **9**, No. 89.
44. *Evangelista, A.* & *Sapio, G.* (1978): "Behaviour of ground anchors in stiff clays". Revue Française de Géotechnique No. 3, pp. 39-46.
45. *Littlejohn, G. S.* (1979): "Ground Anchors: State-of-the-art". Symposium on Prestressed Ground Anchors, The Concrete Society of South Africa. Prestressed Concrete Division, Johannesburg.
46. *Littlejohn, G. S., Jack, B.* & *Sliwinski, Z.* (1971-72): "Anchored diaphragm walls in sand" Ground Engineering 1971 **4** Sept., 14-17 Nov., 18-21, 1972, **5**, Jan. 12-17.

Printed in England by Minerva Press, 9 Tallon Road, Hutton, Brentwood, Essex CM13 1TF

RODGER, A. A. & LITTLEJOHN, G. S. (1980). *Géotechnique* 30, No. 3, 269-293

A study of vibratory driving in granular soils

A. A. RODGER* and G. S. LITTLEJOHN†

Two types of vibratory pile driving have been identified by the Authors termed respectively 'slow' and 'fast' vibrodriving. The occurrence of slow or fast motion is determined by the initial soil density, pile diameter, displacement amplitude and acceleration of vibration. Slow vibrodriving is concluded to be the most widely encountered and the paper, therefore, concentrates on this type of motion. A theory and interactive computer simulation of the case of slow vibratory driving has been developed. The motion is considered to be that of a rigid body subject to viscous–Coulomb side and elasto–plastic end resistance under a combined sinusoidal excitation and static surcharge force. Experimental verification of the theory has been by means of tests on a fully instrumented 0·745 kW (1 hp) model in dry cohesionless soils. The need for further research work to quantify soil resistance and the dependence of this on displacement amplitude and frequency of vibration is stressed. The Authors recommend that this information is obtained from full scale tests using a prototype designed on the basis of the information provided by the present research work.

Deux types de battage de pieux vibratoire ont été identifiés par les auteurs et désignés respectivement par vibrobattage 'lent' et vibrobattage 'rapide'. La 'lenteur' ou la 'rapidité' du mouvement est déterminée par la densité initiale du sol, le diamètre du pieu, l'amplitude du déplacement et l'accélération des vibrations. Selon les conclusions, le type de vibrobattage 'lent' est celui qui se rencontre le plus fréquemment et l'article se concentre donc sur ce type de mouvement. Une théorie et une simulation interactive sur ordinateur du cas du vibrobattage 'lent' ont été mises au point. Le mouvement est envisagé comme étant celui d'un corps rigide soumis à une résistance visqueuse, type Coulomb et extrémité élastoplastique, sous l'action simultanée d'une surcharge statique et d'une excitation sinusoïdale. La théorie a été vérifiée experimentalement par des essais sur un modèle 0·745 kW (1 HP) à instrumentation complète dans des sols secs sans cohésion. L'article insiste sur la nécessité d'effectuer des recherches supplémentaires pour quantifier la résistance du sol et sa dépendance avec l'amplitude du déplacement et la frequence de vibrations. Pour obtenir ces renseignements, les auteurs recommandent d'effectuer des essais a l'échelle à l'aide d'un prototype réalisé sur la base des renseignements obtenus à partir des travaux de recherches actuels.

INTRODUCTION

Vibratory driving is a technique used for driving piles, tubes and rods rapidly into the ground by imparting to the driving unit a small longitudinal vibratory motion of a predetermined frequency and displacement amplitude. The vibrations serve to reduce the ground resistance, allowing penetration under the action of a relatively small surcharge force.

Although under research since 1930 the mechanism which causes reduced ground resistance and penetration has not as yet been identified. This has led to conservative piling design and overdesign of the vibrator unit. This Paper describes the results of a research project undertaken at Aberdeen University to investigate this mechanism.

From this research two types of penetrative motion are identified defined respectively as 'slow' and 'fast' vibratory driving. The research work concentrated on the mechanism of slow vibratory driving since this is considered to be the most widely encountered in practice. A theory and interactive computer simulation of slow vibratory driving is developed using

Discussion on this paper closes 1 December, 1980. For further details see inside back cover.

* Department of Engineering, University of Aberdeen.
† Colcrete Group of companies, Kent.

numerical methods. The validity of this theory is investigated by means of laboratory tests on a fully instrumented model vibratory driver and some monitoring of a 7·45 kW (10 hp) prototype.

NOTATION

A	peak displacement amplitude
B, C, D, E	constants
F_D	dynamic force
F_I	instantaneous dynamic force
F_S	surcharge force
M	mass of vibrator and pile
P_P	power required to sustain penetrative motion
P_V	power required to sustain vibrational motion
R	plastic soil resistance
R_D	dynamic soil resistance
R_E	point resistance to penetrative motion
R_M	minimum dynamic side resistance
R_S	dynamic side resistance
R_{ST}	static side resistance
V	penetration velocity
d	pile diameter
e	base of natural logarithms 2·7183
f	frequency of vibration
f_D	F_D/M
$\bar{f}_D$	f_D/r
f_S	F_S/M
$\bar{f}_S$	f_S/r
g	acceleration due to gravity
k	elastic soil resistance
r	R/M
t	time
x	vibrational displacement
$\bar{x}$	$\omega_n{}^2 x/r$
x_{max}	maximum value of the displacement
y	penetrative displacement
α	constant of dynamic shear strength reduction
η	ratio of amplitude of vibrational acceleration to that of gravity
θ	phase difference between force and displacement
ν	damping constant
τ	$\omega_n t$
ϕ	angle of internal friction of soil
ϕ_d	dynamic angle of internal friction of soil
ϕ_m	minimum dynamic angle of internal friction of soil
ϕ_{st}	static angle of internal friction of soil
ω	angular velocity
ω_n	natural circular frequency
$\bar{\omega}$	ω/ω_n

HISTORICAL DEVELOPMENT

Research into the vibratory driving of piles began in 1930 in Germany and the first commercial application was carried out by Hertwig in 1932. In co-operation with Hertwig the firm Losenhausenwerk registered the German Reich's patent DRP No. 611392, which outlined the principles of vibratory driving. The concept of vibratory driving was discovered almost at the same time in the USSR as a by-product of soil dynamics research by Pavyluk on footing vibrations begun in 1931 and in 1934 Barkan (USSR) showed that the vertical vibration of a pile markedly decreases the skin friction of soils (Barkan, 1949, 1967). This early work was, however, interrupted by the Second World War, and investigations were resumed by the Russians only in 1946 when a study of impact mechanisms was begun by Rusakov and Kharkhevich. Within 2 years work had begun at the Road Scientific Research Institute of the Soviet Union to investigate the use of vibratory hammers to extend the method to cohesive soils. This research discovered that impacts occur between the pile and the soil with the use of low frequency vibratory drivers (Tsaplin, 1953).

Three years later Barkan proposed the use of vibratory drivers for the installation of exploratory boreholes which led to the development of a vibrocorer working at 42 Hz producing a 2 mm displacement amplitude and powered by a 3·5 kW electric motor. In the same year a vibratory driver (BT-5) working between 38 and 45 Hz was used to drive sheet steel piles during the construction of the Gorky Hydroelectric Station. Production rates of 11 m in 2 to 3 min were achieved (Medvedev, 1953; La Technique Moderne, 1955; Barkan, 1962).

The year 1953 saw the appearance of definitive theoretical treatments of vibratory driving and hammering by Neimark, Blekhman and Tsaplin. Further progress in this year included the introduction of the VPP high frequency range of machines which first used a resiliently mounted surcharge load to assist penetration and which could achieve depths of 20 m with piles weighing up to 2 Mg in saturated sands. Penetration of piles of larger point resistance was made possible by water jetting. Also in 1953 the first use of vibratory drivers for the installation of sand drains occurred and in the next few years some 35 000 were emplaced (Barkan, 1962).

In 1955 Tatarnikov designed the VP low frequency range of machines (7–16 Hz) to extend the method to piles of larger point resistance, since at low frequency, large displacement amplitude, separation of the pile tip from the soil and hence impacting could occur which assisted penetration. In 1959 Barkan attempted to increase the capacity of vibratory drivers by using the concept of pile–soil resonance and production machines were developed on this basis. This coincided with the invention of Bodine of the high frequency resonant driver in the USA where the resonance considered is that of the pile rather than the pile–soil system.

German and French engineers encouraged by the reported successes in Russia designed high frequency machines for use in Western Europe, but high rates of wear in motors and bearings later led to a reduction in frequency to 25 Hz. By 1961 vibratory drivers were being produced commercially in West Germany, France, USA, USSR and Japan. In 1962 Bodine obtained a patent for his resonant machine, and two years later research was initiated in the USA to study the resonant machine at model level (Hill 1966; Ghahramani, 1967; Griggs, 1967; Yang, 1967).

To date it has been reported that in the USSR about 400 000 Mg of sheet steel piling has been driven by vibrators and more than 100 million m of exploration boreholes installed using vibratory hammers. The major developments in Western Europe have been confined to the highly specialized installation of medium to large diameter piles, H and sheet steel piles, although it is encouraging to note that more recently some UK companies have begun developing low- and medium-powered vibrators for a variety of geotechnical applications.

Cementation Ground Engineering Ltd have developed rod (40 mm dia.) and casing drivers for alluvial grouting and soil anchor applications. Aimers McLean Ltd working in close collaboration with the Institute of Geological Sciences and the National Engineering Laboratory have produced a commercial gravity coring machine which can be used to obtain sea bed samples up to 14 mm diameter and 6 m long (Kirby, 1972). In 1972 BSP Ltd obtained world rights to the Christiani-Shand variable parameter electro-hydraulic machine and within 2 years produced an improved version (Pearson, 1974).

CURRENT VIBRATORY DRIVING PRACTICE

Vibratory drivers are classified in practice by their range of application, that is, by the maximum side or end resistance which can be accommodated within the range of the machines' dynamic parameters. The two parameters normally used to define the range of application are the displacement amplitude and frequency of vibrations. From the available published information it would appear that the choice of frequency of vibrations should be related to soil type: coarse grained soils, 4–10 Hz; fine-medium sands, 10–40 Hz; cohesive soils, 40–100 Hz; and that a high displacement amplitude (10–20 mm) should be selected for piles with a large

Table 1. **Machine classification**

Cohesive soils	Dense cohesionless soils		Loose cohesionless soils	
High acceleration Low displacement amplitude	Low point resistance	High point resistance	Heavy piles	Light piles
Requires high acceleration for either shearing or thixotropic transformation. Predominant side resistance A	High acceleration	Low frequency. Large displacement amplitude		High acceleration
	Predominant side resistance. Requires high acceleration for fluidization A	Predominant end resistance. Requires high displacement amplitude and low frequency for maximum impact to permit elasto–plastic penetration B		Predominant side resistance. Requires high acceleration for fluidization A
Recommended parameters				
f > 40 Hz $\ddot{x}$: 6–20 g A: 1–10 mm	f: 10–40 Hz $\ddot{x}$: 5–15 g A: 1–10 mm	f: 4–16 Hz $\ddot{x}$: 3–14 g A: 9–20 mm		f: 10–40 Hz $\ddot{x}$: 5–15 g A: 1–10 mm

point resistance, and a small displacement amplitude (1–10 mm) for piles with low point resistance. The explanation for this selection of vibrator parameters follows and is summarized in Table 1, along with recommended parameters collated from published information.

In cohesionless soils of low relative density the application of vibration causes a considerable reduction in the shear strength of the soil, often termed 'fluidization'. The degree of fluidization induced is proportional to both displacement amplitude (A) and frequency (f) of vibration. In loose soils this fluidized zone is assumed to extend beneath the pile. The penetrative motion can, therefore, be considered to be controlled by the applied surcharge force, displacement amplitude and frequency of vibration. The combined influence of the latter two parameters can be defined by amplitude of vibrational acceleration, since this is directly proportional to displacement amplitude and frequency of vibration. A similar motion is assumed to occur with piles of low point resistance driven in more dense cohesionless soils, where the major component of resistance experienced is that at the pile sides.

With piles of larger point resistance in cohesionless soils, it should not be assumed that fluidization occurs beneath the pile tip, since experimental evidence suggests the occurrence of an impact situation, which cannot be satisfactorily explained by the fluidization concept. Penetration, instead, occurs by means of an elasto-plastic motion which is assisted if an impact situation develops (separation of the pile from the soil). A large displacement amplitude and low frequency ensure maximum impact and plastic penetration.

In cohesive soils, the shear strength mobilized is attributable to the high bond strength generated between the small grains, although, adhesion to the pile can exceed these inter-

Table 2. Classification by acceleration of vibration

Class	Acceleration range (g)	Power demand (kW)	Remarks
A	12 to 40	2 to 60	Upper mechanical limit for vibratory driver without including resonance. Application of class A machine is defined by the amplitude of acceleration
B	1 to 12	20 to 200	Threshold value for fluidization of the soil (1 g). Application of class B machines defined by the amplitude of displacement
C	40 to 100	Can be > 750	Resonant driving. Application of class C machines defined by frequency

granular attractive forces. A cohesive soil can usually be considered as a visco-elastic material, and, if vibration occurs at the pile–soil resonant frequency with a large displacement amplitude, all the power will be absorbed by a large volume of the cohesive soil. To minimize this volume it is necessary to vibrate well above the pile–soil resonant frequency, with small displacement amplitude. Penetration of cohesive soils usually occurs by shearing of the soils, but if sufficient moisture is present, thixotropic transformation can occur.

Consideration of Table 1 allows the formulation of the vibratory driver classification system described in Table 2.

SOIL RESPONSE WITH VIBRATORY DRIVING

It is useful at this point to review in more detail the mechanism by which the phenomenon of shear strength reduction or fluidization occurs.

The amplitude of vibrational acceleration is now accepted as the parameter controlling the occurrence of fluidization and that with reference to the effect of this parameter on shearing strength of a cohesionless soil, three distinct physical states may be identified as shown in Fig. 1.

In the first state (acceleration $(d^2x/dt^2) < 0{\cdot}6$ g), which can be defined as the sub-threshold (elastic response) state, interparticle friction does exist but the overburden pressure acting normally on the shear plane is periodic. Normally this would lead to dynamic stability but if the soil density is less than the critical value, compaction will occur. In this state the shear strength has not been found to decrease by more than 5%.

In the second, or trans-threshold (compaction response), state $(0{\cdot}7\ \mathrm{g} < d^2x/dt^2 < 1{\cdot}5\ \mathrm{g})$ the decrease in the shear strength is governed by the exponential function of acceleration of vibration. The parameters of this exponential function are determined by the grain size and shape, and the magnitude of the static normal pressure.

In the third (fluidized response) state the shear strength reduction reaches a maximum. Theoretically this should be achieved at an amplitude of acceleration equal to that of gravity; however, in practice, due to the presence of interparticle friction, the amplitude required is approximately 1·5 g.

These three posited physical states of shear strength reduction have been confirmed by dynamic direct shear tests (see, for example, Mogami and Kubo, 1953).

Barkan (1962) found an exponential relationship between the coefficient of internal friction and amplitude of acceleration, describing the trans-threshold state, of

$$\tan\phi_{\mathrm{d}} = (\tan\phi_{\mathrm{st}} - \tan\phi_{\mathrm{m}})e^{-\alpha_1\eta} + \tan\phi_{\mathrm{m}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

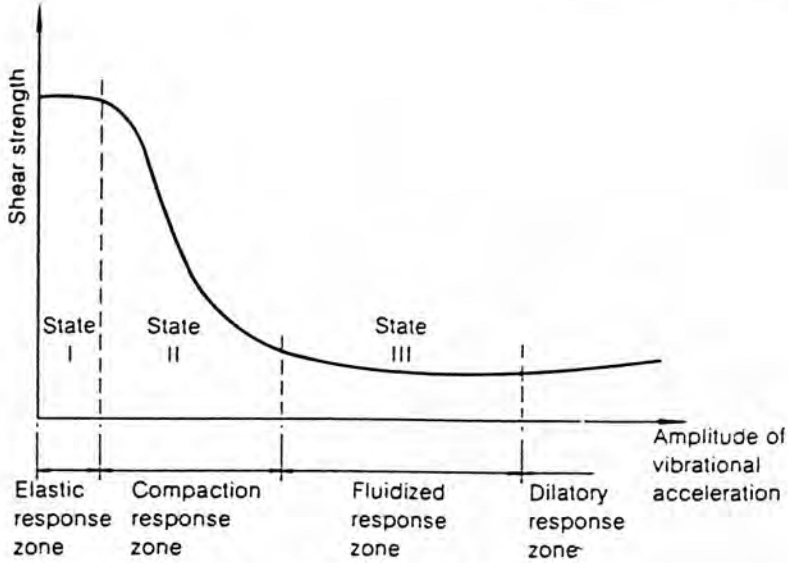


Fig. 1. Relationship between shear strength and amplitude of acceleration

where

$$\tan\phi_d = \text{dynamic value of the coefficient of internal friction}$$
$$\tan\phi_{st} = \text{static value of the coefficient of internal friction}$$
$$\tan\phi_m = \text{minimum value of the coefficient of internal friction}$$
$$\alpha_1 = \text{constant (0·23 for a medium grained dry sand)}$$
$$\eta = \text{ratio of amplitude of vibrational acceleration to that of gravity}$$

This relationship has been verified by subsequent research work by Ermolaev and Senin (1968) and Youd (1967).

It is suggested that it would be valid to apply a relationship of this type when evaluating the soil resistance in the case of class A machines or, the side resistance, only, in the case of class B machines.

The validity of this proposal can be confirmed from the results of Preobrajhenskaja (1956) who undertook extraction tests on full-scale piles from medium dense silts and saturated sands. His results show that the side resistance varied exponentially with the amplitude of acceleration with the maximum reduction in resistance occurring at an accelerational amplitude of 1–2 g.

The following relationship was fitted to the results,

$$R_S = R_M + (R_{ST} - R_M)e^{-\alpha_2\eta} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

where

$$\alpha_2 = \text{constant}$$
$$R_S = \text{dynamic side resistance}$$
$$R_M = \text{minimum dynamic side resistance}$$
$$R_{ST} = \text{static side resistance}$$

Representation of the point resistance encountered by class B machines is much more difficult to formulate. Typical representation has been of the form of either a linear viscous or a linear elasto–plastic model (see Table 3 for classification of representations which have been adopted to date) although, in practice, it is expected that neither the viscosity, elasticity or plasticity is linear. The possibility must also exist of an impact situation developing, with

Table 3. Classification of proposed theoretical solutions

Penetration by modification of soil properties			Penetration with no modification of soil properties		Penetration Assuming a non-rigid pile
Phenomeno-logical solution	Semi empirical solution	Empirical solution	Semi empirical solution	Phenomeno-logical solution	Wave equation
Barkan (1957) Wu (1965)	Shekhter (1955) Barkan (1962) Schmid (1970)	Schmid and Hill (1966, 1967) Bernhard (1963) Kondner and Edwards (1960) Al-Shawaf (1970)	Podol'nyy (1964) Senator (1967) Yang (1967)	Neimark (1953) Blekhman (1954) Koushov and Shliakhtin (1954) Barkan (1962) Savinov and Luskin (1960)	Barkan (1962) Parkin (1961) Ghahramani (1966) Griggs (1966) Hill (1967) Bernhard (1967) Rockefeller (1968) Schmid (1970)

separation of the pile from the soil occurring once per cycle, where the elasto–plastic characteristics of the soil assume lower values due to the vibrations. Further, the formulation of elasto–plastic penetrative motion does not completely rule out the possibility of fluidization occurring beneath the pile point, but instead shows that it is unnecessary to explain penetration of the pile as long as the total dynamic and surcharge forces are sufficient to exceed the soil resistance.

Interdependence of side and point resistance must be considered as the soil beneath the pile will be more compacted than at the sides, requiring a downward flow of soil This would lead to a reduced, localized side resistance and a cone of depression at the surface.

Substantiation for the elasto–plastic impact mechanism has come from the work of Schmid (1969) at Princeton University, from experimental work at Aberdeen University (Rodger, 1976) and from field prototype work undertaken at the National Engineering Laboratory, East Kilbride (Rodger, 1975).

Professor Schmid has found that there are three possible domains for the dynamic force measured at the pile point.

(a) The Sinusoidal Resistance Domain—where the dynamic force is less than the maximum elastic resistance of the soil, allowing no plastic motion and varies as a sinusoidal function in phase with the soil resistance.

(b) The Impact Domain—where the dynamic force is less than the impact threshold but greater than the resistance threshold. The dynamic force is no longer sinusoidal but approaches short periods of impact followed by periods of separation of the pile from the soil.

(c) The Instability Domain—where maximum end resistance is encountered. In this domain the dynamic force has exceeded the impact threshold and a phase difference occurs between the point resistance and the dynamic force.

From a study of theoretical models proposed to date and the above considerations the Authors consider that a valid theory must account for:

(a) The dependence of side resistance on the vibrational parameters including the concepts of fluidized soil viscosity and external friction. Fluidization has to date been accounted for by a linear viscous damping term ($\nu\, dx/dt$) where the damping constant (ν) must be considered as a function of the displacement amplitude and frequency. Only

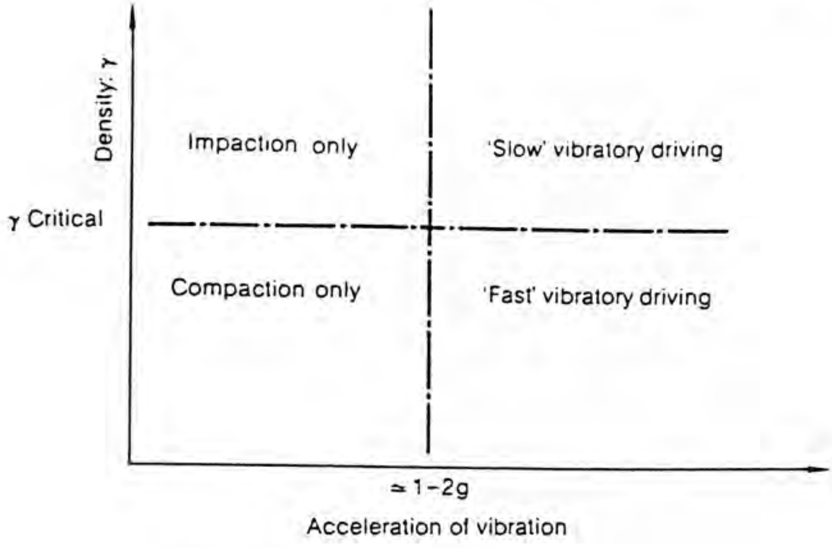


Fig. 2. Determination of the occurrence of vibratory driving motion

Barkan (1962) has considered the external friction of the soil separately as a function of the vibrational parameters.

(b) The complex end resistance which may take the form of a fluidized soil volume beneath the pile.

Fluidization may occur beneath the pile tip but this alone cannot explain experimentally observed phenomena such as the occurrence of an impact domain. This, however, can be accounted for by an elasto–plastic end resistance where the elasto–plastic properties of the soil are modificed by vibration.

A successful solution would therefore have to incorporate a viscous–Coulomb side resistance; and an elasto–plastic end resistance, whose parameters were a function of depth of penetration and vibration.

THEORETICAL SOLUTIONS PROPOSED

It is proposed that two types of vibratory driving motion exist—'slow' and 'fast' vibratory driving. This is because the classical elasto–plastic situation beneath the pile tip, as described above, will only occur when the pile is able to mobilize a threshold value of the soil resistance. The two types of motion can be defined with reference to Fig. 2. Motion is assumed only to occur when the amplitude of acceleration exceeds the threshold value for fluidization of the soil and the amplitude of the force exceeds the soil resistance. It is the relationship between the soil density in situ and the critical soil density which determines whether slow (impact) or fast vibratory driving will occur.

The two types of motion can be defined in the following way.

Fast vibratory driving (see Fig. 3(*a*))

In loose granular soils it is assumed that there is no reversed penetrative, as distinct from vibrational motion, allowing the possibility of treating the two cases of vibratory and penetrative motions separately. Penetrative motion occurs by the progressive compactive collapse of the voids in the soil, the resistance being assumed to be almost wholly due to fluidization of the soil.

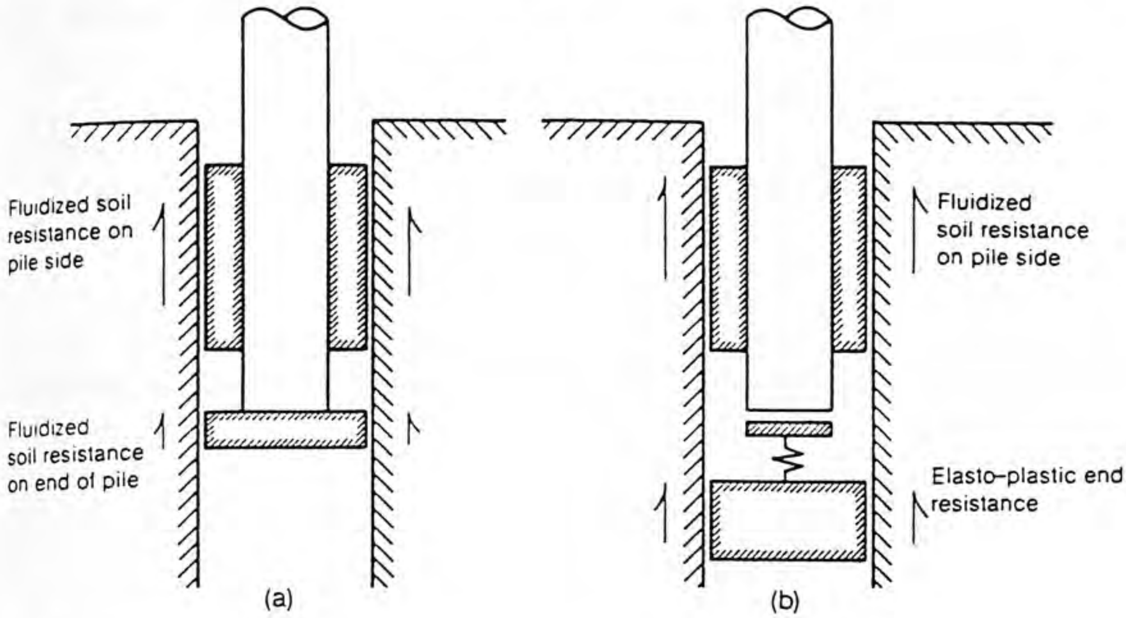


Fig. 3. **Physical modes of vibratory driving motion: (a) fast model; (b) slow model**

Slow vibratory driving (see Fig. 3(b))

In granular soils where the density is greater than critical, if the surcharge force is insufficient to prevent reversal of motion, the end resistance acts only during a portion of each cycle. Penetration is assumed to occur as in a classical elasto–plastic impact situation. Restraint at the pile sides is assumed to be due to external dry frictional and fluidized viscous soil resistance.

Experimental confirmation of the existence of these two types of motion can be found from examinations of the dynamic force waveforms measured at the pile point. In loose granular soils and in vibratory extraction, a sinusoidal force response is obtained, while in dense granular soils there is an impact type of waveform.

THEORETICAL EQUATIONS OF MOTION FOR 'FAST' VIBRATORY DRIVING

Assuming that the relatively small vibratory motion can be separated from the comparatively large penetrative motion and that the sole purpose of the vibratory motion is to reduce the resistance to penetration, the equation of vibratory motion is

$$M\frac{d^2x}{dt^2}+R_D = F_D \sin \omega t \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where

M = mass of vibrator and pile

x = vibrational displacement

R_D = dynamic soil resistance

F_D = dynamic force

ω = angular velocity

t = time

Similarly the equation of penetrative motion is

$$M\frac{d^2y}{dt^2}+R_E = F_S + Mg \quad (4)$$

where

R_E = soil resistance to penetration

F_S = surcharge force

y = penetrative displacement

If it is assumed that the shear resistance per unit length is independent of depth, and that the end resistance is proportional to depth, then resistances may be expressed as

$$R_D = \nu^1 y\frac{dx}{dt} \quad (5)$$

$$R_E = \nu^1 y\frac{dy}{dt}+ky \quad (6)$$

where ν^1 is the viscous shear constant and k the elastic soil resistance (end bearing constant).

The equations of motion therefore become

$$M\frac{d^2x}{dt^2}+\nu^1 y\frac{dx}{dt} = F_D \sin \omega t \quad (7)$$

$$M\frac{d^2y}{dt^2}+\nu^1 y\frac{dy}{dt}+ky = F_S+Mg \quad (8)$$

The shear constant ν^1 is a function of soil properties, pile diameter and the frequency and displacement amplitude of vibration. Experimental observations (Littlejohn, Seager & Rodger, 1974) have shown that ν^1 must increase sharply when the frequency and displacement amplitude falls to the threshold value below which fluidization of the soil does not occur.

Solution of equations (7) and (8) assuming constant penetrative velocity, gives the following expressions for the power demand

P_V = power required to sustain vibratory motion

$$= F_D{}^2/4M\omega \quad (9)$$

P_P = power required to sustain penetrative motion

$$= (\nu^1 yV+ky-Mg)\,V \quad (10)$$

where V is the constant penetrative velocity.

For the vibratory motion the displacement amplitude is given by

$$A = \frac{F_D/\omega}{((M\omega)^2+(\nu^1 y)^2)^{\frac{1}{2}}} \quad (11)$$

from which an assessment of the values of the dynamic parameters needed to sustain penetrative motion or fluidization can be found.

THEORETICAL EQUATIONS OF MOTION FOR 'SLOW' VIBRATORY DRIVING

Figure 4 illustrates the motion considered in the elasto-plastic model of slow vibratory driving. When the pile is penetrating, the resistance is plastic, assuming that the maximum elastic resistance has been overcome. Between periods of penetration there may occur phases

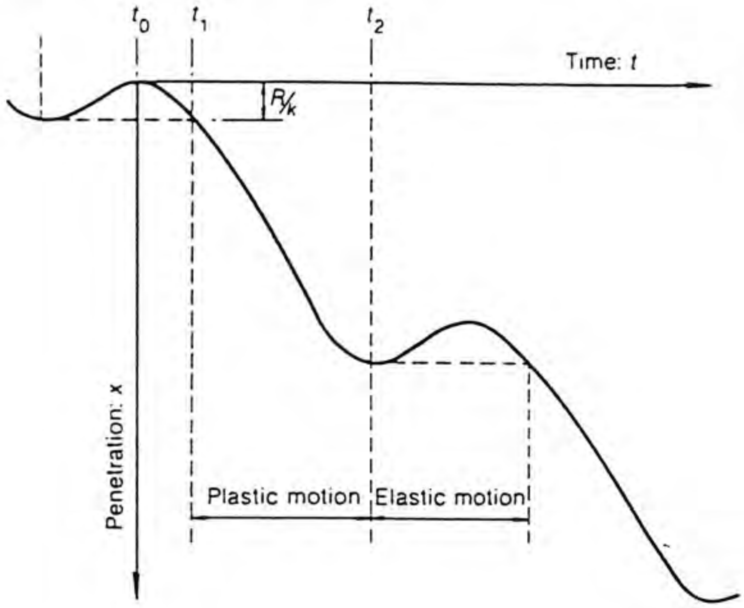


Fig. 4. **Form of the penetration motion**

of reversal and recovery against elastic end resistance. The reversal may be great enough to bring the end of the pile out of contact with the soil. If loss of contact does occur, there is no end resistance during part of the reversal-recovery phase.

The theoretical equations of motion may be defined as follows. If x_{max} is the maximum value of the displacement x, up to any point in time, then in the region of plastic motion ($t_1 < t < t_2$) the end resistance can be represented (see Fig. 5) by

$$R_E = R \quad \text{(see Fig. 5)} \qquad (12)$$

in the region of elastic motion where $x_{max} - R/k < x < x_{max}$ (and $t_0 < t < t_1$) the end resistance is given by

$$R_E = R + k(x - x_{max}) \qquad (13)$$

and in the region of out of contact motion, which occurs wherever

$$x < x_{max} - R/k \qquad (14)$$

($t_0^1 < t < t_0^{11}$ see Fig. 6) the end resistance will be zero. The equation of motion for this system may be formulated as

$$M\frac{d^2x}{dt^2} + R_S + R_E = F_S + F_D \sin(\omega t + \theta) \qquad (15)$$

where

R_S = the side resistance which may be viscous or Coulomb or both

R_E = $[R - k(x_{max} - x)]$ where the square parentheses denote Macauley notation

θ = phase difference between force and displacement.

The best approach to solution of this equation is by numerical methods; however, it is useful to examine a special case to illustrate the form of the solution.

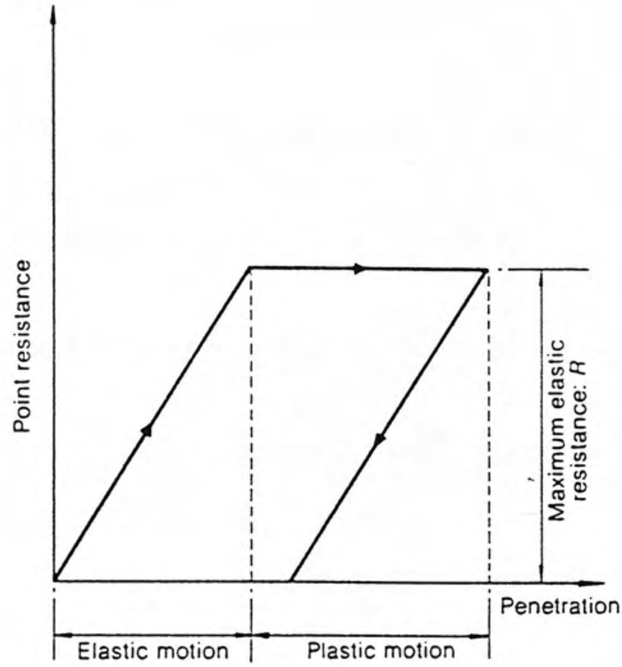


Fig. 5. Formulation of the elasto-plastic end resistance

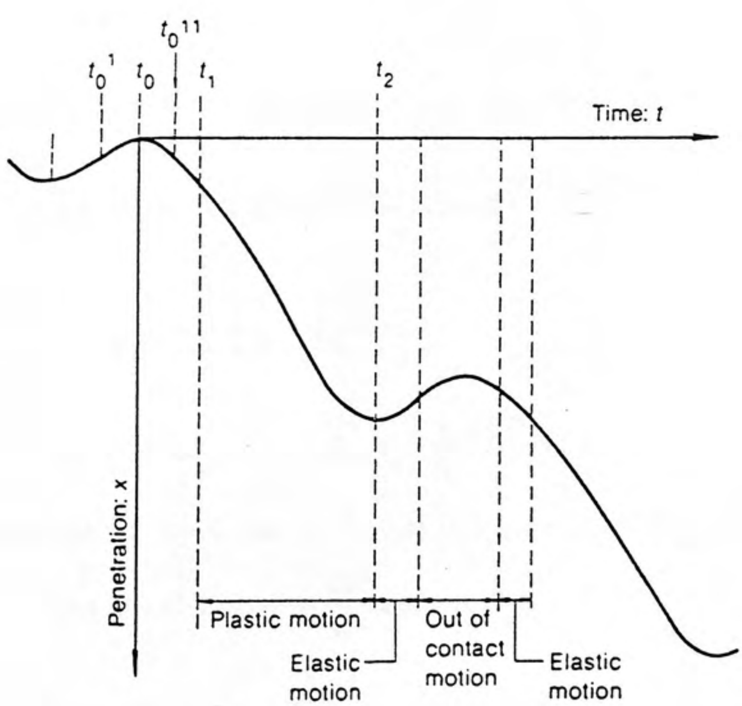


Fig. 6. Motion allowing separation of the pile from the soil

Analysis of motion without side resistance and with no loss of contact

At $t = 0$ (Fig. 6) let the pile be at the start of an elastic phase of motion which continues until $t = t_1$. Take $x = 0$ at $t = 0$ and let the instantaneous value of the dynamic force at $t = 0$ be $F_1 \sin\theta$. Let

$$r = R/M, \quad \omega_n{}^2 = k/M, \quad f_S = F_S/M, \quad f_1 = F_1/M$$

In the elastic region, $0 < t < t_1$, $x_{max} = 0$,

$$d^2x/dt^2 + r + \omega_n{}^2 x = f_S + f_1 \sin(\omega t + \theta) \quad \ldots\ldots \quad (16)$$

with solution,

$$x = B \sin\omega_n t + C \cos\omega_n t + \frac{f_S - r}{\omega_n{}^2} + \frac{f_1}{\omega_n{}^2 - \omega^2} \sin(\omega t + \theta) \quad \ldots\ldots \quad (17)$$

$$dx/dt = \omega_n B \cos\omega_n t - \omega_n C \sin\omega_n t + \frac{f_1 \omega}{\omega_n{}^2 - \omega^2} \cos(\omega t + \theta) \quad \ldots\ldots \quad (18)$$

Initial conditions are $x = dx/dt = 0$ when $t = 0$, therefore,

$$C = -\frac{f_S - r}{\omega_n{}^2} - \frac{f_1 \sin\theta}{\omega_n{}^2 - \omega^2} \quad \ldots\ldots \quad (19)$$

$$B = -\frac{f_1(\omega/\omega_n)\cos\theta}{\omega_n{}^2 - \omega^2} \quad \ldots\ldots \quad (20)$$

writing

$$\bar{x} = \frac{\omega_n{}^2 x}{r}, \quad \bar{f}_S = \frac{f_S}{r}, \quad \bar{f}_1 = \frac{f_1}{r}$$

$$\dot{x} = \frac{dx}{dt}, \quad \dot{x} = \frac{\omega_n \bar{\dot{x}}}{r}, \quad \tau = \omega_n t, \quad \bar{\omega} = \frac{\omega}{\omega_n}$$

then

$$\bar{x} = \bar{B} \sin\tau + \bar{C} \cos\tau + (\bar{f}_S - 1) + \frac{\bar{f}_1}{1 - \bar{\omega}^2} \sin(\bar{\omega}\tau + \theta) \quad \ldots\ldots \quad (21)$$

$$\bar{\dot{x}} = \bar{B} \cos\tau - \bar{C} \sin\tau + \frac{\bar{f}_1 \bar{\omega}}{1 - \bar{\omega}^2} \cos(\bar{\omega}\tau + \theta) \quad \ldots\ldots \quad (22)$$

where

$$\bar{B} = \frac{\omega_n{}^2 B}{r} = -\frac{\bar{f}_1 \bar{\omega} \cos\theta}{1 - \bar{\omega}^2}$$

$$\bar{C} = \frac{\omega_n{}^2 B}{r} = -(\bar{f}_S - 1) - \frac{\bar{f}_1 \sin\theta}{1 - \bar{\omega}^2}$$

At the point of maximum reversal $\bar{\dot{x}} = 0$ and $\bar{\ddot{x}} > 0$. Let this occur at $\tau = \tau_0$, then

$$\bar{B} \cos\tau_0 - \bar{C} \sin\tau_0 + \frac{\bar{f}_1 \bar{\omega}}{1 - \bar{\omega}^2} \cos(\bar{\omega}\tau_0 + \theta) = 0 \quad \ldots\ldots \quad (23)$$

$$\bar{B} \sin\tau_0 + \bar{C} \cos\tau_0 + \frac{\bar{f}_1 \bar{\omega}^2}{1 - \bar{\omega}^2} \sin(\bar{\omega}\tau_0 + \theta) < 0 \quad \ldots\ldots \quad (24)$$

In order for there to be no loss of contact it is necessary that $\bar{x}(\tau_0) > -1$, that is

$$\bar{B}\sin\tau_0+\bar{C}\cos\tau_0+\bar{f}_S+\frac{\bar{f}_1}{1-\bar{\omega}^2}\sin(\bar{\omega}\tau_0+\theta)>0 \quad . \quad . \quad . \quad . \quad . \quad . \quad (25)$$

The elastic region ends at $\tau=\tau_1$ where $\dot{\bar{x}}(\tau_1)=0$, that is

$$\bar{B}\sin\tau_1+\bar{C}\cos\tau_1+(\bar{f}_S-1)+\frac{\bar{f}_1}{1-\bar{\omega}^2}\sin(\bar{\omega}\tau_1+\theta)=0 \quad . \quad . \quad . \quad . \quad . \quad (26)$$

In the plastic region where $t_1<t<t_2$,

$$\ddot{x}+r=f_S+f_1\sin(\omega t+\theta) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (27)$$

therefore

$$\dot{x}=(f_S-r)t-\frac{f_1}{\omega}\cos(\omega t+\theta)+D \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (28)$$

$$x=\tfrac{1}{2}(f_S-r)t^2-\frac{f_1}{\omega^2}\sin(\omega t+\theta)+Dt+E \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (29)$$

or, in dimensionless form

$$\dot{\bar{x}}=(\bar{f}_S-1)\tau-\frac{\bar{f}_1}{\bar{\omega}}\cos(\bar{\omega}\tau+\theta)+\bar{D} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (30)$$

$$\bar{x}=\tfrac{1}{2}(\bar{f}_S-1)\tau^2-\frac{\bar{f}_1}{\bar{\omega}^2}\sin(\bar{\omega}\tau+\theta)+\bar{D}\tau+\bar{E} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (31)$$

Initial conditions are $\bar{x}=0$, $\dot{\bar{x}}=\bar{V}_1$ when $\tau=\tau_1$, therefore

$$\bar{D}=\bar{V}_1-(\bar{f}_S-1)\tau_1+\frac{\bar{f}_1}{\bar{\omega}}\cos(\bar{\omega}\tau_1+\theta) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (32)$$

$$\bar{E}=-\bar{C}\tau_1-\tfrac{1}{2}(\bar{f}_S-1)\tau_1^2+\frac{\bar{f}_1}{\bar{\omega}^2}\sin(\bar{\omega}\tau_1+\theta) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (33)$$

End of plastic region occurs when $\tau=\tau_2$ where

$$\dot{\bar{x}}(\tau_2)=0$$

Motion with loss of contact follows a similar form of analysis the equation being: at $t=0$, $\bar{x}=\dot{\bar{x}}=0$.

$$\text{Elastic motion:}\quad \ddot{\bar{x}}+\bar{x}=\bar{f}_S-1+\bar{f}_1\sin(\bar{\omega}\tau+\theta) \quad . \quad . \quad . \quad . \quad . \quad . \quad (34)$$

$$\text{Out of contact:}\quad \ddot{\bar{x}}=\bar{f}_S+\bar{f}_1\sin(\bar{\omega}\tau+\theta) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (35)$$

$$\text{Plastic motion:}\quad \ddot{\bar{x}}=\bar{f}_S-1+\bar{f}_1\sin(\bar{\omega}\tau+\theta) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (36)$$

Having considered the special case of zero side resistance over only one cycle of motion the need for the introduction of numerical methods is evident. The problem in formulating a numerical solution is the selection of the initial soil parameters and the variation of these parameters with depth of penetration. These parameters have not as yet been defined accurately enough for assessment by calculation. Any approach must, therefore, be semi-empirical in nature until sufficient data are available to either construct design charts to predict values of soil parameters, or to evolve a satisfactory relationship between the soil parameters and acceleration and displacement amplitude of vibration.

The approach to the solution of the problem was two-fold. Firstly, an interactive computer simulation of the theoretical equations of motion was developed and secondly on a model level

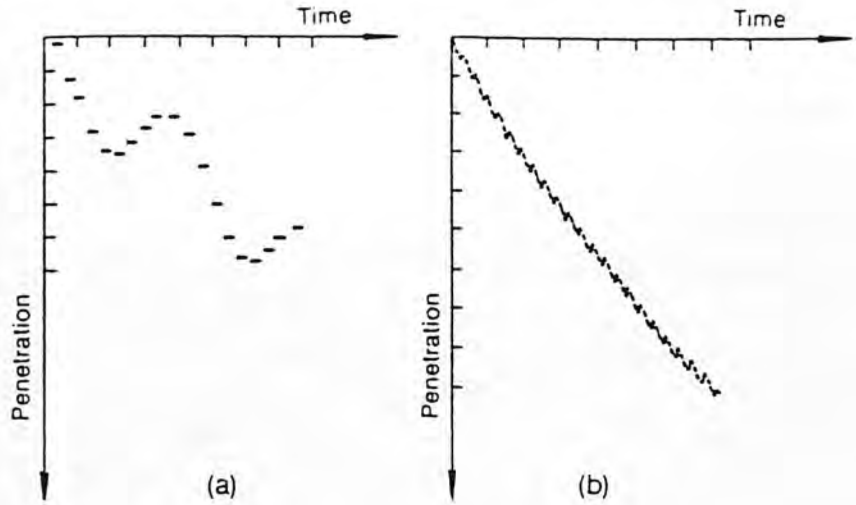


Fig. 7. **Computer simulation: (a) initial cycle of motion; (b) complete motion**

the influence of the various parameters was investigated. The theoretical simulation was fitted to the variable parameter test results, thereby defining possible values of the soil resistance—the solution being thus semi-empirical in nature.

For the purpose of the simulation a linear variation with depth was chosen for all forms of the soil resistance, while the side frictional resistance was found using a relationship similar to that of Preobrajhenskaja,

$$R_S = R_M e^{-\alpha\eta}\pi d x \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (37)$$

where πdx represents the perimeter of the pile at depth x. It was also assumed that the displacement per cycle was sufficiently small to consider constant values of the soil resistance over one cycle of motion. Fig. 7 shows a typical computer simulation.

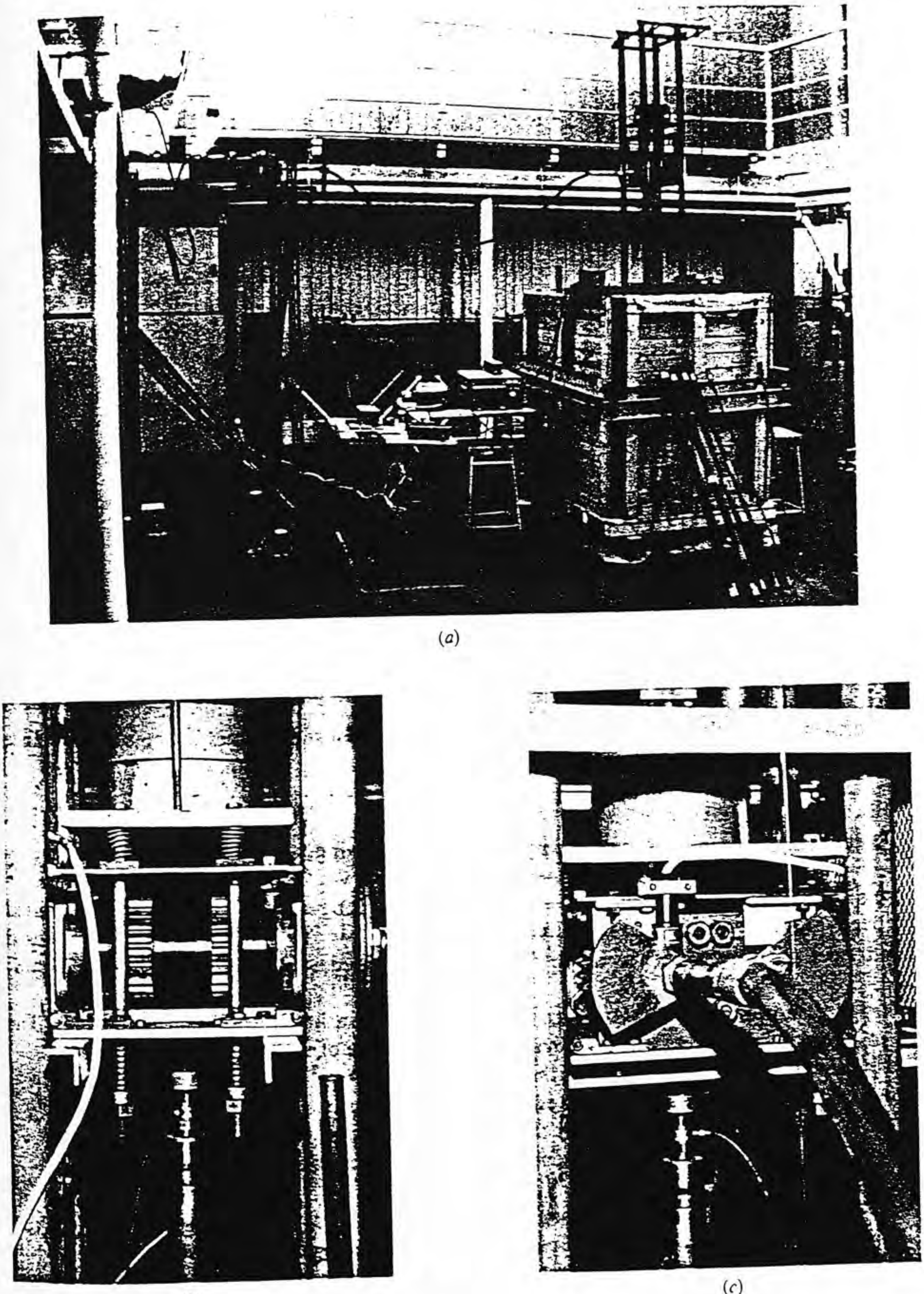
EXPERIMENTAL VERIFICATION OF THEORY

To confirm this semi-empirical theory of vibratory driving, the 0·745 kW model vibratory driver shown in Fig. 8 was constructed. This machine was capable of: exciting a force of constant magnitude independent of frequency, and exciting a force which could be varied independent of frequency. The machine was used to drive an instrumented 38 mm o.d., closed ended steel pile into a bed of dense fine–medium grained sand ($U = 1{\cdot}2$; $D_{10} = 0{\cdot}29$ mm; $D_r = 71{\cdot}5\%$; $\phi = 41°$).

Figure 9 shows a block diagram of the instrumentation and transducers employed. The purpose of the transducers was to provide a direct measure of the following parameters.

(a) Acceleration of vibration of the pile cap and tip.
(b) Triaxial acceleration of the resiliently mounted surcharge load.
(c) Dynamic and static forces at the pile cap and tip.
(d) Phase angle between displacement, force and acceleration of vibration.
(e) Penetrative displacement.
(f) Dynamic and static soil pressure.

Since Fourier analysis of the complex signals from the transducers was essential, all the signals were fed into an FM magnetic tape recorder. Fourier analysis was then accomplished using a standard algorithm—the Fast Fourier Transform (FFT). The instrumentation required for this form of analysis is shown in Fig. 10. The experimental test results may be summarized as follows.



(a)

(b)

(c)

Fig. 8. (a) General view of model vibratory driver; (b) and (c) detailed views of model vibrator unit

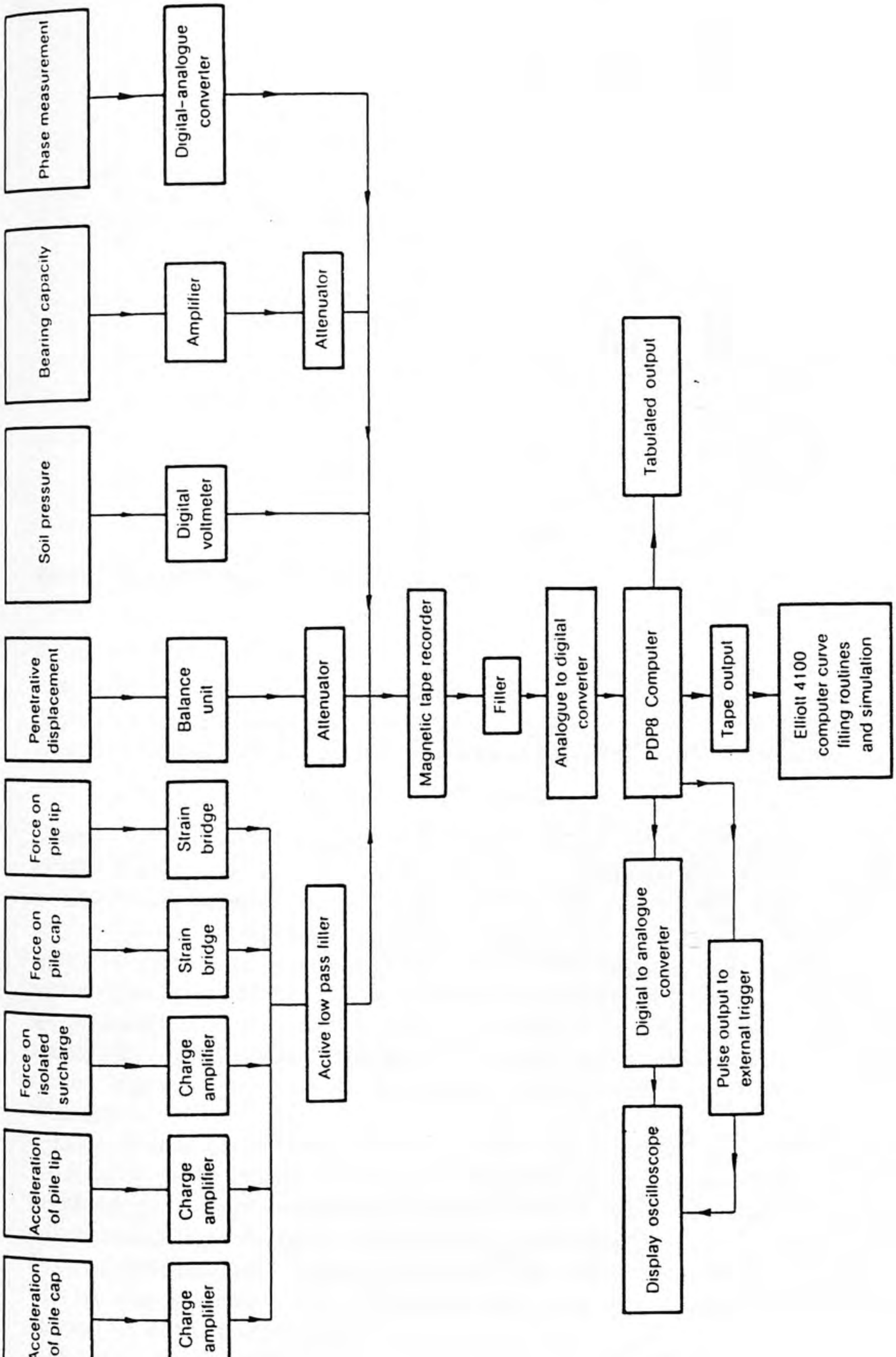


Fig. 9. Block diagram of instrumentation

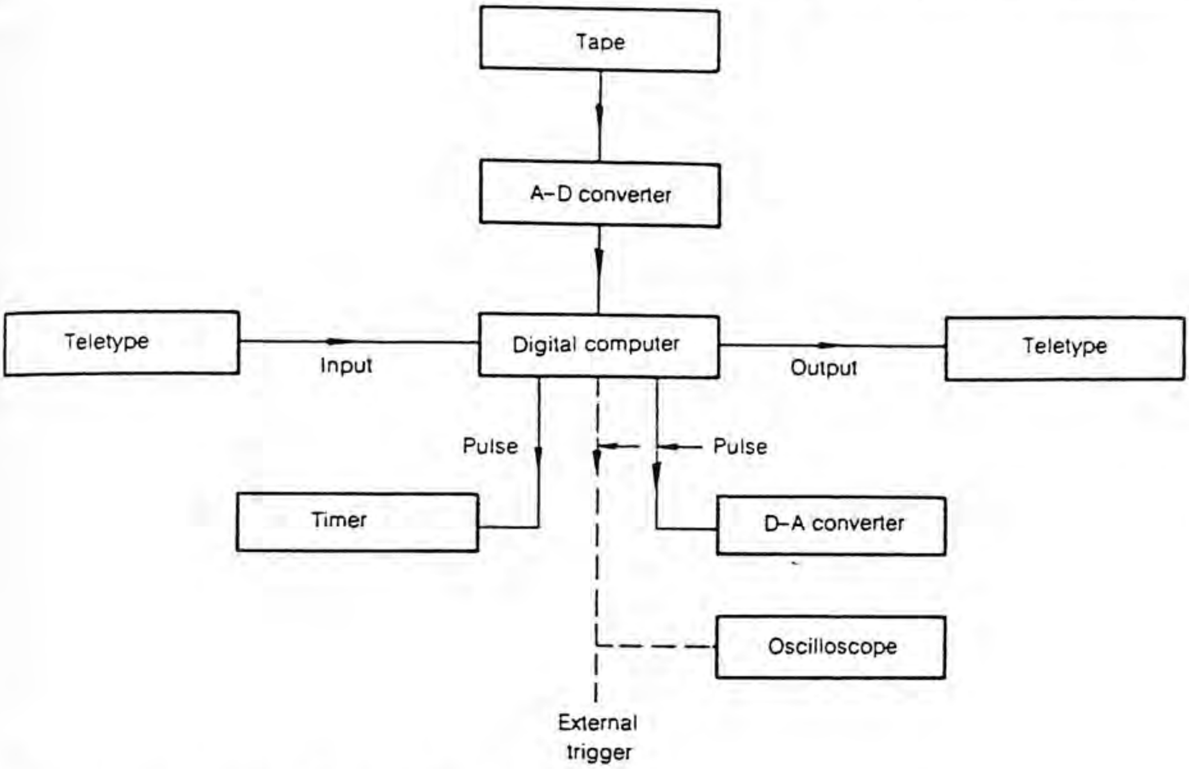


Fig. 10. Equipment required for spectral analysis

Frequency of vibration tests

A series of tests were conducted where the eccentric moment of the vibrator was held constant while the frequency was varied between 20 and 50 Hz. It was found that the penetration–time profiles could be represented by an exponential relationship of the form:

$$y = A^1 - B^1 e^{-c^1 t} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (38)$$

where A^1, B^1 and C^1 are constants. During the period of steady-state motion the relationship between frequency and penetration velocity was linear exhibiting a threshold value, below which no penetration occurred, for the frequency of 18 Hz. This corresponded to a free vibrational acceleration of 1·6 g and a peak dynamic force of 550 N. This threshold value may be explained with reference to the exponential variation of soil resistance with amplitude of vibrational acceleration described earlier. Its existence is consistent with the concept of fluidization and the formulation of an elasto–plastic end resistance the properties of which are reduced in magnitude by the vibration. This threshold value is identical to the resistance threshold defined earlier. Figure 11 indicates the occurrence of the threshold with reference to acceleration of vibration.

The existence of three domains of the dynamic force was also confirmed. Penetration occurred when the dynamic force first exceeded the resistance threshold of 550 N. A second threshold, the impact threshold, of between 550 and 725 N was identified beyond which the pile separated from the soil. At 800 N an effective upper limit to the dynamic force was also found, a limit threshold beyond which the ultimate depth of penetration no longer increased with increasing dynamic force. These threshold values are not absolute but are related to the dimensions of the pile employed.

Over the 'steady-state' motion period, when the dynamic force was in the pre-impact range, the first harmonic of the dynamic force was predominant (see Fig. 12(a, b)). When the force

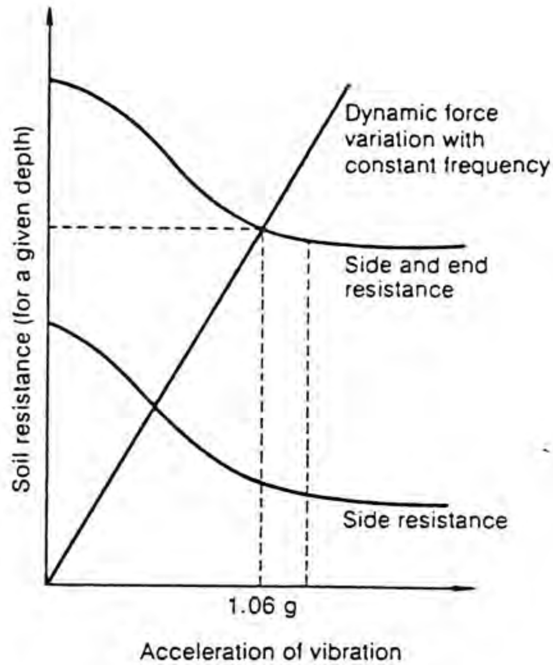


Fig. 11. **Illustration of threshold condition: variable frequency tests.**

level was increased beyond the impact threshold an impact 'spike' formed (see Fig. 12(c)) which decreased in duration and increased in magnitude up to the limit threshold, when a secondary impact spike developed in each cycle of motion.

In variable frequency extraction tests the motion was represented by the same exponential relationship indicated in the penetration tests. Extraction velocity was found to be independent of frequency of vibration with constant eccentric moment, consistent with the formulation of a viscous–Coulomb side resistance and independence of vibratory and extractive motions.

No impact profiles were observed in the force waveforms although the force level was sufficient to exceed the impact threshold earlier defined. This implies that insufficient soil resistance was mobilized to allow an impact situation, which is again consistent with the formation of a viscous side resistance.

Over the frequency range investigated the force response was almost purely sinusoidal—at 50 Hz the second harmonic representing only 8% of the first. From the variable frequency tests it may be postulated that the motion of the pile during penetration was slow vibratory driving and fast vibratory driving during extraction.

Eccentric moment tests

One series of tests was conducted at a constant frequency of 30 Hz while the eccentric moment was varied between 22 and 90 kg mm. The relationship between penetration velocity and eccentric moment was linear with a threshold value, below which no penetration occurs, of 200 N—a value which corresponds to a free vibrational acceleration of 1·06 g and a displacement amplitude of 0·3 mm. The threshold value, as in the variable frequency tests, may be explained with reference to the exponential variation of soil resistance with amplitude of acceleration as shown in Fig. 13.

Observations of the dynamic force again confirmed the existence of the three force domains.

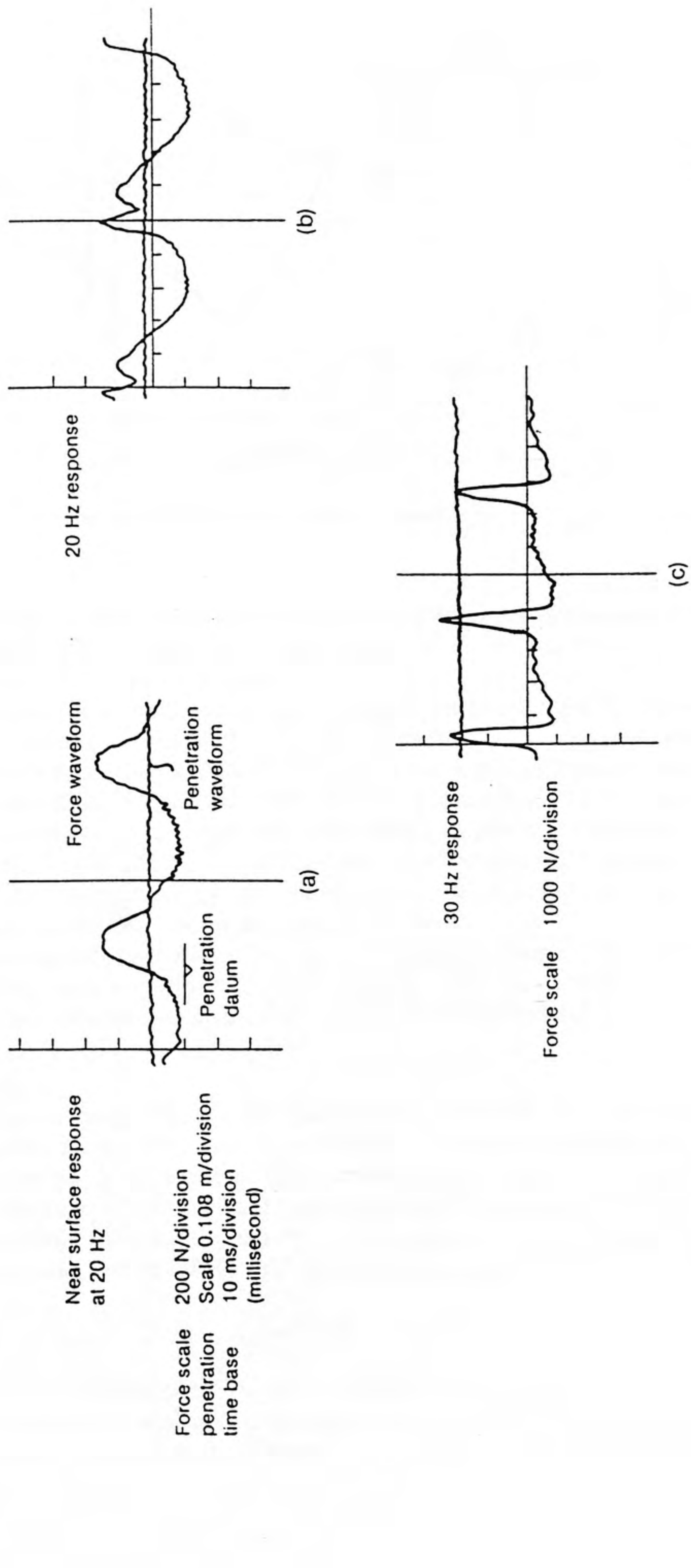


Fig. 12. Waveforms of dynamic force

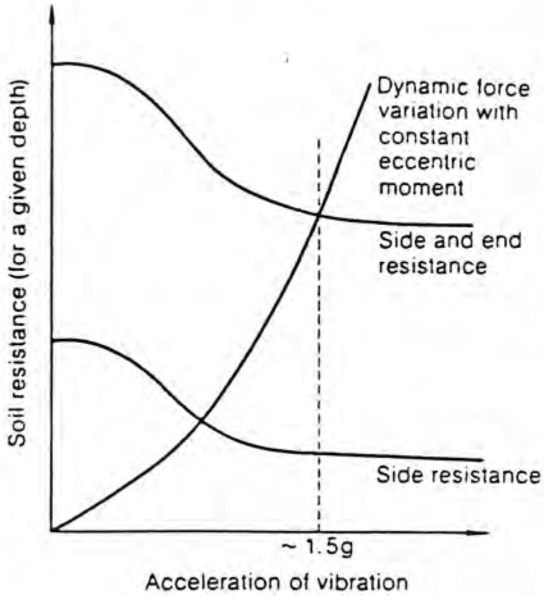


Fig. 13. **Illustrations of threshold conditions: variable eccentric moment tests**

In the pre-impact range, the first harmonic of the dynamic force was predominant (60% greater than any other harmonic). Beyond the impact threshold an impact spike formed which increased in magnitude to the limit threshold.

In the variable eccentric moment extraction tests the extraction velocity increased with increased eccentric moment—consistent with the formation of a viscous–Coulomb side resistance. The force waveforms were found to increase linearly with eccentric moment with no indication of significant impaction. With increasing eccentric moment, increased soil resistance was mobilized due to the increased displacement amplitude. This caused the force waveforms measured in the pile to include second- and third-order harmonics although remaining predominantly sinusoidal. In all the tests the force level was below the threshold value of the dynamic force measured in the penetration tests.

The static force required for extraction was 6% of the predicted static value and was found to vary exponentially with acceleration of vibration in both the variable frequency and eccentric moment tests, after the manner of the tests of Preobrajhenskaja.

Surcharge force tests

Over the range of surcharge forces investigated (F_S/F_D from 0·13 to 0·22) increasing the surcharge force caused an increase in both penetration depths and velocity. No threshold value for the surcharge was identified since this would only occur when the dynamic force was less than the soil resistance. An upper limit to the surcharge force occurred when it exceeded the dynamic force and vibration was suppressed. From published literature (e.g. see Hill, 1966, model tests; Savinov and Luskin, 1960, field tests) it appears that an optimum value occurs when

$$F_D = 2F_S \qquad (39)$$

COMPARISON OF THE EXPERIMENTAL AND THEORETICAL RESULTS

Comparison of the experimental and theoretical work is restricted by the present lack of quantitative information on the empirical constants necessary to define exactly the dynamic

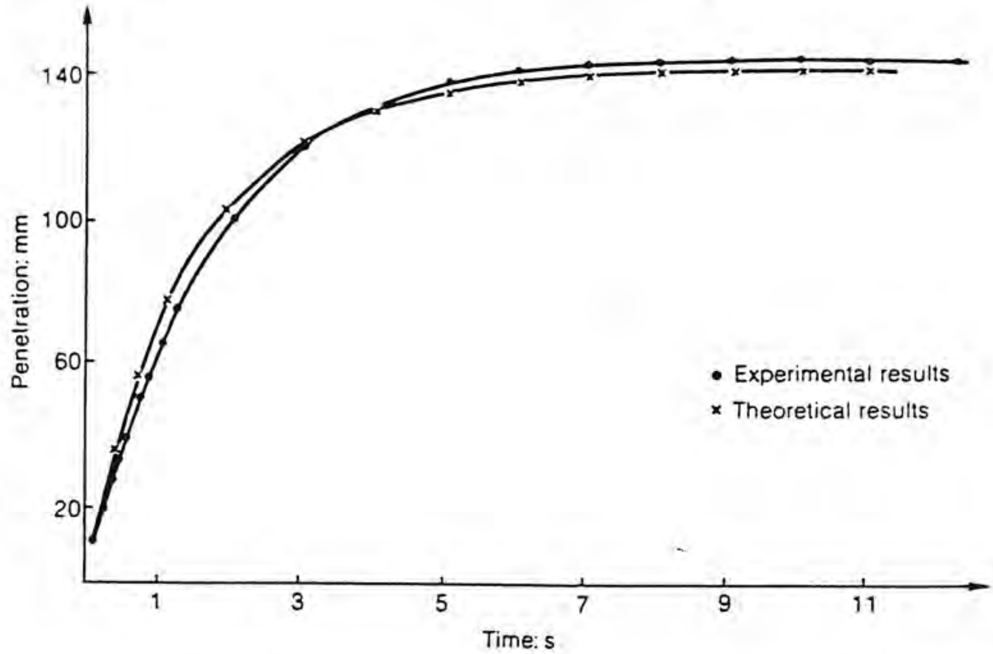


Fig. 14. Comparison of the theoretical and experimental penetration-time curves

soil resistance. To illustrate this, the equation of motion with viscous–Coulomb side resistance is

$$M\ddot{x}+R_{S_1}+R_{S_2}+R-k(x_{max}-x) = F_S+F_D\sin\omega t \quad . \quad . \quad . \quad . \quad . \quad (40)$$

From equation (2)

$$R_{S_1} = R_M+(R_{ST}-R_M)\,e^{-\alpha_2\eta}$$

while $R_{S_2} = \nu\dot{x}$, where α_2 and ν are functions of depth, frequency and displacement amplitude of vibration. In comparing experimental and theoretical results there is therefore the problem of predefining the soil parameters R_M, ν, α_2, R and k and, in addition, the variation of these with depth of penetration.

By means of an interactive computer display, which allowed the variation of soil resistance parameters it was found possible to reproduce the exponential type of penetration-time curve. Figure 14 shows the variable x_{max} (the maximum displacement at any time) fitted to one of the curves from the variable frequency test series. This simulation involved a viscous side resistance and elastic end resistance where the soil resistance resulting from parameters ν and R was found to follow the exponential behaviour shown in Fig. 15.

Figure 16 shows the theoretical behaviour of penetration velocity as a function of displacement amplitude, showing that a similar trend to that described in the experimental work can be obtained. The trend is towards increased penetration depth and velocity with increased displacement amplitude. On increasing the surcharge force it was found that the trend was also towards increased penetration depth and velocity. Once a threshold value was exceeded it was found that the vibrational motion was suppressed. This was found to occur theoretically when $F_S/F_D = 1{\cdot}2$ for the model vibratory driver.

Once a fit between experimental and theoretical results had been achieved a Fourier analysis of the theoretical solution was undertaken. From this it was found possible to generate harmonics similar to those observed in the experimental results.

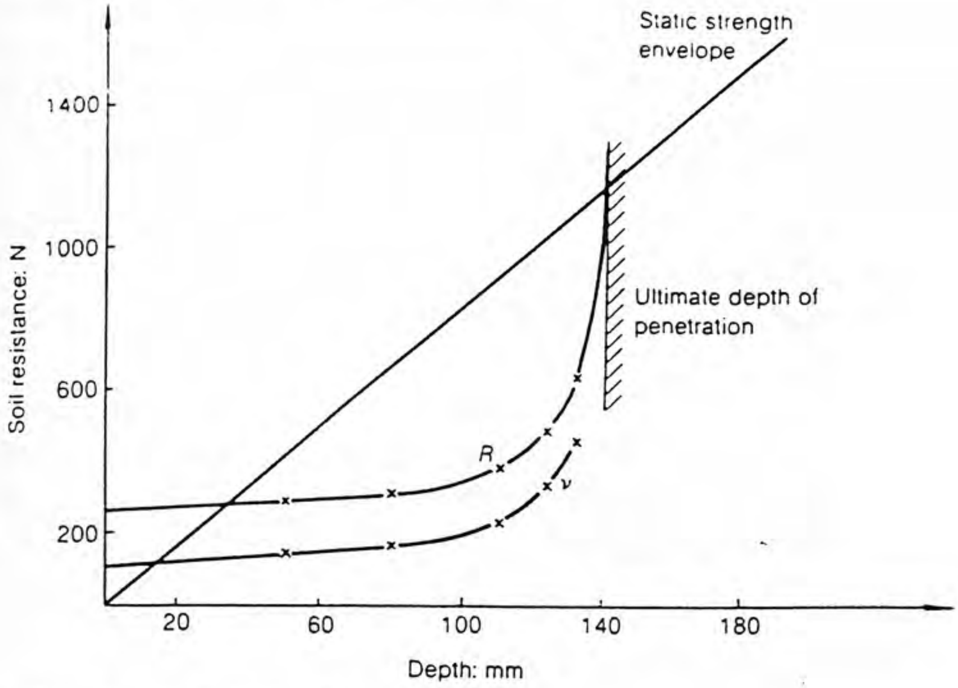


Fig. 15. **Variation of soil resistance with depth of penetration**

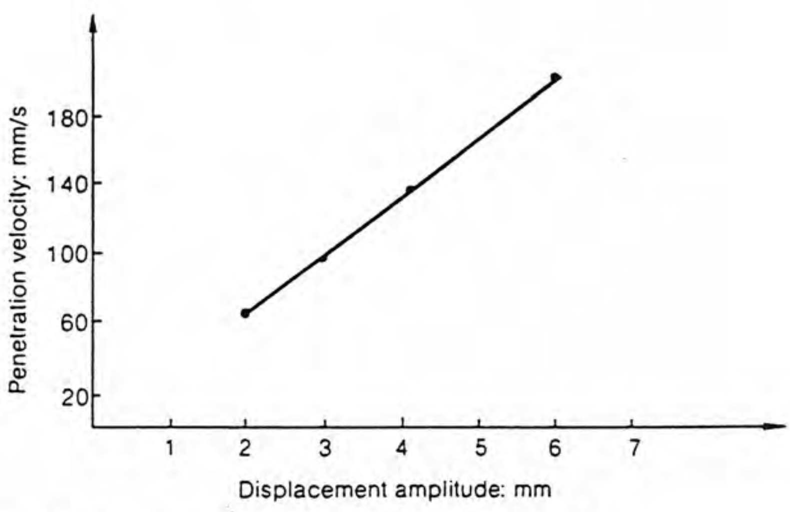


Fig. 16. **Initial penetration velocity as a function of displacement amplitude**

CONCLUSIONS

A theoretical solution to the most commonly encountered form of vibratory driving has been proposed. It has been found that with suitable choice of soil resistance parameters the theory can successfully predict experimentally derived results. To develop the solution further the soil resistance requires to be quantified together with its dependence on frequency and displacement amplitude, preferably at field prototype level. The research work to date has provided sufficient information on the behaviour of the dynamic parameters involved to allow the specification and design of a field prototype on which such tests can be conducted. The values found from full scale tests can then be applied in the theory of slow vibratory driving, and a comparison of penetrative and vibrational motions in theory and practice obtained.

In anticipation of such work being initiated, the research is at present being extended to examine the possibility of representing the end response by a conventional static function, assuming that in the course of slow vibratory driving, end response is not substantially altered by the vibration. If, as early results indicate, the end response can be thus represented, the research to date will be of immediate practical use in optimizing the vibratory driving process.

ACKNOWLEDGEMENTS

The Authors wish to acknowledge the technical and financial support provided by the Department of Engineering, University of Aberdeen, and the financial aid awarded by the Science Research Council.

REFERENCES

Al-Shawaf, N. A. (1971). *High frequency vibratory pile driving*. MSc thesis, Bristol University.
Anon. (1955). Utilisation de la vibration pour le foncage en masse de palpanche en acier. *La Technique Moderne* **10**, No. 1, 24–25.
Barkan, D. D. (1957). Foundation engineering and drilling by the vibration method. *Proc. 4th Int. Conf. Soil Mech. Fdn Engng, London* **2**, 3–7.
Barkan, D. D. (1960). *Dynamics of bases and foundations*. McGraw-Hill, New York. Original Russian (1948). Moscow.
Barkan, D. D. (1962). Development of vibrational procedures in the construction of foundations in the USSR. *Annales de L'Institut Techn. du Batiment et des Travaux Publics*, No. 165, 890.
Barkan, D. D. (1967). Developments in soil dynamics. *Proc. Earth Materials Symp*. New Mexico University, 599–606.
Bernhard, R. K. (1967). *Fluidization phenomenon in soils during vibro-compaction and vibro-pile-driving and pulling*. US Army. Cold Regions Research and Engineering Laboratory, Sp. Rep. 106.
Blekhman, I. I. (1954). Investigation of the process of vibration driving of piles. *Inzhenerniyi Sbornik Akad.* Nauk. SSR **19**. Translated by Assoc. Techn. Services Inc., New York. RJ–4524.
Crockett, J. H. A. (1968). Vibration piles, their control and future development. *Nat. Engng Lab. Report* No. 434, 194.
Ermolaev, N. N. & Senin, N. V. (1968). Shear strength of soil with vibration transmission. *Soil Mech. Fdn Engng* **1**, 15–18. Translated by Consultants Bureau.
Ghahramani, A. C. (1967). *Vibration pile driving, ultimate penetration and bearing capacity*. PhD thesis, Princeton University.
Griggs, F. E. (1967). *The pile problem with special emphasis on the vibratory emplacement technique*. Dr Eng. thesis, Rensselaer Poly. Inst.
Hill, H. T. (1966). *Frictional resistance in vibratory pile driving*. PhD thesis, Princeton University.
Kirby, R. (1972). The UCS vibrocorer 1972. NERC Report UCS/10.
Kondner, R. L. & Edwards, R. J. (1960). The static and vibratory cutting and penetration of soils. *Proc. Highway Res. B* **39**, 583–604.
Koushov, M. Y. & Shliaktin, A. V. (1954). On the theory of vibration impact driving of a cylindrical rod into an elasto-plastic medium. *Izvestiya An*, USSR **1**.
Littlejohn, G. S., Seager, D. L. & Rodger, A. A. (1974). Current studies of vibrodriving in ground engineering. *Proc. Symp. Exploitation of Vibration, Nat. Engng Lab.*
Medvedev, S. R. (1953). The use of vibrators for driving sheet steel piling. *Civ. Engng Pub. Wks Rev.* **48**, No. 659, 61–62.
Mogami, T. & Kubo, K. (1953). The behaviour of soil during vibration. *Proc. 3rd Int. Conf. Soil Mech. Fdn Engng, Berne*, **1**, 152–155.
Neimark, Y. I. (1953). Theory of vibration driving. *Tech. Pap. Acad. Science.* (USSR) **5**, 15. Translated by B.L.L.
Parkin, B. P. (1961). Impact waves in sand—implications of an elementary theory. *Am. Soc. Civ. Engrs* S.M.4, 147–163.
Paunescu, M. (1969). Sinking and extracting different structural elements by vibration. *Acta. Techn. Acad. Sci. Hungary* **64**, No. 1–2, 193–204.
Pearson, R. L. F. (1974). Pile driving trials with an electrohydraulic vibrator. *Proc. Symp. Exploitation of Vibration. Nat. Engng Lab.*
Podol'nyy, O. A. (1964). Simulation of vertical drag of the ground during vibratory pile driving. *Acad. Nauk* SSSR Izvestiya, *Techn. Kaya Kibenetika* **4**, 191–192.
Preobrajhenskaja, N. A. (1956). The influence of vibration factors on the penetration of piles and sheet piles. *Pap. of Meeting of Inst. of Fdns.*
Rockefeller, W. C. (1968). Mechanical resonant systems in high power applications. ASME VIBR–S1.

Rodger, A. A. (1975). *Preliminary report on vibrocoring trials held at the National Engineering Laboratory.* Geotechnics Research Group Rep. V/3/75. Aberdeen University.

Rodger, A. A. (1976). *An experimental and theoretical investigation of the parameters influencing the vibrational penetration of dry cohesionless soils.* PhD thesis, Aberdeen University.

Savinov, O. A. & Luskin, A. Y. (1960). *Vibration pile driving and its application in construction practice.* Moscow.

Schmid, W. E. (1969). *Driving resistance and bearing capacity of vibrodriven model piles.* American Society of Testing and Materials Special Techn. Publ. **444**, 362–375.

Schmid, W. E. & Hill, H. T. (1967). A rational dynamic equation for vibro driven piles in sand. *Symp. Dynamic Properties of Earth Materials. New Mexico University*, 349.

Senator, M. (1967). Vibratory penetration of soils. *ASME Engng for Industry* 759–765.

Shekhter, O. Y. (1955). *The effects of the dynamic characteristics of a vibrator on the forced vertical vibrations of piles.* Translated by Nat. Res. Council, Canada (1969), 1502.

Smorodinov, M. I. (1967). Vibro pile driving and hammers. CIRIA Translation 25.

Tsaplin, S. (1953). *Vibratory impact mechanisms for road and bridge construction.* Autotranzizdat, Moscow. Nat. Engng Lab. translation.

Weisflog, W. E. (1968). *Vibration pile driving.* London: Rolba Ltd.

Wu, P. K. (1965). The resistance of soils in vibrosinking of precast reinforced concrete pipe piles of large diameter. *Proc. 6th Int. Conf. Soil Mech. Fdn Engng, Ottawa* **2**, 352–355.

Yang, E. L. C. (1967). *An analytical and experimental investigation of superimposed longitudinal vibration on the rate of penetration of a pile.* PhD thesis, Ohio University.

Youd, T. L. (1967). *The engineering properties of cohesionless soils during vibration.* Thesis, Iowa University.

Acceptance criteria for the service behaviour of ground anchorages

by G. S. LITTLEJOHN*, BSc(Eng), PhD, CEng, FICE, MIStructE, FGS

1. Introduction

WHILST DEGREE OF proof loading and acceptable limits for load-extension behaviour are generally in close agreement throughout the world, by contrast acceptance criteria related to service behaviour are widely divergent in regard to duration of monitoring, and whether load relaxation or creep displacement should be monitored.

Engineers in countries such as Britain, USA, South Africa and Australia tend to favour relaxation criteria, e.g. a prestress loss of up to 5% in 24 hours (Britain), whereas in South America, Continental Europe and Eastern Block countries, engineers prefer creep criteria, e.g. a creep displacement of up to 4mm in 72 hours (France), or a creep rate of less than 0.135mm/m of free tendon for every tenfold increase in time (Czechoslovakia). All these criteria have been used as upper thresholds of acceptability in practice, but it is widely recognised by the specialists concerned that the figures are arbitrary in nature and often incompatible except for a specific free tendon length, cross-sectional area and elastic modulus.

For economic as well as operational reasons the time involved in stressing and testing anchorages on a construction site should be minimised. Thus many engineers have attempted to classify ground which is susceptible to creep, e.g. fine grained as opposed to coarse grained soils in DIN 4125, in order to reduce the period of monitoring down to 1 hour. Since these particle size distinctions are not always reliable for this purpose, a standard sequence of time intervals is ideally required so that only the behaviour of the anchorage dictates the overall period of monitoring and not a prior judgement of the type of ground.

This Paper discusses the interpretation of short-term service behaviour in relation to on-site suitability and routine acceptance tests, with the objective of recommending universally applicable criteria based on load relaxation or an equivalent creep displacement. In addition, it is suggested that short duration acceptance tests of less than 1 hour are possible provided that the accuracy of the monitoring equipment is sufficient to record a trend towards stabilisation.

On-Site Suitability Tests are carried out on anchorages constructed under identical conditions as the working anchorages and loaded in the same way to the same level. The period of monitoring should be sufficient to ensure that prestress or creep fluctuations stabilise within tolerable limits. These tests indicate the results which should be obtained from the working anchorages.

Routine Acceptance Tests are carried out on every anchorage and demonstrate the short-term ability of the anchorage to support a load which is greater than the design working load and the efficiency of load transmission to the fixed anchor zone. A proper comparison of the short-term results with those of the On-Site Suitability Tests provides a guide to longer term behaviour.

*Technical Director, Colcrete Ltd., Rochester, Kent

2. General proposals

For the service monitoring of complete anchorages as part of On-Site Suitability Testing the period of observation should be long enough to provide a predictive capacity for long-term service behaviour. With this background of information equivalent monitoring under Acceptance Testing need only confirm progressive stabilisation and a similar pattern in the short term as that indicated by the On-Site Suitability Tests.

Both load relaxation and creep displacement are important but load is proposed as the major parameter to be monitored since anchorages are designed for structural purposes in the main and working loads with load safety factors are specified. Thus the client or engineer is concerned if load reduces. In addition, load is relatively simple to monitor and also sensitive to fixed anchor displacement, so that both parameters can be measured, creep indirectly. Thus, for a typical tendon having a free length of 10m, a working stress of 1kN/mm² and a Young's modulus of 200kN/mm², a 3mm change of extension is equivalent to a 6% change of load. For a time interval of 1 day it is noteworthy that both these figures are similar to arbitrary limits which are already established in practice (Littlejohn & Bruce, 1977).

It is further proposed that the time intervals are based on Δt equal to 5 minutes, and a sequence of Δt, $3\Delta t$, $10\Delta t$, $30\Delta t$, $100\Delta t$, etc. (Huder, 1978). These intervals may permit short-term acceptance testing of 50 minutes if accurate monitoring ($< 1\%$) is applied, and for each interval a single relaxation or creep criterion can be established which will automatically ensure stabilisation. In such a case the readings when plotted against log time will give a straight line. Whilst the duration of the test and the intermediate time intervals proposed are based on field experience and simplicity, the recommendations should not preclude different observation periods provided that sufficient data are accumulated to permit an accurate assessment of service performance in relation to the acceptance criteria.

A 6% load loss figure is specified in Table I at 1 day based on proximity to current practice, and for the time intervals recommended the rate of prestress loss should reduce to 1% initial residual load or less before the period of monitoring is terminated.

As an alternative to monitoring load relaxation, the creep displacement criteria of Table II are proposed, where 1% Δ_e is the displacement equivalent to the amount of tendon shortening caused by a prestress loss of 1% of initial residual load:

$$\Delta_e = \frac{\text{initial residual load} \times \text{free tendon length}}{\text{area of tendon} \times \text{elastic modulus of tendon}}$$

Based on these concepts the following recommendations are presented for On-Site Suitability Tests and routine Acceptance Tests.

TABLE I. ACCEPTANCE CRITERIA FOR RESIDUAL LOAD-TIME BEHAVIOUR

Period of observation (minutes)	Permissible loss of load (% initial residual load)
5	1
15	2
50	3
150	4
500	5
1 500 (say 1 day)	6
5 000 (say 3 days)	7
15 000 (say 10 days)	8

3. On-Site Suitability Tests

3.1 General

Provision should be made within the terms of a contract for on-site tests to prove the suitability of the anchorages for the conditions on site.

They should be constructed in exactly the same way and located in the same ground as the working anchorages and should be used as standards against which the performance of the working anchorages can be judged.

At least the first three anchorages should be subjected to Suitability Tests

TABLE II. ACCEPTANCE CRITERIA FOR DISPLACEMENT-TIME BEHAVIOUR AT RESIDUAL LOAD

Period of observation (minutes)	Permissible displacement (% of elastic extension, Δ_e, of tendon at initial residual load)
5	1
15	2
50	3
150	4
500	5
1 500 (say 1 day)	6
5 000 (say 1 days)	7
15 000 (say 10 days)	8

TABLE III. RECOMMENDED LOAD INCREMENTS AND PERIODS OF OBSERVATION FOR ON-SITE SUITABILITY TESTS

Temporary anchorages		Permanent anchorages		Period of observation (minutes)
Load increment (% T_w)		Load increment (% T_w)		
1st load cycle*	2nd & 3rd load cycles	1st load cycle*	2nd & 3rd load cycles	
20	20	20	20	5
	40		40	5
50	60	50	60	5
	80		80	5
100	100	100	100	5
	120		120	5
			140	5
125	125	150	150	15
100	100	100	100	5
50	50	50	50	5
20	20	20	20	5

*For this load cycle there is no pause other than that necessary for the recording of extension data

with further tests for each category of anchorages envisaged in the works. Anchorages are categorised by (*a*) geometry, e.g. vertical or inclined, and (*b*) ground type, e.g. clay, or gravel.

3.2 Proof loads

The maximum proof load should generally be 125% T_w and 150% T_w for temporary and permanent anchorages, respectively, where T_w is the working load of the anchorage.

3.3 Load-extension data

Load-extension data should be plotted continuously over the range 20 to 125% T_w for temporary anchorages (20 to 150% T_w for permanent anchorages) with load increments not greater than 20% T_w where extensions are being carefully monitored. During unloading, extensions at not less than two load decrements in addition to datum, should be measured preferably occurring at one third points with respect to the proof load (Table III).

Each stage loading in the 2nd and 3rd cycles should be held for at least 5 minutes and the extension recorded at the beginning and end of each period. For proof loads this period is extended to at least 15 minutes with an intermediate extension reading at 5 minutes. On completion of the 3rd load cycle, reload in one operation to 110% T_w and lock-off. Re-read the load immediately after lock-off to establish the initial residual load. This moment represents zero time for monitoring load/displacement-time behaviour (3.6, 3.7).

3.4 Proof load-time data

If the proof load has not reduced during the 15 minutes by more than 5% after allowing for any temperature changes, and movements of the anchored structure, the anchorage may be deemed to have satisfied this stage. If a greater loss of prestress is recorded, this should be investigated and a diagnosis recorded.

3.5 Displacement-time data at proof load

As an alternative to **3.4** the proof load can be maintained by jacking and the anchor head displacement monitored after 15 minutes. If the creep is less than 5% Δ_e the anchorage may be deemed to have satisfied this stage.

If a greater displacement is recorded, this should be investigated and a diagnosis recorded.

3.6 Residual load-time data

Load-time data should be monitored commencing at 110% T_w and continuing for 10 days with observation periods in accordance with Table I and using either load cells or grade *A* pressure gauges.

Where the load has not attained a constant value after allowing for temperature, structural movements and relaxation of the tendon, the above test should be extended by monitoring at 7-day intervals approximately for a period up to 30 days or until the load becomes constant, whichever is the lesser period.

Readings within the first 1 500 minutes should only be attempted where the monitoring equipment has a relative accuracy* of at least 0.5%. Where the monitoring involves a stressing operation, e.g. lift-off check without load cell, an absolute accuracy† less than 5% is unlikely and the observation periods are 1, 3 and 10 days, although more frequent observations may be made if considered appropriate.

Where the loss of load is monitored accurately the rate of loss from the initial residual load should reduce to 1% or less per time interval for the observation periods (Table I). Alternatively, where less accurate monitoring is applied, losses should not exceed 6%, 7% or 8% of initial residual load at 1, 3 and 10 days, respectively. For prestress gains see **4.10**.

3.7 Displacement-time data at residual load

As an alternative to **3.6** displacement-time data may be monitored commencing at 110% T_w and continuing for 10 days with observation periods in accordance with Table II and using dial gauges or steel rule.

Where the displacement has not reached a constant value after allowing for temperature, structural movements and creep of the tendon, the above test should be extended by monitoring at 7 day intervals approximately for a period up to 30 days or until the displacement becomes constant, whichever is the lesser period.

Restressing or constant load methods may be used to monitor the displacement at initial residual load. At each monitoring period the anchorage may be restressed and the increment of tendon displacement (ram extension may be sufficient if the bearing plate is fixed) to regain the lock-off load (initial residual load) is recorded after which the stressing load is released. Alternatively, the load can be held constant with the aid of the jack pump and the displacement of the tendon with time may be measured direct (Fig. 1). This method is particularly suited to short duration testing. In both cases, however, the datum for the displacement readings, e.g. bearing plate for restressing system or the tripod base (Fig. 1) for the constant load system, should be surveyed accurately for movement, otherwise the displacement readings may be erroneous.

Rate of displacement should reduce to 1% Δ_e or less per time interval for the observation periods in Table II.

Where less accurate monitoring is applied, displacement should not exceed 6% Δ_e, 7% Δ_e or 8% Δ_e at 1, 3 and 10 days, respectively.

3.8 Number of load or displacement measurements

In order to minimise errors, particularly where a restressing operation is involved without a load cell, e.g. at 1, 3 and 10 days, each reading for **3.6** or **3.7** should be taken at least three times and the results averaged.

3.9 Final lock-off

If the anchorages are to be used in the works, and on completion of the on-site suitability test the cumulative relaxation or creep has exceeded 5% initial residual load or 5% Δ_e respectively, the anchorage should be restressed and locked-off at 110% T_w.

4. On-Site Acceptance Tests

4.1 General

Every anchorage used on a contract should be subjected to an acceptance test in accordance with **4.2-4.7** with the exception of low capacity tensioned rock bolts used in secondary reinforcement,

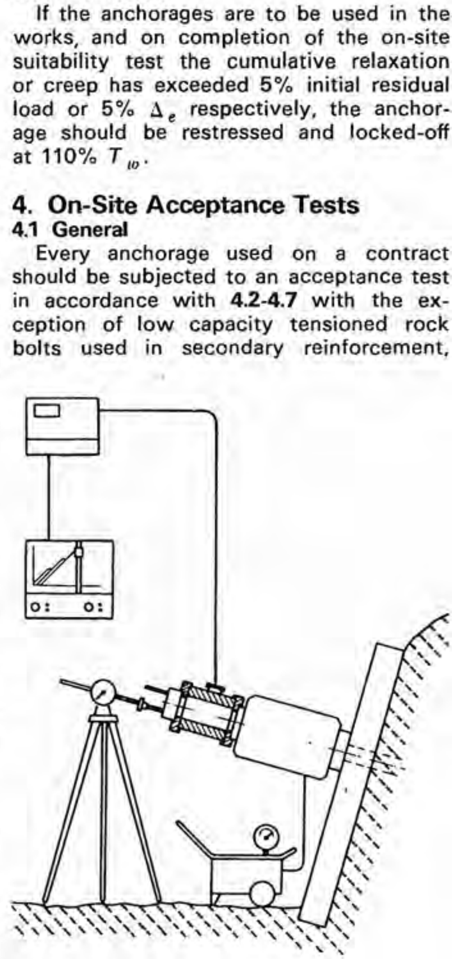


Fig. 1. Typical method of measuring tendon displacement using a dial gauge

* *Relative accuracy* refers to the deviation from the measured value, i.e. the error in measurement where small changes in load or displacement are monitored against time.

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

where the anchorage may be loaded to the proof load **(3.2)**, checked for fixed anchor displacement and then locked off at 110% T_w. For guidance the permanent fixed anchor displacement should not exceed 20mm and 5mm for mechanical anchorages, e.g. expansion shell, and straight shaft anchorages, e.g. cementitious or resin cartridge, respectively, otherwise an investigation as to the cause and need for additional anchorages should be undertaken.

4.2 Proof loads

The maximum proof load should be in accordance with **3.2.**

4.3 Load-extension data

Load-extension data should be plotted continuously over the range 20 to 125% T_w for temporary anchorages (20 to 150% T_w for permanent anchorages) using load increments not more than 25% T_w where extensions are being carefully monitored. During unloading, extensions at not less than two load decrements, in addition to datum, should be measured preferably occurring at one-third points with respect to proof loads (Table IV).

Each stage loading in the 2nd cycle should be held for at least 5 minutes and the extension recorded at the beginning and end of each period. For proof loads this period is extended to at least 15 minutes, with an intermediate extension reading at 5 minutes.

On completion of the 2nd load cycle, reload in one operation to 110% T_w and lock-off. Re-read the load immediately after lock-off to establish the initial residual load. This moment represents zero time for monitoring load/displacement-time behaviour.

4.4 Proof load-time data

The proof load-time data should be in accordance with **3.4.**

4.5 Displacement-time data at proof load

The displacement-time data should be in accordance with **3.5.**

4.6 Residual load-time data

Using accurate monitoring equipment the residual load may be monitored at 5, 15 and 50 minutes.

If the rate of load loss reduces to 1% or less per time interval for the specific observation periods above after allowing for temperature, structural movements and relaxation of the tendon in accordance with the manufacturer's data, the performance of the anchorage is satisfactory. If the rate of load loss exceeds 1%, further readings may be taken at observation periods up to 10 days (Table I).

Alternatively, where less accurate monitoring is applied, e.g. lift-off check without load cell, if the total loss at 1 day does not exceed 6% of initial residual load the performance of the anchorage is satisfactory. If the load loss exceeds 6%, further observations may be taken at 3 days, and if necessary at 10 days, when the total loss should not exceed 7% or 8% respectively.

If, after 10 days the anchorage fails to hold its load in accordance with Table II, the anchorage should be deemed to have failed.

Following an investigation as to the cause of failure and dependent upon the circumstance the anchorage should be (*i*) abandoned and replaced, (*ii*) reduced in capacity, or (*iii*) subjected to a remedial restressing programme **(4.10)**.

4.7 Displacement-time data at residual load

As an alternative to **4.6** displacement-time data may be obtained at the specific observation periods of **4.6.** Restressing or constant load methods may be used to monitor the displacement at initial residual load **(3.7)**.

Using accurate monitoring equipment, if the rate of displacement reduces to 1% Δ_e or less per time interval for the observation periods 5, 15 and 50 minutes, after allowing for temperature, structural movement and creep of the tendon in accordance with the manufacturer's data, the performance of the anchorage is satisfactory. If the rate of displacement exceeds 1% Δ_e, further readings may be taken at observation periods up to 10 days (Table II).

Where less accurate monitoring is applied, e.g. lift-off check without load cell, if the total displacement at 1 day does not exceed 6% Δ_e, the performance of the anchorage is satisfactory. If the displacement exceeds 6% Δ_e, further observations may be taken at 3 days, and if necessary at 10 days, when the total displacement should not exceed 7% Δ_e or 8% Δ_e respectively.

If after 10 days the anchorage fails to hold the displacement in accordance with Table II the anchorage should be deemed to have failed, and subsequent actions should be in accordance with **4.6.**

4.8 Final lock-off

On completion of the acceptance test, if the cumulative relaxation or creep exceeds 5% initial residual load or 5% Δ_e, respectively, the anchorage should be restressed and locked-off at 110% T_w.

4.9 Interaction of anchorages

Where fixed anchors are closely spaced e.g. less than 1m, or anchor heads are located on a single waling or structural unit, or a group of anchorages ties back a re-entrant corner, interaction between anchorages may occur during stressing and subsequent service. When testing an isolated anchorage in such circumstances it may be prudent to check adjacent anchorages during the same period, preferably one day, even if an acceptance test has already been carried out on some of the anchorages in question (Littlejohn & Macfarlane, 1974).

4.10 Remedial action for failed anchorages

Where an anchorage fails at the ground/grout interface, a first estimate of the new load may generally be taken as the maximum load at failure divided by 1.6 or 2.0 for temporary and permanent anchorages, respectively.

Where the anchorage has passed its proof-loading and failure is solely related to the relaxation or creep criterion **(4.6** or **4.7)** a provisional reduction divisor of 1.2 is tentatively recommended in the absence of field data at the present time, and service monitoring should be repeated at the new reduced load in accordance with **4.6** or **4.7.**

Where a remedial stressing programme is considered appropriate, the initial residual load (110% T_w) is regained by stressing, and service monitoring **(4.6 or 4.7)** is repeated. This principle has been applied successfully in stiff/hard clay where the preliminary stress history provides a preloading effect (Littlejohn, 1970) thereby consolidating the ground local to the fixed anchor, which in turn gives an enhanced performance during subsequent service.

Where prestress gains are recorded monitoring should continue to ensure stabilisation of prestress within a load increment of 10% T_w. Should the gain exceed 10% T_w a careful diagnosis is required to ascertain the cause and it will be prudent to monitor the overall structure/ground/anchorage system. If, for example, overloading progressively increases due to insufficient anchorage capacity in design or failure of a slope, then additional support is required to stabilise the overall anchorage system. Destressing to working load values should be carried out as prestress values approach proof loads, e.g. 120% and 140% T_w in the case of temporary and permanent anchorages, respectively, accepting that movements may continue until additional support is provided.

TABLE IV. RECOMMENDED LOAD INCREMENTS AND PERIODS OF OBSERVATION OF ON-SITE ACCEPTANCE TESTS

Temporary anchorages		Permanent anchorages		Period of observation
Load increment (% T_w)		Load increment (% T_w)		
1st load cycle*	2nd load cycle	1st load cycle*	2nd load cycle	(minutes)
20	20	20	20	5
50	50	50	50	5
	75		75	5
100	100	100	100	5
			125	5
125	125	150	150	15
100	100	100	100	5
50	50	50	50	5
20	20	20	20	5

*For this load cycle there is no pause other than that necessary for the recording of extension data

5. Relationship between relaxation and creep acceptance criteria

Table V illustrates by worked example the relationship between the acceptance criteria for load-time (Table I) and displacement-time (Table II), and their respective sensitivities to initial residual load (100kN and 1 000kN) and free tendon length (5m, 10m and 20m) for observation periods of 5 min, 15 min, 50 min and 1 500 min (say 1 day).

Tendon details:

Nominal area of single strand	=	100mm^2
Elastic modulus	=	200kN/mm^2
Initial residual load (1 strand)	=	100kN
Initial residual load (10 strands)	=	1 000kN

TABLE V. RELATIONSHIP BETWEEN LOAD-TIME AND DISPLACEMENT-TIME ACCEPTANCE CRITERIA

Period of observation (minutes)	Free tendon length (metres)	Limiting loss of load		Limiting creep displacement	
		Single strand (kN)	Ten strands (kN)	Single strand (mm)	Ten strands (mm)
5	5	1	10	0.25	0.25
	10	1	10	0.5	0.5
	20	1	10	1	1
15	5	2	20	0.5	0.5
	10	2	20	1	1
	20	2	20	2	2
50	5	3	30	0.75	0.75
	10	3	30	1.5	1.5
	20	3	30	3	3
1 500 (1 day, say)	5	6	60	1.5	1.5
	10	6	60	3	3
	20	6	60	6	6

For the common range of free tendon lengths quoted either acceptance criterion may be applied quite independently. For short free tendon lengths (< 5m), rate of prestress loss becomes the more appropriate criterion, whilst for long free tendon lengths (> 30m) it is clear that rate of displacement is the more important parameter to limit and therefore more appropriate as an acceptance criterion. To take account of free tendon length in the example quoted, a single creep criterion of 0.05mm/m of free tendon length per time interval would be appropriate. On some contracts with a wide variety of tendon lengths it may be more convenient to specify a limiting creep criterion in such units.

6. Stressing and monitoring equipment

6.1 General

As a consequence of reducing the period of monitoring for acceptance tests, more accuracy and control are required on site, which implies careful choice of appropriate equipment and regular calibration.

6.2 Stressing equipment

Stressing equipment for wire, bar and strand tendons should preferably tension the whole of the tendon in one operation. However, both single unit and multi-unit operations are used in practice.

The design of the jack should permit the tendon elongation at every stage to be measured to an accuracy appropriate for the test requirements. Accuracy of reading may be as low as ± 0.2mm for short duration (< 1 hour) testing of rate of relaxation or creep but for conventional proof-loading cycles or long duration testing (> 1 day), an accuracy of ± 1mm should normally be sufficient.

Hydraulic pumps should be rated to operate through the pressure range of the stressing jack. The controls of the pump should allow the tendon extension to be easily adjusted to the nearest millimetre whether the jack is opening or closing. The pressure gauge should be mounted such that it is reasonably free of vibration during pumping.

6.3 Load cells

Where the basic characteristics of a load cell are being established by the manufacturer, consideration should be given to the following series of tests in order to simulate the service conditions to which the load cell may be subjected, e.g. eccentric loading effects (McLeod & Hoadley, 1974).

(*i*) Routine calibration using centric loading and rigid flat platens at 20°C, say.
(*ii*) As in (*i*) but using (*a*) concave inclined platens, (*b*) convex inclined platens and (*c*) 0.3mm sheets with irregular spacing to simulate uneven bedding (Fig. 2).
(*iii*) Eccentric loading between rigid flat plates, with eccentric distance up to 10% cell diameter.
(*iv*) If torsion is anticipated during service, an appropriate torque should be applied during a test between rigid flat platens to gauge the effect.
(*v*) Inclined platens up to 1° with centric loading.
(*vi*) On completion of the appropriate series of tests, the cell should finally be subjected to a repeat routine calibration (*i*).

For routine calibration the load cell should be delivered to the laboratory at least one day before the test to permit sufficient time for the cell to attain the correct ambient temperature (20°C). The cell should be subjected to centric loading between rigid flat platens using a testing machine with an absolute accuracy not exceeding 0.5%.

Bearing in mind that the load cell may not have been used for some time, it may be prudent to load cycle the cell two or three times over its full loading range until the zero and maximum readings are consistent. The load increments and decrements should not exceed 10% of the cell's rated capacity and short pauses at these intervals need only be long enough to take careful readings.

To measure the specific effects of temperature, a centric loading test using rigid flat platens should be carried out at temperatures above and below ambient (20°C), say 40°C and 0°C, respectively.

For each individual test the absolute accuracy should be monitored. Where a worst combination of circumstances is envisaged this situation should be simulated since the total error is not necessarily the sum of the individual errors.

The information created from the series of tests above should be compiled into a basic specification, together with any long-term stability results. In addition a recommended operating range should be indicated, e.g. 10-100% of rated capacity.

The resolution of the read-out equipment should be appropriate for the accuracy specified, and accuracies down to 1-10kN are available. Wherever possible read-out equipment should be calibrated along with the load cell.

Load read-out or recording instruments should not have more than 10m of electrical cable and should be calibrated with the actual cable to be used on site. The instrument should be provided with input voltage indicators whether mains or battery operated.

6.4 Frequency of calibration

Jacks should be calibrated at least every year using properly designed test equipment with an absolute accuracy not exceeding 0.5% and the test records should tabulate the relationship between the load carried by the jack and the hydraulic pressure when the jack is in the active mode with load both increasing and decreasing.

The jack calibration should be checked prior to the start of tensioning on each contract and a calibration curve prepared for each jack.

The calibration should extend from zero over the full working range of the jack and should be established for the opening (load rising) and closing (load falling) operation of the jack so that the friction hysteresis can be known when repeated *(concluded on page 36)*

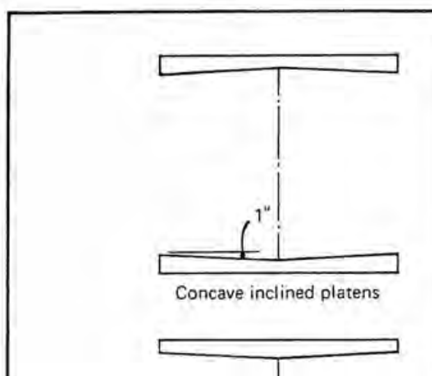


Fig. 2. Typical types of platen to simulate uneven bedding

loading cycles are being carried out on the tendon.

Pressure gauges should be calibrated either every 100 stressings or after every 30 days, whichever is the more frequent, against properly maintained Class *A* gauges, or whenever they have been subjected to shock. If a group of three gauges is employed in parallel this frequency of calibration does not apply.

Load cells should be calibrated every 200 stressings or after every 60 days use, whichever is the more frequent, unless complementary pressure gauges used simultaneously indicate no significant variation, in which case the interval between calibrations may be extended up to a maximum of one year when a routine calibration should be carried out using properly designed test equipment with an absolute accuracy not exceeding 0.5%.

7. Final remarks

During acceptance testing of production anchorages one of the prime objectives is to ensure that the service load locked-off after stressing is stable.

The alternative methods employed in practice of monitoring rate of load relaxation or rate of creep displacement are made compatible in these proposals, and a standard series of time intervals is recommended when monitoring either parameter.

The shorter the time scale the greater the accuracy of measurement required. Where a relative accuracy of 0.5% can be provided the minimum period of monitoring is 50 minutes c.f. one day for simple lift-off checks.

To give a background of service behaviour against which to judge the performance of production anchorages, at least three On-Site Suitability Tests are recommended where accurate high frequency testing over a period of hours is combined with a minimum overall period of observation of 10 days.

It is hoped that this routine collection of data related to relaxation or creep for different types of ground and anchorage load and geometry will improve understanding of the service behaviour of anchorages and lead to improved design procedures in future. In the short term such data can establish that overload allowances applied to the working load at initial lock-off are adequate. At the present time an overload of 10% T_w is commonly applied which appears to be realistic in most cases.

References

Huder, J. (1978): "Boden-und Felsanker: Andforderungen, Prufung und Bemessung — Die neue Norm SIA 191". Schweizerische Bauzeitung 96, Jahrgang Heft 40 (5 Oct), 753-761.

Littlejohn, G. S. (1970): "Soil anchors". Proc. Conf. on Ground Engineering, pp. 33-44. Institution of Civil Engineers, London.

Littlejohn, G. S. & *Macfarlane, I. M.* (1974): "A case history study of multi-tied diaphragm walls". Proc. Conf. on Diaphragm Walls and Anchorages, pp. 113-121. Institution of Civil Engineers, London.

Littlejohn, G. S. & *Bruce, D. A.* (1977): "Rock anchors: state-of-the-art". Foundation Publications Ltd., High Street, Brentwood, Essex, CM14 4AH, England.

MacLeod, J. & *Hoadley, P. J.* (1974): "Experience with the use of ground anchors". Proc. Tech. Session on Prestressed Concrete Foundations and Ground Anchors, pp. 83-85. 7th FIP Congress, New York.

Design of Cement Based Grouts

by G.S. Littlejohn
BSc(Eng), PhD, CEng., FICE, MIStruct.E, FGS
Technical Director, Colcrete Ltd, Rochester,
England

Abstract
Following a description of the main constituent materials of cement based grouts, the paper discusses major design characteristics such as bleeding, fluidity, setting, shrinkage, thermal properties, strength and durability, and provides predictive equations, together with test results.

1. INTRODUCTION

Cement grouts should be sufficiently fluid to allow efficient pumping and injection, and sufficiently stable to resist displacement and erosion after injection. Where a grout is used to waterproof zones of fractured rock or soil, fluidity, particle size and bleeding are important, whereas for grouted aggregate concrete, shrinkage, heat of hydration and strength may be the more dominant parameters in design. Cement grouts are formed basically from ordinary portland cement (Type I) and water. Occasionally other cements are used to obtain high early strength (Type III) low heat of reaction (Type IV) or resistance to chemical attack (Type V). Other solid materials such as fine sand, flyash or clay are added for economy or to obtain special grout characteristics. In this regard chemical admixtures designated according to their action as anti-bleed agents, fluidifiers, accelerators, retarders and expansion agents, may also be incorporated.

The principal variable affecting the properties of cement grouts is the water/cement ratio (w), the amount of water determining the rate of bleeding, subsequent plasticity and ultimate strength of the grout. The extent to which these, and also fluidity, are related to w of neat Type I cement grout is shown in Fig 1. Excess of water causes bleeding, low strength, increased shrinkage and poor durability.

The purpose of this paper is to provide a simple guide to the main factors determining grout properties and thereby provide a logical basis for ascertaining the optimum mix design for a specific grouting application.

2. CONSTITUENT MATERIALS

Water

As a general rule water which is suitable for drinking, except for the presence of bacteria, is suitable for cement grout formulation. On the other hand water containing sulphate (>0.1%), chloride (>0.5%), sugars, suspended matter such as algae, or high alkali content is technically dangerous particularly for high strength applications in the presence of steel e.g. duct grouting. If structural steel is not involved seawater can be employed e.g. underbase grouting of concrete platforms offshore. Satisfactory field experience of seawater concrete in fact dates back some forty years[1].

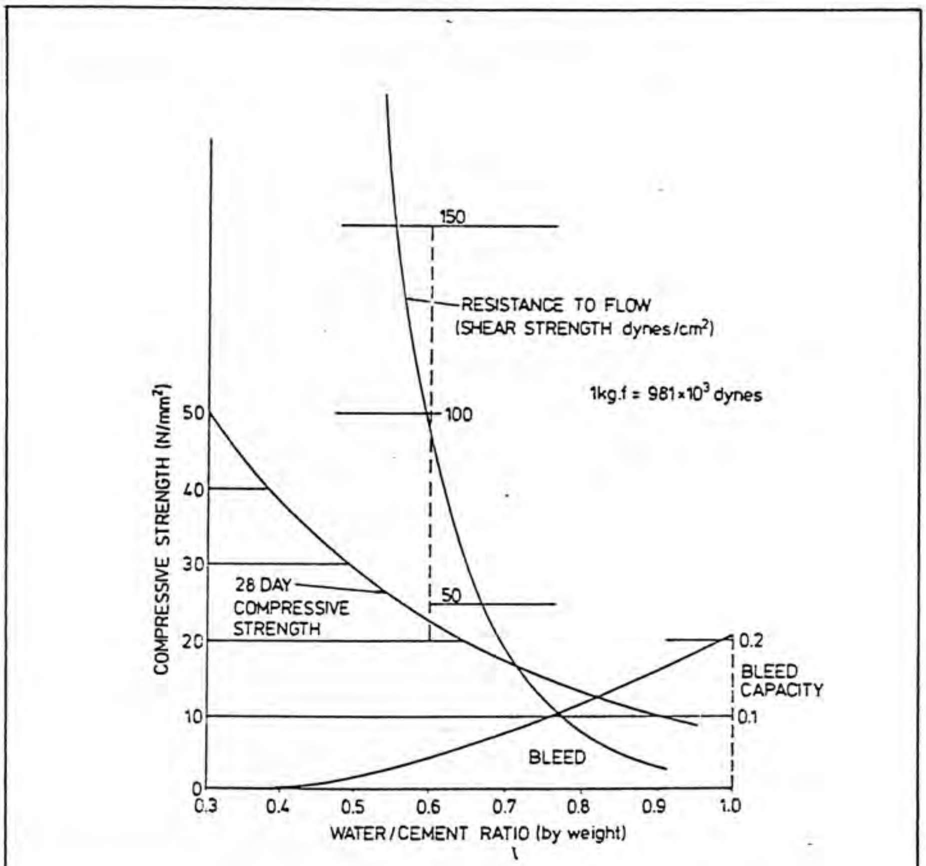


Fig 1. Effect of Water Content on Grout Properties

	C_3S	C_2S	C_3A	C_4AF	Others
Ordinary (ASTM-I)	45	27	11	10	7
High Early Strength (ASTM-III)	55	17	11	9	8
Low Heat (ASTM-IV)	30	46	5	13	6
Sulphate Resisting (ASTM-V)	45	35	4	10	6

Table 1. Typical Percentage Compositions of Portland Cements

Cement

The most common form of hydraulic cement is composed of the following compounds:

Tricalcium silicate ($3CaO.SiO_2$) – C_3S

Dicalcium silicate ($2CaO.SiO_2$) – C_2S

Tricalcium aluminate ($3CaO.Al_2O_3$) – C_3A

Tetracalcium aluminoferrite ($4CaO.Al_2O_3.Fe_2O_3$) – C_4AF

Gypsum is also included to slow down the set. Magnesium oxide, free lime and silica are present in minor quantities.

The main compounds behave quite differently and their relative proportions dictate cement properties (Table 1). C_3A virtually flash sets, great heat is evolved and some strength is attained at 1 day but no increase thereafter. C_4AF sets in minutes, with some strength development but less heat is evolved c.f. C_3A. C_3S sets in a few hours and attains almost full strength in 7 days. By contrast, C_2S sets very slowly with low heat but ultimately attains the strength of C_3S.

For most cements the maximum practical size (99% passing) ranges from 44-100μ (Fig 2) and these particle sizes limit penetration of cement grout to soils with a permeability less than $5x10^{-4}$m/sec, or fissures in rock of width less than 160μ unless fracturing pressures are used.

Fillers

Often the main purpose of fillers is to reduce the overall cost of the grout, without affecting significantly its physical properties. Certain fillers, however, give technical advantages, e.g. reduced bleeding, improved fluidity or retardation. Three basic types are used; namely, pozzolans, sands and clays. Of these sands and clays are basically inert while pozzolans are reactive.

Clays with their capacity to absorb water and ability to form gel structures even at low concentrations as well as their small particle size are used to stabilise the cement, thus preventing its bleeding. The clay performs no important function in the final chemical set resulting from hydration of the cement, but development of strength is slow and there is no well-defined setting time. The special characteristics favourable to grout formulation are possessed most markedly by the sodium, and to a lesser extent by the calcium montmorillonites rather than by the kaolinites and illites which are mainly used as fillers.

In rock grouting and consolidation of soil where strength is required, the clay content is kept to a minimum by using bentonite (2-5% by wt. of water) for w = 1-3. For clay/cement grouts where high proportions of clay (50%) are employed, there is little demand for strength and the clay filler simply increases the volume yield per unit weight or cost of material.

Pozzolans, as silicates and aluminosilicates, are not themselves cementitious, but will react with free lime cement in the presence of water to form a cementitious compound. Naturally occurring pozzolans include finely ground shale, pumicite and diatomite. Flyash a combustion by-product of pulverised coal, and ground blast-furnace slag are examples of artificial pozzolans. Since flyash and slag are waste products they are normally used as cheap bulk fillers in low strength grouts for cavity filling eg consolidation of old coal workings. Precipitator flyash is preferred to lagoon ash since the latter is more variable. In this regard chemical composition is a useful indicator of variability but it is more usual to simply monitor grading, specifically maximum particle size and % passing the 45μ sieve, moisture content and % combustibles.

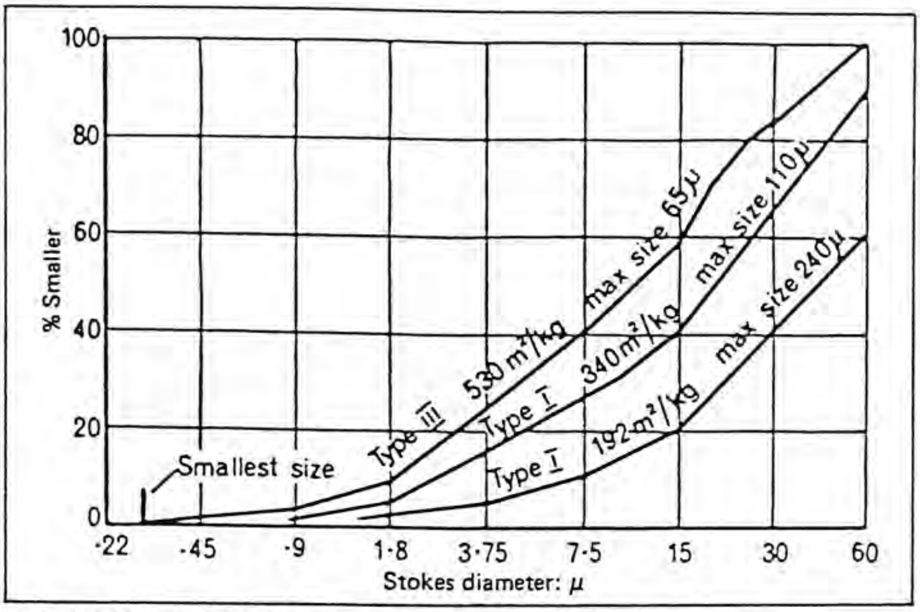


Fig 2. Size Analysis Curves for Cement

Admixture	Chemical	Optimum Dosage (% cement wt)	Remarks
Accelerator	Calcium Chloride	1-2	Accelerates set and hardening
	Sodium Silicate	0.5-3	Accelerates set
	Sodium Aluminate		
Retarder	Calcium Lignosulphonate	0.2-0.5	Also increases fluidity
	Tartaric Acid	0.1-0.5	
	Sugar	0.1-0.5	
Fluidifier	Calcium Lignosulphonate	0.2-0.3	
	Detergent	0.05	Entrains air
Air Entrainer	Vinsol Resin	0.1-0.2	Up to 10% of air entrained
Expander	Aluminium Powder	0.005-0.02	Up to 15% pre-set expansion
	Saturated Brine	30-60	Up to 1% post-set expansion
Anti-Bleed	Cellulose Ether	0.2-0.3 (for w < 0.7)	Equivalent to 0.5% of mixing water
	Aluminium Sulphate	Up to 20% (for w < 5)	Entrains air

Table 2. Common Cement Admixtures

A maximum particle size of 0.5mm is recommended, and the amount retained at 45μ can vary from 15 to 25%. Fineness is also related to pozzolanicity, thus finer ashes (less than 10% retained at 45μ) react exothermically, contributing much earlier to strength than do coarser ashes (between 20 and 30% retained at 45μ). These coarser ashes contribute little to early heat, but do not give such significant water reductions as the finer ashes. As a further quality control, there is good grouting experience where % loss on ignition <5%, and moisture content <10%.

Fine sands can be added to neat cement/water suspensions to form an economical grout particularly where a high solids, low water grout with relatively high frictional shear strength is required. Sand is chosen as for concrete in relation to durability, shrinkage, and alkali reaction, and in general hard bulky crushed rock is preferred to flat, angular or flaky material which gives poor fluid handling properties. Evenly graded sands are preferred (5mm down to 75μ) but for long pumping distances in excess of 300 metres the maximum size should ideally be reduced to 0.5mm and the maximum sand/cement ratio limited to 3 to maintain the particles in suspension and avoid segregation.

Admixtures can be added in relatively small quantities to modify grout properties (Table 2). Most commercial admixtures are compatible with Type I and III Portland cements, but many are incompatible with High Alumina and Supersulphated cements. Admixtures should not be regarded as a replacement for good grout practice nor be used indiscriminately. In general their suitability should be varified by trial mixes, and if two or more admixtures are proposed for a mix, the manufacturer of each should be consulted.

3. BLEEDING

During mixing the cement particles are dispersed and suspended in water. Except in the case of a very dense paste, the resultant suspension is initially unstable and the cement particles settle under gravity. This bleeding mechanism is important in grout design particularly the bleeding rate and the final volume of bleeding (bleed capacity). The initial rate of bleeding (Fig 3) can be estimated from D'Arcy's Law for $w<1$ assuming acquiescent conditions.

$$q = \frac{9}{32}\,\frac{dm^2}{Y}\,\frac{Dc-Dw}{Dw}\,\frac{w^2}{(1+3w)} \quad \ldots\ldots\ldots \quad 1$$

where q = initial rate of bleed, dm = equivalent spherical particle diameter of cement in suspension, Y = kinematic viscosity of water, Dc = density of cement, Dw = density of water and w = w/c ratio by weight.

In less dense suspensions where particles are settling rapidly an analysis based on Stokes Law is more realistic. Fig 4 shows results of bleed tests on high w grouts.

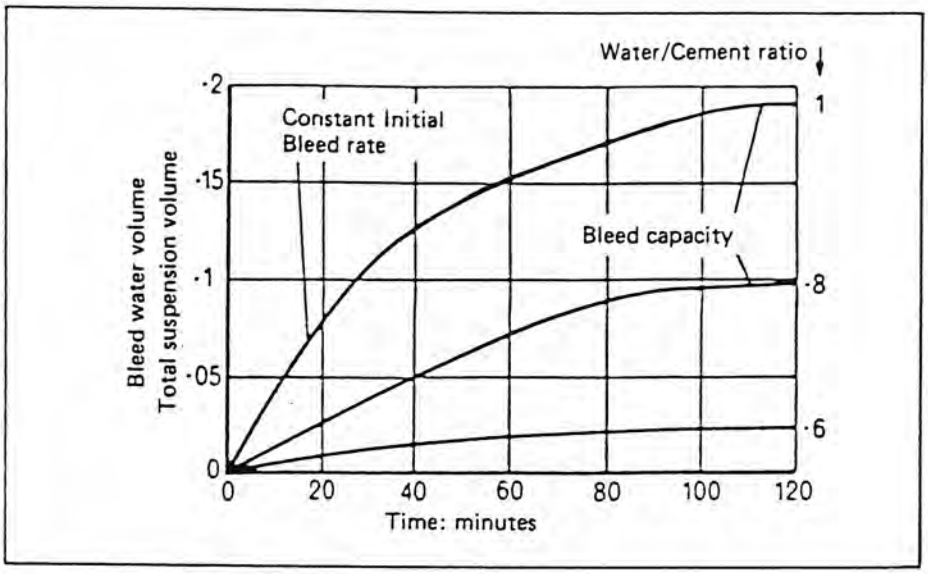


Fig 3. Rates of Bleed (Type I)

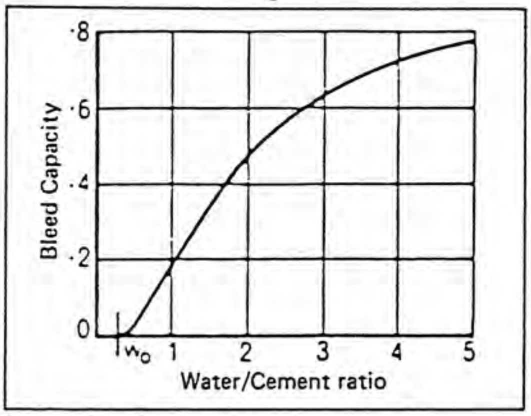


Fig 4. Effect of w on Bleed Capacity

4. FLOW PROPERTIES

Flow properties are affected principally by dynamic interparticle forces of attraction and repulsion and, in dense grouts, by dilatency of the moving particles. A dense grout can only be pumped easily when it contains sufficient fluid to prevent expansion of the particle matrix during shear and, generally speaking a wide well graded range of particles is preferred since the better the grading the lower the critical porosity at which the grout becomes pumpable. A reasonable percentage of fine particles is also desirable to increase the specific surface of the grout particles and thereby slow the separation of the liquid and solid phases.

Under conditions of laminar flow, cement grouts behave as Bingham fluids. The shear stress (T) necessary to cause the grout to flow at a constant rate of strain is given by

$$T = T_s + Y_p\frac{dS}{dt} \quad \ldots\ldots\ldots\ldots\ldots \quad 2$$

where T_s = initial shear strength,
Y_p = coefficient of plastic viscosity and
$\frac{dS}{dt}$ = rate of shear strain or velocity gradient.

Typical flow curves in Fig 5 indicate a rapid increase in viscosity and shear strength at $w<0.9$. The relationship between plastic viscosity and w may be expressed approximately in the form of the Arrhenius equation[2].

$$Y_p = Y_o e^{k/w} \quad \ldots\ldots\ldots\ldots\ldots \quad 3$$

where Y_o = viscosity of water (0.01 poise) and K = constant (range = 1.6-2.2, and 1.8 assumed for Table 3).

Based on the work of Ish Shalom & Greenburg[3] and Papadakis[4], a similar relationship can be established between shear strength and w.

$$T_s = T_o e^{k/w} \quad \ldots\ldots\ldots\ldots\ldots \quad 4$$

where To = 3.5 dynes/cm² and K = 2.1.

From equations 3 and 4, Table 3 has been compiled to relate the critical flow properties of a neat cement grout to its water/cement ratio. Bearing in mind that the most sophisticated chemical grouts e.g. resorcinol formaldehyde, have viscosities no lower than 0.015 poise, the traditional practice in fissure grouting of gradually thickening the cement grout starting from w = 20 hardly seems justified. As a starting point w = 5 would seem quite adequate from Table 3 and the volume of water for transport of cement is adequate with bleeding at ·5 (Fig 4).

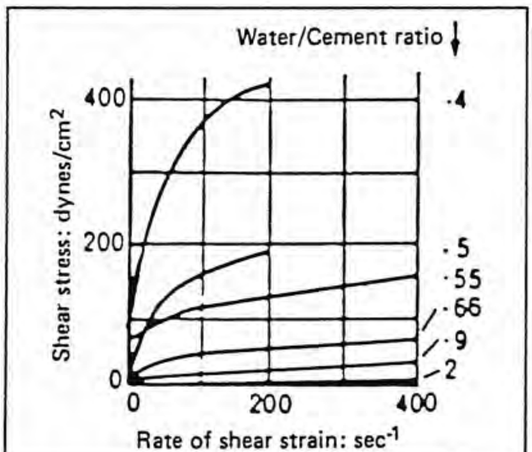


Fig 5. Flow Curves for Neat Cement Grouts

w	Shear Strength Ts (dynes/cm²)	Plastic Viscosity Yp (poise)
0.3	3840	4.03
0.4	670	0.90
0.5	230	0.37
0.6	120	0.20
0.7	70	0.13
1.0	29	0.06
2.0	10	0.025
5.0	5.3	0.014
10.0	4.3	0.012
20.0	3.9	0.011
Water	0	0.010

Table 3 – Shear Strength and Plastic Viscosity of Neat Cement Grouts

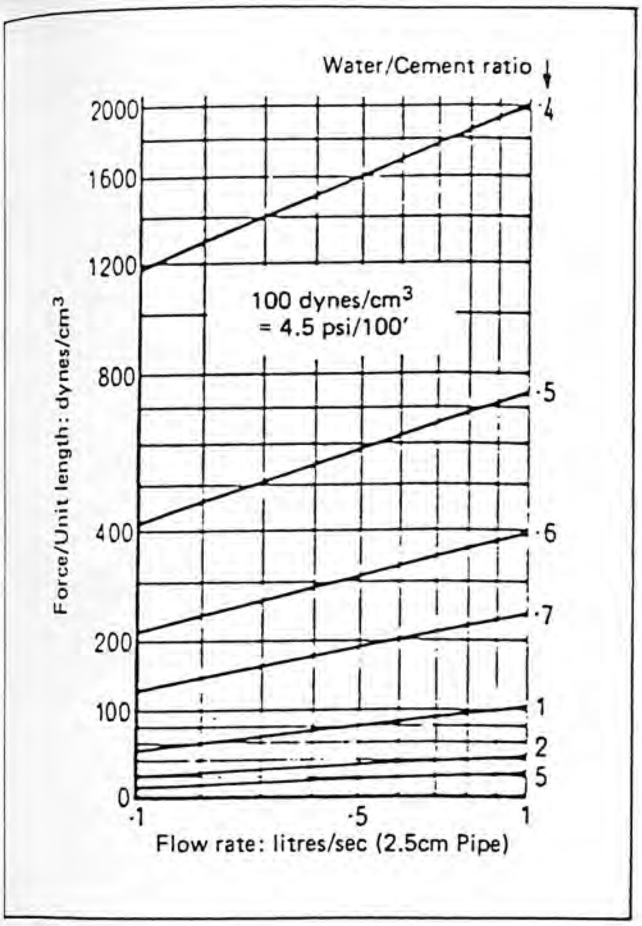


Fig 6. Pipe Flow Curves for Neat Cement Grouts

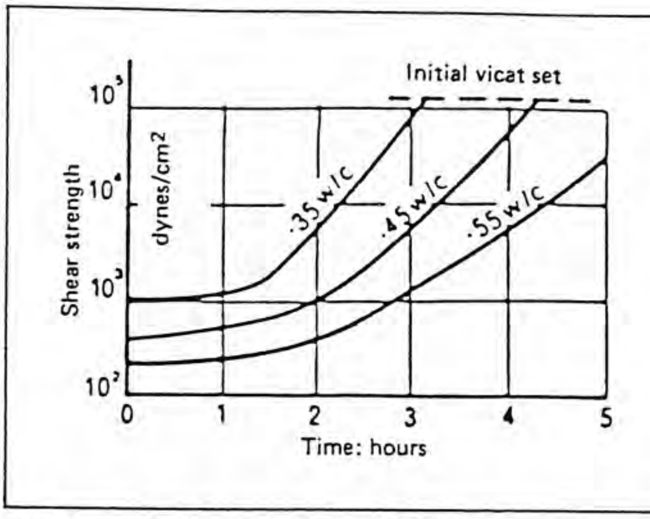


Fig 7. Setting Times for Type I Grouts (18°C)

Specific Surface Area (m^2/kg)	Initial Vicat Set (min)
1320	36
797	63
594	180
270	315

Table 4. Effect of Cement Fineness on Initial Set (Type I – 18°C)

For laminar flow in pipes it may be shown that

$$V = \frac{D}{8}\frac{dS}{dt} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 5$$

and

$$\frac{dP}{dL} = \frac{4T}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 6$$

where V = flow velocity, D = pipe diameter and dP = pressure increment/unit length of pipe dL.

Substituting for T and $\frac{dS}{dt}$ in equation 2 gives

$$\frac{dp}{dL} = \frac{4}{D}\left(Ts + Yp\frac{32Q}{\pi D^3}\right) \quad \ldots\ldots\ldots \quad 7$$

where Q = flowrate, $Ts = 3.5e^{2.1/W}$ dynes/cm² and $Yp = 0.01e^{1.8/W}$ poise.

This gives a series of laminar flow curves (Fig 6) for estimating line pumping distances.

5. SETTING

The setting process has two stages, an initial stage in which the fluidity of the grout decreases to a level at which it is no longer pumpable and a second stage in which the set grout hardens and increases in strength. Generally speaking rates of setting and hardening are not related.

Ish Shalom & Greenburg[3] have indicated a near exponential increase in shear strength and viscosity with time (Fig 7) which means that during the initial period following mixing these increases are small and do not affect pumping and grouting operations. For w = 0.35 an initial period of 1 hour is shown which extends to 2 and 3 hours for w = 0.45 and 0.55, respectively. These periods can be extended by continuous agitation of the grout. The rate of development of shear strength is also affected by age and particle size of the cement, which determine rate of hydration. This is low for coarse or old partly hydrated cement and high in new finely ground cement and Table 4 shows how initial Vicat set, equivalent to a shear strength of 170,000 dynes/cm² i.e., 2.5 psi, is accelerated by fineness of cement.

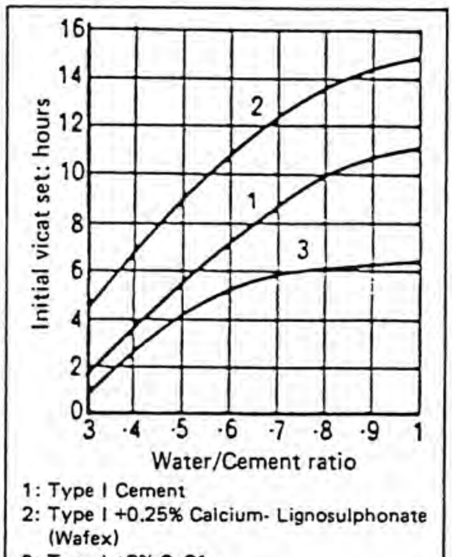


Fig 8. Effect of Retarder and Accelerator on Setting time (18°C)

Rate of shear strength development can most readily be changed by the use of retarders or accelerators (Fig 8). In the absence of test results calcium chloride reduces setting time by an amount roughly equal to 90 minutes per 1% added by weight of cement, but it is important to ensure that the $CaCl_2$ is evenly distributed by dissolution in the mixing water. Although an excellent accelerator $CaCl_2$ has several important side effects. It may affect sulphate resistance, can corrode steel in contact with the grout and increases drying shrinkage. Where instantaneous "flash" setting is required High Alumina/Type I cement mixes may be appropriate, and Fig 9 shows the relationship between setting time and proportion of Type I cement.

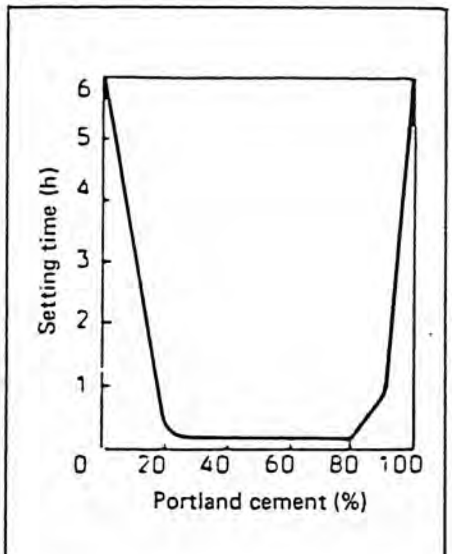


Fig 9. Setting Times for Type I High Alumina Mixes

Retarders are normally used to reduce setting rates in hot conditions and when pumping over long distances. Sugar or tartaric acid in quantities of 0.05% by weight of cement can increase setting time by 100%, but trial mixes are recommended since results are variable.

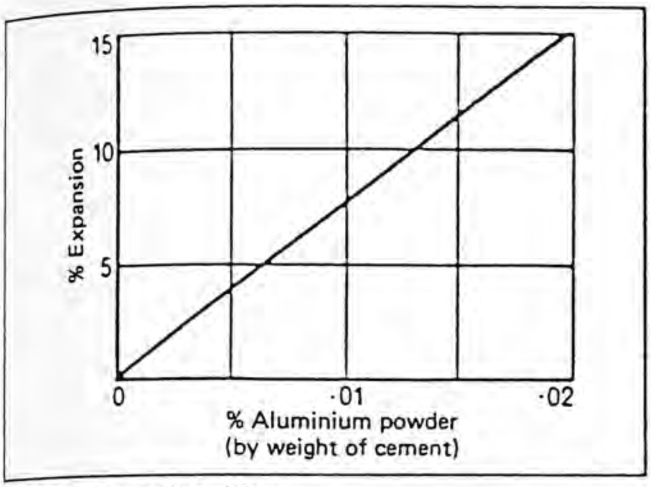


Fig 10. Expansion of Type I Grout

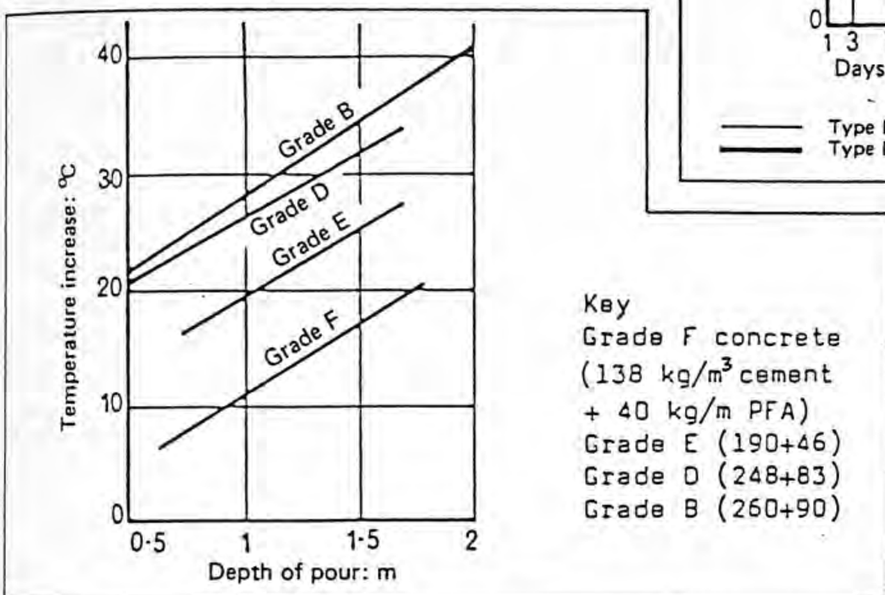


Fig 11. Relationship between maximum temperature increase and lift height: Cement content[6]

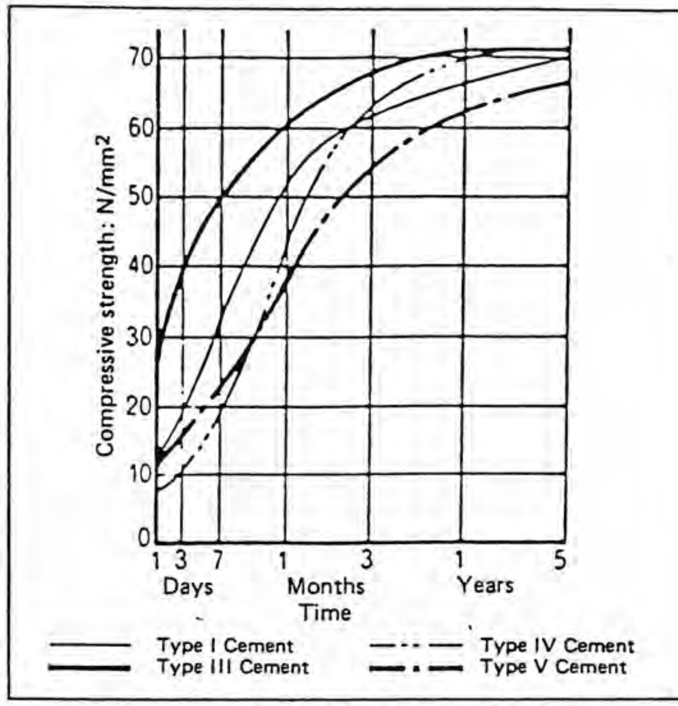


Fig 12. Gain in Strength of Set Grouts

Time (days)	Shrinkage (−%)
1	+ 0.08
2	+ 0.11
3	+ 0.11
7	− 0.72
8	− 0.94
10	− 1.30
14	− 2.37
17	− 3.24
21	− 4.19
28	− 5.14

Table 5. Shrinkage of Neat Type I grout (w = 0.4 at 18°C and 70% relative humidity)

6. SHRINKAGE

Shrinkage of cement grout is related principally to the amount of water removed. Thus a moist cured grout remaining moist throughout its life, will not shrink and may in fact expand slightly with time. On the other hand, a cement grout dry cured or allowed to dry out after moist curing will shrink (Table 5). For rich large volume grouts autogenous shrinkage (self dessication) is known[5], but even for very rich mixes the maximum shrinkage recorded is only 60 microstrain, and under submerged conditions the mechanism tends to be self healing.

Shrinkage is not normally a serious problem in ground engineering where the environment is often damp or submerged. However, where shrinkage leading to formation of microfissures is likely to affect the permeability of a waterproofing grout, positive steps may be taken to counteract shrinkage, by introducing expanding agents into the grout (Table 2). Fig 10 shows the effect of aluminium powder on unrestrained expansion during the pre-set period.

7. THERMAL PROPERTIES

Temperatures in a grout mass which are induced by heat of hydration are dictated largely by type and fineness of cement, cement content, placing temperature and insulation. Where thermal cracking is a major concern, low heat cements, low cement content, cool mixing water and cool constituent materials have been used with success to reduce the placement temperature and heat of hydration.

For thermal cracking field experience[6] indicates that cracks in pours up to 1.5m deep are initially formed by a temperature differential of 25-28°C. Provided that the mass is less than 1.5m thick the thermal problem is short term, the main temperature differentials dissipating within 7 to 10 days, the greatest temperature rises taking place in the first 3 days[7]. In the long term there is a gradual cooling of the grout mass to ambient temperatures.

For mass grout or concrete pours the heat generated by the cement hydration process is controlled by using a low cement content, usually less than 300kg/m³. It is often desirable to reduce heat even further substituting some of the cement with replacement material. In this regard ACI studies[8,9] indicate that the early heat contribution of flyash may be conservatively estimated to be 5% to 35% of equivalent cement.

For Type I cement with flyash replacement Fig 11 shows the maximum temperature rise which increases with pour depth. The results clearly show the temperature rise (per kg/m³) of cement content decreases with decreasing cement content. There is also a distinct change in the amount of heat generated at a placing temperature of 16-18°C. Below this temperature the actual temperature rise is about 12°C per 100 kg/m³ cement content (a common figure employed in design for Type I cement) whereas at 21°C say, the temperature rise increases to 16°C per 100 kg/m³. In this connection for every 10°C increase in placing temperature the rate of heat generation is doubled.

Fineness of cement also has a considerable effect on rate of heat generation, although not total heat[10]. When the specific surface is increased from 250^2/kg up to 350 and 450m²/kg, the rate of heat evolution is increased by 45% and 80% respectively. The linear coefficient of thermal expansion of grout and grouted aggregate concrete ranges from about 5 to 15 microstrain per °C depending on richness of mix and aggregate quality. The lowest coefficients being related to lean grouts injected into preplaced flint or granite aggregate.

8. GROUT STRENGTH

The most important variables affecting grout strength are the original w, the pore space of the set grout and, in the case of times up to 28 days, the type of cement and presence of admixtures.

The dominant parameter is w and the unconfined compressive strength may be expressed in the form of Abrams' Law:

$$UCS = \frac{A}{B^{1.5w}} \qquad 8$$

where UCS = unconfined compressive strength, A = strength constant of 14,000 lb/in² and B = dimensionless constant depending on characteristic of cement at age of test. For Type I cement at 28 days, B = 5. Since full strength is only generated by complete hydration, Abrams' Law is best restricted to $w > 0.3$ and in grout subject to minimal bleed i.e. $w < 0.7$.

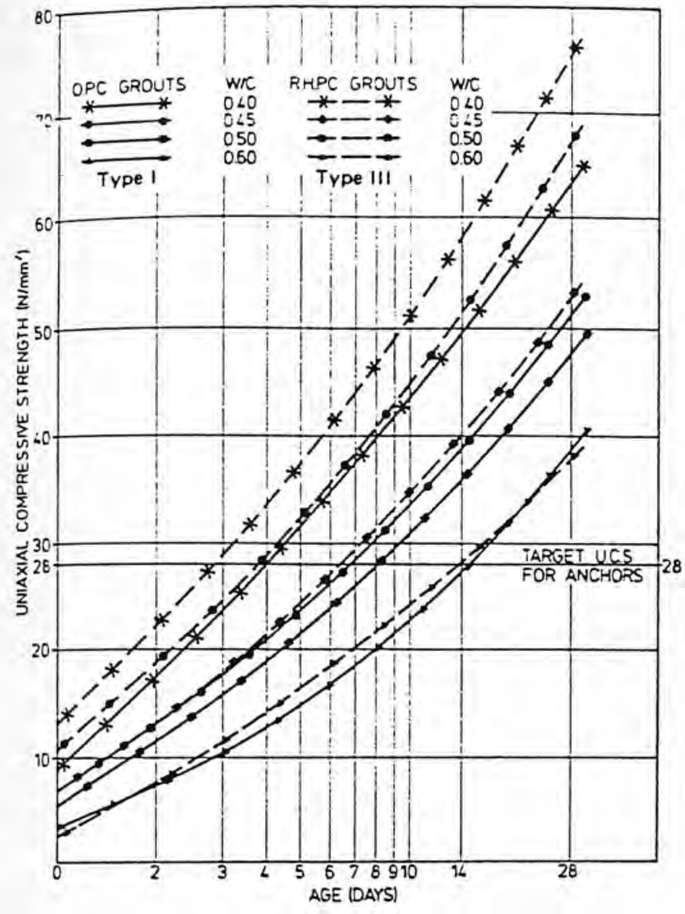


Fig 13. Effect of w on Strength Development (Types I & III)

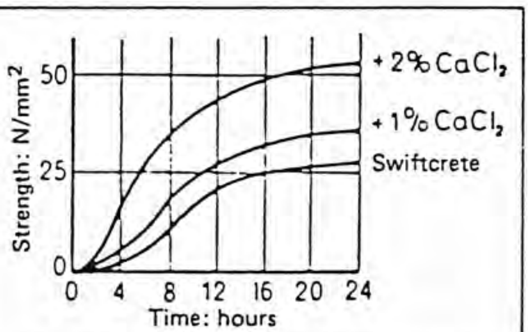


Fig 14. Strength Development of Finely Ground Type I Grout (w = 0.45)

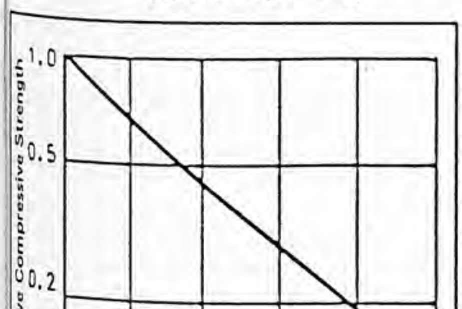


Fig 15. Effect of Gas Entrainment on Strength

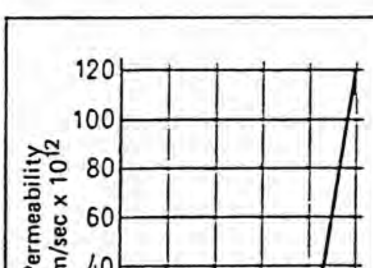


Fig 16. 28 day Permeability of Cement Grout

Under optimum curing conditions and not subject to chemical attack a set grout will continue increasing in strength over a prolonged period (Fig 12) and, irrespective of its short term strength gain, will attain a constant ultimate strength. There is a tendency for cements with a low rate of hardening to have a higher ultimate strength due to slow formation of denser gel during initial stages of setting. In general a Type I grout will have a set strength at 28 days equivalent to approximately 60-70% ultimate. This proportion may be obtained in 3 days with a Type III cement. The strength development curves of Types I and III cement are plotted in Fig 13 to illustrate the influence of w but also to highlight that at w > 0.6 Type I cement will give a strength development equivalent to that using Type III.

Addition of fluidifiers permits a reduction of w of the grout at a given fluidity and causes a proportionate increase in set grout strength. Many fluidifiers also act as retarders, increasing the initial set and decreasing early strength. For early strenth gain, tests show that $CaCl_2$ gives 30% increased strength at 24 hours when added at 1% of cement weight, although the gain reduces to 10% at 28 days. Finely ground Type I cement (830m²/kg) gives approximately the same strength increase and can be used safely in the presence of steel. The most rapid controlled acceleration is obtained by combining finely ground cement and $CaCl_2$ (Fig 14).

Porosity of Grouts

A relationship between void volume and strength for brittle materials was originally developed by Feret[11] in the form:

$$UCS = K \frac{c}{c + e + a} \qquad \text{. . . . . 9}$$

where K = constant and c e and a are the respective volumes of cement, water and air in the grout. In a grout containing little air, the porosity is determined by w, e.g. 6.5% and 3.5% for w of 0.55 and 0.4 respectively, thus relating strength to w. Where air or gas is released into the grout there is an additional effect on strength, e.g. 10% gas content can reduce strength by up to 50% (Fig 15).

Modulus of Elasticity

As for concrete an approximate relationship exists between compressive strength and static modulus for neat and normal sand/cement grouts giving in the latter case E values of 20-40 kN/mm² over the strength range 20-70 N/mm². The relative value of static to dynamic modulus is 50-80% for the strength range 20-45 N/mm². Grouts are not normally designed on the basis of E and where knowledge of this elastic property is important experimental determination is necessary using the desired materials and proportions. In this regard for most normal sand/cement grouts the higher the sand/cement ratio, the higher is the E value for a given strength, but E is sensitive to type of filler and may reduce considerably for lightweight materials.

9. DURABILITY

Cement grouts are durable under most normal conditions, but deterioration may be caused by abnormal environmental conditions, especially where there are deficiencies in grout quality, e.g. low density and high permeability. Of the adverse environmental conditions leading to grout deterioration the most common are chemical attack, notably through sulphates contained in ground water, and large scale temperature fluctuations.

The permeability of a mature cement grout is related, like its strength to its original w (Fig 16). In a fresh or setting grout, permeability is

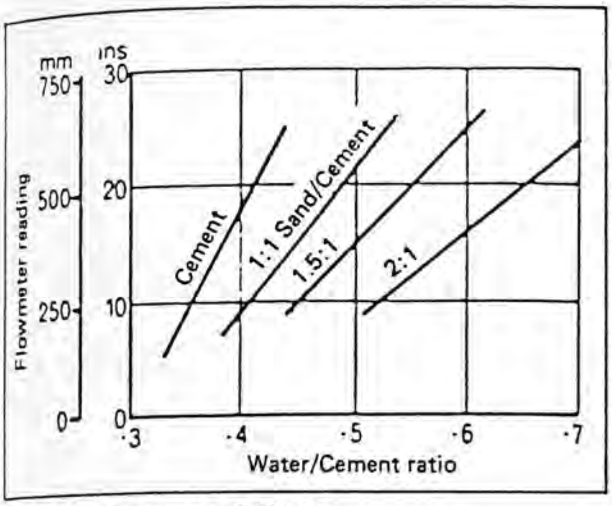


Fig 17. Colcrete Flowmeter Calibration

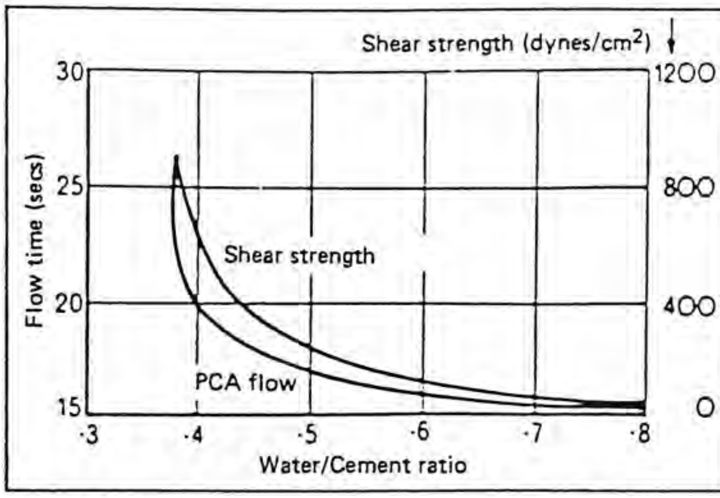


Fig 18. PCA Cone Calibration

Age (days)	Coefficient of Permeability (cm/sec)
Fresh	2×10^{-4}
5	4×10^{-8}
6	1×10^{-8}
8	4×10^{-9}
13	5×10^{-10}
24	1×10^{-10}
Ultimate (estimated)	6×10^{-11}

Table 6. Permeability of a Type I Grout (w = 0.7)

related to the age of the grout (Table 6) and the subsequent degree of hydration which ultimately determines the extent of pores in the grout. Whilst there are no positive guidelines on grout design for durability, a minimum cement content is recommended to provide acceptable durability under the appropriate conditions of exposure as in conventional concrete technology e.g. 400Kg cement/m^3 for severe exposure to sea water, driving rain, alternate wetting and drying and to freezing whilst wet.

Long and short term deterioration of cement grouts due to chemical attack may be caused by the presence of dissolved sulphates or acids in ground water, or by prolonged exposure to sea water. Since Type I cements have a poor resistance to chemical attack, this may be increased by use of blast furnace, sulphate resisting (for 0.5-1.0% total SO_3 in ground) or ultimately aluminous cements (for $\geqslant$ 2% total SO_3), but special cements should be considered primarily as an adjunct to high density and low permeability in increasing durability. Where grouts are subjected to frost attack, w should be less than 0.4 to reduce the capillary pore space and permeability of the grout. Alternatively, a rapid setting high heat of hydration cement might be appropriate, or entrainment of air if high water/cement ratios are considered inevitable.

10. QUALITY CONTROL

Variations in grout properties arise from three principal causes: (a) inadequate mixing, (b) variations in grout materials quantities and quality, and (c) apparent variations arising from the testing procedure. In order to obtain a satisfactory basis for grout mix design it is essential, prior to any contract, that methods of storage, batching, mixing and testing of materials be rigidly specified.

Mixing of cement grout leads to a sequence of exothermic reactions in four distinct stages, namely (a) initial highly exothermic reaction lasting 5-10 minutes, (b) a dormant period of up to 2 hours with low rate of heat evolution, (c) an increasing rate of reaction leading to final set after 6+ hours and (d) a continuing decreasing rate of reaction after setting. During the dormant period, a cement grout should maintain a consistent physical state, when its properties can be measured. To achieve this state and at the same time avoid false sets, mixing for 5-10 minutes is normally required. Under most field applications this should be achieved by agitation during storage and pumping and placement after mixing.

Accuracy of measurement of grout properties is also an important factor in determining the variability of grout properties in the field. Some property measurements such as bleeding have been developed from laboratory tests, e.g. Powers Float Test and ASTM method, but bleed capacities >0.5% can be easily detected in graduated wide, low containers. Laboratory measurements of grout fluidity in terms of shear strength and plastic viscosity are normally carried out with a rotating disc or coaxial cylinder viscometer. Two field instruments are the Colcrete flowmeter (which expresses fluidity in terms of horizontal slump – Fig 17) and the Portland Cement Association cone (in terms of flow time – Fig 18). Supplementary checks of w can be made on site by measuring the grout specific gravity using a Baroid mud balance. Hydrometers are not recommended since at low w errors of 25% may be introduced due to the thixotropy and particulate structure of the grouts. In most grouts the hydrogen ion concentration can reflect chemical contamination, thus pH is another useful parameter to monitor. For setting the Vicat apparatus is appropriate and for strength development characteristics 150mm or 75mm cubes can be crushed, the former invariably giving higher early strength (1-7 days) due to heat evolution but lower 28 day results due to sample size in relation to friction of platen faces.

As a principle in quality control, emphasis should be placed on those tests which permit the grout to be assessed prior to placement e.g. fluidity and density.

11. FINAL REMARKS

General guidance on factors determining grout properties has been provided for design purposes. Final designs should be confirmed, and adapted where necessary, through testing. Inevitable variations in the constituent materials and test conditions must be reflected in the quantitative data quoted and a coefficient of variation of at least 10% should be allowed. To obtain predictable and reproducible results will require:

a) obtaining cement, fillers and chemical admixtures from a reliable source,
b) storage of cementitious materials under dry and constant conditions,
c) accurate monitoring of moisture content of fillers,
d) use of fresh cement,
e) weigh batching of materials,
f) controlled water/cement ratio,
g) adequate rate and time of mixing,
h) early pumping and placement of grout after mixing, and
i) rigid supervision of all operations.

12. REFERENCES

(1) **Oza, H.P.** "Concrete in Sea Water", Indian Concrete Journal, Feb. 1965, p.43.

(2) **Powers, T.C.** "The Properties of Fresh Concrete", J. Wiley & Son, Inc., New York, 1968.

(3) **Ish-Shalom, M. & Greenberg, S.E.** "The Rheology of Fresh Portland Cement Pastes", Proc. 4th Int. Symp. on Chem. of Cement Nat. Bur. Std. U.S. Monograph 43, Vol. 2, 1962, pp. 731-743.

(4) **Papadaxis, M.** "Rheology of Cement Suspensions", Tech. Publ. No. 72. Centre d'Etudes et de Recherches de l'Industrie des Liants Hydraulic, Paris, 1955.

(5) **Troxell, G.E., Davis, H.E. and Kelly, J.W.** "Composition and Properties of Concrete", McGraw-Hill, New York, 1968.

(6) **Dunstan, M.R.H. & Mitchell, P.B.** "Results of a Thermocouple Study in Mass Concrete in the Upper Tamar Dam", Proc. Instn. Civ. Engrs., Vol. 60, No. 1, 1976, pp.27-52.

(7) **Handcock, M.G.** Discussion to Paper, "Shrinkage and Thermal Cracking in a Reinforced Concrete Retaining Wall", by E.P. Evans & B.P. Hughes, Proc. Instn. Civ. Engnrs, Vol. 38, Jan. 1968, pp.111-125.

(8) **ACI**, see "Features of Lednock Dam, Including the Use of Flyash", Proc. Instn. Civ. Engrs, Vol. 13, June 1970, p.179.

(9) **ACI**, see "Mass Concrete for Dams and Other Massive Structures", J. Amer. Con. Inst., Vol. 67, April 1972, pp.273-309.

(10) **Forrester, J.A.** "A Conduction Calorimeter for the Study of Cement Hydration", Cement Technology, Vol. 1, May/June 1970, pp.95-99.

(11) **Neville, A.M.** "Properties of Concrete", Pitman, London, 1963.

Reprinted from Proceedings of the Conference on Grouting In Geotechnical Engineering. ASCE, New Orleans, Lousiana, February 10-12, 1982.

Proc. Instn Civ. Engrs, Part 1, 1987, **82**, Jun. 645–662
TN476

Ground anchorages: corrosion performance

G. S. LITTLEJOHN, BSc(Eng), PhD, FICE*

Introduction

Millions of highly stressed tendons have been installed around the world and to date the FIP Working Group on Ground Anchorages has collected 35 case histories of failure by tendon corrosion, only one of which led to the collapse of the complete structure–ground–anchorage system, but under circumstances related to installation procedures and corrosion protection which would not be acceptable by today's standards.

2. Of the 35 cases, 24 related to permanent anchorages (protected and unprotected) and 11 applications were temporary with no designed protection other than cement grout cover for the fixed length and on occasions a decoupling sheath over the free length.

3. Generally speaking, corrosion failures are not well documented and seldom are they investigated methodically to diagnose root cause.

Review of results

4. Table 1 summarizes the key data accumulated from practice based on reported case histories of tendon corrosion notably Portier,[1] Herbst,[2] Nurnberger,[3] Weatherby,[4] and FIP.[5] The following observations and comments are presented.

5. Corrosion is localized and appears irrespective of tendon type in that nine incidents involved bar, 19 involved wire and eight involved strand, the period of service before failure ranging from a few weeks to many years for each tendon type. Short-term failures (after a few weeks) have been due to stress corrosion cracking or hydrogen embrittlement.

6. In terms of duration of service, nine failures occurred within six months, ten in the period six months to two years, and the remaining 18 over two years and up to 31 years.

7. With regard to failure location 19 incidents have occurred at, or within 1 m of the anchor head, 21 incidents in the free length and two incidents in the fixed length.

Fixed anchor problems

8. Both fixed anchor problems were caused by inadequate grouting of the tendon bond length, which in one case exposed 3 m of tendon to aggressive groundwater containing sulphides and chlorides. In this particular case failure of three rock anchorages tying back an abutment occurred after five years in service

---

Written discussion closes 14 August 1987; for further details see p. ii.
* University of Bradford; member of the FIP Commission on Practical Construction; Chairman of the Working Group on Ground Anchorages.

Table 1. Data of reported case histories of tendon corrosion

Date of installation	Time in service at failure	Geographical location	Type of structure	General environment	Ground conditions	Type of tendon	Working load or (stress level)
1934	31 years	Algeria	Anchored dam	Dry air	Masonry overlying sandstone	630N° wires 5 mm diameter (1100–1300 N/mm²)	10 000 kN (65% UTS)
1952	Few months	France	Anchored dam	Temperate climate	Concrete overlying rock	Multi wire	13 000 kN (67% UTS)
1955	16 years	Czechoslovakia	Prestressed concrete dam	Humid air	Concrete overlying rock	4·5 mm dia. smooth patented wires	4 MN
1955	26 years	Sweden	Underground power station crane beam	Humid air	Rock underlying concrete with reported water leakage (pH = 7–8)	26 mm dia. bar (80/105)	300 kN
1959	10 months	West Germany	Underground power station	Temperate climate	Rock. Below water table. Water contains very little chlorides i.e. fairly non-aggressive	15 oval ribbed wires (1570 N/mm²)	(74% of elastic limit)
1961	2 years	USA	Anchored cofferdam	—	Soil overlying rock in the presence of saltwater	35 mm diameter bar	—

Anchorage category	Corrosion protection	Corrosion number	Failure location	Remarks diagnosis	Case
Permanent	Coated tarpaulin covered by mixture of grease and bitumen with outer tarpaulin sheath over free length. Cement grout cover in fixed length	4 anchorages	Beneath anchor head	Floodwaters and repeated tensioning tore tarpaulin fabric, exposed internal bitumen cover melted under high ambient temperatures, removing protection. Localized corrosion	1
Permanent	Coated tarpaulin covered by mixture of grease and bitumen with outer tarpaulin sheath over free length. Cement grout cover in fixed length	Wires in 2 anchorages	Beneath anchor head	Corrosion failure under tension linked to type of steel. Decision taken to limit working stresses to 55% UTS thereafter, but to increase proof loading up to 1.5 times working load on occasions.	2
Permanent	Grease impregnated glass fibre bandage and outer asphalt wrapping over free length. Cement grout cover in fixed length	4 anchorages	Beneath anchor head	Fully corroded wires exposed in spite of protective wrapping. By contrast, on same site and in same location, steel tendons comprising 37 bundles of 19 wires of 2.9 mm. dia. were undamaged. Here internal spaces of ropes were filled in the factory with red lead sealing compound	3
Permanent	Bitumen coating of anchor head; cement grout cover over tendon length	1 anchorage	In Fully bonded length 2.5 m up from crane beam anchor head	Virtually no trace of grout cover. Significant pitting and typical reduction in cross sectional area was 6.8%. Depth of deepest crack = 1·3 mm. Failure attributed to intergranular stress corrosion. Steel judged to be sensitive to cracking	4
Permanent	No protection at anchor head. Jute wrapping impregnated bitumen in free length	17 anchorages	5 in anchor head. 12 in free length of which 5 were within 0·5 m of anchor head	Deep localized corrosion where bitumen protection was missing (differential aeration postulated). This protection could not withstand damage during installation or environmental attack. Steel judged to be sensitive to corrosion	5
Temporary	No protection over free length. Cement grout cover in fixed length	A few anchorages	Free length	Brittle failure. Groundwater corrosive due to presence of sulphuric acid formed from cinders falling for many years from steam locomotives. Brine may also have contributed.	6

Table 1 (Continued)

Date of installation	Time in service at failure	Geographical location	Type of structure	General environment	Ground conditions	Type of tendon	Working load or (stress level)
1963	Few years	West Germany	Anchored retaining wall adjacent to river	Temperate climate	Soil. Below water table. Water contained industrial pollutants and high chloride ion content	22 8 mm dia. wires (ST 135–150)	—
1964	Few weeks up to 1 year	Algeria	Anchored dam	Dry air	Masonry overlying sandstone	54 7 mm. dia. cold drawn wires (1265–1432 N/mm²)	1960 kN (75% of elastic limit)
1965	7 years 9 years	UK	Rock face stabilization	Temperate climate	Limestone	44 high tensile steel wires	2000 kN
1960s	8 years	West Germany	Anchored retaining wall	Temperate climate	—	5.2 mm diameter wires (alloy steel)	—
1960s	3 months	UK	Anchored floor of dry dock	Temperate climate and saline atmosphere	Rock	Quenched and tempered low alloy bars (1500 N-mm²)	(67% UTS)

Anchorage category	Corrosion protection	Corrosion number	Failure location	Remarks/diagnosis	Case
Permanent	Tendon encased in cement grout	3 anchorages	85% of ruptured wires failed in vicinity of concrete deadman-tendon interface	Surface corrosion and pitting observed in tendons. Insufficient grout cover and presence of chlorides. Stress corrosion and cracking. Tendon bending due to ground movement	7
Permanent	Road oil loaded with red lead but anchor head waiting several weeks before protective filling placed	Several individual wires	Button headed wires at anchor head	Brittle failure under tension. Button heads were cold forged on site	8
Permanent	Bituminous infilling as a surround for the free length (piped in hot). Cement grout cover in fixed length.	24 wires in 3 anchorages. A further 13 wires in same anchorages 2 years later	0·6 to 1 m beneath anchor head	Stress corrosion due to an aqueous environment	9
Permanent	Tendon painted with bitumen over free length Cement grout cover in fixed length	3 anchorages	Free length	Although no corrosion producing elements found, stress corrosion postulated where bitumen protection had broken down. Surface corrosion and heavy pitting observed on wires. Some pits contained small fissures	10
Permanent	Bare bar, ungrouted over free length. Cement grout cover in fixed length	2 anchorages	Free length	Corrosion pitting leading to hydrogen induced stress corrosion cracking at failure. Free length grouting actioned in 1977 since when no corrosion failures have been observed	11

Table 1 (Continued)

Date of installation	Time in service at failure	Geographical location	Type of structure	General environment	Ground conditions	Type of tendon	Working load or (stress level)
1967–68	6–18 months	West Germany	Underground pumped storage scheme	Temperate climate	Rock	18 8 mm diameter wires (1500–1700 N/mm²)	900–1700 kN
1968	Within 3 years	Switzerland	Underground power station	Temperate climate	Rock	12 8 mm diameter wires	650 kN
1968	11 years	France	Anchored foundation blocks	Temperate climate	Coal mine waste fill. Above water table	8/12 mm dia. oval ribbed wires (1450–1600 N/mm²)	1720 kN (67% of elastic limit
Before 1969	A few days and 100 days	West Germany	Anchored retaining wall supporting a rail track	Temperate climate	Soil fill	6 12·2 mm dia. wires	—
1968–69	—	USA	Anchored retaining wall	—	Landfill with high organic content overlying mica schist. Brackish groundwater	12·7 mm dia. strand of 4.2 mm wire (270 K grade)	Up to 2640 kN
1969	10 years	USA	Anchored retaining wall	Acidic. Adjacent to waste acid neutralization plant	Fill (clays and silts)	32 mm dia. high strength bars (1033 N/mm² ultimate)	636 kN

Anchorage category	Corrosion protection	Corrosion number	Failure location	Remarks/diagnosis	Case
Permanent	A chemical filler (oil based unsaturated fatty acid polymer) surrounding the free length. Cement grout cover in fixed length	Majority of 133 anchorages installed	Free length	Stress corrosion due to leaching out of nitrate ions from chemical protective filler	12
Test	Tendon installed in borehole with anchor heads at either end. No protection by design for test purposes	2 anchorages (11 wires and 10 wires	Stressed length between anchor heads	Presence of sulphides caused embrittlement of the steel	13
Permanent	OPC and polyethylene outer tube over free length	4 anchorages	0·2 to 2·5 m beneath anchor head	Brittle failure under tension, initiated at surface oxidized local defects	14
Temporary	Tendon unprotected over free length. Cement grout cover in fixed length	3 anchorages	Free length	Tendons not heavily corroded. Failure judged to be due to corrosion fatigue as result of bending due to fluctuating loads from railway being transmitted through frozen ground. Cracks in steel noted at failure location	15
Temporary	Bentonite–cement grout cover plus outer steel pipe in free length. In addition a sacrificial zinc ribbon anode was installed with each tendon. Cement grout cover in fixed length	—	Beneath anchor head	Brittle corrosion failure of tendon where bentonite–cement grout cover had dropped 1–1·2 m. Hydrogen sulphide was present in the soil and the sacrificial anode was consumed near the anchor head of the failed tendons	16
Permanent	No protection of anchor head. In free length grease, paper wrapping and plastic sleeve embedded in cement grout. Cement grout cover in fixed length.	8 anchorages	Beneath anchor head	Heavy pitting leading to brittle failure of unprotected tendon	17

Table 1. (Continued)

Date of installation	Time in service at failure	Geographical location	Type of structure	General environment	Ground conditions	Type of tendon	Working load or (stress level)
1969	Few weeks	France	Anchored retaining wall	Temperate climate	Above water table. Chlorides and sulphates in water from sewer leakages	8/12 mm dia. ribbed wires (1450–1600 N/mm^2)	1030 kN (63% of elastic limit)
1969	5 years	Malaysia	Rock strengthening	Humid	Rock	36 7 mm dia. wires	700 kN
1970	28 months	New Zealand	Anchored retaining wall	—	Clay overlying sandstone	42 7 mm. dia. wires	UTS = 2570 kN initial tensioning to 48% UTS. Tendon designed to work at up to 66% UTS
Before 1971	—	West Germany	Anchored retaining wall	Temperate climate	—	15 5·2 mm diameter wires (alloy steel)	—

Anchorage category	Corrosion protection	Corrosion number	Failure location	Remarks/diagnosis	Case
Permanent	OPC grout and mild steel outer tube in free length. Cement grout cover in fixed length	6 anchorages	0·1 to 0·5 m beneath anchor head	Brittle failure under tension. Decarboned steel at wire perimeter. Incomplete filling of protective grout beneath anchor head	18
Permanent	Anchor outer head protected by sealing cap infilled with grease and injected under pressure. Polypropylene sheathed wires surrounded by bitumen placed in situ over free length. Polypropylene sheathed wires with stainless steel end barrels surrounded by cement grout in fixed length	1 anchorage comprising 36 wires of which 33 were broken	Underside of anchor head and at bare section of wires immediately above plastic	Stress corrosion cracking of wires. Inadequate filling of inner head region with bitumen. Exposed bare wires subject to wetting and drying cycles. Groundwater of low pH suspected	19
Permanent	Polypropylene extruded sheathing of individual wires with outer plastic tube infilled with a mastic sealant. Ribbed alkathene tube and epoxy resin cover in fixed length	5 wires	In free length 1 to 8 m below anchor head	Surface corrosion cracking. Mastic filler found to be hygroscopic and it was suspected that the mineral oil softened the polypropylene sheathing. Also speculated that the polypropylene sheathing may have been damaged during transport and installation. 1 m. of polypropylene sheathing stripped off below anchor head before tendon installation and stressing	20
Temporary	Tendon unprotected over free length. Cement grout cover in fixed length	2 anchorages	Free length	Heavy pitting and occasional cracking of wires noted. Chemical analysis of corrosion products indicated 0·25% sulphur content but no chlorides	21

Table 1. (Continued)

Date of installation	Time in service at failure	Geographical location	Type of structure	General environment	Ground conditions	Type of tendon	Working load or (stress level)
Before 1971	Within 1 year	West Germany	Anchored retaining wall	Temperate climate	—	5·2 mm dia. wires (alloy steel)	—
1971	6 weeks	USA	Anchored retaining wall	—	Acidic soil embankment comprising mainly blast furnace slag. Soil moist adjacent to tendon	32 mm dia. bar hot rolled, drawn and stress relieved. (1100 N/mm^2 ultimate)	—
1971	4 weeks	USA	Anchored retaining wall	—	Moist soil with low pH	35 mm dia. bar, hot rolled, drawn and stress relieved	—
1972	2 years	South Africa	Restraint for cantilevered grandstand	Seasonal wetting and drying	Fill	5 12.2 m dia. strands	450 kN

Anchorage category	Corrosion protection	Corrosion number	Failure location	Remarks/diagnosis	Case
Temporary	Tendon encased in cement grout	5 anchorages	Free length	Heavy corrosion and pitting in certain zones where there was no adhering cement grout. Other sections of tendon which were completely grout free displayed general corrosion. No corrosion where tendon still bonded to grout. Brittle failure recorded. Tendon bending and overstressing also induced by ground deformations. Analysis of corrosion products indicated 0·63% sulphates but no chlorides or sulphides	22
Temporary	Tendon unprotected over free length. Cement grout cover in fixed length	4 anchorages	Free length	Stress corrosion cracking postulated	23
Temporary	Tendon unprotected over free length. Cement grout cover in fixed length	—	Free length	Stress corrosion cracking postulated	24
Permanent	Polypropylene sheathed and greased strands in free length. Cement grout cover in free length	No failure, but one tendon located with unacceptable corrosion i.e. pitting and all 9 anchorages condemned	Fixed anchor zone	Some doubts expressed over efficacy of grouting of fixed anchor length where no special precautions had been taken. When one anchorage excavated, grout cover in fixed zone ranged from nil to 6 mm, and pitting up to 1 mm in depth was measured	25

Table 1. (Continued)

Date of installation	Time in service at failure	Geographical location	Type of structure	General environment	Ground conditions	Type of tendon	Working load or (stress level)
1972-73	In the early stages of contract	New Zealand	Anchored retaining wall	—	Clays and silts overlying sandstone	Multi-wire tendon	490–1050 kN (50% UTS)
1973	11 years	UK	Anchorage restraint of abutment which was yielding initially	Temperate climate	Fill overlying clay and weak rock	4–5 15·2 mm dia. strands	350 kN (50% UTS) maximum)
1974	—	New Zealand	Anchored bridge abutment	—	Rock	34 7 mm dia. wires	1000 kN (50% UTS)
1974	5 years	Algeria	Concrete dam raising	Dry air	Concrete	36 15·2 mm diameter strands	—
1975	6 months	France	Anchored retaining wall	Temperate climate	Above water table. Nothing suspicious	32 mm dia. ribbed bars (1079–1225 N/mm^2)	640 kN (74% of elastic
1976	5 years	Switzerland	Anchored abutments for pipeline bridge	Temperate climate	Fill overlying sands and gravels overlying rock	10 12·7 mm diameter steel strands	1130–1150 kN

Anchorage category	Corrosion protection	Corrosion number	Failure location	Remarks/diagnosis	Case
Temporary	Unprotected in free length. Cement grout cover in fixed length	—	—	Ground movements created severe overloading of tendons in certain locations. Corrosion of tendon	26
Permanent	No anchor head protection. Greased and sheathed strands over free length. Cement grout cover in fixed length	1 strand in each of two anchorages	Beneath anchor head	Failure due to stress corrosion	27
Permanent	Polypropylene sheathing over wires with a secondary protection of outer tube and mastic infilling in free length. Corrugated tube/grout encapsulation over fixed length	—	—	Protective ducting in free length damaged during transportation permitting leakage of mastic filler which had softened at the high ambient temperature. Protected tendons stored several months on site before installation	28
Permanent	Free length annulus grouted with acrylamide chemical. Cement grout cover in fixed length	—	Beneath anchor head	Where duct had not been filled properly with acrylamide grout, tar epoxy was poured in to fill upper 0·5 m. Failure occurred at the base of the tar epoxy	29
Temporary	Polyethylene outer tube over free length. Cement grout cover in fixed length	2 anchorages	3 and 8 m beneath anchor head	Brittle failure under tension	30
Permanent	Polyethylene sheathed strands in free length with asphalt filling. Cement grout cover in fixed length	3 anchorages	In fixed anchor length within 500 mm of free length	Bridge collapse due to failure of anchored abutment. Severe corrosion of strands in proximal zone of fixed anchor length which was only partially grouted. The tendon was exposed to aggressive groundwater containing sulphides and chlorides in the fill and sandy gravel. Poor construction practice and lack of quality controls such as water testing lead to inadequate grouting. Fixed anchor straddled permeable soil and rock	31

Table 1 (Continued)

Date of installation	Time in service at failure	Geographical location	Type of structure	General environment	Ground conditions	Type of tendon	Working load or (stress level)
1977	Within 3 years	Hong Kong	Anchored retaining wall	Humid and slightly saline	Non-aggressive fill overlying completely weathered granite which improves with depth to moderately strong granite	7 12·9 mm diameter Supa strands	1050 kN
1977	4 months	West Germany	Anchored retaining wall	Temperate climate	Fill consisting of slag and ash. Sulphate content = 200 mg/l. No tendon contact with groundwater	32 mm diameter hot rolled and threaded bars (1100 N/mm^2 ultimate)	—
1978	4 years	South Africa	Slope stabilization	Humid	Weathered sedimentary rock	4–6 15.2 mm diameter strands	590 kN 890 kN (60% UTS)
1980	1-3 years	Hong Kong	Stabilization of rock	Humid and slightly saline	Rock	High tensile steel bars	500–650 kN

Anchorage category	Corrosion protection	Corrosion number	Failure location	Remarks diagnosis	Case
Permanent	Anchor head encased in concrete. Grease and plastic sheathing over free length. Cement grout cover in fixed length	1 anchorage (2 strands)	Beneath anchor head and in free length	No corrosion protection was provided immediately beneath the anchor head. Considerable delays were experienced between stressing and concrete encasement of the anchor head. Metallographic examination of tendon wires in 45 anchorages showed up to 2·7% and 12% loss of diameter for delay periods of 1–8 months and 16–36 months respectively. It was also speculated that strands had been stored on site for some time (allowing corrosion to develop) prior to greasing and sheathing of the free length	32
Temporary	No protection at anchor head. Polythene tube over free length. Cement grout cover in fixed length	2 anchorages	50 mm beneath anchor head. Middle of free length	Failure adjacent to anchor head due to brittle fracture at a deep pit. Second failure attributed to hydrogen embrittlement. Ground deformations also present leading to bending and overstressing. Lack of protection and use of corrosion susceptible steel highlighted overall. Sulphur compounds present as corrosion products	33
Permanent	Grease filled or cement grouted outer anchor head. PVC sheathed and greased strands in free length. Cement grout cover and epoxy resin coating over fixed length	2 anchorages	Underside of anchor head	Ground movement after service increased tendon loads by up to 20%. Grease filling and capping of anchor head inadequate to stop infiltration of surface water inner head. Stray currents from adjacent electrified rail line (15–20 m distance) identified. Sulphate reducing bacteria located in annulus between strand and PVC sheathing in some cases	34
Permanent	Cement grout plus sheath over free length and tendon bond length. Grease at bar couplers	10 anchorages	Up to 20 m beneath anchor head but always adjacent to a coupling joint	All fractures occurred over a small area where neither grout nor grease was in contact with the bar. This small air void resulted from the method of encapsulation. Metallurgical examination showed pitting corrosion and hydrogen embrittlement. Traces of chloride salts were present on the bar after assembly which probably initiated pitting	35

and led to the collapse of a pipeline bridge in Switzerland (Case 31, Table 1). The following points are recorded.

(*a*) No borehole was sunk at the abutment; rock head was deduced from a borehole 25 m away.
(*b*) Drilling was subcontracted, poorly supervised and drill logs were not produced.
(*c*) No water or pregrouting tests were carried out before tendon installation. Such tests would have highlighted the presence of permeable gravels at the top of the tendon bond length.
(*d*) Grout injection procedures did not provide a grout flow return—in other words, a fixed quantity of grout was preplaced sufficient only for tendon bond.
(*e*) No protective sheath was applied over the tendon bond length.

Free length failures

9. Failures in the free length are recorded under a variety of individual and combined circumstances such as

(*a*) tendon overstressing caused by ground movement leading to tendon cracking, sometimes augmented by pitting corrosion or corrosion fatigue
(*b*) inadequate or no cement grout cover in the presence of chlorides, e.g. industrial waste fills or organic materials
(*c*) breakdown of bitumen cover due to lack of durability
(*d*) inappropriate choice of protective material, e.g. chemical grout containing nitrate ions and hygroscopic mastic
(*e*) use of tendon stored on site for a long period in an unprotected state.

Anchor head failures

10. Failures at, or adjacent to the anchor head are due to various causes ranging from absence of protection (even for only a few weeks in aggressive environments) to inadequate cover due to incomplete filling initially or slumping of the protective filler during service.

11. In one example in Hong Kong (Case 32, Table 1) where the delay between stressing and concrete capping of the anchor head was one to eight months, a loss of wire diameter up to 2.7% was measured. Where the delay was 16–36 months, the maximum loss monitored was 12%.

Discussion of results

12. From all the case histories reviewed, it is apparent that corrosion incidents are somewhat random in terms of cause, with the possible exception of choice of steel. In this regard, various studies have highlighted that quenched and tempered plain carbon steels and high strength alloy steels are more susceptible to hydrogen embrittlement than other varieties. Accordingly, these named steels should be used with extreme caution where environmental conditions are aggressive.

13. Bearing in mind the exposure of the anchor head to the atmosphere, which often subjects this component to greater risk of corrosion than the embedded free and fixed lengths, it is surprising that the quality of the anchor head protection is not generally to a higher standard than the remainder of the anchorage. In fact, the opposite appears to be true.

14. Although rather unusual, there is also one case of overstressing of steel wires due to poor design and bad alignment of the tendon at the anchor head. In modern practice, application of national standards covering dynamic and static load efficiencies of anchor head assemblies should eliminate this problem.

15. The fact that 19 failures occurred within two years of installation confirms that where the environment is aggressive, temporary anchorages should be given appropriate protection. The corrosion protection for the anchor head should also be applied as soon as practicable after grouting, whatever the service life. Where a delay is likely, consideration should be given to temporary protection in the form of plastic paint, grease impregnated tape or some type of cover. In spite of the above, there appears to be no evidence to suggest that the current limit of two years for the service period of temporary anchorages, should be reduced or extended.

Conclusions

16. 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.

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

18. Since there is no certain way of predicting localized corrosion rates, where aggressivity is recognized, 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.

19. 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.

20. For corrosion resistance, the anchorage should be protected overall as partial protection of the tendon may only induce more severe corrosion of the unprotected part. To achieve overall protection, great attention must be paid to the design and construction detail. Junctions between the fixed length, free length and anchor head are particularly vulnerable, as are joints and couplers.

21. Out of millions of prestressed ground anchorages which have been installed around the world, 35 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 high standards both in design and construction in order to ensure satisfactory performance during service.

References

1. PORTIER J. Protection of tie-backs against corrosion. *Proc. tech. session on prestressed concrete foundations and anchors*, 7th FIP Congress, New York, 1974, 39–53.

2. Herbst T. F. Safety and reliability in manufacture of rock anchors. *Int. symp. on rock mechanics related to dam foundations. Rio de Janiero, 1978.*
3. Nürnberger U. Analysis and evaluation of failures in prestressed steel. *Forschung, Straßenbau und Straßenverkehrstechnik*, 1980, **308**, 1–195
4. Weatherby D. E. *Tiebacks*. Report FHWA/RD-82/047, US Dept. of Transportation, Federal Highway Administration, Washington, DC, 1982.
5. Federation Internationale de la Précontrainte. *Corrosion and corrosion protection of prestressed ground anchorages.* State of the art report, Thomas Telford, London, 1986.

MONITORING THE INFLUENCE OF BLASTING ON THE PERFORMANCE OF ROCK BOLTS AT PENMAENBACH TUNNEL

G.S. Littlejohn, B.Sc.(Eng.), Ph.D., C.Eng., F.I.C.E., M.I.Struct.E., F.G.S.
(University of Bradford)
A.A. Rodger, B.Sc.(Eng.), Ph.D., C.Eng., M.I.C.E., F.G.S.
(University of Aberdeen)
D.K.V. Mothersille, B.Eng.
(University of Bradford)
D.C. Holland, B.Sc.
(University of Aberdeen)

The rock mass classification and associated permanent rock bolt support system for Penmaenbach Tunnel are briefly described, along with a scheme for studying the influence of blasting on the performance of resin bonded bolts. Particular attention is given to the instrumentaiton employed to measure instantaneous fluctuations in bolt prestress, vibrational movement of bolts and surrounding rock, and the data recording and analysis systems. Typical monitored data are presented and from a preliminary analysis, fully bonded prestressed bolts are shown to have accommodated accelerations up to 130 g without distress.

INTRODUCTION

Where tunnelling rock demands drill and blast methods of excavation, there is no predictive capacity for optimising the distance from the tunnel face to a safe location for the installation of permanent rock bolts. Current estimates are usually based upon conservative distances derived from precedent practice or a limiting dynamic parameter e.g. peak particle velocity of 100 mm/s.

Wherever rock conditions demand bolt support within a specified 'safe' distance, the bolting is generally classified as temporary, because of lack of confidence concerning potential breakdown of bond, loss of integrity of the corrosion protection system or even bolt damage. In such circumstances costly duplication of bolting occurs as the permanent reinforcement advances behind the face, and the temporary bolts become redundant.

By measuring instantaneous and residual load changes in bolts under blast loading at various distances from the tunnel face together with the characteristics of the blasting vibrations, it is hoped to provide a more fundamental appreciation of the dynamic response of the rock mass and the support system and thereby establish more appropriate criteria for estimating the optimum safe distance for permanent rock bolts subjected to close proximity blasting. Since current predictions are judged to be conservative, it is also anticipated that the new criteria will lead to more economical permanent tunnel support.

TUNNEL PROJECT

The Penmaenbach Tunnel has been commissioned by the Welsh Office to provide a new carriageway for westbound traffic on the A55 North Wales coast road. It is driven by drilling and blasting through the rhyolite extrusion which forms the Penmaenbach Headland, two miles to the West of Conway. The completed tunnel is 640 m long and a typical cross section is shown in Fig. 1. A top heading and bench extraction method was employed to advance the tunnel face, and further rock extraction to form service trenches, created multiple blast sources within the tunnel on occasions.

ROCK MASS CLASSIFICATION AND TUNNEL SUPPORT

The headland is composed mainly of fairly competent rhyolite of Ordovician age. The rhyolite is slightly weathered fine grained very strong material with narrow to wide fracturing (spacing typically 0.2 to >0.5 m). The range of properties is listed in Table 1.

Intact Rock	
Bulk Density	2.52-2.66 Mg/m^3
Uniaxial Compressive Strength	85-339 MN/m^2
Point Load Index	2.3-10.2 MN/m^2
Static Elastic Modulus	35-60 kN/mm^2
Dynamic Elastic Modulus	68.5-75 kN/mm^2
Poissons Ratio	0.1-0.2
Rock Mass	
Static Elastic Modulus	10-40 kN/mm^2
Dynamic Elastic Modulus	20-50 kN/mm^2

TABLE 1. GEOTECHNICAL PROPERTIES OF THE RHYOLITE

The designers of the rock support at Penmaenbach Tunnel have employed two well established empirical methods, namely the NGI tunnelling quality index (Q) developed by Barton (ref. 1) and the CSIR Rock Mass Rating (RMR) scheme proposed by Bieniawski (ref. 2). Both methods were used to compare and check the recommended amount and type of permanent support required for stability.

At Penmaenbach three classes of rock were established (Table 2) each with a standard form of support which involved combinations of spot bolting, patterned rock bolting, and sprayed concrete, with and without fabric reinforcement.

Rock bolting was the predominant means of support, and the fully bonded two-stage resin type was chosen for its known resilience to blasting. Bolt length, fixed anchor length and spacing were designed according to rock class and location of bolt on the tunnel perimeter. Thus the more fractured rock required greater fixed anchor lengths (bond length for fast setting resin, ranged from 1.5 to 3 m) and in the crown areas which were judged to be inherently more unstable, overall lengths were increased form 3.5 to 7 m. Although the rock classification provided the basic rock support parameters, detailed mapping

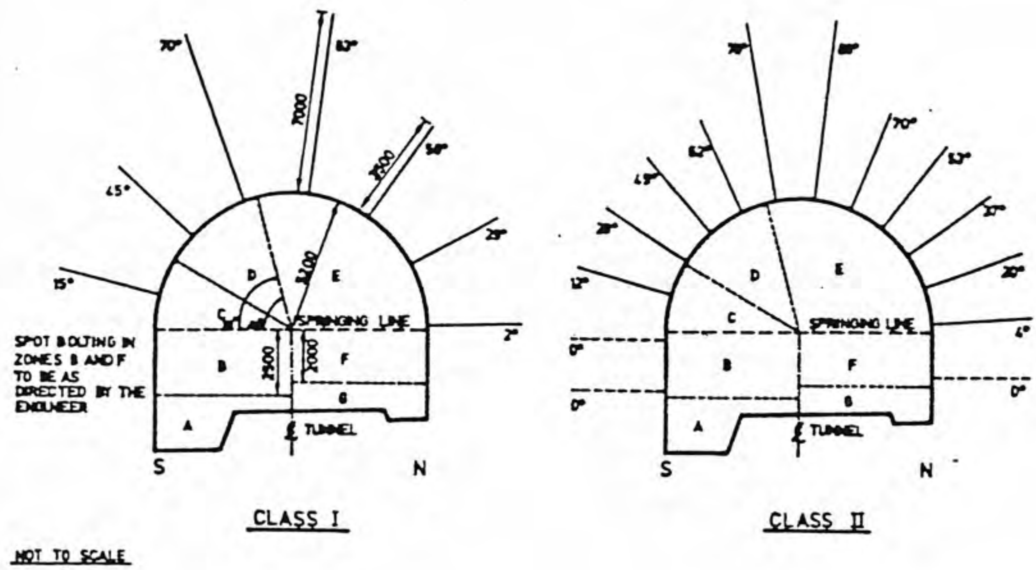


FIG. 1. TYPICAL PRODUCTION BOLT ARRAY RELATED TO ROCK CLASS

<table>
<tr><th rowspan="2">TUNNEL PERIMETER ZONE</th><th rowspan="2">ROCKBOLT LENGTH (m)</th><th rowspan="2">ROCKBOLT ORIENT-ATION</th><th colspan="3">ROCKBOLT FIXED ANCHOR LENGTH (m)</th><th colspan="3">ROCK BOLT SPACING (m)</th><th colspan="3">ADDITIONAL SUPPORT</th></tr>
<tr><th>CLASS I</th><th>CLASS II</th><th>CLASS III</th><th>CLASS I</th><th>CLASS II</th><th>CLASS III</th><th>I</th><th>II</th><th>III</th></tr>
<tr><td>A. (S.SIDEWALL)</td><td colspan="8">NO ROCKBOLTS REQUIRED</td><td colspan="2" rowspan="2">NONE</td><td rowspan="2">50mm SPRAYED CONCRETE</td></tr>
<tr><td>B. (S.SIDEWALL)</td><td rowspan="2">3.5</td><td>HORIZ-ONTAL</td><td colspan="2" rowspan="2">1.5</td><td rowspan="2">2.0</td><td>SPOT BOLTING</td><td>2.0</td><td>1.5</td></tr>
<tr><td>C. (CROWN)</td><td rowspan="3">RADIAL</td><td rowspan="3">2.5</td><td rowspan="3">1.5</td><td rowspan="3">1.0</td><td rowspan="3">NONE</td><td rowspan="3">25mm SPRAYED CONCRETE</td><td rowspan="3">50mm FABRIC REIN-FORCED SPRAYED CONCRETE</td></tr>
<tr><td>D. (CROWN)</td><td>7.0</td><td colspan="2">2.0</td><td>3.0</td></tr>
<tr><td>E. (CROWN)</td><td rowspan="2">3.5</td><td colspan="2" rowspan="2">1.5</td><td rowspan="2">2.0</td></tr>
<tr><td>F. (N.SIDEWALL)</td><td>HORIZ-ONTAL</td><td>SPOT BOLTING</td><td>2.0</td><td>1.5</td><td colspan="2" rowspan="2">NONE</td><td rowspan="2">50mm SPRAYED CONCRETE</td></tr>
<tr><td>G. (N.SIDEWALL)</td><td colspan="8">NO ROCKBOLTS REQUIRED</td></tr>
</table>

a) SUPPORT REQUIREMENTS

CLASS	R.Q.D.	DISCONTINUITY SPACING	(CSIR) RMR RANGE	(NGI) Q RANGE
I	90-100%	> 0.5m	81 - 100	> 20
II	60-90%	0.2 - 0.5m	61 - 80	20 - 4

b) ROCK CLASS DEFINITIONS

TABLE 2 SUPPORT REQUIREMENTS RELATED TO ROCK CLASS of the rock face as the tunnel advanced dictated actual bolt location.

RESEARCH PROJECT

In order to study the performance of rock bolts when subjected to blasting, experimental fully bonded two-stage resin bolts (Fig. 2) were installed initially at decreasing distances (20 m down to 3 m) from the tunnel face. The bolts (25 mm dia.) were placed on the south wall of the tunnel (zone B in Fig. 1) at 3.5 m centres to coincide with face advances of 3.5 to 4 m per blast (Table 3). Thereafter, experimental bolts were located to within 1 m of the face and two arrays of 3 bolts (wall, haunch and crown) were monitored to assess the influence of bolt location. At the end of this phase of the work ad hoc tests were run on decoupled bolts (slow setting resin omitted) and shortened bolts (3.5 m c.f. 6 m) to permit comparison with production bolt behaviour.

In all cases bolts were post-tensioned to a nominal load of 100 kN when the fast setting resin had set, after which the bolt prestress was "locked-in" by the slow setting resin, where employed, except for a 700 mm length below the bolt bearing plate. This length was decoupled by the use of grease impregnated wrapping tape to permit some load adjustment on the bolt in the event of surface spalling of the rock after blasting.

For each blast which was classified in terms of hole pattern, charge weight per hole, group delay and type of explosive, service loads on the bolts before and after blasting were recorded together with the instantaneous fluctuation in prestress using load cells. Accelerometers attached to the load cells recorded the vibrations from the blast to identify the signature of each blast and permit correlation between blast pattern and bolt load fluctuation. On completion of the blasting sequence, load extension characteristics were recorded to investigate bolt debonding. Currently, the vibration records are being used to study movement of the bolt/rock system and wave propagation through the rock.

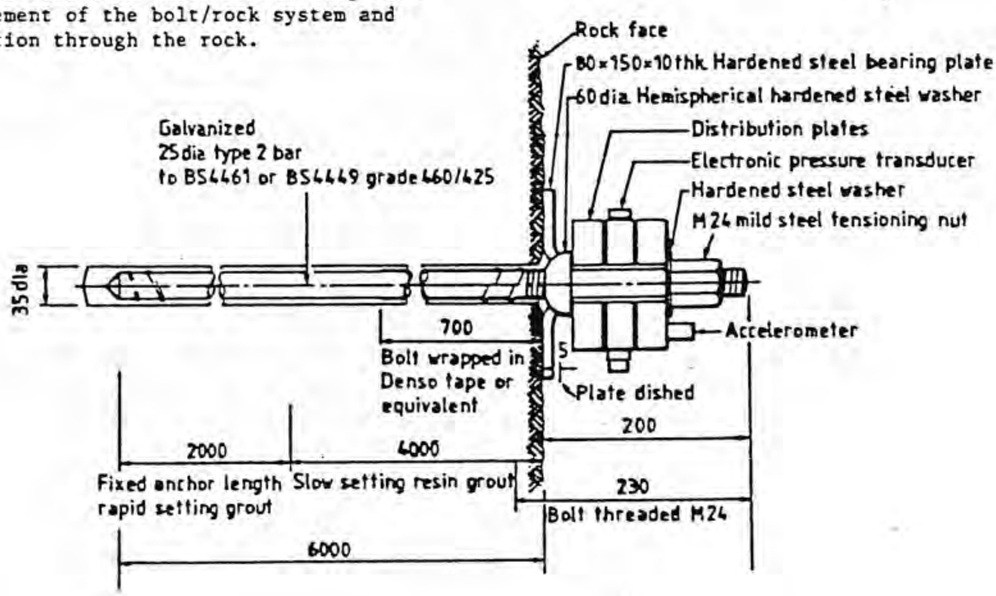
INSTRUMENTATION SYSTEMS

The instrumentation employed in this project may be categorised into the system associated with monitoring load fluctuations in the rock bolts, the system for monitoring vibrational movement of the bolts and surrounding rock, and the data recording and analysis system.

Load Monitoring

Fluctuations in rock bolt prestress were measured by means of annular Glotzl hydraulic load cells fitted with electronic pressure transducers to permit remote reading. Each cell consists of a sensitive pressure pad formed by joining together two very stiff steel discs at their edges, the space inside the cell being filled with de-aired hydraulic fluid. When a load is applied to the cell, the pressure in this fluid changes, and this change in pressure is transferred to the strain gauged diaphragm of the electronic pressure transducer. Figs. 2 and 3 show the installed load cell configuration. Load distribution plates coated with epoxy based paint to provide corrosion protection were employed both above and below the cells. The load measuring range of this system was 0-250 kN with a rated accuracy of ±1% of full scale.

Although these transducers had been used widely in static applications little information was available on their dynamic response. Consequently, prior to commencement of the project, tests were undertaken using an ESH impulse testing machine to assess their frequency response in comparison with a Kistler type 9071 piezoelectric load cell as shown in Fig. 4. Fig. 5 shows comparative results of a typical rapid unloading test from a precompression load of 70 kN confirming satisfactory performance of the hydraulic cell. Similar satisfactory results were obtained where a compressive preload of 10 kN was applied to the load cells and a compressive impulse load of 80 kN superimposed on this.

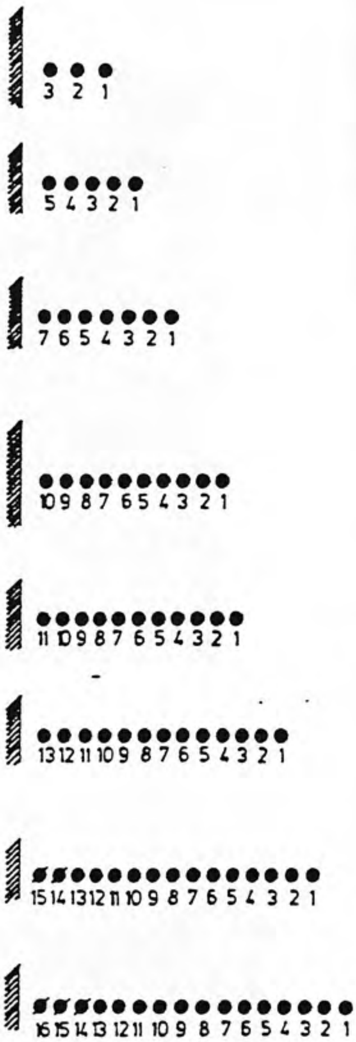


All dimensions in millimetres

FIG. 2. EXPERIMENTAL ROCK BOLT INSTALLATION

Face Chainage (m)	Chainage of Experimental Bolt (m)
2835.4	1. 2816 2. 2818.7 3. 2822.9
2839.4	4. 2826.6 5. 2830.2
2783.2 (bench)	6. 2832.9 7. 2835.5
2851.0	8. 2840 9. 2843.4 10. 2846.2
2854.6	11. 2850.4
2858.4	12. 2853.7 13. 2856.1
2862.3	14. 2859.1 15. 2862.1
2874.0	16. 2866.2

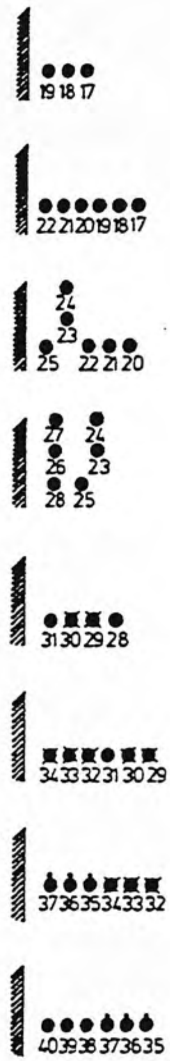
Layout



a) Phase 1

Face Chainage (m)	Chainage of Experimental Bolt (m)
2904.3	17. 2901 18. 2902 19. 2903
2908.6	20. 2905 21. 2905.8 22. 2907
2912	23. 2910 24. 2910 25. 2911
2915.8	26. 2914.6 27. 2914.6 28. 2914.6
2919.6	29. 2917.2 30. 2918.2 31. 2918.8
2923.7	32. 2921 33. 2921.8 34. 2922.8
2927.1	35. 2924.6 36. 2925.6 37. 2926.4
2931.6	38. 2927.6 39. 2928.9 40. 2929.9

Layout



b) Phase 2

Key: ● 6 m long bolt
bolt installed - not monitored
single stage bolt
3½ m long bolt

TABLE 3. EXPERIMENTAL BOLT INSTALLATIONS

Additional vibrational loading tests were conducted at frequencies of up to 60 Hz on production hydraulic load cells using an Instron testing machine (Fig. 6) which also confirmed satisfactory performance. Further tests are planned to investigate the frequency response of the cells, up to the maximum frequency expected in this project of 5 kHz, by means of an electrodynamic vibration exciter. Using this method, force is calculated from an acceleration signal from a given axis and compared with the load cell output from the same axis making evident the frequency dependent error (ref. 3).

FIG. 3. LOAD CELL ARRANGEMENT PRIOR TO FITTING OF ACCELEROMETER

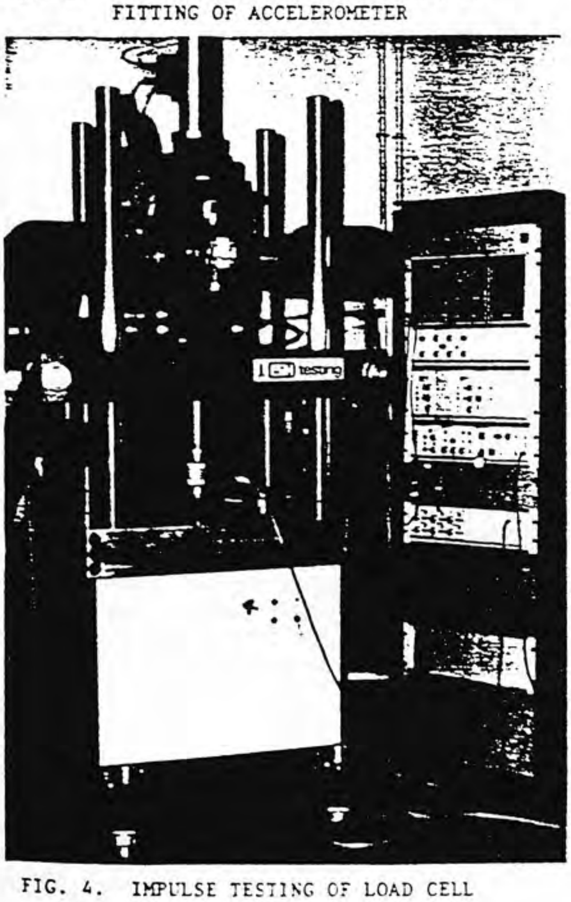


FIG. 4. IMPULSE TESTING OF LOAD CELL

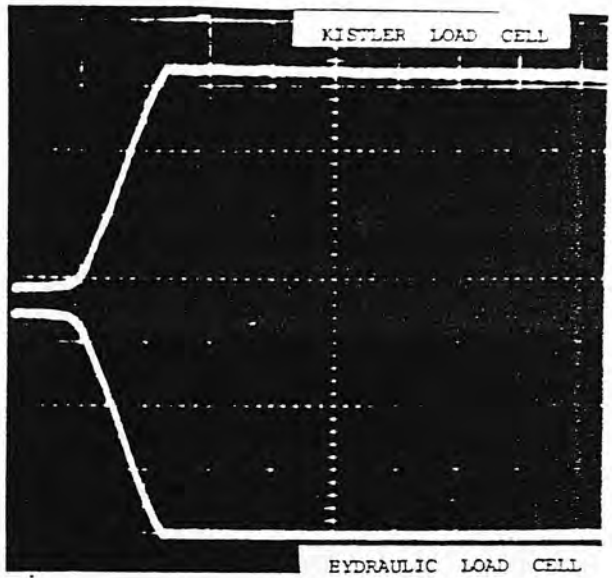


FIG. 5. RESULTS OF RAPID UNLOADING TEST FROM 70 KN PRECOMPRESSION

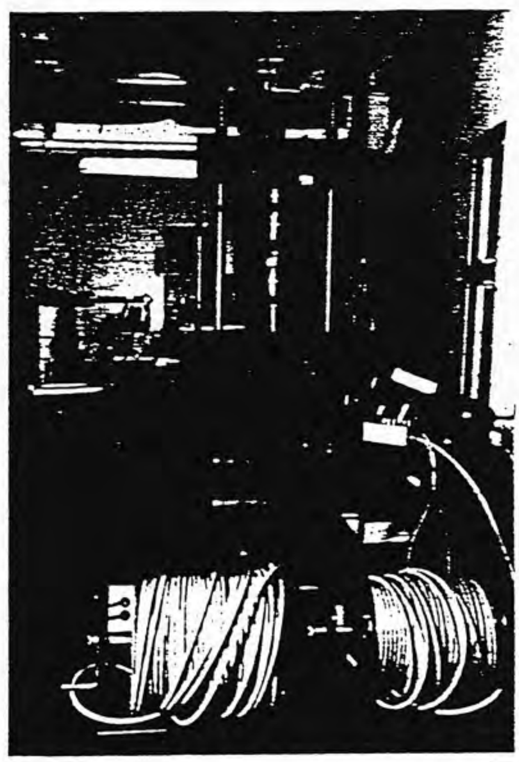


FIG. 6. VIBRATION TESTING OF LOAD CELL

Subsequent to the laboratory tests recording instrumentation was housed on site in a cabin positioned approximately 100 m from the tunnel entrance to ensure blast protection. To prevent excessive signal loss during its transmission through 400 m long cables, a signal conditioning and amplification system had, however, to be provided within the tunnel.

In consultation with the suppliers of the load cells, Geotechnical Instruments Ltd., the system outlined in Fig. 7 was devised. Each load cell is connected via waterproof transducer cable to its own 100 m cable drum which connects in turn into a single, larger 200 m cable drum of multiway cable which also houses a 10-channel amplifier and precision power supply unit used to energise the load cells and amplifiers. The complete installation is sealed to IP65 standard. The output from the amplifiers, which have a frequency response from DC to 5 kHz, is passed into the single 16 mm diameter multiway cable contained in the larger drum. The multiway cable is connected into a third drum containing a further 100 m of cable which terminates in the cabin with a sealed ABS box containing 10 BNC sockets for recorder connection.

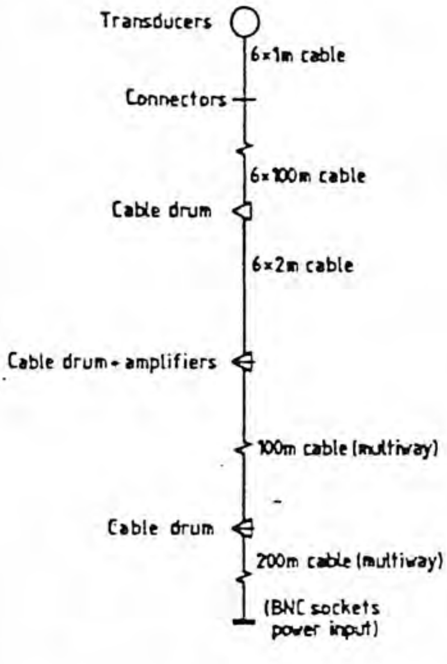


FIG. 7. LOAD CELL CABLE ARRANGEMENT

With this configuration the load cell conditioning and amplifier units were designed to be placed at least 100 m from the blast face on the assumption of a full face extraction. After manufacture of the systems the Contractor adopted bench and heading extraction which required provision of reinforced timber protection for the drum containing the amplifiers. Transducer installations were protected by steel box sections fabricated on site and cable protection was effected by steel channel sections or Armco ducting attached by rock dowels to the tunnel wall. At the tunnel portal, cable was housed inside sheet metal racks (Fig. 8) and from the rock face to the instrumentation cabin, the cable was placed in concrete ducting at ground level (Fig. 9). Power to the load cell system was provided in the site cabin by means of a lead-acid battery via the multiway cable.

Vibration Monitoring

Vibration was monitored by means of a single piezoelectric accelerometer disposed axially with respect to each rock bolt and mounted on the upper load distribution plate. These measurements were supplemented in the latter stages by triaxial measurements of vibration on the surrounding rock mass. Insitu calibration of the vibration monitoring installation was accomplished by means of a portable vibration generator.

FIG. 8. CABLE PROTECTION SYSTEM BEING INSTALLED OVER TUNNEL PORTAL

FIG. 9. CONCRETE DUCTING AND INSTRUMENTATION CABIN

In recording blast vibrations it is important to adjust the system to ensure a good quality recording without either overloading the recording system or introducing an unacceptable signal to noise ratio. To achieve this two types of accelerometer were selected with a 10:1 ratio in sensitivities, the lower one being suitable for measurements close to the blast. All vibration monitoring equipment was of Bruel & Kjaer Ltd. manufacture providing a frequency range from 0.1 Hz to 5 kHz. To prevent signal loss in the long cable lengths necessary, a line drive amplifier was attached to the top of each accelerometer with power supplied along each transducer cable from the cabin. At the site cabin the output from the system was a signal at a level of either 10 or 1 $mV/m/s^2$ dependant upon the accelerometer employed. Additional signal conditioning was also provided in the site cabin by charge amplifiers modified for voltage input. These amplifiers provided two stages of integration to give velocity or displacement signals for further processing.

Data Recording and Analysis

All transducer signals were recorded on FM magnetic tape to permit further processing either on site or at the Universities. On site data analysis was effected by means of a microcomputer based transient capture system with a memory size of 16 k samples per channel. The software for the system permitted Fast Fourier Transform analysis of stored waveforms or the display of power spectra. A link was also devised between the on site computer and the more comprehensive facilities at the Universities using modems, the telephone line and the inter-University network (JANET).

PRELIMINARY RESULTS

To date no detailed analysis of results has taken place, but Fig. 10 illustrates dynamic load changes in a research bolt positioned 1.9 metres from the tunnel face. 76 milli-seconds of activity is displayed which represents prestress fluctuations due to a single delay in the "fanned burn cut" blast pattern. The duration of the dynamic response of the rock bolt was 7 seconds for the full blast. In Fig. 10 the maximum increase and decrease in load are 13% and 8% respectively, of the initial lock-off load, and although not shown on the graph, the residual service load in the bolt was unchanged from its

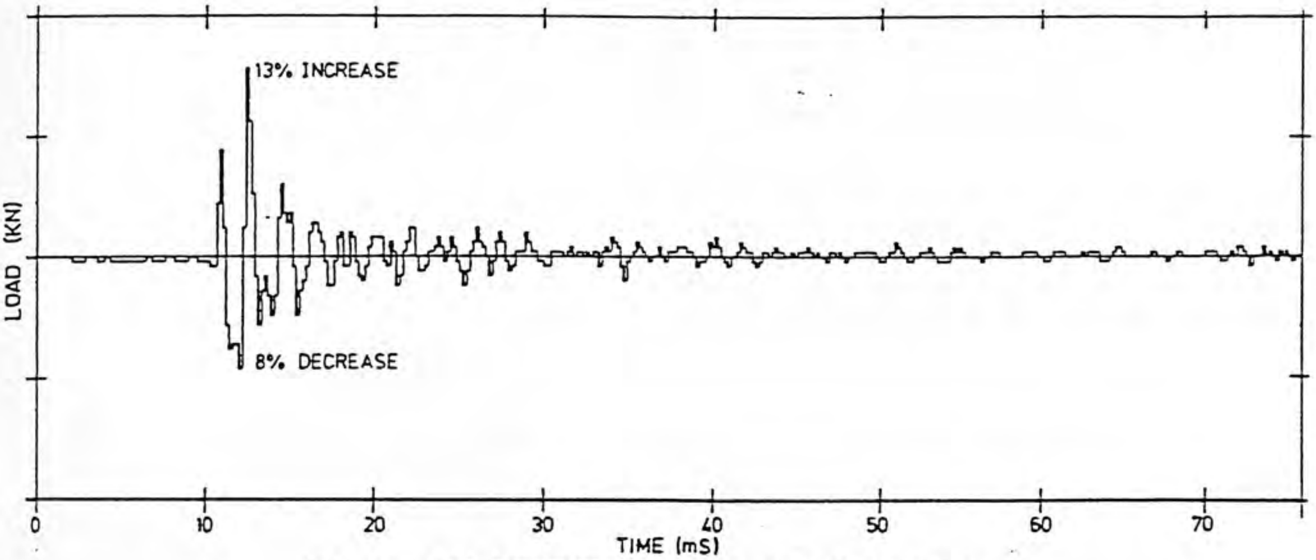


FIG. 10. LOAD FLUCTUATION WITH TIME FOR A SINGLE DELAY

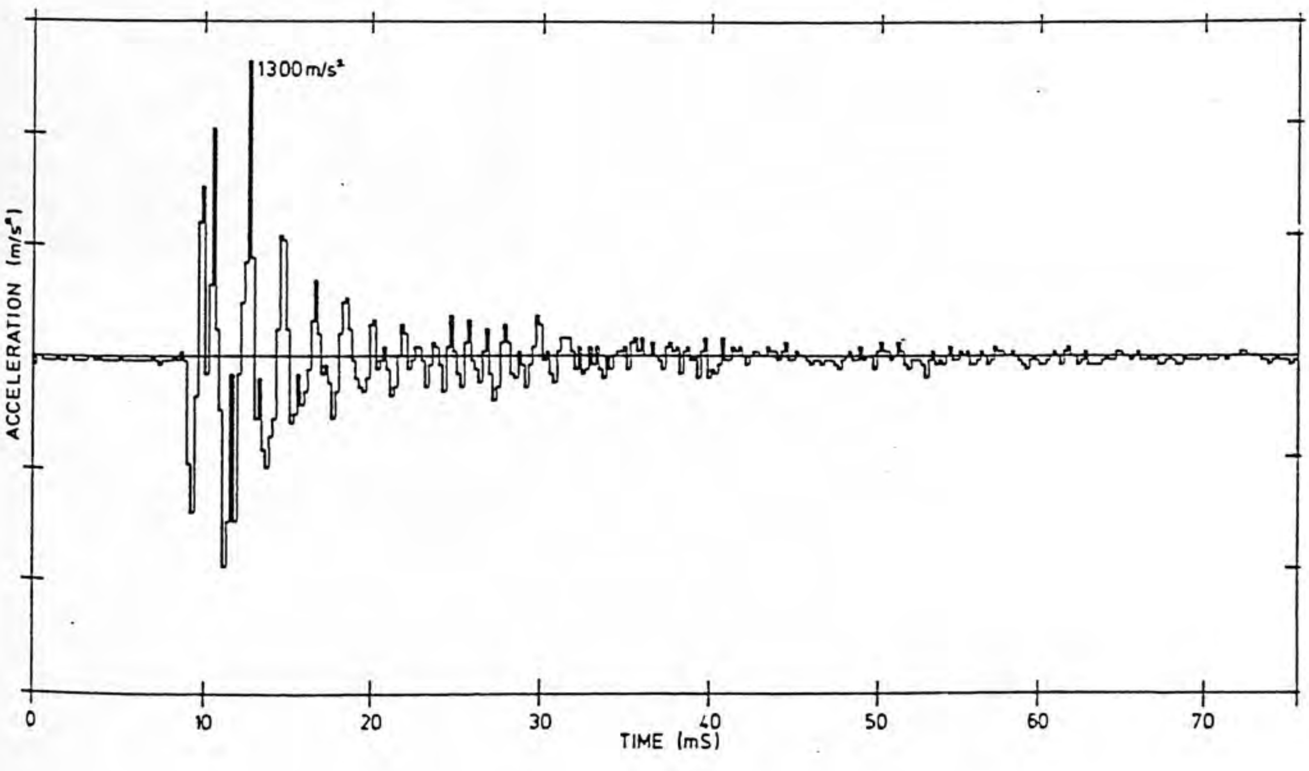


FIG. 11. ACCELERATION PLOT FOR A SINGLE DELAY

SPRAYED CONCRETE FOR UNDERGROUND SUPPORT

Author 3

Professor G.S. Littlejohn
Department of Civil Engineering
University of Bradford
England.

1. INTRODUCTION

Sprayed concrete is essentially a mixture of cement, aggregate and water, which is projected at high velocity from a nozzle into place to produce a dense mass. The original "Cement-Gun" pneumatic placing machine was invented in America by Carl Akeley in 1907 and George Rice, chief engineer of the Pittsburg Bureau Mines is credited with the first practical underground application of sprayed concrete at the Brucetown experimental mine in 1914.

With over 70 years of commercial success (ACI, 1966; ACI, 1976), sprayed concrete is an established engineering expedient which can be indispensable in deep excavations, tunnels, shafts, caverns and mines as a temporary or permanent support and lining (Fig. 1).

FIG. 1. Shotcreting a tunnel lining.

For underground works in rock, sprayed concrete normally serves two principal functions:

(i) for massive blocks sprayed concrete penetrates and covers joints to encourage composite block action and to prevent seepage or leaching out of joint infilling, and

(ii) in the case of heavily fractured zones, sprayed concrete forms a layer spanning between reinforcing elements such as steel ribs or rock bolts, and prevents surface degradation.

Thus the main advantage of sprayed concrete for underground support is its unique capability to prevent early loosening and disintegration of excavated surfaces, by providing sufficient shear resistance to the crown arch and wall of the excavation. The main economy and advantage of sprayed concrete is that it becomes an integral part of the excavation cycle when progressing through difficult or unstable ground formations.

2. DEFINITIONS

Although many terms have evolved throughout the world, the following definitions, published by the Concrete Society in 1979, are recommended for modern practice:

"Gunite" is a term used for sprayed concrete where the maximum aggregate size is less than 10 mm.

"Shotcrete" is used where the maximum aggregate size is 10 mm or more.

"Dry Process" is a mixture of cement and aggregate weigh or volume batched, thoroughly mixed "dry" and fed into a purpose-made machine wherein the mixture is pressurised, metered into a dry air stream and conveyed through a pipeline to a nozzle before which water as a spray is introduced to hydrate the mix which is projected without interruption into place.

"Wet Process" is a mixture of cement and aggregate weigh batched and mixed with water at site or in mixer trucks prior to being conveyed through a pipeline to a nozzle where air is injected and the mix projected without interruption into place.

"Flash Coat" is a term used for sprayed concrete applied as a thin layer to protect or prime the surface.

"Layer" is a term used for a discrete thickness of sprayed concrete built up from a number of passes of the nozzle and allowed to set.

"Rebound" is a term used for all material having passed through the nozzle which does not conform to the definition of sprayed concrete.

3. ROCK SUPPORT

In underground excavations the need for sprayed concrete, usually in association with other forms of support such as rock bolts, is often assessed initially using empirical methods (LINDER, 1963; DEERE et al, 1969) or classification schemes (BARTON et al, 1974; BIENIAWSKI, 1974; HOEK & BROWN, 1981).

As an example, CECIL (1970) proposed the empirical relationship between Rock Quality Designation (RQD) and support requirements in Table 1, based on 100 case studies in Swedish tunnels. Fig. 2 shows support requirements related to a rock classification.

RQD*	Support
>90%	Minimum (rock bolts or none)
60-90%	Intermediate (rock bolts and one shotcrete layer, 50-75 mm thick)
<60%	Maximum (rock bolts, steel mesh and multiple shotcrete layers, up to 200 mm total thickness)

*RQD = % core recovered with core pieces greater than 100 mm

TABLE 1. Support requirements related to RQD.

Generally speaking, there are no widely accepted criteria for quantifying the optimum area or thickness of sprayed concrete or the amount of any additional reinforcement which may be required. Rock classification systems are often highly subjective and leave a wide margin for personal engineering judgement (CECIL, 1970). Equally, it is dangerous to extrapolate empirical rules, which are based on local experience with limited geological conditions.

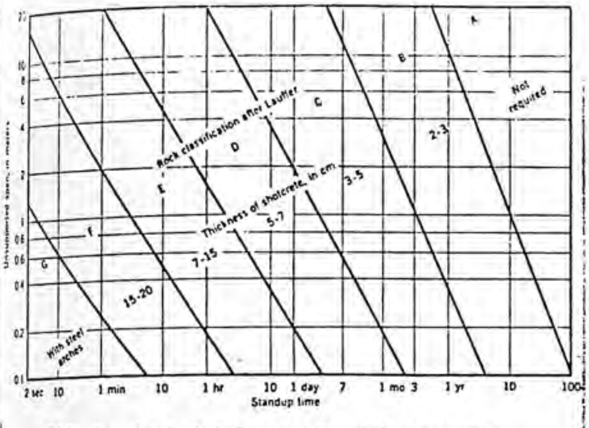


FIG. 2. Rock reinforcement with shotcrete (after Linder, 1963)

(A) No reinforcement required.

(B) 2-3 cm shotcrete; or rock bolts on 1-1.5 m spacing with wire net, occasionally reinforcement needed only in arch.

(C) 3-5 cm shotcrete; or rock bolts on 1-1.5 m spacing with wire net, occasionally reinforcement needed only in arch.

(D) 5-7 cm shotcrete with wire net; or rock bolts on 0.7-1 m spacing with wire net and 3 cm shotcrete.

(E) 7-15 cm shotcrete with wire net, rock bolts on 0.5-1.2 m spacing with 3-5 cm shotcrete sometimes suitable; or steel arches with lagging.

(F) 15-20 cm shotcrete with wire net and steel arches; or strutted steel arches with lagging and subsequent shotcrete.

(G) shotcrete and strutted steel arches with lagging.

Precedent practice in underground support indicates typical sprayed concrete thicknesses of 50 to 150 mm, with up to 250 mm for ribs, the greater thicknesses being built up in layers. Mesh reinforcement is specified for rock areas that are fractured to such an extent that spalling is probable, and mesh sizes of 50 mm to 200 mm using wire diameters of 2.5 mm to 5.0 mm are common. On occasions double meshes are used to accommodate thick layers of sprayed concrete due to overbreak.

Where required, sprayed concrete is ideally applied, at least as a flash coat, immediately after a local excavation or face advance and following mapping of the rock discontinuities. Early support is desirable to minimise initial rock convergence since it is inherently more difficult to stabilise the opening once rock movements are under way. In this regard, sprayed concrete can creep with the rock, and deformations of several cm have been recorded over periods of months without visible cracks or loss of strength in the lining, e.g. the Canadian National Railways Tunnel in Vancouver.

Fig. 3 compares a variety of support systems in terms of roof displacement in a 3.3 m diameter experimental tunnel in mudstone. At this site a combination of untensioned fully bonded resin rock bolts (1.8 m long at 0.9 m centres) and reinforced sprayed concrete (50 mm x 50 mm x 3.2 mm diameter steel mesh) provided the most stable solution.

For economy combined with early safe support the stiffening and strengthening rates of the concrete need to be optimised in relation to any dilating properties of the rock and the timing and positions of construction operations.

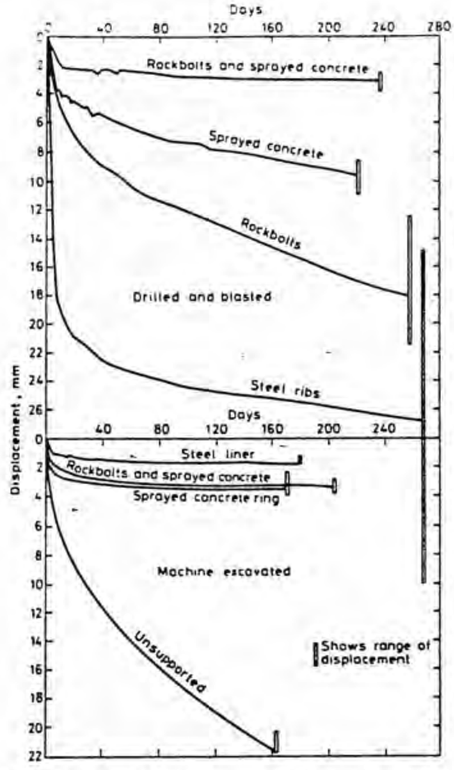


FIG. 3. Typical displacements and range of displacements of rock 0.3 m above crown in all supporting systems (after Ward et al, 1976)

4. DRY PROCESS

4.1 Equipment

(a) The feed wheel type is a double chamber machine in which the top chamber acts as an air lock to feed the material into the bottom chamber in which high air pressure is maintained. The material in the bottom chamber falls around the rotating feed wheel which moves pockets of material round in its spokes until each pocket comes opposite the outlet. At this point, the pocket is picked up by a high pressure air stream and is carried into and along the delivery hose. The machines are robust in construction and generally not expensive to maintain. Outputs range from 0.3 to 3 m^3/hour.

(b) The rotating barrel type (Fig. 4) consists of a number of cylindrical chambers set between two perfectly plain and parallel plates. As the barrel revolves, each chamber in turn becomes charged with material falling in from above; the chamber is sealed by passing into a blanked off area, and is then discharged by coming under air pressure from above which forces the material into the outlet where a further air supply blasts the material into the hose. The machines have a higher output and can cope with larger aggregates than the double chamber type. However, frequent replacement of plate wearing surfaces can be expensive. Outputs generally range from 1 to 5 m^3/hour.

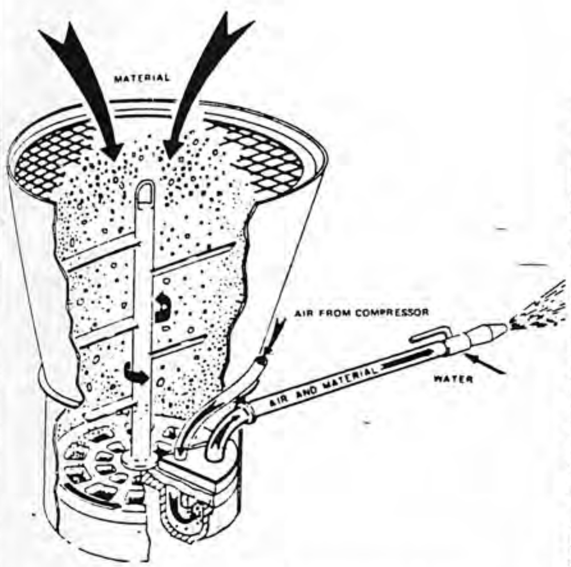


FIG. 4. Operating principle of rotating barrel machine

4.2 Mix Design

For fine aggregate gunite, Table 2 indicates the sand grading commonly recommended in North America.

Sieve Size (mm)	Percentage Passing (by weight)
9.5	100
4.8	95-100
2.4	80-100
1.2	50-85
0.6	25-60
0.3	10-30
0.15	2-10

TABLE 2. Recommended grading limits for gunite and fine aggregate in shotcrete.

For coarse aggregate shotcrete, Fig. 5 shows grading limits typically specified in Sweden, and natural well rounded aggregates are preferred.

For the dry process the ideal aggregate moisture content is 5-8% by weight, and the grading is normally restricted to 5 mm down in feed wheel equipment.

Finer sands result in great drying shrinkage while coarser sands will give more rebound. As dust coats the aggregate and can inhibit bond, fines below 0.1 mm are limited to 2% occasionally.

Aggregate:cement proportions by weight can vary from 5 down to 2, but 3.5 is most common and, combined with a water:cement ratio of 0.45, gives average crushing strengths of 10, 15 and 30 N/mm² at 3, 7 and 28 days respectively.

Table 3 lists typical 28 day physical properties for a 3.5:1 sand:cement gunite shot onto a vertical face under production conditions.

Property	Value
compressive strength	24-58 N/mm²
tensile strength	1.9-3.6 N/mm²
flexural strength	4.9-7.3 N/mm²
elastic modulus	25-33 kN/mm²
drying shrinkage	(290-570) 10^{-6} m/m
expansion coefficient	(6.3-11.8) 10^{-6} m/m°C
contraction coefficient	(11.5-13.4) 10^{-6} m/m°C
water absorption	5-9% (by wt. of dry material)
water penetration at 100 lN/m²	3-22 mm
density	2.0-2.2 Mg/m³

TABLE 3. Typical properties of production gunite.

The variable compressive strengths are due primarily to stratification within the gunite and local variations in the sand:cement and water:cement ratios. Given correct batching of materials, the longer the mixing time the smaller will be the variation in sand:cement ratio. The influence of sand:cement ratio on strength is shown in Table 4. Water:cement ratio is more difficult to control since fluctuations in flow rate occur at a frequency of 5 per second and the nozzle operator can only cater for average flow conditions with his water control valve. However, the addition of water 5 metres back from the nozzle assists in producing a good homogeneous mix.

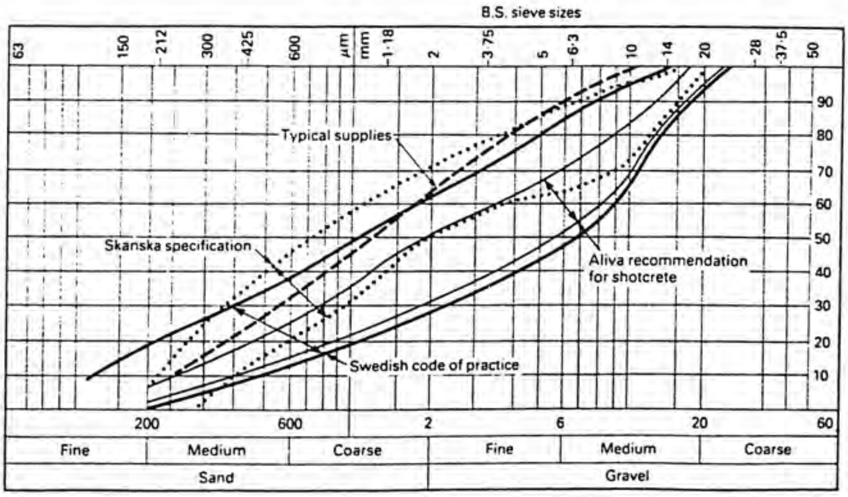


FIG. 5. Typical grading curves for shotcrete (after Waller, 1980).

Sand:Cement Ratio	Average Compressive Strength (N/mm^2)	Average Tensile Strength (N/mm^2)
3.25	42	2.9
3.50	37	2.5
3.75	33	1.9

TABLE 4. The influence of sand:cement ratio on 28 day strength.

4.3 Practical Procedures

Whenever possible, except when enclosing major reinforcing steel (up to 25 mm diameter), the nozzle should be held at right angles to the surface at a distance of 0.8 to 1.2 m. When enclosing steel, the nozzle should be held so as to direct the material around the reinforcement. For high quality structural work a nozzle operator's helper equipped with an air jet should blow out all rebound which may have lodged on the gunite or reinforcement.

For efficient spraying Table 5 indicates typical air requirements for hose lengths of 50 metres with the nozzle not more than 7 metres above the delivery equipment (gun). Operating pressures are generally increased by 35 kN/m^2 for each additional 15 metres (horizontally) or 7.5 metres (vertically). Air pressures should be held constant during spraying and water pressure at the nozzle is normally 100 to 150 kN/m^2 above line air pressure.

Air Flow (m^3/min)	Hose Diameter (mm)	Nozzle Diameter (mm)	Gun Pressure (kN/m^2)
7	25	19	275
10	38	32	380
21	50	44	590

Note: Compressed air should be dry.

TABLE 5. Typical air requirements for gunite.

Gunite mixes can be transported up to 200 metres horizontally but for longer distances segregation of the "dry" materials can occur prior to hydration at the nozzle. Gunite can also be shot at a height of 100 metres above the gun using 21 m^3/min at 690 kN/m^2. A minimum hose length of 20-25 metres is required to allow the materials to accelerate to the nozzle velocity, and there should be as few bends as possible with no bend radius less than 1 metre. The effect of air pressure on gunite performance is shown in Fig. 6.

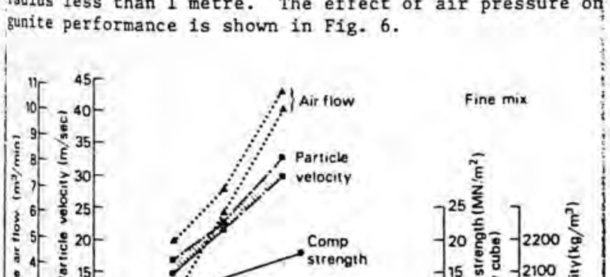


FIG. 6. Effect of air pressure on particle velocity, density and strength of gunite (after Ward & Hills, 1977).

The amount of rebound varies with position of spraying, air pressure, cement content, maximum size and grading of aggregate and layer thickness. Typical vlaues are 5-15% for floors and shallow slopes, 15-30% for steep slopes and vertical faces, and 20-50% for overhead work. Initially, the % rebound is large but it reduces after a soft layer has been built up. Rebound is much leaner and coarser than the original mix e.g. 3.5% cement by weight and 80% of the material exceeds 0.6 mm. Dust emission and rebound can be reduced by adding the water at high pressure (up to 1500 kN/m^2) some 3 to 5 metres back from the nozzle, or by the use of accelerators (see 6).

5. WET PROCESS

5.1 Equipment

Modern positive displacement piston pumps have been developed for sprayed concrete since the 1950s along with pure pneumatic feed guns. Piston pump machines require the premixed wet concrete to be more plastic than the pneumatic feed type guns, and with the higher water:cement ratios, accelerators are necessary for successful concrete plcement on vertical and overhead work. As a consequence, the rotating roller squeezcrete pump (Fig. 7) was developed in 1965 to enable low slump (<50 mm) shotcrete to be conveyed into place.

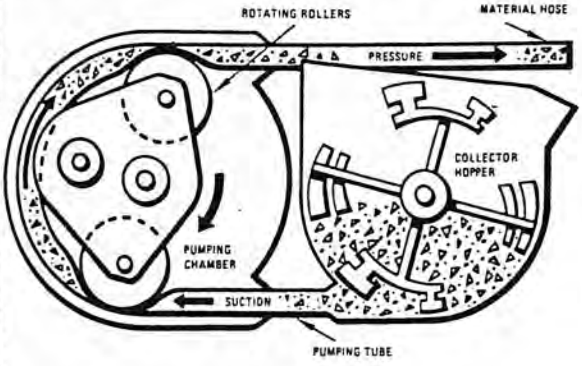


FIG. 7. Operating principle of squeez-crete pump (after Littlejohn, 1980).

For 50 mm diameter line and a hand held nozzle, outputs range from 5 to 10 m^3/hour although at the higher rate the nozzle operator needs to be spelled during the shift.

For a 75 mm line and mechanically held 75 mm nozzle, rates up to 15 m^3/hour can be attained.

5.2 Mix Design

Concrete mixes for shotcreting have similar physical properties to standard mixes but tend to be richer in cement and are often designed for high early strength. Since the mixes are normally pumped through 50 mm lines, mix designs contain more sand, usually 55 to 65% of the total aggregates by weight.

Tables 2 and 6 show the typical gradings for fine and coarse aggregates, respectively.

Sieve Size (mm)	Percentage Passing (by weight)		
19	100	100	90-100
12.5	100	90-100	-
9.5	85-100	40-70	20-55
4.8	10-30	0-15	0-10
2.4	0-10	0-5	0-5
1.2	0-5	-	-

TABLE 6. Recommended coarse aggregate grading limits for shotcrete.

Generally speaking, washed natural sands are preferred. The very fine sand (% passing 0.3 mm) together with the cement and water, provides the lubricant between the distribution lines and the mix during pumping. Optimum mixes contain fine components comprising from 15 to 30% that pass the 0.3 mm sieve and 5 to 10% that pass the 0.15 mm sieve. Coarse aggregates comprise either gravels or crushed stone.

The amount of water is selected on the basis of slump or specified strength. For general support a slump of 75 to 100 mm is practical, but may be reduced to 50 mm where no accelerator is permitted. The upper range is 150 to 175 mm because at higher values the mix loses cohesion and coarse aggregate tends to settle out leading to line blockages. Table 7 includes some typical water contents for specified maximum aggregate sizes to give a 75 to 100 mm slump. An increase or decrease of 5 litres per cubic metre of concrete will increase or decrease the tabulated slump by about 25 mm.

Water Quantity (litres/m^3)	Maximum Size of Aggregate (mm)
227	9.5
218	12.5
209	19

TABLE 7. Required mixing water per cubic metre of concrete (including moisture in aggregate).

In regard to strength it is always advisable to allow a 15 to 20% margin of safety above the actual specified strength to compensate for any variations during production. Table 8 suggests water:cement ratios for various 28 day compressive strengths based on field experience using ordinary Portland cement and damp curing conditions.

28 day Compressive Strength (N/mm^2)	Water:Cement Ratio
35	0.48
28	0.57
21	0.68
14	0.82

TABLE 8. Suggested water:cement ratios for shotcrete.

The strength after 7 days of curing is generally equal to two thirds of the 28 day strength. The usual range of cement content is 280 to 450 kg/m^3 depending on strength and other requirements such as shrinkage. Below 280 kg/m^3 the lack of cement/water paste affects lubrication whilst above 450 kg/m^3 cost and shrinkage are important considerations.

5.3 Practical Procedures.

The performance of the concrete pump is largely determined by the concrete pipeline system, taken in conjunction with the pump's output capacity and maximum discharge pressure. Based on the results of field experience Fig. 8 relates pumping rate to line pressure, line diameter, line distance and slump. Thus for example, a line pressure of 14 bars (1400 kN/m^2), a 100 mm slump and a pumping distance of 91 metres, permits a maximum pumping rate of 15 m^3/hour with a 75 mm diameter line. Since Fig. 8 is based on pumpable mix designs using gravel aggregate, line pressures are increased by 12%, for crushed stone mixes.

In computing the effective pumping distance vertical lengths take twice the pressure of equivalent horizontal lengths. In practice, the maximum effective line distances for wet process shotcreting are 100 and 200 metres for 50 and 75 mm diameter lines, respectively, using 150 mm slump concrete.

For production spraying the nozzle is positioned 0.6 m to 1.5 m from the surface, although larger distances are feasible. For vertical surfaces it is normal to commence work at the bottom to avoid trapping rebound. In underground work however, depressions in the rock surface in any particular area are generally filled first. No sprayed concrete should be sprayed on any overhanging surface, other than by remotely controlled nozzles, until the surface has been inspected and declared safe.

With accelerated mixes, layer thicknesses up to 300 mm can be sprayed, but without an accelerator 50 mm is a practical limit.

In general, rebound is minimal and with accelerated mixes nil, 5% and 10% rebound has been monitored for horizontal, vertical and overhead surfaces, respectively.

In order to reinforce the extreme end of a previously applied shotcrete lining, and to avoid damage resulting from the proximity of blasting operations, it is advisable to overlap successive applications by about 600 mm.

6. ACCELERATORS

In rock support work it is often necessary to attain concrete strength as soon as possible. This is normally achieved by the use of accelerators in either liquid or powder form. Fig. 9 from the Dinorwic Power Station in Wales shows the effects of two types of accelerator in varying proportions by weight of cement on strength development for dry process shotcrete.

For wet process shotcrete in general the following setting times and strengths have been specified using 1 to 3% Sigunite by weight of cement.

Initial set of cement/admixture paste = 3 min
Final set of cement/admixture paste = 12 min
Compressive strength of concrete at:
8 hours = 4 N/mm^2 — 7 days = 22 N/mm^2
24 hours = 10 N/mm^2 — 28 days = 35 N/mm^2

Initial sets of 30 to 200 seconds can be attained but a 3 minute set avoids too rapid a heat generation.

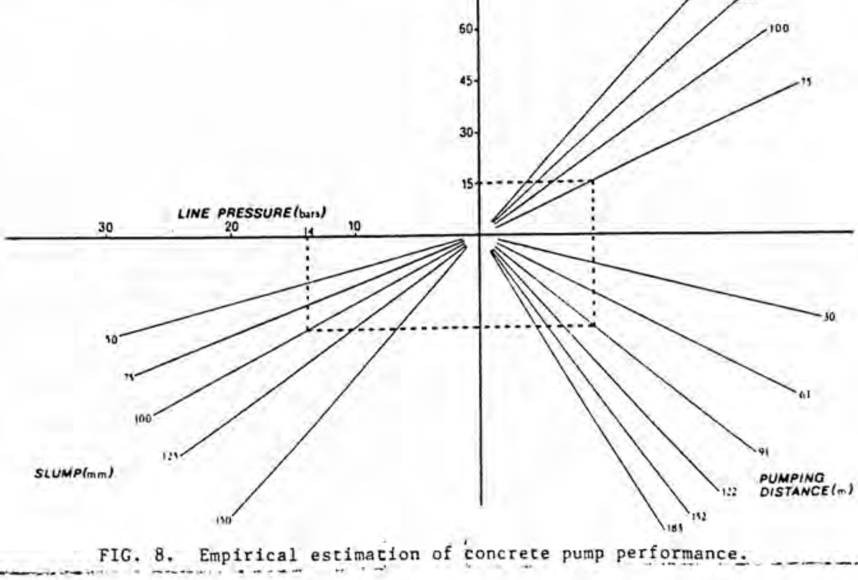


FIG. 8. Empirical estimation of concrete pump performance.

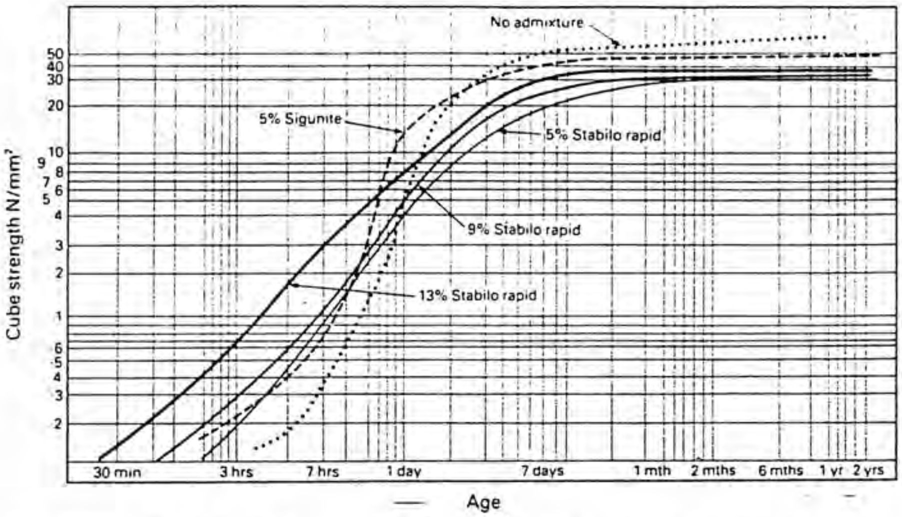


FIG. 9. Comparison of early strength development using different accelerator concentrations (after Waller, 1980).

There are several advantages and disadvantages to the use of these admixtures. Advantages include high early strength, reduction in rebound, ability to stem moderate water flows by flash set and an ability to build up thick layers quickly. Disadvantages include lower long term strength, highly caustic nature, difficulty in removing rebound material and limited data on long-term durability. In this regard, only the first primary lining requires accelerators in underground support.

7. SURFACE PREPARATION

The rock surface should be cleaned of loose material, mud and other foreign matter which might prevent bond of the sprayed concrete onto the rock surface. After washing down, surfaces should be damp but exhibiting no free water prior to the application of sprayed concrete.

Where flow of water could interfere with the application of sprayed concrete or cause leaching of cement, the water should be led by pipes, gutters, or permeable membranes to some point where it may be plugged off or transported away, after the application of sprayed concrete.

Porous surfaces should be kept damp for several hours before concrete spraying. In practice the nozzle operator usually scours clean the area ahead of the application with an air-water jet, then the water is shut off and all free water is blown away by compressed air.

8. QUALITY CONTROLS

During the contract plywood moulds (thickness ≮ 20 mm) for unreinforced test panels 750 mm x 750 mm x 100 mm thick should be rigidly fixed alongside the production work in positions and numbers judged appropriate (Concrete Society, 1979). The moulds should be sprayed and cured under working conditions.

100 mm diameter cores should be cut from test panels at right angles to the plane of the panel approximately 48 hours after the panel has been sprayed. Cores should not be taken within 125 mm of the edges of the panel. These cores permit the crushing strength to be tested at 3, 7 and 28 days, by which time a minimum strength of 20 N/mm^2 should have developed.

For 100 mm cores insitu, the concrete must be of sufficient thickness to allow an acceptable height:diameter ratio (1.0-2.0). Tests of this type may be specified on a weekly basis, or for high production works every 50 m^3 of concrete placed.

When required 25 mm diameter cores can be taken to determine concrete thickness and quality of the sprayed concrete.

The preparation of test panels and coring represent a time consuming and costly procedure however, and simple insitu tests which can be carried out at a high frequency are advantageous.

Soundness can be tested by hand hammer, a hollow response indicating a possible lack of bond or other defect. The compressive strength of the sprayed concrete can be assessed by the Schmidt hammer, calibrated by reference readings taken on sprayed concrete where cores for strength assessment have also been obtained. At least ten readings should be used for each strength assignment.

More recently the Windsor Probe System has been introduced for strength testing based on the penetration of a steel probe driven into the hardened concrete (KOPF et al, 1978).

In addition to performance testing, full records of all material quantities supplied and used should be maintained, including aggregate gradings, moisture contents and cement certificates.

Lighting also influences quality and a minimum lighting intensity of 50 lux should be maintained in the spraying area.

9. CONCLUSIONS

Technical literature provides convincing evidence of many successful applications of sprayed concrete for underground support over the past 70 years.

The technique is flexible in that the area and thickness of the sprayed concrete can be adjusted quickly on site and used in conjunction with a variety of other support systems to meet the demands of varying ground conditions and excavation geometries.

To achieve satisfactory results skilled operators are necessary and the work requires to be well planned and properly supervised.

Since a variety of techniques are available, specifications for sprayed concrete should be designed so that the end product is clearly defined, leaving the greatest possible degree of freedom to the contractor to use his experience and expertise to achieve the desired finished concrete for the project.

REFERENCES

1. AMERICAN CONCRETE INSTITUTE (1966)
Shotcreting.
ACI Publication SP-14,
ACI, P.O. Box 4745, Detroit, Michigan, U.S.A.

2. AMERICAN CONCRETE INSTITUTE (1976)
Shotcrete for Underground Support.
ACI Publication SP-54,
ACI, P.O. Box 4745, Detroit, Michigan, U.S.A.

3. BARTON, N., LIEN, R. & LUNDE, J. (1974)
Engineering Classification of Rock Masses for the Design of Tunnel Support.
Rock Mechanics, 6(4), 189-236.

4. BIENAWSKI, Z.T. (1974)
Geomechanics Classification of Rock Masses and its Application in Tunnelling.
Proc. 3rd Congress, ISRM, Denver, Vol. IIA, 27-32.

5. CECIL, O.S. (1970)
Shotcrete Support in Rock Tunnels in Scandinavia.
Civil Engineering (ASCE), January, 74-79.

6. CONCRETE SOCIETY (1979)
Specification for Sprayed Concrete.
The Concrete Society, Grosvenor Gardens, London.

7. DEERE, D.U. et al (1969)
Design of Tunnel Liners and Support Systems.
U.S. Dept. of Transport Contract 3-0152,
University of Illinois.

8. HOEK, E. & BROWN, E.T. (1981)
Underground Excavation in Rock.
Institute of Mining and Metallurgy, London.

9. KOPF, R.J., COOPER, C. & WILLIAMS, F. (1978)
Insitu Strength Evaluation of Concrete Case Histories and Laboratory Excavations.
Symposium on Insitu Strength Evaluation of Concrete,
ACI, Houston, Texas.

10. LINDER, R. (1963)
Technologie des Spritzbetons.
Beton und Stahlbetonbau, Vol. 58, Heft 2-3.

11. LITTLEJOHN, G.S. (1980)
Wet Process Shotcrete.
Proc. Symp. on Sprayed Concrete,
Concrete International 80, pp. 18-35.
The Construction Press, London.

12. WALLER, E. (1980)
Dinorwic Tunnels and Caverns.
Proc. Symp. on Sprayed Concrete,
Concrete International 80, pp. 36-51,
The Construction Press, London.

13. WARD, W.H., COATS, D.J. & TEDD, P. (1976)
Performance of Tunnel Support Systems in the Four Fathom Mudstone.
Building Research Establishment CP.25/76, Garston, England.

14. WARD, W.H. & HILLS, D.L. (1977)
Sprayed Concrete : Tunnel Support Requirements and the Dry Mix Process.
Building Research Establishment CP.18/77, Garston, England.

THE THIRD INTERNATIONAL CONFERENCE ON
UNDERGROUND SPACE AND EARTH SHELTERED BUILDINGS

1989

ROCK ANCHORAGES FOR UNDERGROUND SUPPORT

Author 1
Professor G.S. Littlejohn
Department of Civil Engineering
University of Bradford
England.

Author 2

Author 3

1. INTRODUCTION

Early reports of anchoring bars into rock to secure a roof date from 1918 in the Mir mine of Upper Silesia in Poland, and by 1926 faces of an inclined shaft in shales in Czechoslovakia were secured against caving by grouted bars installed in a fan pattern. Over the past 70 years millions of anchorages have been installed in most classes of rock to stabilise and facilitate the construction of deep excavations, tunnels, shafts, caverns and mines.

Rock reinforcement by anchorages strengthens the rock mass surrounding an excavation by preventing the detachment of loose blocks, by increasing the shear resistance of discontinuities, and by enhancing the interlocking nature of individual blocks. This results in a reinforced zone within the rock mass which maintains the integrity of the excavated surface, possesses sufficient flexibility to allow for the redistribution of stresses around the excavation, and has enough stiffness to minimize the dilation of discontinuities within the rock mass surrounding the excavation.

In the specific case of localised reinforcement to overcome spalling, grouted or mechanical rock bolts and dowels are a traditional solution e.g. Seelisberg tunnel in Switzerland. The use of rock bolts in the People's Republic of China has been described by Liu and Huang, 1983. In difficult rock conditions, long post-tensioned anchorages may be required e.g. the 14 km Arlberg tunnel in the Austrian Alps where tendons up to 12 m were used.

On a larger scale, high capacity rock anchorages may be employed to provide or improve the overall stability of caverns e.g. Roncovalgrande, Italy, El Toro, Chile, Waldeck, West Germany (Fig. 1), Drakensburg, South Africa and Dinorwic, Wales.

Anchorages may be used as the sole means of providing stability or they may be used in conjunction with concrete linings or steel arches.

FIG. 1. Self supporting cavern 106 m long, 54 m deep and 33.5 m wide using 784 permanent rock anchorages installed to lengths of 20 to 28 m and post-tensioned to working loads of 940 to 1300 kN, at Waldeck II Pumped Storage Scheme in West Germany.

2. DEFINITIONS

"Rock Anchorage" is an installation that is capable of transmitting an applied tensile load to load bearing rock. The installation consists basically of an anchor head, free anchor length and fixed anchor.

"Rock Bolt" is a specific form of rock anchorage tensioned during installation, where a steel bar is fixed in rock.

"Rock Dowel" is a specific form of untensioned rock anchorage where a steel bar is fixed in rock.

All three types of anchorage may be required in a reinforcement pattern for highly fractured rock above a wide span (Fig. 2).

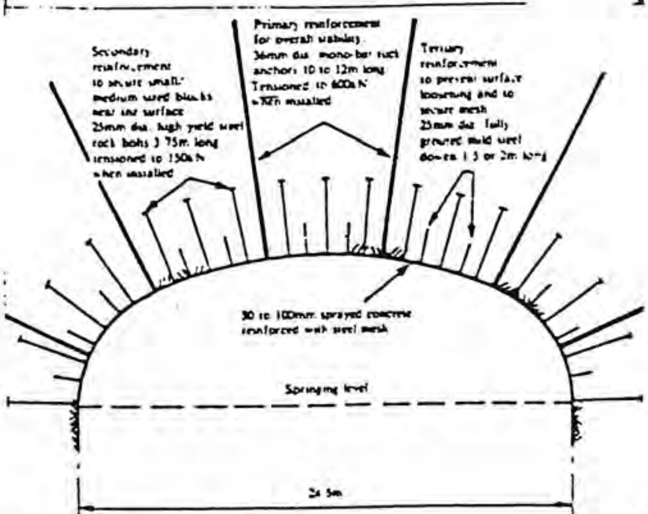


FIG. 2. Primary, secondary and tertiary reinforcement (after Douglas et al, 1979).

3. ROCK SUPPORT

Instability of an underground excavation may be related to the following:

(i) movement of blocks or wedges of rock under the action of external forces, particularly gravity and pressure due to water;

(ii) movement of blocks or wedges of rock under the action of in situ stresses;

(iii) failure of intact rock due to overstressing;

(iv) deterioration or chemical degradation of rock material exposed to atmospheric conditions.

In general, in situ (pre-excavation) stress levels increase with depth below ground surface. Thus, at shallow depths, stability of an underground opening is generally controlled by the three-dimensional geometry of the excavation and by the properties of the rock structure. At greater depths, the stability of the excavation is controlled by the limitation of stress induced failures of the intact rock or block movement along pre-existing discontinuities.

Assessment of stability should be based on detailed geotechnical investigation of the rock structure using stereographic projection or other graphical methods (Hoek & Brown, 1981) in conjunction with the determination of stress conditions relative to rock mass strength using elastic and elasto-plastic stress analysis methods.

Typical modes of failure are indicated in Fig. 3 together with an indication of the function of rock anchorages in maintaining stability.

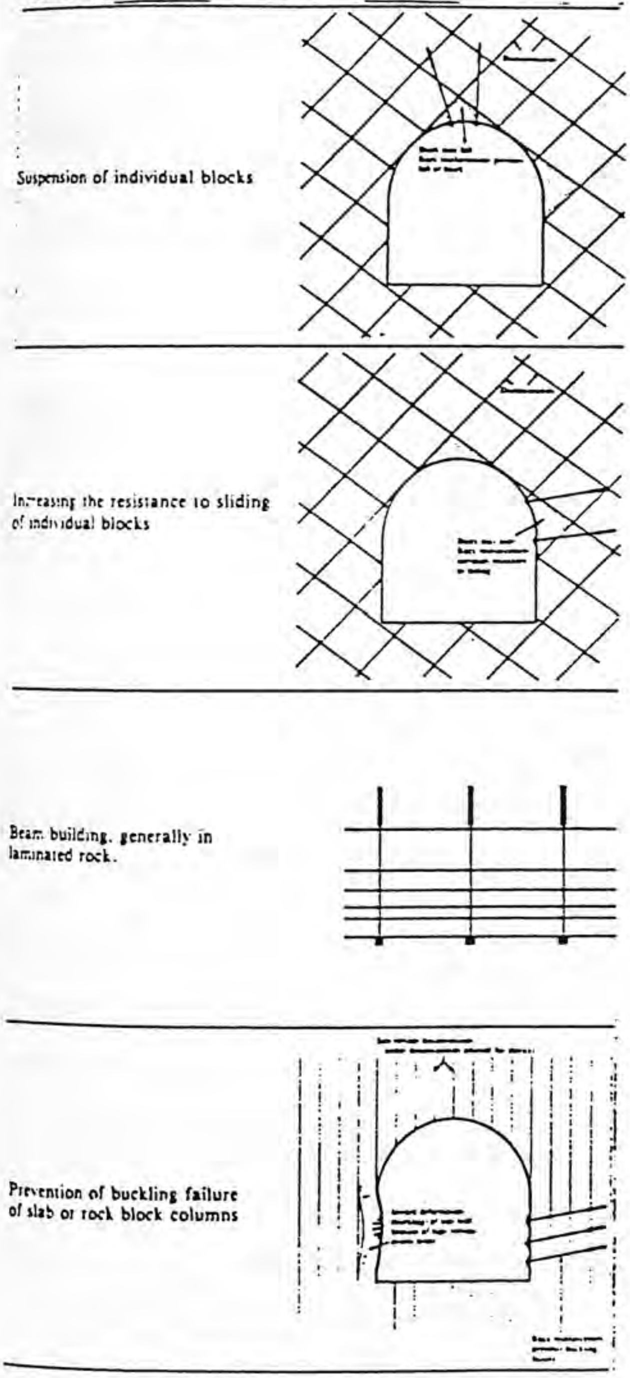


FIG. 3. Theoretical mechanisms of rock reinforcement (after CIRIA, 1983).

The selection of anchorages in a given situation is largely a matter of experience and judgement. Comparison with previous experience, together with considerations of classification schemes and empirical guidelines can provide useful assistance. (Hoek and Brown, 1981; Barton et al, 1974; Farmer and Shelton, 1980).

Anchorages should be selected to suit the conditions encountered (Hoek and Brown, 1981; US Army Corps of Engineers, 1980). The range of application is summarized in Table 1.

Rock dowels

Use: (a) to ensure stability of areas very near to the excavated surface;

(b) to reinforce rock to be removed at a later stage;

(c) for general support when installed very close to an advancing face, when tension is developed after installation;

(d) for pre-reinforcement prior to an excavation.

Rock: suitable for all types. In weak rocks sufficient length should be allowed to develop the tensile strength of the dowel.

Rock bolts

Use: for general support in all types of underground opening;

Rock: (a) bolts with mechanical fixed anchors are suitable for use in hard rocks only;

(b) bolts with grouted fixed anchors may be used in all rock types. In soft rocks, or where clay infilling has a tendency to line the drillholes, there may be insufficient anchorage capacity available for resin-grouted fixed anchors.

Maximum support pressure[1]: 300 kN/m^2.

Rock anchorages

Use: for reinforcement of large openings which require high support pressures and long length of reinforcement.

generally used in combination with bolts, dowels or sprayed concrete.

Rock: suitable for all rock types but care should be exercised in rocks with a combination of low RQD[2], heavily jointed or crushed rock, smooth slickensided or filled joints, high water inflows, high in situ stresses, swelling or squeezing rock.

In weak rocks, anchorages appropriate to soils may be required.

Minimum support pressure: 200 kN/m^2
Maximum support pressure: 600 kN/m^2

NOTES: 1. Support pressure = support pressure available at time of installation

2. RQD = Rock Quality Designation, i.e. % core recovered with core pieces greater than 100 mm.

TABLE 1. Use of anchorage reinforcement in rock (after BS.8081).

Initial dilation of discontinuities following excavation may lead quite rapidly to loss of integrity of the mass, and intact blocks progressively slide and rotate relative to adjacent blocks. Reinforcement should be installed, therefore, as soon as possible after excavation.

An important distinction should also be made between bonded and unbonded reinforcement (Fig. 4). An anchorage can either be bonded to the surrounding rock over its full length or decoupled over the free anchor length. A fully bonded anchorage, which is that normally used, provides restraint along the full free length, thus minimising the dilation of joints and providing the greatest potential for achieving the desired combination of flexibility and stiffness. A decoupled free length is generally only used in cases where anchorages are to be restressed during service where substantial post tensioning movement is expected or where a substantial amount of localised movement is expected which may overstress the anchorage. In current practice fully bonded bolts are preferred to expansion bolts which are useful mainly as local safety measures.

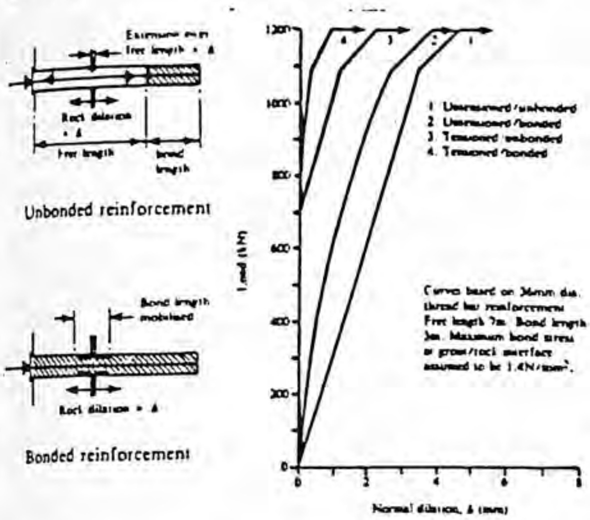


FIG. 4. Comparison of reinforcement systems (after CIRIA, 1983).

Reinforcement may be tensioned when it is installed or tension may be introduced by rock movements following installation. If elements are installed and grouted very close to an advancing face, there may be no need to tension the reinforcement, as load is induced by subsequent rock deformation. However, if it is not possible to ensure that installation and grouting are carried out quickly enough to prevent excessive rock movement, useful reinforcement can be achieved only if tensioned at the time of installation.

4. ROCK ANCHORAGE DESIGN

4.1 Fixed Anchor Design: Cement Grouted Anchorages

Whilst there is a wide variety of anchorage types in rock, the straight shaft tremie or packer grouted type is the most popular in current practice on the basis of cost and simplicity of construction. For such rock anchorages, designs are based on the assumption of uniform bond distribution (Littlejohn & Bruce, 1977). Thus the pull-out capacity of the fixed anchor T_f, in kN, is estimated from equation (1):

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

where: τ_{ult} is the ultimate bond or skin friction at the rock/grout interface (in kN/m^2)

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

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

Equation (1) is based on the following assumptions:

(i) Transfer of the load from the fixed anchor to the rock occurs by a uniformly distributed stress acting over the whole of the perimeter of the fixed anchor.

(ii) The diameters of the borehole and the fixed anchor are identical.

(iii) Failure takes place by sliding at the rock-grout interface (smooth borehole) or by shearing adjacent to the rock-grout interface in the weaker medium (rough borehole).

(iv) There is no local debonding at the rock-grout interface.

The assumption of a uniformly distributed stress along the fixed anchor may require careful consideration in terms of the likely stress concentrations at the proximal end of the fixed anchor in weak, deformable rock. Under such conditions it may be necessary to base the design directly on proving test results.

4.2 Rock/Grout Interface

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

For guidance, Littlejohn and Bruce, 1977, and Barley, 1988 provide rock-grout bond (skin friction) values that have been recommended for design or used in practice.

4.3 Grout/Tendon Interface

Bearing in mind the compressive strength (30 N/mm^2) required for cement based grouts prior to stressing, the ultimate bond assumed to be uniform over the tendon bond length should not exceed

1.0 N/mm^2 for clean plain wire or plain bar;
1.5 N/mm^2 for clean crimped wire;
2.0 N/mm^2 for clean strand or deformed bar;
3.0 N/mm^2 for clean locally noded strands.

The above values may be applied to single unit tendons and to parallel multi-unit tendons where the clear spacing is not less than 5 mm. For noded strands or tendons that can mobilise mechanical interlock or the shear strength of the grout, the minimum spacing does not apply. For resinous grouts ultimate bond values should be obtained from site proving tests in the absence of relevant documented test data.

4.4 Concentration of Steel Tendon in Borehole

In high capacity anchorages (>2000 kN) debonding may occur as the ductile tendon transfers stress to the brittle cement grout (Fig. 5). The subject demands more study, but since tendon density appears to be influential, the cross sectional area of the steel tendon should not normally exceed 15% of the borehole area of parallel multi unit tendons and 20% of the borehole area for single unit tendons e.g. bar.

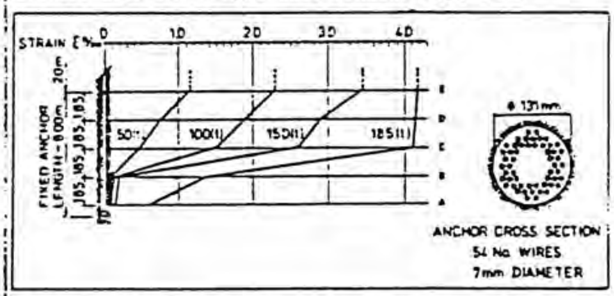


FIG. 5. Strain distribution along tendon in fixed anchor zone of a 2200 kN capacity anchorage (after Muller, 1966).

For rock bolts, the ratio of tendon area to borehole area should be in accordance with manufacturer's recommendations. In the case of resin capsules for example, the annular space between bar and borehole is critical for efficient mixing.

4.5 Fixed Anchor Length

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

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

The fixed anchor length should not exceed 10 m.

4.6 Fixed Anchor Design:Mechanically Anchored Rock Bolts

No design rules are provided for mechanically anchored rock bolts. The working or ultimate loads are generally given by the manufacturer; such values are usually based on the result of pull-out tests. As the conditions under which such tests are performed are not normally available, the working load should be ascertained on site from tests. These should investigate, in particular, the effect of changes in borehole orientation with respect to natural features in the rock and the effect of vibrations, where excavation is by drill and blast.

4.7 Fixed Anchor Design:Resin Bonded Rock Bolts

For rock bolts grouted using resin capsules, the bond length has been estimated using the following empirical relation (Daws, 1979).

$$L = 2.5\ P + 50$$

where L = bond length (mm)

P = working load of reinforcing element (kN).

The bond length should not be less than 400 mm.

Where limited information is available proving tests are recommended, especially in weak, deformable rocks (see 8). This will confirm the adequacy of the adopted fixed anchor lengths in the prevailing conditions.

4.8 Safety Factors

In current practice, the load safety factor of an anchorage is the ratio of ultimate load-holding capacity to the working load (T_w). The proof load factor which provides a measured margin of safety in the field is the ratio of proof load to the working load. Table 2 indicates typical factors employed at present. The minimum safety factors are applied to those anchorage components known with the greatest degree of accuracy and therefore invariably relate to the steel tendon.

5. CORROSION PROTECTION

The safety of people and property in the event of anchorage failure should be balanced against cost of providing protection. Since unprotected steel tendons will probably corrode in time, it is also necessary to decide whether the rate of corrosion merits the expense of protection. In practice, corrosion rates vary enormously according to anchorage and working mode, and there is no certain way of identifying corrosive circumstances with sufficient precision to predict corrosion rates. Consequently, for all permanent rock anchorages corrosion protection appropriate for the circumstances should be provided (Table 3). The design solution may range from double protection, implying two physical barriers to corrosion, in permeable highly fractured rocks in an aggressive environment, to simple cement grout cover in non-aggressive rocks having a mass permeability less than 10^{-10} m/sec. In general for corrosion resistance, the anchorage has to be protected overall as partial protection of the tendon may only induce more severe corrosion on the unprotected part. Accordingly, protective systems usually aim to exclude a moist gaseous atmosphere around the steel tendon by totally enclosing it within an impervious covering or sheath.

Grout injected in situ to bond the tendon to the rock does not constitute a part of a double protection system because the grout quality and integrity cannot be assured. Furthermore, when tendons in cement grouts are stressed, cracks tend to occur at 50-100 mm intervals and with widths of 1 mm or more. In current practice simple grout cover is used where low capacity

Anchorage category	Minimum safety factor: Tendon	Rock/grout or grout/tendon interface	Proof load factor
Temporary anchorages where a service life is less than six months and failure would have no serious consequences and would not endanger public safety.	1.40	2.0	1.10
Temporary anchorages with a service life of say up to two years where, although the consequences of failure are quite serious, there is no danger to public safety without adequate warning.	1.60	2.5†	1.25
Permanent anchorages and temporary anchorages where corrosion risk is high and/ or the consequences of failure are serious.	2.00	3.0*	1.50

*May need to be raised to 4.0 to limit creep in weak rock.

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

Notes:

1. Minimum safety factors for the rock/grout interface generally lie between 2.0 and 3.0. However, it is permissible to vary these, should full scale field tests (trial anchorage tests) provide sufficient additional information to permit a reduction.
2. The safety factors applied to the rock/grout interface are invariably higher compared with the tendon values, the additional mangitude representing a margin of uncertainty.

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

rock bolts are employed solely as secondary reinforcement. For high capacity permanent anchorages installed in low permeability rock, designers should insist on at least one physical barrier to protect the tendon against corrosion (single protection). Nevertheless, satisfactory performance has been observed where an alkaline environment is the only protection against corrosion (FIP, 1986).

Double protection implies the supply of two barriers where the purpose of the outer second barrier is to protect the inner barrier against the possibility of damage during tendon handling and placement. The second barrier therefore provides additional insurance.

Anchorage capacity	Class of protection
Temporary	Temporary
	Temporary with single protection
	Temporary with double protection
Permanent	Permanent with single protection
	Permanent with double protection

TABLE 3. Proposed classes of protection for ground anchorages (after FIP, 1986).

6. CONSTRUCTION

6.1 General

In anchorage construction the importance of skilled operatives cannot be over-emphasised, since quality of workmanship greatly influences subsequent performance. This workmanship factor also limits the ability to predict anchorage performance accurately solely on the basis of empirical rules and ground investigation data. Quality controls and record keeping are therefore strongly recommended during the construction stage.

6.2 Drilling

Any drilling procedure may be employed that can supply a stable hole, but preferably the method should minimise disturbance to the surrounding rock. Care should be taken not to use high pressures with any flushing media in order to minimise the risk of hydro-fracture. In this regard, an open return to the surface is desirable. The drilling or flushing method should not give rise to excessive loss of ground compared with the nominal volume of the drill hole.

6.3 Hole Geometry

Holes for rock anchorages should be drilled to the diameter, length, alignment and position shown on the drawings, but subject to the following permissible deviations, unless otherwise specified.

Entry point	±75 mm
Alignment	±2.5°
Deviation	1 in 30
Diameter	not less than designed diameter
Length	designed length + 0.3 to 0.7 m

6.4 Water Testing

On completion of drilling rock, it is prudent to assess the likelihood of grout loss prior to tendon installation. Grout loss from around the tendon in the fixed anchor zone is of prime importance in relation to efficient distribution of load and corrosion protection. A water test using a falling head or packer injection technique may be used or alternatively cement grout injection may be permitted to assess loss or gain. In the case of cement based grouts, pre-grouting is not required if leakage or water loss in the fixed anchor section of the hole does not exceed 5 litres per minute at an excess head of 100 kN/m^2. Where there is a measured water gain under artesian conditions, care should be taken to counteract this flow by the application of a back pressure prior to grouting. If the flow cannot be stabilised in this way, pre-grouting is required irrespective of the magnitude of the water gain.

6.5 Tendon

Prestressing steel is normally supplied in accordance with national standards. A film of rust on the tendon is not necessarily harmful and may improve bond, but tendons showing signs of pitting should not be used.

Steel tendons in the bare condition or with protective coating should be stored in clean dry conditions and be suitably protected against mechanical damage or contamination. For lifting, only fibre rope or webbing slings should be used, and for long bars, cradles are recommended to prevent excessive bending. Kinked or sharply bent steel should be rejected because load-extension characteristics may be adversely affected.

Anchorage tendons should be fabricated in a workshop or in the field under a covered area, using trained personnel. The prestressing steel should not be subjected to any metallic coating process, heat treatment or welding. In multi-unit tendons, centralizers should ensure a minimum grout cover of 10 mm and where the applied tensile load is transferred by bond, the spacers should ensure a minimum clear spacing of 5 mm. For tendons with local or general nodes that provide mechanical interlock, occasional contact between tendon units is permissible.

Immediately prior to installation tendons should be carefully inspected for damage to components and corrosion. Given a satisfactory condition, the tendons should be homed at a steady controlled rate, and for heavy tendons weighing in excess of 200 kg, mechanical handling equipment should be employed.

6.6 Grouting

Batching of dry materials should be by weight and mixing should be carried out mechanically after which the grout should be pumped to its final position as soon as practicable. Water cement ratios generally lie in the range 0.35 to 0.50. Grout injection should be performed in one continuous operation with the aid of a positive displacement pump, care being taken to expel air from the pump and line. Injection pressure should preferably be limited to avoid distress in the surrounding rock and where high pressures are permitted that could cause hydrofracture, careful monitoring of grout pressure and quantity is recommended together with surveying of the rock face.

Quality controls related to batching and mixing should include initial fluidity and density, initial setting time and bleed during the stiffening stage and cube crushing strengths at 7, 14 and 28 days.

7. STRESSING

Depending on requirements stressing equipment may range from torque wrenches to direct pull hydraulically operated mono or multi unit jacks. Where torque wrenches or percussive torque wrenches are employed, the maximum load in the bar should not exceed 33% and 50% of the characteristic strength respectively.

Under direct pull no tendon should be stressed beyond either 80% of the characteristic strength or 95% of the characteristic 0.1% proof strength. For cement grouted fixed anchors, stressing should not commence until the grout crushing strength has attained 30 N/mm^2.

Details of all forces, extensions, seating and other losses observed during all stressing observations and the times at which the data were monitored should be recorded in an appropriate form for every anchorage.

8. TESTING

8.1 General

There are three classes of tests for all anchorages as follows:

(a) Proving tests;
(b) on-site suitability tests;
(c) On-site acceptance tests.

Proving tests may be required to demonstrate or investigate, in advance of the installation of working anchorages, the quality and adequacy of the design in relation to the rock conditions and materials used and the levels of safety that the design provides. The tests may be more rigorous than on-site suitability tests and the results, therefore, cannot always be directly compared, e.g. where short fixed anchors of different lengths are installed and tested, ideally to failure. In such cases where the rock capacity is being investigated, loads are quoted in terms of characteristic strength of tendon and the appropriate working load is deduced from the proving test results.

On-site suitability tests are carried out on anchorages constructed under identical conditions as the working anchorages and loaded in the same way to the same level. These may be carried out in advance of the main contract or on selected working anchorages during the course of construction. The period of monitoring should be sufficient to ensure that prestress or creep fluctuations stabilize within tolerable limits. These tests indicate the results that should be obtained from the working anchorages.

On-site acceptance tests are carried out on all anchorages except rock bolts and demonstrate the short term ability of the anchorage to support a load that is greater than the design working load and the efficiency of load transmission to the fixed anchor zone. A proper comparison of the short-term results with those of the on-site suitability tests provides a guide to longer term behaviour. A representative sample (1% to 5%) of all rock bolts should be subject to acceptance tests, except where rock bolts are used as the principal or only means of support when a higher proportion (50% to 100%) should be subject to such tests.

Where two-speed resin type rock bolts are proposed the slow setting resin may have to be omitted over the free length to allow cyclic loading to be carried out. Separate investigations should be carried out to verify that the test results have not been influenced by differences in installation procedure.

8.2 On-site acceptance tests

As already indicated, every anchorage used on a contract should be subjected to an acceptance test involving proof loading to show a margin of safety, load-displacement analysis to confirm that the resistance to withdrawal is being mobilised correctly in the fixed anchor zone, and short term monitoring of the service behaviour to ensure reliable performance in the long term. The maximum proof loads are dictated by Table 2 and load-displacement data should be plotted continuously over the loading and unloading cycles in accordance with Table 4. On completion of the second load cycle, the anchorage is re-loaded in one operation to 110% T_w and locked off.

Temporary anchorages Load increment (% T_w)		Permanent anchorages Load increment (% T_w)		Minimum period of observation
1st load cycle* %	2nd load cycle %	1st load cycle* %	2nd load cycle %	min
10	10	10	10	1
50	50	50	50	1
100	100	100	100	1
125	125	150	150	15
100	100	100	100	1
50	50	50	50	1
10	10	10	10	1

*For this load cycle, there is no pause other than that necessary for the recording of displacement data.

TABLE 4. Recommended load increments and minimum periods of observation for on-site acceptance tests (after BS.8081).

The apparent free tendon length is calculated from the elastic displacement curve (Fig. 5) using the manufacturer's values for elastic modulus. The apparent length should not be less than 90% of the free length intended in the design nor more than the intended free length plus 50% of the intended bond length or 110% of the intended free tendon length. The latter upper limit takes account of relatively short encapsulated tendon bond lengths and fully decoupled tendons with an end plate or nut.

In terms of service behaviour either loss of prestress or creep displacement can be monitored in the short term in accordance with the acceptance criteria of Table 5 where

$$\Delta_e = \frac{\text{initial residual load x free tendon length}}{\text{area of tendon x elastic modulus of tendon}}$$

Using accurate monitoring equipment the minimum period of observation for on-site acceptance testing is 50 minutes.

For rock bolts, the permanent displacement of the fixed anchor should not normally exceed 20 mm and 5 mm for expansion and grouted fixed anchors respectively. Where such displacements are exceeded additional cyclic loading is recommended to ensure reproducible behaviour and, if necessary, to establish a more appropriate acceptance criterion for displacement.

For those remaining rock bolts not subject to on-site acceptance tests, they may simply be loaded directly to 110% T_w and locked off without reference to displacement behaviour, unless the bolt yields. In such circumstances, the bolt may have to be derated or replaced, as appropriate.

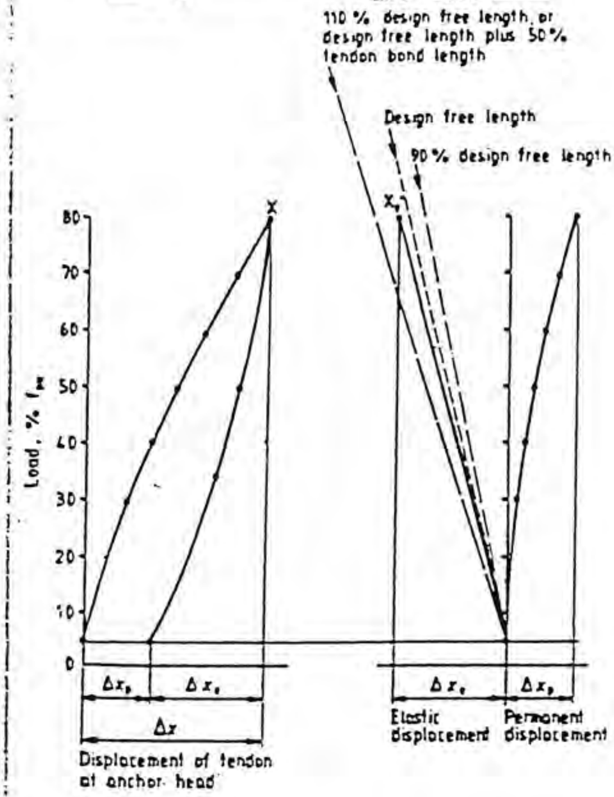


FIGURE 5. Acceptance criteria for displacement of tendon at anchor head.

Period of observation	Permissible loss of load (% initial residual load)	Permissible displacement (% of elastic extension Δ_e of tendon at initial residual load)
min	%	%
5	1	1
15	2	2
50	3	3
150	4	4
500*	5	5
1500 (approx. 1 day)	6	6
5000 (approx. 3 days)	7	7
15000 (approx.10 days)	8	8

*500 min reading is not observed in routine practice.

TABLE 5. Acceptance criteria for service behaviour at residual load.

8.3 Monitoring Service Behaviour

As for buildings, bridges and dams, monitoring of certain underground anchorage support systems will be appropriate on occasions. Monitoring may be by two methods, either measurement of individual anchorage loads or measurement of the excavated face as a whole, the latter being preferred. Variations up to 10% of working load do not generally cause concern, but higher losses unless the reasons are known should be investigated to diagnose the causes and consequences. Remedial action, which may involve partial destressing or additional anchorages, is recommended where prestress gains exceed 20% T_w and 40% T_w for temporary and permanent anchorages, respectively.

9. CONCLUSIONS

Over the past 70 years the use of rock anchorages has become widespread for both temporary and permanent applications and it is reassuring to note that millions of anchorages have been installed successfully.

For these applications, anchorages can be employed to solve problems involving direct tension, sliding, overturning, dynamic loading and ground prestressing, which in turn demand a variety of design, construction and testing techniques.

Since construction techniques and workmanship greatly influence subsequent performance, quality controls and record keeping are strongly recommended during the construction phase. Furthermore, each anchorage once installed, should be subjected to some form of performance testing for on-site acceptance.

REFERENCES

1. BARLEY, A.D. (1988)
   10,000 Anchorages in Rock.
   Ground Engineering (April/May issues) (in press).
2. BARTON, N., LIEN, R. & LUNDE, J. (1974)
   Engineering Classificaiton of Rock Masses for the Design of Tunnel Support.
   Rock Mechanics 6(4), 189-236.
3. BRITISH STANDARDS INSTITUTION (1988)
   Ground Anchorages.
   BS.8081 (in press).
   British Standards Institution, 2 Park Street, London.
4. CIRIA (1983)
   A Guide to the Use of Rock Reinforcement in Underground Excavations.
   Report No. 101, CIRIA, 8 Storey's Gate, London.
5. DAWS, G. (1979)
   Resin Rock Bolting.
   Civil Engineering 7, 39-43 & 53.
6. DOUGLAS, T.H., RICHARDS, R.L. & ARTHUR, L.J. (1979)
   Dinorwic Power Station - Rock Support of Underground Caverns.
   Proc. 4th Congress, I.S.R.M., Montreux, 1, 361-370.
7. FARMER, I.W. & SHELTON, P.D. (1980)
   Factors that Affect Underground Rockbolt Reinforcement Systems Design.
   Trans. Inst. Mining & Metallurgy, 89, A68-A83 & A106.
8. FEDERATION INTERNATIONALE DE LA PRECONTRAINTE (1986)
   Corrosion and Corrosion Protection of Prestressed Ground Anchorages.
   Thomas Telford Ltd., London.
9. HOEK, E. & BROWN, E.T. (1981)
   Underground Excavation in Rock.
   Inst. of Mining & Metallurgy, London.
10. LITTLEJOHN, G.S. & BRUCE, D.A. (1977)
    Rock Anchors : State-of-the-Art.
    Foundation Publications Ltd., Brentwood, Essex, England.
11. MULLER, H. (1966)
    Erfahnungenmit Verankerungen System BBRV in Fels- und Lockergesteinen.
    Schweizerische Bauzeitung, 84, (4), 77-82.
12. U.S. ARMY CORPS OF ENGINEERS (1980)
    Engineering and Design : Rock Reinforcement.
    Engineer Manual EM.1110-1-2907,
    Office of the Chief of Engineers, Washington, D.C., U.S.A.

35. Instrumentation used to monitor the influence of blasting on the performance of rock bolts at Penmaenbach Tunnel

A. A. RODGER, BSc, PhD, MICE, FGS, D. C. HOLLAND, BSc, University of Aberdeen, G. S. LITTLEJOHN, BSc, PhD, FICE, FIStructE, FGS, FRSA, and D. K. V. MOTHERSILLE, BEng, University of Bradford

SYNOPSIS. Joint research is being undertaken by the two universities to provide a fundamental understanding of the dynamic response of rock bolt systems. The work has involved a combination of field measurements of the dynamic response of rock bolts installed during the construction of the new Penmaenbach Tunnel in North Wales, along with complementary laboratory model and finite element computer studies. This paper describes the instrumentation systems devised to measure bolt performance when subjected to dynamic loading. Important results arising from the field work are presented which should ultimately lead to significant improvements in design practice.

INTRODUCTION

1. The use of resin bonded rock bolts for the support of tunnels formed by drill and blast methods is becoming common practice. Yet, a review of published literature has indicated that there is a dearth of information concerning the behaviour of resin bonded rock bolts under impulsive loading conditions. Research to date appears to have concentrated on the performance of mechanically anchored bolts (refs 1-2) and cement grouted rock reinforcement (refs 3-5). The research into the performance of resin bonded bolts which has been conducted has shown the bolts to be resilient to dynamic loading. Dunham (ref. 6) monitoring the long term behaviour of 20mm diameter, two-speed resin bolts with fixed anchor lengths of 350 and 650mm and prestress levels of between 50 and 120kN encountered load losses of at most 14kN when placed as close as 5m from the blast face. For fixed anchor lengths of 350mm, Beveridge (ref. 7) found that 20mm diameter bolts, installed 10-12m from the face in a limestone tunnel, experienced prestress losses of 15-20% under prolonged blast induced

vibrations whereas mechanically anchored bolts suffered losses of up to 80%. No information appears to be available on the behaviour of bolts installed closer than 5m to the face.

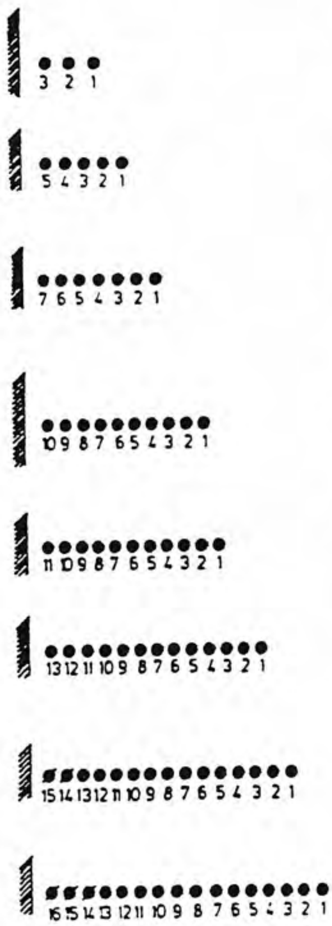
2. Currently therefore there is no basis for the production of a rational design procedure for defining safe distances for the installation of rock bolts close to a blast face. Design at present is based on conservative distances derived from precedent practice. The project described in this paper was devised to produce a more fundamental understanding of the dynamic response of resin bonded rock bolts by undertaking fully instrumented field measurements during the construction of the Penmaenbach Tunnel in North Wales. Complementary laboratory and finite element studies were also initiated. This paper describes the instrumentation devised for measuring the dynamic response of the rock bolts and some important results arising from the measurements.

PENMAENBACH TUNNEL

3. The Penmaenbach Tunnel was commissioned by the Welsh Office to provide a new carriageway for westbound traffic on the A55 North Wales coast road. Work commenced on the tunnel in 1986 and it was driven by drilling and blasting through the rhyolite extrusion which forms the Penmaenbach headland. The completed tunnel is 640m in length, 8m high and 10m wide. The rhyolite was slightly weathered, fine grained and very strong with narrow to wide fracturing (spacing typically 0.2 to >0.5m). Full details of this rock have already been published in ref. 8. Depending on the condition of the rock encountered various combinations of resin bonded rock bolts and sprayed concrete were employed for support within the tunnel.

4. For the purpose of the research work fully bonded, two-speed resin bolts were installed in the tunnel wall initially in a linear pattern at distances of 20m down to 3m from the blast face. The lateral spacing between these research bolts was 3.5m to coincide with the expected face advance per blast. Subsequent tests involved bolts, installed within 1m of the face and arrays of bolts installed in the wall, haunch and crown of the tunnel to assess the influence of bolt location. The bolts employed in the test work were 25mm diameter, 6m in length and were prestressed up to 100kN. Tests were also conducted with shortened and single speed bolts. Fig. 1 shows the location of the experimental rock bolts which were monitored.

Face chainage (m)	Chainage of experimental bolt (m)	Layout
2835·4	1. 2816 2. 2818·7 3. 2822·9	
2839.4	4. 2826·6 5. 2830·2	
2783·2 (bench)	6. 2832·9 7. 2835·5	
2851·0	8. 2840 9. 2843·4 10. 2846·2	
2854·6	11. 2850·4	
2858·4	12. 2853·7 13. 2856·1	
2862·3	14. 2859·1 15. 2862.1	
2874·0	16. 2866.2	



a) Phase 1

Face chainage (m)	Change of experimental bolt (m)	Layout
2904·3	17. 2901 18. 2902 19. 2903	
2904·3	20. 2905 21. 2905·8 22. 2907	
2912	23. 2910 24. 2910 25. 2911	
2915·8	26. 2914·6 27. 2914·6 28. 2914·6	
2919·6	29. 2917·2 30. 2918·2 31. 2918·8	
2923·7	32· 2921 33. 2921·8 34. 2922·8	
2927·1	35. 2924·6 36. 2925.6 37. 2926.4	
2931·6	38. 2927·6 39. 2928·9 40. 2929·9	

b) Phase 2

Key: ● 6m long bolt
bolt installed – not monitored
single stage bolt
3½m long bolt

Figure 1 : Experimental rock bolt locations

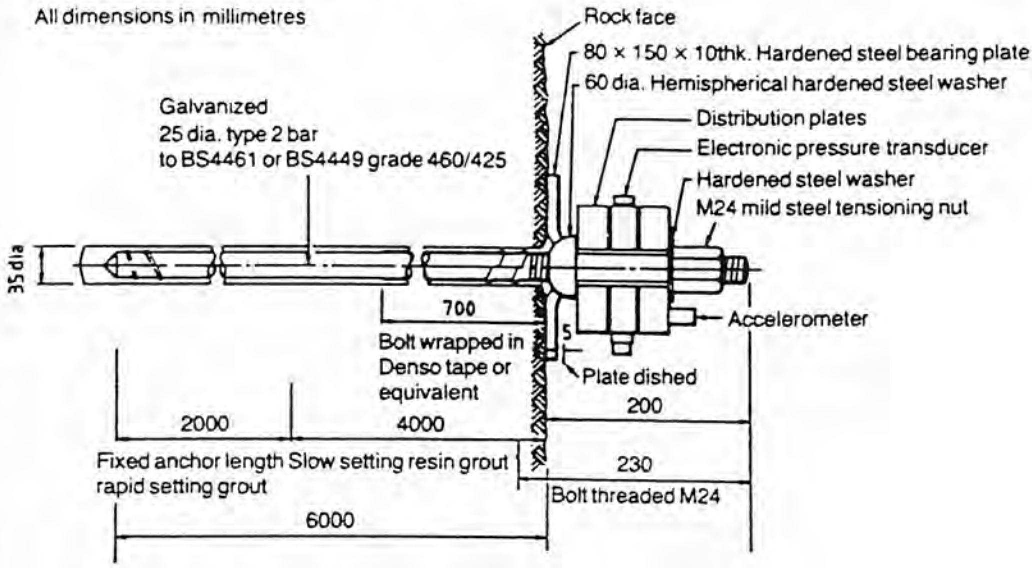


Figure 2 : A typical experimental rock bolt installation

INSTRUMENTATION

5. For each test, the service loads on the bolts were recorded before and after blasting together with instantaneous fluctuation in the prestress using load cells. Accelerometers attached to the load cells enabled the recording of blast induced vibrations. On completion of the blasting sequence, load-extension behaviour was investigated. The suites of instrumentation employed to effect these measurements consisted of three separate systems:

Load Cell System

6. This comprised annular Glotzl hydraulic load cells, fitted with electronic pressure transducers to permit remote reading, which were connected via a network of cables to a 7 channel FM tape recorder. The cells were attached to the bolts such as to measure axial load as shown in Fig. 2. The load cells were designed to measure from zero to 250kN with an accuracy of 1% and a frequency response of up to 2kHz. To prevent excessive signal loss in the 400m of cable necessary between the load cells and the monitoring station, an in-line amplifier system was devised to be situated 100m from the cells. The amplifiers were designed to operate from dc to 5kHz, were able to compensate for the voltage drop along the remaining 300m of cable leading to the monitoring station, and had a low drift with regard to temperature. The amplifiers were powered by a 12v lead acid battery, contained at the monitoring station, through the same multi-way cable as used for the transducer signals. Static and dynamic calibration of the load cell

system was effected in the laboratory using an Instron, and an ESH impulse, testing machine. The dynamic calibration involved both rapid loading and unloading, and steady state vibrational loading tests. Details of the calibration procedure are described in ref. 8. Fig. 3 shows schematically the field arrangement of the load cell system.

Accelerometer System

7. Vibration was monitored by means of a single piezo-electric accelerometer mounted on the load distribution plate of each rock bolt and disposed axially along the line of action of the bolt. The signals from these accelerometers were also recorded on FM magnetic tape for subsequent analysis. Two types of accelerometer were employed which differed in sensitivity by a factor of 10. For bolts nearest to the blast face a lower sensitivity accelerometer was employed which produced $1pC/m/s^2$

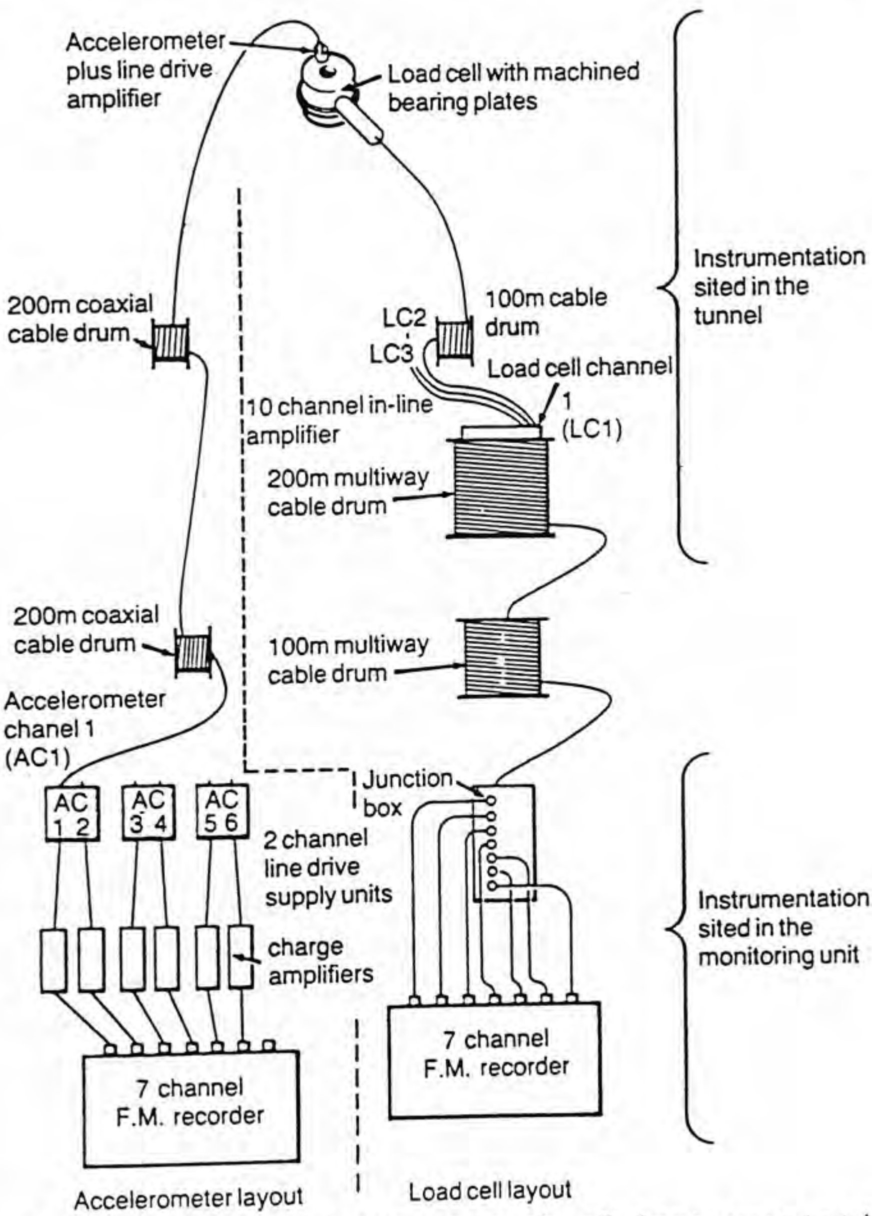


Figure 3 : Field arrangement of instrumentation

with an upper frequency limit of 12kHz and transverse sensitivity of 1.5%. Each accelerometer was fitted with a line drive amplifier powered through the transducer cable from the monitoring station. The line drive amplifier unit attaches directly to the accelerometer and consists of two components, the first amplifies the charge output from the accelerometer and the second uses this signal to modulate the current of the supply voltage enabling the signal to be transmitted over the cable carrying the power supply. This system presents a very low impedance allowing the use of very long cables without altering the accelerometer sensitivity. To prevent the occurrence of ground loops and 'cross-talk' between the load cell and accelerometer systems, the accelerometer was mounted using an insulating stud and mica washer. An additional stage of amplification was provided in the monitoring station with units which also permitted integration of the signals before or after recording to give velocity or displacement values.

8. In situ calibration of the system was effected by a portable vibration generator which produced a reference acceleration of $10m/s^2$ peak at 80Hz. Fig. 3 shows the field arrangement of the instrumentation. Substantial blast protection systems needed to be devised for load cell and accelerometer cables within the tunnel. This took the form of heavy steel channel sections connected to the tunnel wall by dowels. Prefabricated steel boxes were used to protect the bolt heads and transducers.

Data Recording, Analysis & Communication System

9. All signals were recorded in the monitoring station on magnetic tape at a speed which allowed a recorded bandwidth of dc to 2.5kHz. This speed was selected for the recordings after preliminary test-work revealed that all signals fell within this bandwidth. An on site microcomputer, a transient recorder and an analogue to digital convertor provided a means for preliminary analysis of recorded data and enabled communication with remote sites using a modem. Time or frequency domain analysis could be performed using the microcomputer with the results being output to a line printer. The transient recording system had the facility for digitising 50k samples per second and could store 16k samples within the unit itself. Control of the transient capture system was effected by the microcomputer.

ANALYSIS OF RESULTS

10. Throughout the period of the site work the experimental data was analysed using the site facilities to provide a check on data quality. In the laboratory analysis was effected by initially producing a complete time history of bolt response to each blast by displaying the load cell and accelerometer results by means of an ultra-violet (UV) recorder. The need for examination of the complete time history becomes apparent when the mechanics of the blasting process are considered. Each blast involved detonation of a pattern of charges covering the tunnel face. The charges were detonated at successive time intervals in groups with the time elapsed before each detonation being designated a 'delay'. This detonation method reduces the blast induced vibration and increases blasting efficiency. Typically twenty three delays were involved in each blast at Penmaenbach with delay times ranging from 100 to 6000 milliseconds. Examination of the time history permitted identification of the bolt response associated with each group of delays. Following study of the time history of the bolt response, the characteristics of the response waveforms were examined in detail in the time and frequency domains using a high resolution signal analyser. Fig. 4 shows a typical set of load and acceleration waveforms corresponding to a single explosive detonation.

11. Analysis of the results undertaken to date has shown that all bolts under test exhibited only elastic response to the blast including bolts positioned as close as 0.7m to the face. Fig. 5 shows the maximum dynamic load induced in the bolts, expressed as a percentage of the prestress load, and the corresponding vibrational acceleration. For a bolt close to the blast face and prestressed to 39kN, a load change of over 40% is indicated in this graph with a corresponding acceleration of 6400m/s^2. For bolts prestressed to 100kN the highest load change was 12% with an acceleration of 1000m/s^2. A velocity of motion of 300mm/s was determined for this bolt positioned 0.8m from the face.

12. In practice permanent rock bolts are designed to withstand load change of 50% without damage. In the absence of reliable damage criteria, a velocity limit of 200mm/s was employed at Penmaenbach to control the safe installation of permanent resin bonded rock bolts. The peak overloads shown in Fig. 5 coupled with the elastic nature of the bolt response in the local rhyolite, suggest that higher velocities can be safely

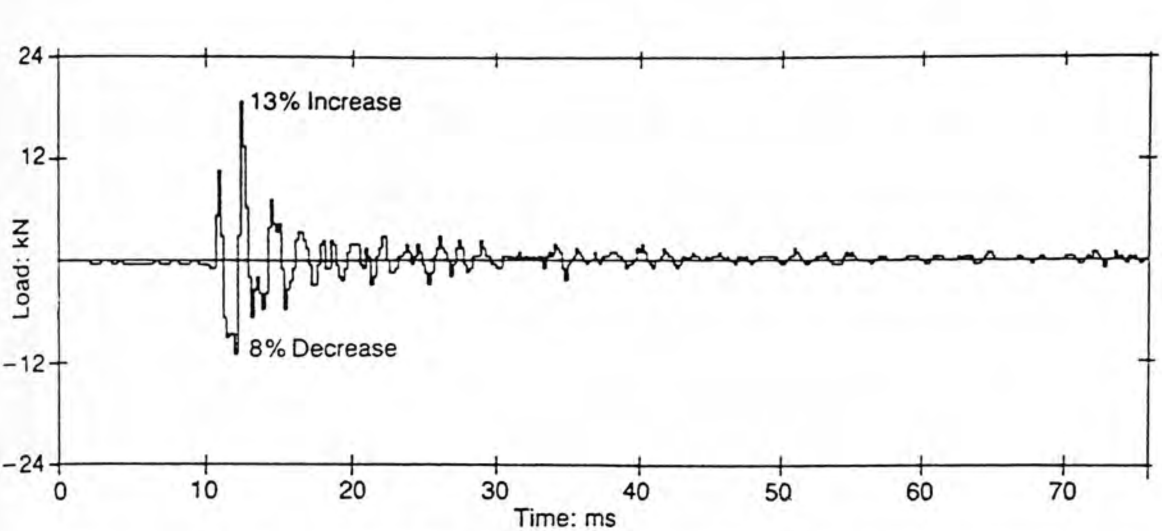


Figure 4(a) : Load fluctuation with time for a single delay

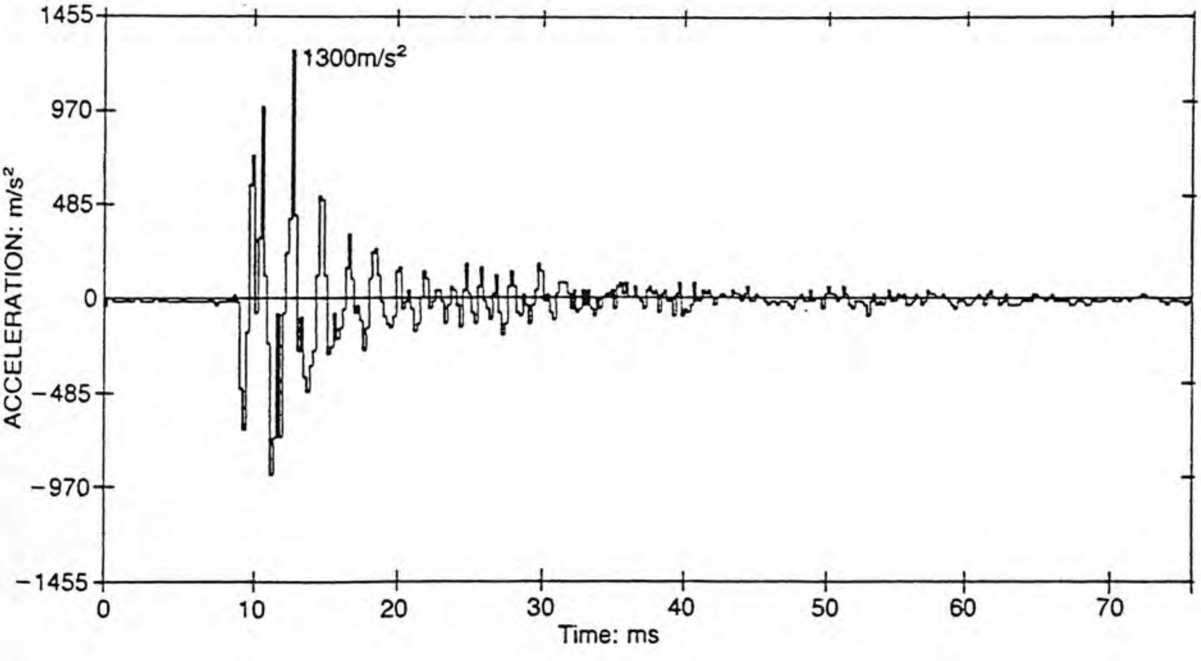


Figure 4(b) : Acceleration plot for a single delay

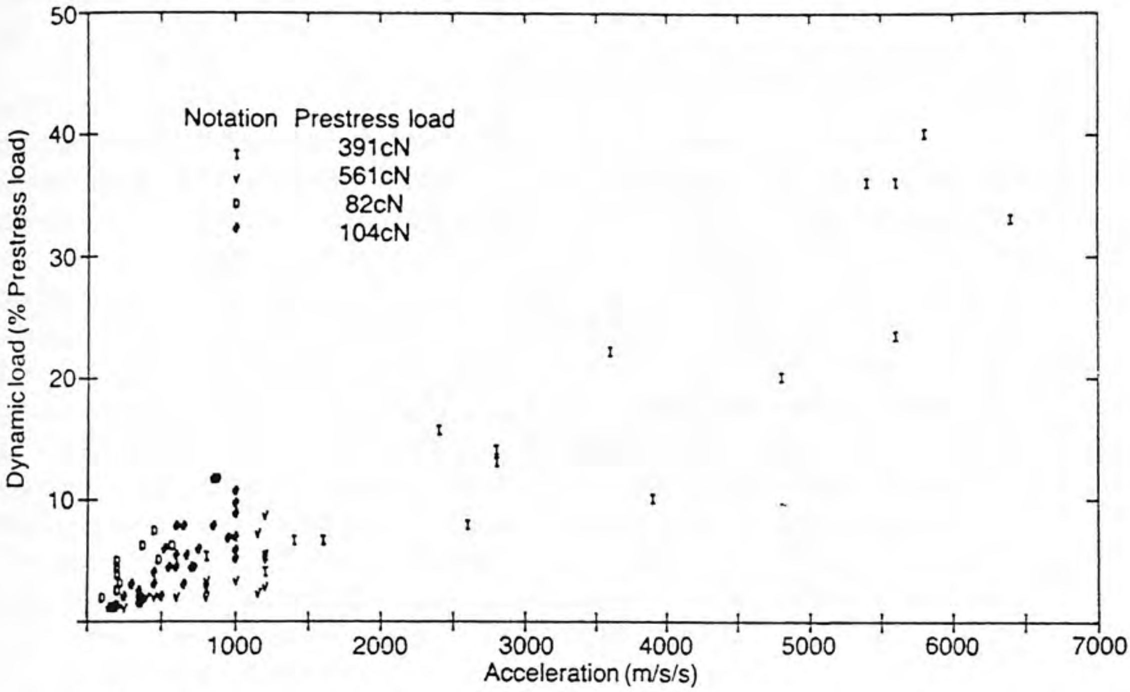


Figure 5 : Relationship between dynamic load and acceleration for two-speed bolts at different prestress loads

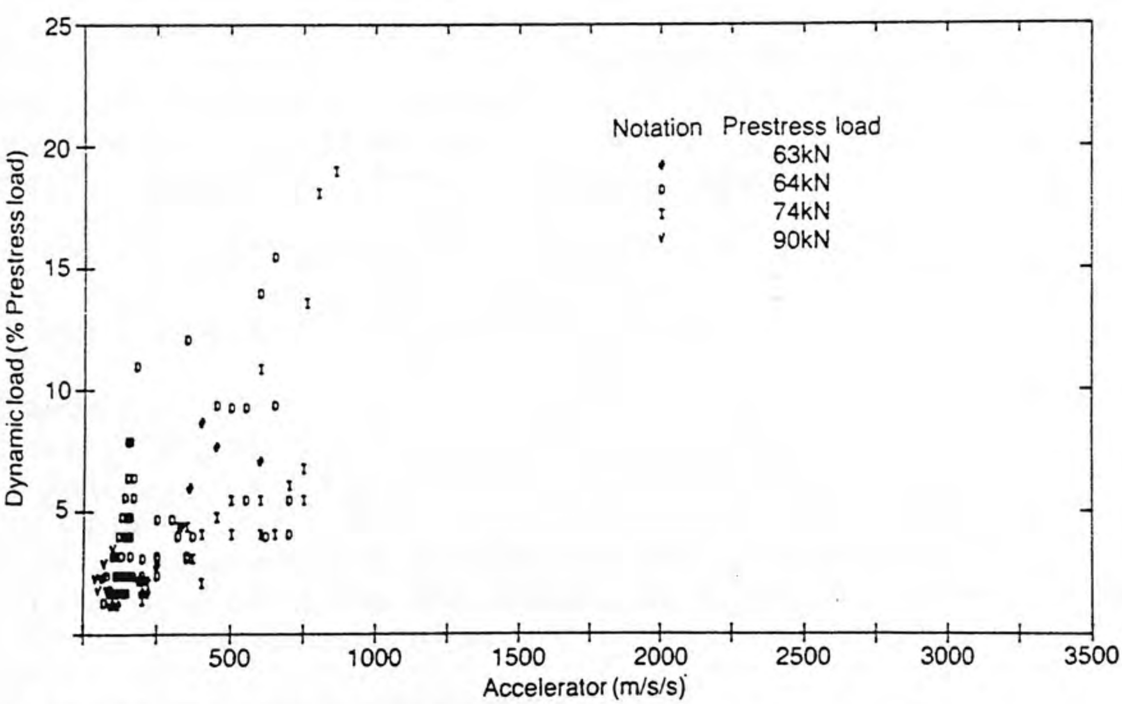


Figure 6 : Relationship between dynamic load and acceleration for single-speed bolts at different prestress loads

accommodated. Furthermore, results presented in this form could provide a means of predicting the change in bolt load by measurement of acceleration of vibration at the bolt head.

13. Fig. 5 also shows that increasing the prestress load in the bolt decreases the amplitude of acceleration experienced by the bolt. This is assumed to occur as a consequence of the greater mass of rock mobilised by the higher prestress load. No appreciable difference was observed between the dynamic load response of 3.5 and 6m long bolts. Fig. 6 shows the corresponding results for the single-speed resin bonded bolts where a greater load change is experienced for a given acceleration. These bolts were not positioned so close to the face and therefore the acceleration values are lower. The greater load change arises from the larger free length of the single-speed bolts. This result highlights the advantages to be gained from the use of two-speed bolts in dynamic load environments.

14. Fig. 7 shows the dynamic load change relative to 'scaled distance'. The term scaled distance is a parameter which enables linearisation of the relationship between induced load change and distance and incorporates the charge weight involved in the test. Also shown on the graph is the prestress load involved in each test. Graphs of this kind are often employed to predict potential damage and employ peak particle velocity as the damage criterion. The attenuation relationships resulting from this work are:

$$PPV = 190(x)^{-0.615} \tag{1}$$

$$PDLR = 4(x)^{-0.633} \tag{2}$$

where,

PPV is the peak particle velocity

PDLR is the ratio of the peak dynamic load to pre-stress load expressed as a percentage

x is the scaled distance, being a ratio of distance from the blast face to the square root of the charge mass

LABORATORY MODEL RESEARCH

15. This work was aimed at investigating aspects of dynamic response of rock bolts which at this stage in the research could not be examined in the field such as the nature and pattern of the load distribution in the fixed anchor. The model was devised to simulate a single-speed resin bonded bolt subject to impulsive loading. The results of

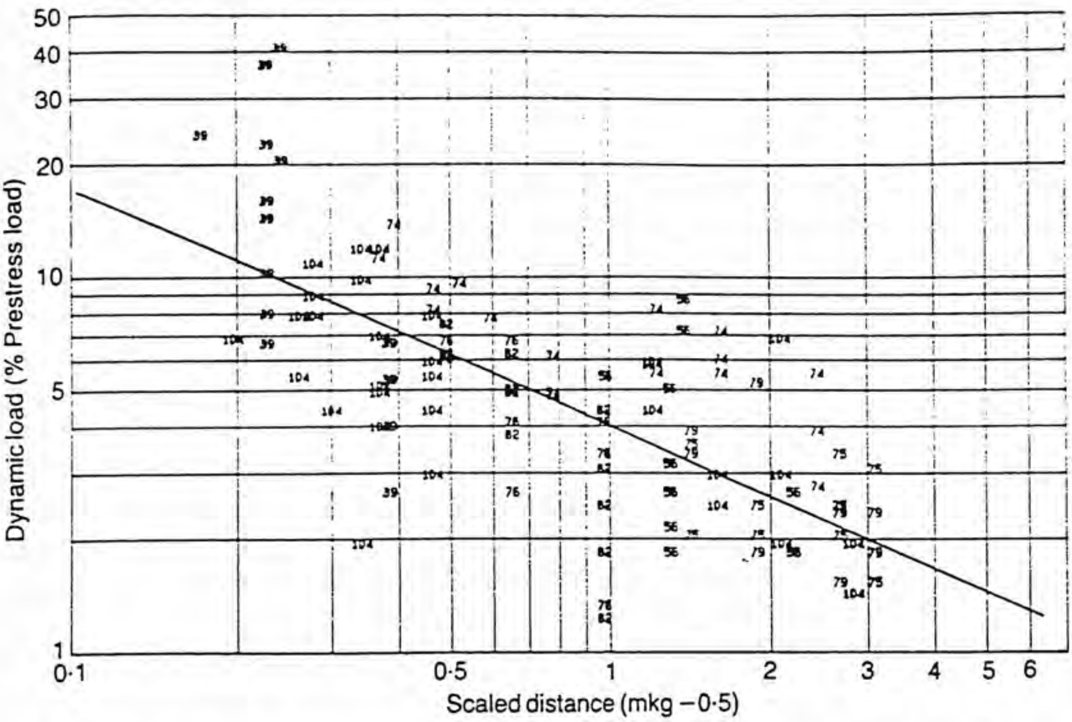


Figure 7 : Relationship between dynamic load and scaled distance for 6 m two-speed bolts. (The numbers on the graph indicate the prestress loads).

the testwork have shown that the load distribution decays exponentially with distance from the proximal end of the fixed anchor. Increasing the magnitude of the impulse was found to lead to an increased rate of attenuation of load with distance along the fixed anchor. The effects of prestress load on overall bolt behaviour were significant. Bolts with greater initial prestress were found to sustain less dynamic stress increase confirming the important result observed in the field test work.

COMPUTER SIMULATION

16. The simulation is being developed as a means of generalising, and providing theoretical corroboration of, the field and laboratory experimental results. The approach being adopted is based on the dynamic finite element method which enables changes to be effected in the system geometry, applied loading, material properties and stress-strain relationships. The transient response calculations are being undertaken using the Newmark β method with applied loadings consisting of an initial prestress and a superimposed impulse. With respect to load distribution in the fixed anchor

similar trends to those obtained from the model tests have been obtained to date. Currently work is in progress to extend the simulation to model the field situation.

CONCLUSIONS

17. Analysis of the field results have produced valuable information on the transmission of vibrations in rock resulting from blast loading and the corresponding response of rock bolts installed in the rock. Prestressed resin bonded rock bolts have been found to be remarkably resilient to dynamic loading even when installed as close as 0.7m from a blast face. Based on the initial results of the work it has proved possible to reduce the 'safe' distance from an advancing face to 3m in the proposed Pen y Clip Tunnel in North Wales. If the 'safe' distances observed at Penmanebach of 0.7-1m are confirmed at Pen y clip, the practice of temporary bolting may in future be able to be restricted to the face itself which would result in significant savings in construction practice.

18. The complex nature of the problem, however, makes generalisation of the results difficult. The problem requires, for example, a characterisation of the method of blasting, assessment of the vibration transmission properties of the rock involved, including the effect of discontinuities, and, determination of the effect of the transmitted vibration on the rock bolts taking account of their method of construction. With this in mind it is considered that the most valuable use of the field results is to assist in the development of a comprehensive finite element simulation. Significant advances have been made in the development of such a simulation. Further research is currently planned to confirm the results obtained to date in a wider range of rock types and to obtain field corroboration of the load transfer mechanism predicted from the laboratory model results. Once fully developed, such a simulation will provide a powerful design tool for predicting the dynamic response of rock masses and anchorage systems.

ACKNOWLEDGEMENTS

This paper is published by kind permission of the Welsh Office. The project was funded by the Science & Engineering Research Council in collaboration with consultants R. Travers Morgan & Partners and Howard Humphreys & Partners. Installation of the experimental bolts was carried out as part of the Penmaenbach Tunnel contract by

Balfour Beatty Construction Ltd. The load monitoring system was supplied by Geotechnical Instruments Ltd. and the vibration monitoring equipment by Bruel & Kjaer (UK) Ltd. The authors wish to thank all parties concerned for their assistance and support.

REFERENCES

1. STETILIK, C.J. Mine roof rock and bolt behaviour resulting from nearby blasts, US Bureau of Mines, Dept. of Interior, Report of Investigations No. 6372, 1964.
2. PARSONS, E.W. and OSEN, L. Load loss from rock bolt anchor creep, US Bureau of Mine, Dept. of Interior, Report No. 7220, 1969.
3. LITTLEJOHN, G.S., NORTON, P.J., and TURNER, M.J. A study of rock slope reinforcement at Westfield open pit and the effect of blasting on prestressed anchors, Proc. Conf. on Rock Eng., Newcastle Upon Tyne, 1973, 293-310.
4. FULLER, P.G. Reinforcement of cut and fill slopes, Conf. on Application of Rock Mechanics to Cut and Fill Mining, Inst. Min. & Metallurgy, London, 1981, 55-63.
5. STILLBORG, B. Experimental investigation of steel cables for rock reinforcement in hard rock, Ph.D. Thesis, Div. of Rock Mechanics, University of Lulea, Sweden, 1984.
6. DURHAM, R.K. Field testing of resin anchored rock bolts, Colliery Guardian, May 1974, 146-151.
7. BEVERIDGE, R.L.N. Repairs and extensions to concrete structures using resin anchored bars, Civil Eng. and Public Works Review, July 1973, 609-617.
8. LITTLEJOHN, G.S., RODGER, A.A., MOTHERSILLE, D.K.V., HOLLAND, D. Dynamic response of rock bolts at the Penmaenbach Tunnel, Proc Int Conf on Foundations & Tunnels, 1987, Vol. 2, 99-106, Eng. Technics Press, Edinburgh.

G.S. Littlejohn, B.Sc.(Eng.), Ph.D., C.Eng., F.I.C.E., F.I.Struct.E., F.G.S., F.R.S.A. (University of Bradford)

A.A. Rodger, B.Sc.(Eng.), Ph.D., C.Eng., M.I.C.E., F.G.S. (University of Aberdeen)

D.K.V. Mothersille, B.Eng., Ph.D. (University of Bradford)

D.C. Holland, B.Sc., M.Sc. (University of Aberdeen)

The paper describes the performance of post-tensioned resin bonded rock bolts, when subjected to close proximity blasting during the construction of the new Penmaenbach Tunnel in North Wales. For the very strong rhyolite encountered on this site attenuation relationships are established for peak particle velocity and peak dynamic load (expressed as a % of prestress load). Prestressing the bolt serves to decrease the effect of vibrational loading on the bolt. For accelerations ranging from 10 g up to 640 g all deformations were found to be elastic, and no significant load loss or resin/bolt debonding was registered on bolts even when located within 1 m of the blast face.

INTRODUCTION

This paper describes the main results arising from the field monitoring phase of an ongoing research project which has the aim of investigating the performance of rock bolts when subjected to blast loading in order to produce a rational design methodology. The field work reported in this paper was undertaken during the construction of the new Penmaenbach Tunnel in North Wales.

This 640 m long tunnel was constructed by drilling and blasting through a slightly weathered fine grained very strong rhyolite with narrow to wide fracturing (spacing typically 0.2 to >0.5 m). For tunnel support, 6 m long fully bonded two-speed resin rock bolts were installed routinely and post-tensioned to 100 kN.

The field experimental programme comprised the monitoring of standard 6 m long bolts at distances of 20 m down to 0.7 m from the tunnel face, plus some single speed resin bolts and 3.5 m long fully bonded bolts. Axial load and acceleration were measured at the head of each bolt using an annular Glotzl hydraulic cell and a piezo-electric accelerometer, respectively, and a novel signal conditioning system was developed to permit remote monitoring via 400 m of cables to FM tape recorders. Details of the tunnel geometry, rock mass properties and the instrumentation systems involved have already been published (1,2).

CURRENT PRACTICE

Prior to commencement of this project there had been little research into the response of rock bolts to dynamic loading. Where tunnels are to be constructed using blasting methods, the need for safe support often necessitates the placing of rock bolts in close promixity to the blast source. At present there are no reliable and economic criteria for determining how near to the blast a rock bolt can be placed without diminishing its capacity for permanent support. Current design relies on past experience with rock bolts combined with information borrowed from research into the response of structures to blast induced vibrations. A maximum permissible limit is usually set to the blast induced peak particle velocity (PPV). The distance from the blast face at which this limiting PPV is expected is designated the safe distance beyond which structural elements are judged to have no risk of blast induced damage.

At Penmaenbach Tunnel a PPV of 200 mm/s was selected as the safe vibration limit for permanent production rock bolts. A vibration survey undertaken in advance of tunnel construction led to the prediction that the corresponding safe distance for bolt installation would be 5 m from the blast face. The results of the field research work showed that following blasting bolts located as close as 0.7 m to the blast face were in fact undamaged.

EXPERIMENTAL ARRANGEMENT

Standard 6 m long, two-speed fully bonded rock bolts were installed at 3.5 m centres but at gradually decreasing distances (20 m down to 3 m) from the blast face. Subsequent bolts were located at 1 m centres (3 m down to 0.7 m) from the face and two arrays of bolts were positioned on the wall, haunch and crown of the tunnel at 1 and 2 m from the face. The monitoring of single-speed and shorter (3.5 m) bolts were also conducted at 0.7 to 2.7 m from the face. Figure 1 shows the instrumentation system used to monitor the performance of these bolts.

The tunnel was excavated using a top heading and bench method, the heading being advanced 3.5 to 4 m per blast. As the top heading was up to 6.5 m high (area = 54 m²) a burn cut blasting pattern was used, typically of 97 holes and detonated in 23 arrays (see Figure 2a). The central cut comprised three 100 mm diameter void holes surrounded by eight closely spaced shot holes with a charge mass of 23.6 to 57.1 kg (see Figure 2b). Each delay interval was 100 ms so that the duration of the cut sequence was 800 ms. Following formation of the central cavity, the main blasting sequence comprise charge masses of 7.1 to 35.8 kg per delay, delay intervals of 100 to 500 ms and an overall duration of 700 ms. The explosive used to extract the main body of rock was Quarrex 'A', a high energy nitroglycerine powder with a detonation velocity of 2500 to 4800 m/s. To minimise overbreak, a lower energy combination was used in the perimeter holes (0.85 kg Gurit + 0.25 kg Gelamex per hole).

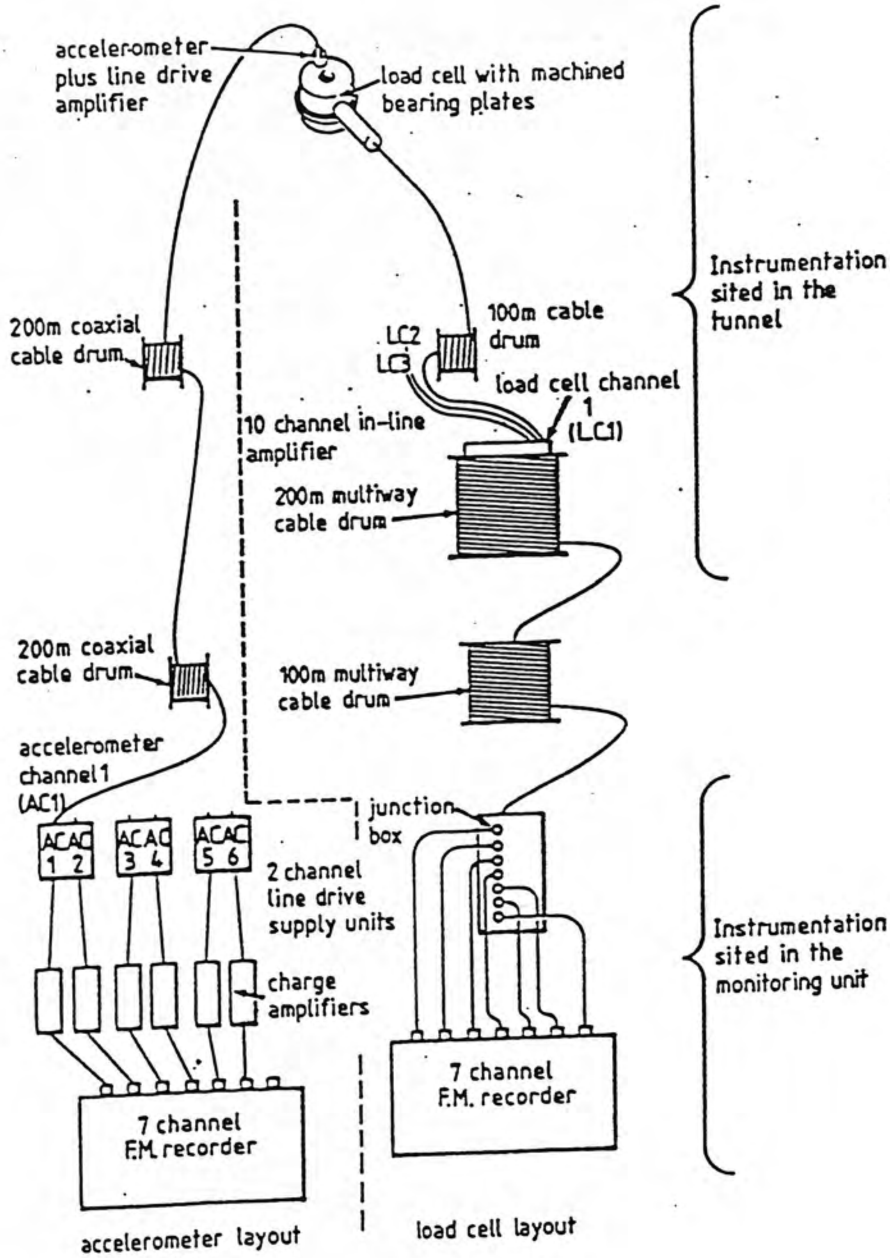
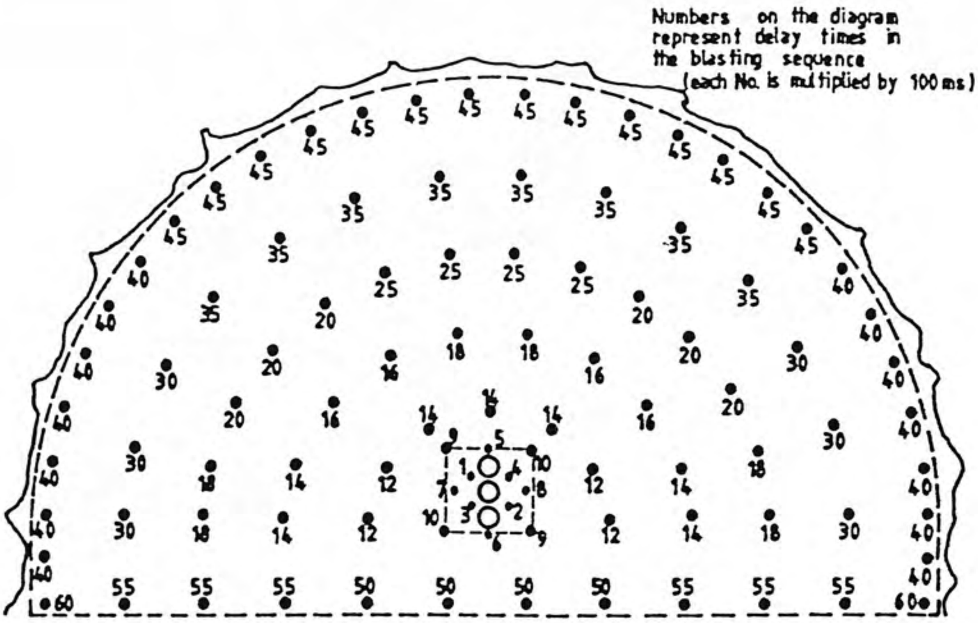
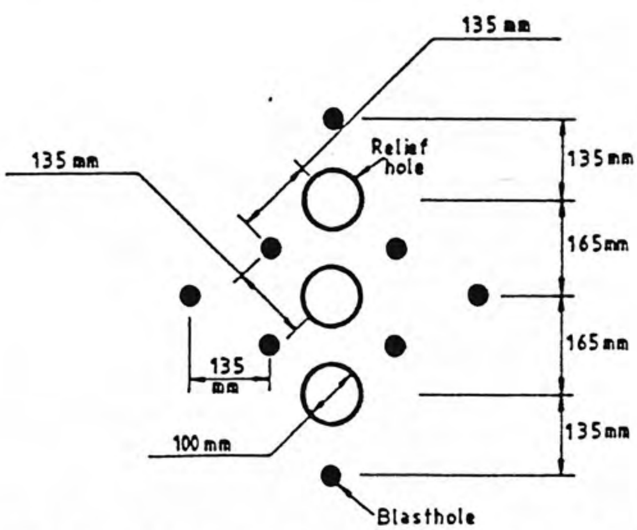


Figure 1: Layout of instrumentation.



(a) Typical blasting pattern employed for top heading advances (NTS)



(b) Details of the central cut (all holes drilled perpendicular to the face)

Figure 2: Details of blasting pattern.

ANALYSIS OF RESULTS

All field results were recorded on FM magnetic tape in order to permit detailed analysis of dynamic response waveforms. Figure 3 shows the instrumentation used in the laboratory for analysis of waveforms in the time and frequency domains. Where no overlap of vibrations occurred between successive delays, it was found possible to link the response waveforms of load and acceleration measured at the bolt head to the detonation of individual delays. Figure 4 shows a dynamic load and acceleration waveform relating to a single delay explosive detonation.

In addition to recording dynamic response waveforms the static performance of the bolts was assessed following each blast to establish which bolts had sustained blast induced damage. By direct stressing the prestress load of each bolt was recorded together with the load-extension behaviour both before and after blasting.

DYNAMIC LOAD RESPONSE OF BOLTS

Comparison of the load-extension behaviour of the monitored rock bolts before and after blasting showed that no significant load loss or resin-bolt debonding was registered for any bolt, even those

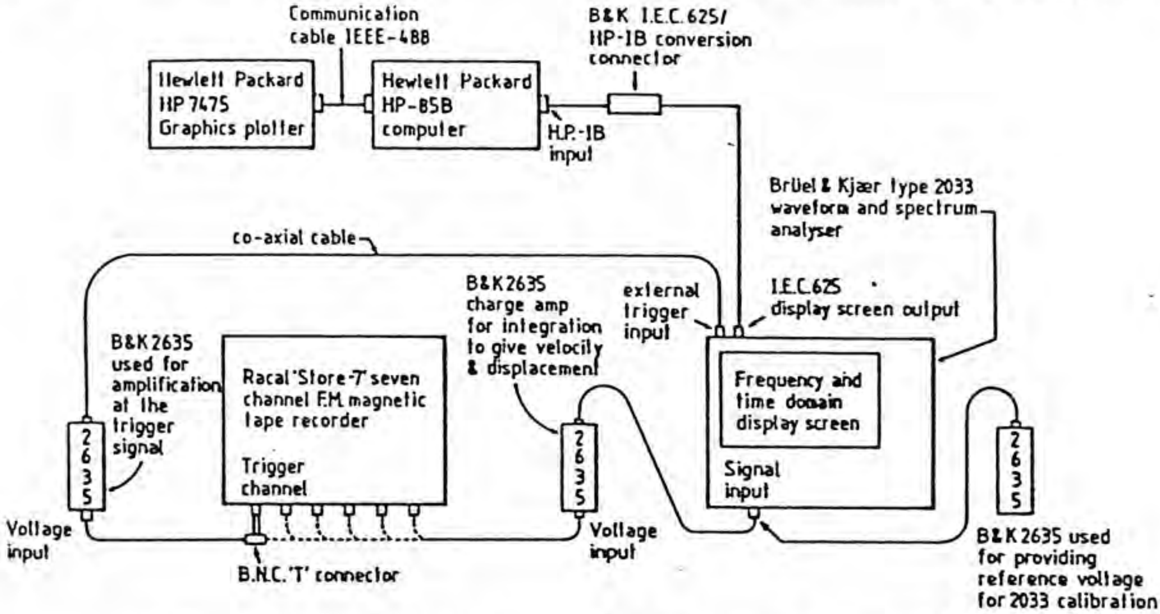


Figure 3: Arrangement of equipment for the digital display of data

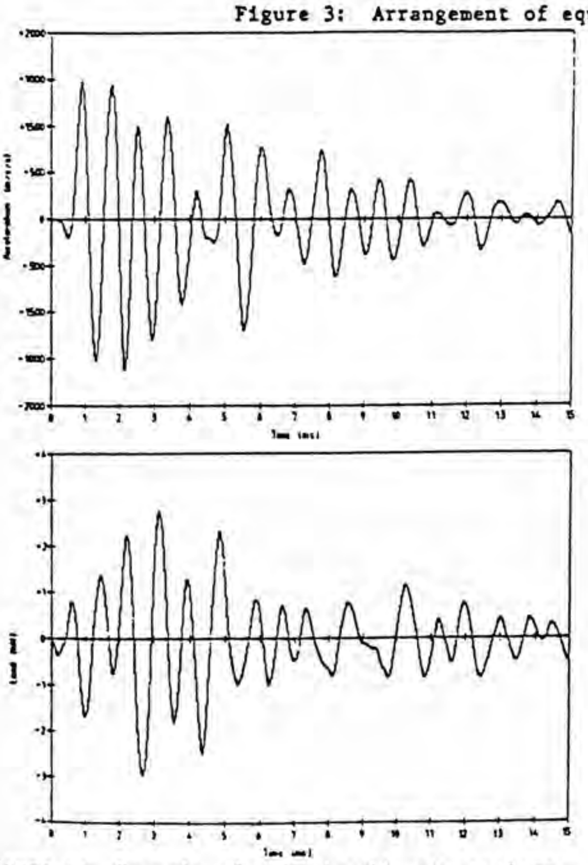


Figure 4: Example of a load cell and accelerometer response to a single delay detonation

positioned within 1 m of the blast face. This indicated that on site a large number of bolts scheduled to be replaced after each blast, due to their proximity to the blast face, were in fact undamaged.

Figure 5 shows the experimental relationship between the peak dynamic load, expressed as a percentage of the prestress load, and peak acceleration for 6 m long, two-speed rock bolts subjected to a range of prestress loads. The general trend is towards a linear relationship between load and acceleration with the amplitude of acceleration decreasing with increase in prestress load for monitoring positions located at the same distance from the blast face. The results shown in Figure 5 relate to measurements obtained from bolts positioned at various distances from the blast face (I = 0.7 m, * = 0.8 m, O = 2.0 m, V = 4.0 m). Comparison of the results obtained with prestress loads of 32 and 104 kN highlights the effect of prestressing, as these bolts were positioned at similar distances from the blast face.

For the 104 kN prestress the range of accelerations experienced by the bolt extend to 120 g with changes in the load of up to 12%. For the case of a 39 kN prestress, the acceleration range is over 600 g with a load change of up to 40%. This result had been anticipated, as it was expected that increasing the prestress load would mobilise a greater mass of surrounding rock, however, no prior experimental justification of this had been available. These results have also been confirmed by laboratory

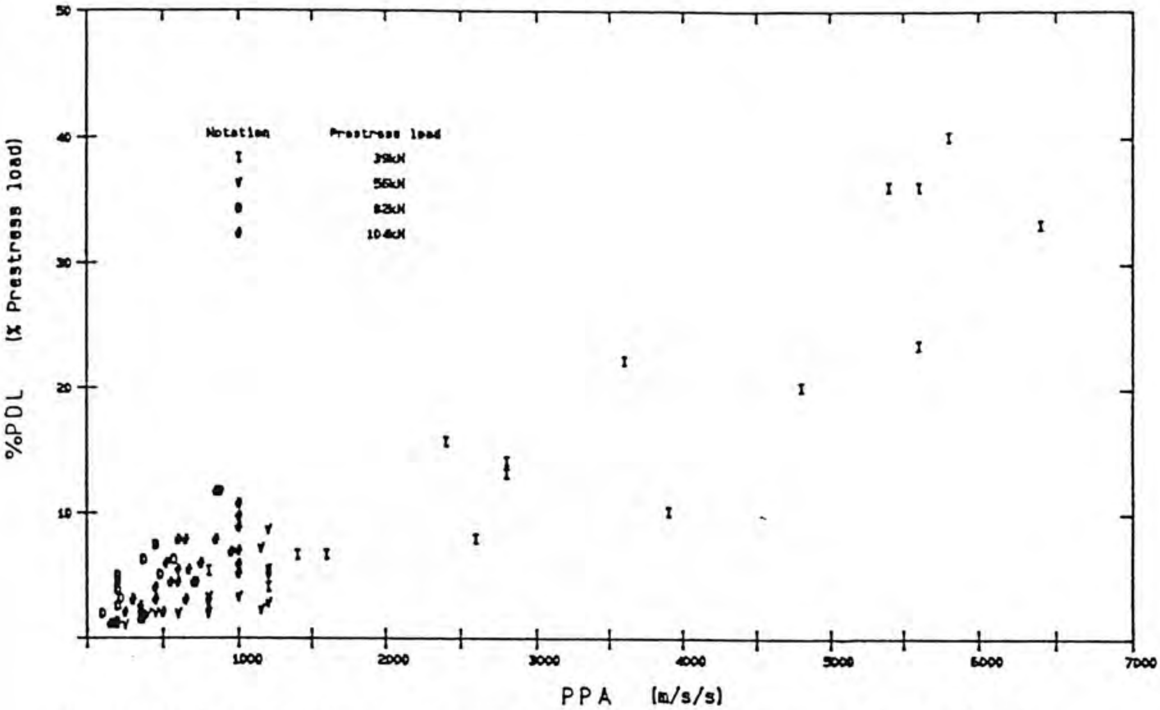


Figure 5: The relationship between peak dynamic load increase (PDL) and peak acceleration (PPA) for 6 m two speed resin bolts installed in rhyolite.

model tests conducted subsequent to the field tests at Penmaenbach (3).

No appreciable difference was observed between the peak dynamic load response of 3.5 and 6 m long two-speed bolts while the single speed bolts were found to experience twice the dynamic load of the two-speed. The latter result arises directly from the longer free length of the single speed bolt and highlights the improvement to be gained from the use of two-speed fully bonded bolts in a dynamic environment. Figure 6 shows the results obtained from monitoring the performance of the single-speed bolts. The free lengths were 0.7 m and 4 m for the 6 m two-speed bolts and 6 m single-speed bolts, respectively.

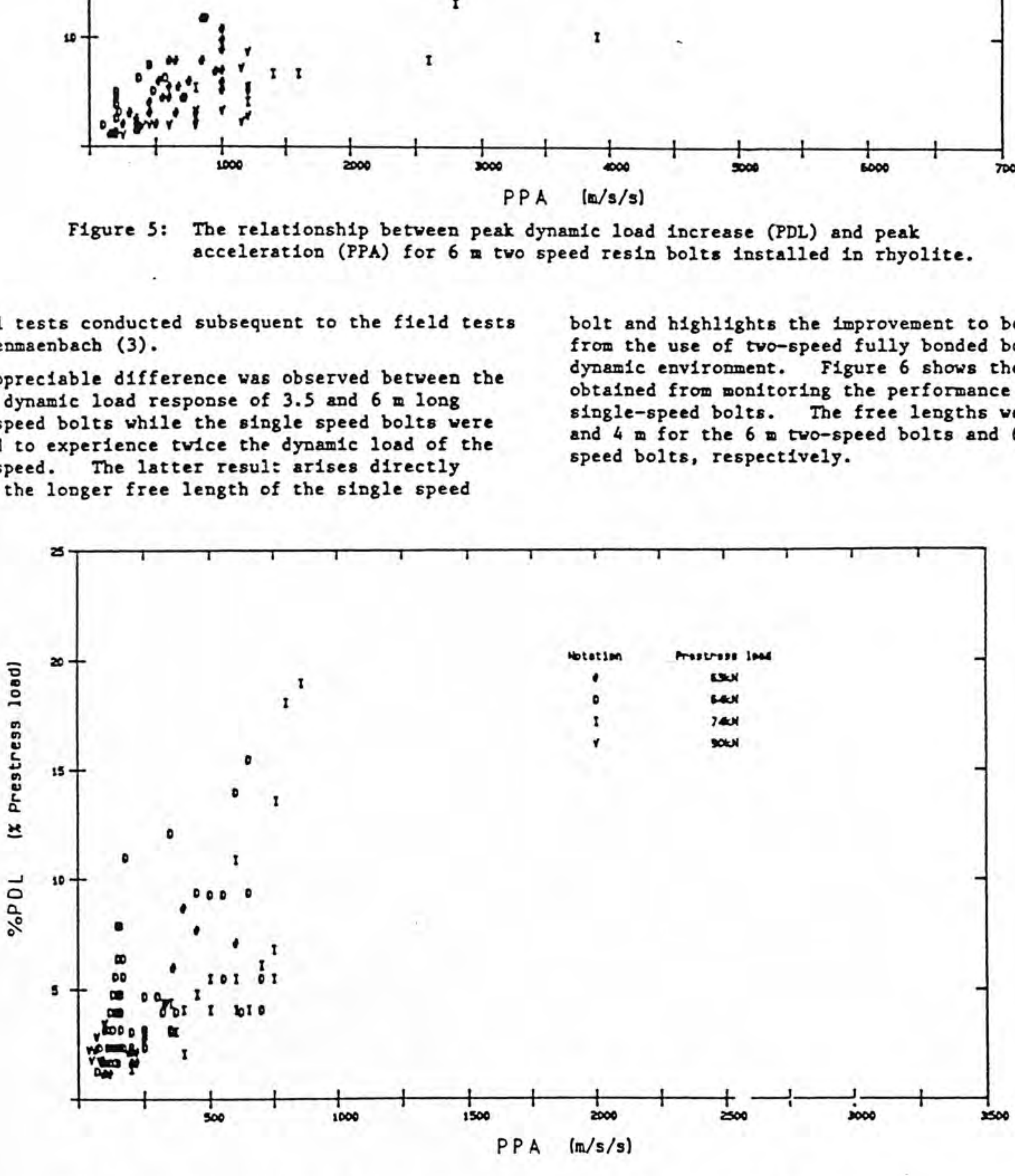


Figure 6: The relationship between peak dynamic load increase and peak acceleration for 6 m single speed resin bolts installed in rhyolite.

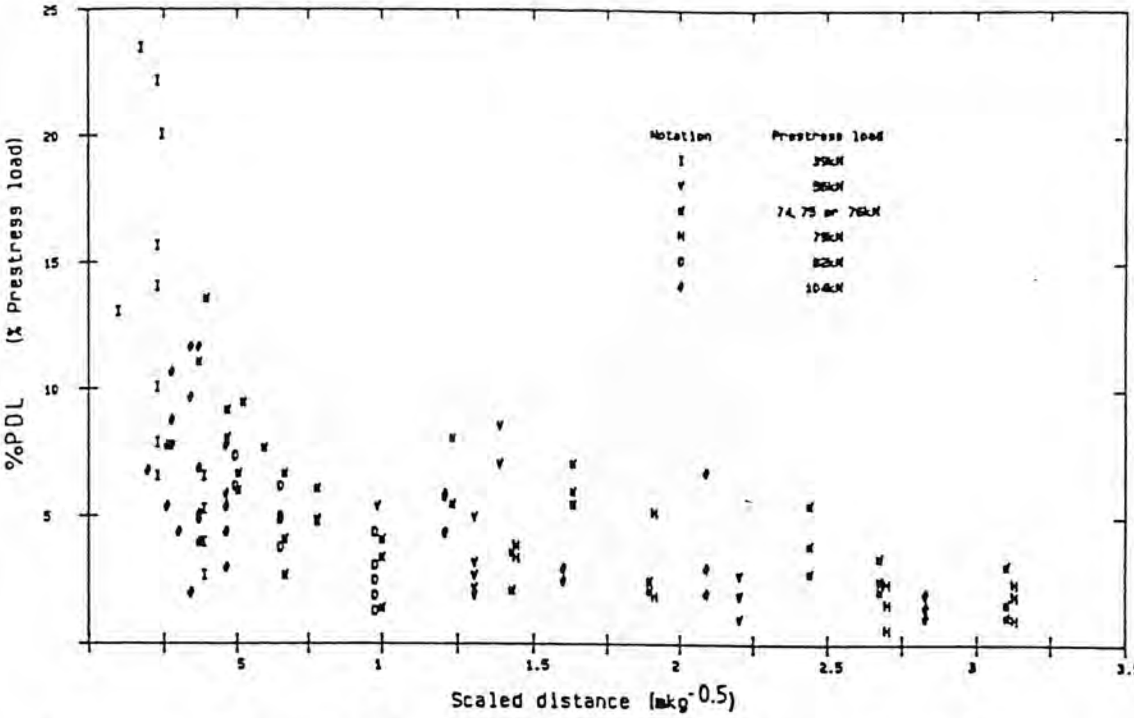


Figure 7: The relationship between peak dynamic load increase and scaled distance (distance/√ charge mass) for 6 m two speed resin bolts installed in rhyolite.

TRANSMISSION OF VIBRATION

Figure 7 shows the relationship between the peak dynamic load expressed as a percentage of the prestress load and distance along the tunnel wall. In this figure an attempt has been made to incorporate the influence of charge mass by dividing the distance from the tunnel face by the square root of the charge mass, to give what is commonly referred to as a scaled distance. Graphs employing a scaled distance parameter are often employed in current practice to predict damage potential of vibrations using peak particle velocity as the damage criterion.

By plotting the results on a semi-logarithmic graph the linear relationship shown in Figure 8 may be obtained. Analysis of the results has led to the development of the following attenuation relationships for the Penmaenbach Tunnel site:

$$PPV = 190(x)^{-0.615} \qquad \ldots\ldots (1)$$

$$PDL = 4(x)^{-0.633} \qquad \ldots\ldots (2)$$

where, PDL = peak dynamic load expressed as a percentage of prestress load

x = scaled distance.

A significant amount of scatter may be observed in the data associated with these relationships. This arises as a result of variations in the nature of the rock surrounding the bolts, the greater mass of rock mobilised at higher prestress loads, changes in blast face geometry and variations in the actual amount of charge associated with each delay. To give the scatter some perspective in Figure 8 the shaded envelope covers a range of ±10% of the prestress load which is not considered excessive in practice.

CONCLUSIONS

Valuable information on the response of rock bolts to dynamic loading has been gained from the field work. For accelerations ranging from 10 g up to 640 g, all deformations were found to be elastic. No significant load loss or resin/bolt debonding was registered on the experimental bolts, even when located within 1 m of the blast face. Such bolts were subjected to a maximum charge mass (Quarrex 'A') in the range 16.5 to 35.8 kg per delay.

No appreciable difference was observed between the dynamic load responses of fully bonded 3.5 m and 6 m bolts. Single-speed 6 m bolts experienced twice the dynamic loads of the equivalent two-speed resin bonded bolts due to the longer decoupled length of the single-speed resin bolts.

With regard to attenuation of vibrations with distance from the blast face, relationships have been established with respect to peak dynamic load (expressed as a percentage of prestress load) and peak particle velocity for the Penmaenbach Tunnel.

An increase in the bolt prestress serves to decrease the effect of vibrational loading on the bolt - a result which has also been confirmed in associated laboratory testwork.

Based on the results at Penmaenbach, the safe distance for permanent rock bolt installation has been reduced to 3 m for the new Pen y Clip Tunnel in North Wales, where the rock is microdiorite with a discontinuity spacing of 0.1 to 0.2 m. Confirmatory testing of the rock bolts will be carried out during the construction of this tunnel and the load transfer mechanism along the fixed anchor length of the bolts will also be studied in detail.

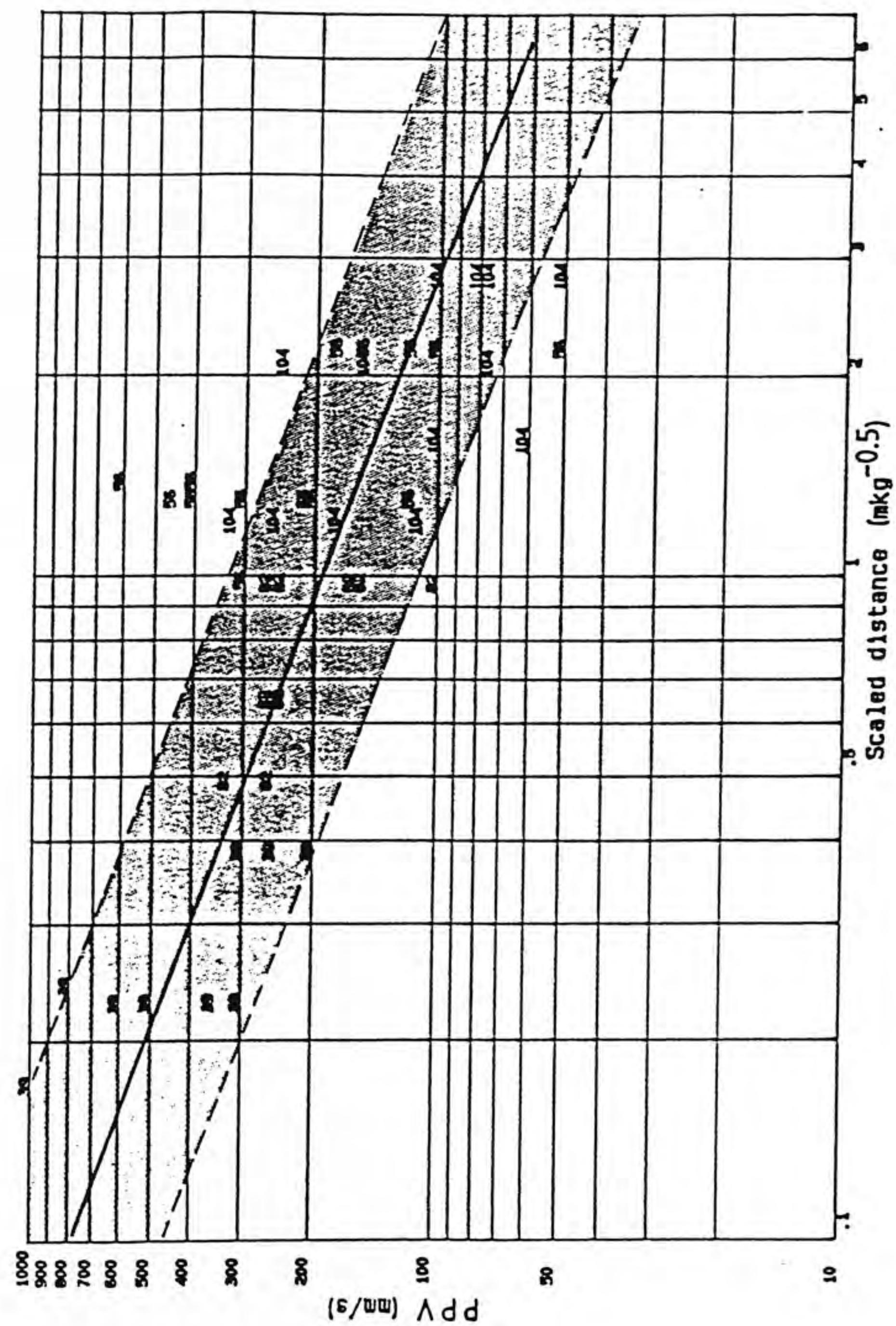


Figure 8: The relationship between peak dynamic load increase and scaled distance (distance/√ charge mass) for 6 m two speed resin bolts installed in rhyolite.

If the safe distances observed at Penmaenbach of 0.7 - 1 m are confirmed at Pen y Clip, the practice of temporary bolting could be restricted to the face itself, where fully bonded resin bolts are employed. In world practice duplication of temporary bolts by permanent primary support could be reduced with savings of up to 50%.

Assessment of the dynamic response of rock bolts is a multi-faceted problem. The development of a rational design methodology which may be used generally in practice requires characterisation of the method of blasting, assessment of the vibration transmission properties of the rock incorporating the effect of discontinuities, and a knowledge of the load-transfer mechanisms appropriate to the form of bolt to be employed. To that end a dynamic finite element model is now being developed with the objective of providing theoretical corroboration of the experimental results and to provide a means of generalising the results to act as a basis for a design predictive capacity.

ACKNOWLEDGEMENTS

This paper is published with the kind permission of the Welsh Office. The project was funded by the Science and Engineering Research Council in collaboration with consultants R. Travers Morgan and Partners and Howard Humphreys and Partners. Installation of the experimental bolts was carried out as part of the Penmaenbach Tunnel contract by Balfour Beatty Construction Ltd. The load monitoring system was supplied by Geotechnical Instruments Ltd. and the vibration monitoring equipment by Bruel and Kjaer (UK) Ltd. The authors wish to thank all parties for their assistance and support.

REFERENCES

1. Littlejohn, G.S., Rodger, A.A., Mothersille, D.K.V. and Holland, D.C., "Monitoring the influence of blasting on the performance of rock bolts at Penmaenbach Tunnel", Proc. Int. Conf. on Foundations & Tunnels, 1987, Vol. 2, pp.99-106.
2. Rodger, A.A., Littlejohn, G.S., Holland, D.C. and Mothersille, D.K.V., "Instrumentation used to monitor the influence of blasting on the performance of rock bolts at Penmaenbach Tunnel", Proc. Int. Conf. on Instrumentation in Geotechnical Engineering, 1989, pp.267-279.
3. Mothersille, D.K.V., "The influence of close promixity blasting on the performance of resin bonded rock bolts", Ph.D. thesis, 1989, University of Bradford.

Papers

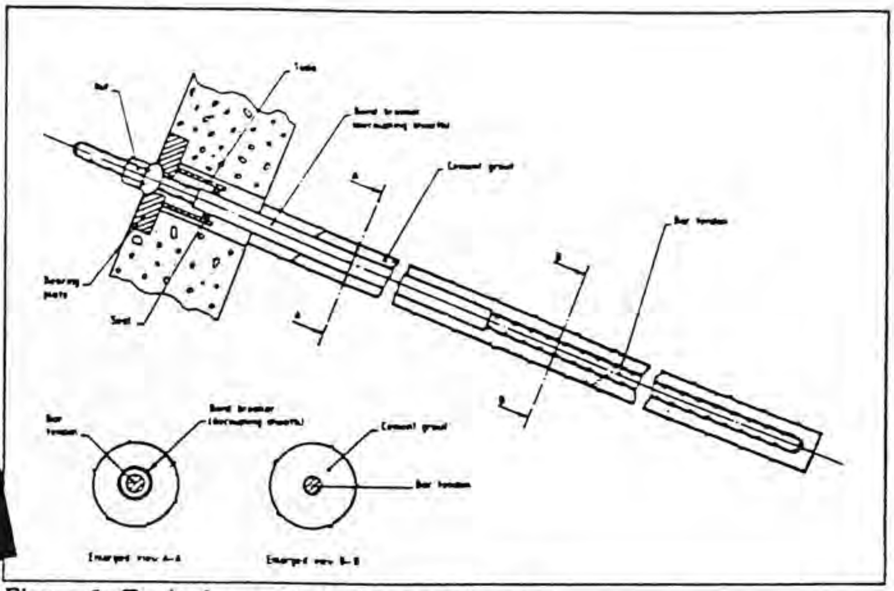


Figure 1: Typical unprotected bar anchorage.

Corrosion protection of steel tendons for Ground Anchorages

by GS Littlejohn*

1. Introduction

In view of the number of ground anchorages currently being installed around the world, where cement grout cover is considered to provide adequate protection against corrosion of the steel tendon, the purpose of this paper is to review the tendon corrosion performance of ground anchorages and, given the lessons learned, to highlight the principles of protection which should be considered by designers of permanent anchorages, and temporary anchorages exposed to an aggressive environment. Guidelines are provided for the recognition of aggressive ground conditions together with examples of the protective systems recommended by the Fédération Internationale de la Précontrainte (1986) and the British Standards Institution (1989).

2. Extent and nature of tendon corrosion

In 1986 FIP published 35 case histories of anchorage failure by tendon corrosion, which included 24 permanent anchorage projects (protected and unprotected tendons) and 11 temporary anchorage projects where the tendons had no designed protection other than cement grout cover for the fixed length and on occasion a decoupling sheath over the free length (see ***Figure 1***).

Analysis of the results shows that for post tensioned prestressing steel corrosion is invariably localised, taking the form of pitting, hydrogen embrittlement or stress corrosion cracking. In such circumstances there is no certain way of predicting localised corrosion rates and the case histories of tendon corrosion indicate that failure can occur after service of only a few weeks or many years. Short term failures (after a few weeks) have been due to stress corrosion cracking or hydrogen embrittlement.

For localised corrosion no tendon type (bar, wire or strand) appears to have a special immunity in that nine incidents involved bar, 19 involved wire and eight involved strand, the period of service before failure ranging from a few weeks to many years for each tendon type.

These observations invalidate the traditional or intuitive view of some designers that an increase in steel tendon diameter will secure the designed service life of post tensioned anchorages.

In this regard, it is important to note that the current recommendations of the Bureau Securitas (1989) to take account of losses of steel through corrosion, and the guide reference to annual loss of material of 0.01mm to 0.1mm/year for driven steel piles, only applies to steels for non-prestressed anchorages (passive anchorages). It is the author's view that caution should still be exercised even for passive anchorages, since nature may demand that the tendons mobilise tensile stresses during service.

With regard to failure location, 19 incidents occurred at, or within 1m of the anchor head, 21 incidents in the free length and two incidents in the fixed length.

Both fixed anchor problems were caused by inadequate grouting of the tendon bond length which exposed the tendon to an aggressive environment.

Failures in the free length were recorded under a variety of individual or combined circumstances such as:
(i) tendon overstressing caused by ground movement,
(ii) little or no cement grout cover in the presence of chlorides,
(iii) inappropriate choice of protective material,
(iv) use of tendon after a long period of storage in an unprotected state.

Failures at, or adjacent to the anchor head were due to causes ranging from absence of protection (even for only a few weeks in aggressive environments) to inadequate cover due to incomplete filling initially or slumping of the protective filler in service.

From all the case histories reviewed, it is apparent that corrosion incidents are somewhat random in terms of cause, with the possible exception of choice of prestressing steel. Various studies (eg Burdekin & Rothwell, 1981) have highlighted that quenched and tempered plain carbon steels and high strength alloy steels are more susceptible to hydrogen embrittlement than other varieties. Accordingly, these named steels should be used with extreme caution where environmental conditions are aggressive.

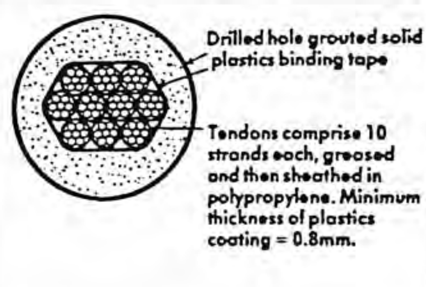


Figure 2: Typical free length detail for single protection of strand tendon.

*Professor of Civil Engineering, University of Bradford, Chairman of BSI Technical Committee on Ground Anchorages, Chairman of FIP working group on ground anchorages.

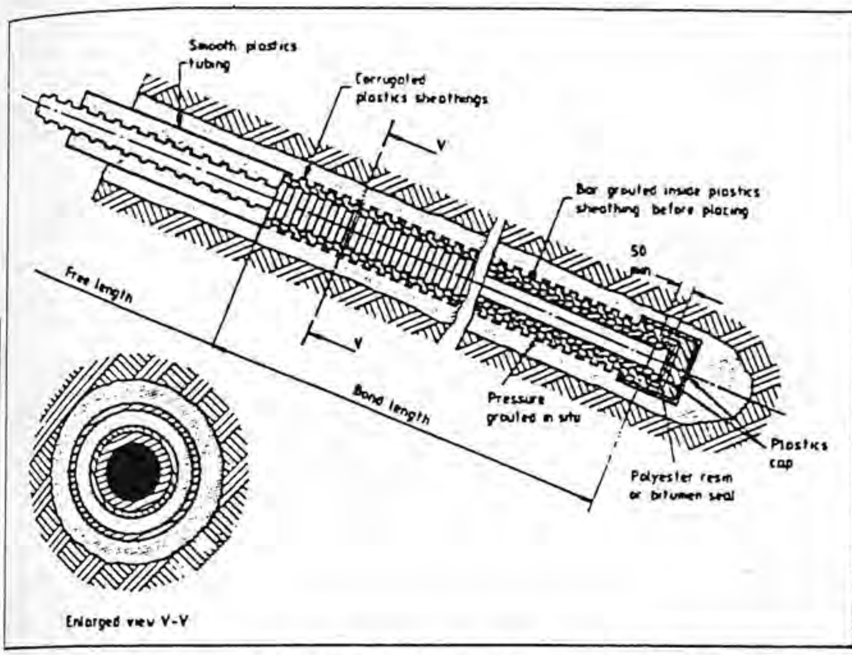


Figure 3: Typical double protection of bond length of smooth or ribbed bar tendon using a double corrugated sheath.

It is also noteworthy that Uhlig (1971) indicates that high-strength steels with yield strengths greater than 1240 N/mm^2, or a Rockwell C hardness value greater than 40, are susceptible to stress corrosion cracking. If sulphides are present, Phelps (1967) has stated that the equivalent Rockwell C threshold is reduced to 22.

3. Corrosion of steel tendon in hydraulic cement

Steel is protected against corrosion when maintained in a high pH environment free of aggressive ions. Such an environment is provided by hydrated hydraulic cement (pH = 11-13), which will give protection over the long term while the high level of alkalinity remains. However, loss of protection to the steel tendon or anchor head can occur as a result of lowering alkalinity, through cracks, carbonation, or the presence of aggressive ions, especially chloride.

When steel tendons in cement fixed anchor grouts are stressed, cracks tend to occur at about 50mm to 100mm apart and of widths up to 1mm or more (see Graber, 1981 and Meyer, 1977). Such cracks are unacceptable in a protective barrier.

Although there is little field evidence to indicate what crack widths are acceptable in a cementitious barrier, an upper limiting crack width of 0.1mm has been proposed by several researchers (FIP, 1986). In this regard, field evidence of the performance of ribbed bar (Ostermayer & Scheele, 1977) has illustrated that the ribs can control the frequency of cracking within a corrugated duct encapsulation, to such an extent that the crack widths are less than 0.1mm. For this situation it may be argued that the inner cracked grout will provide a physical barrier to corrosion. The field work was carried out in compact gravelly sands (D_r = 76%; Ø = 42°) and looking to the future more research is required to confirm that ribs can limit crack widths to 0.1mm in poor ground with a low lateral restraint.

4. Aggressivity of ground and ground water

Whilst the mechanisms of steel corrosion are understood, the aggressivity of the ground towards steel is seldom quantified at the site investigation stage.

Although water content, aggressive ion content, eg chloride, sulphide and nitrate ions, and permeability of the ground all influence corrosion, it is apparent that some generalised measure of redox potential and soil resistivity can provide guidance for the assessment of potential ground corrosiveness to embedded metals (see ***Table 1***).

The quantitative assessment of redox potential provides guidance on the risk of microbiological corrosion which most often results from the metabolic processes of sulphate-reducing bacteria (SRB) utilising sulphate in anaerobic conditions. Suitable anaerobic conditions are found in spaces isolated from atmospheric oxygen, particularly in sulphate-bearing clay or organic soils below the water table. SRB are most active at pH values of 6.2 to 7.8 and microbiological corrosion is frequently characterised by pitting attack.

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

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

Table 1. Corrosiveness of soils related to values of resistivity and redox potential (after King, 1977).

Table 1 provides guidance for soils of single composition and special precautions may be necessary where the anchorage passes through strata of differing composition to avoid the development of differential embedment cells, eg where tendons pass from an aerated soil such as gravel to a non-aerated soil such as clay.

At the present time there is no standard method for measuring redox potential and consequently care is required both in the performance of the test and interpretation of the results (see also Department of Transport, 1986). Similarly, methods of measuring resistivity vary but all involve passing a known current through the ground and measuring the voltage drop along the line of current flow (Palmer, 1974). For soils that are well graded and homogeneous, the resistivity values should adequately predict the corrosion hazard. Non-cohesive soils are the most reliable to measure, ie there is less scatter of results.

ASTM (1979) provides useful data on the corrosion processes and measurements involved underground. In general, fills and disturbed soils demand careful investigation.

Another important consideration is the potential for stray current corrosion, since high tensile steel is more sensitive to such attack than mild steel.

The presence in the ground of stray electrical currents arising adjacent to electrical plant, eg electrified railways and cathodic protection systems, can cause corrosion of a steel tendon if this becomes the anode in a galvanitic process. Insulation of the anchorage

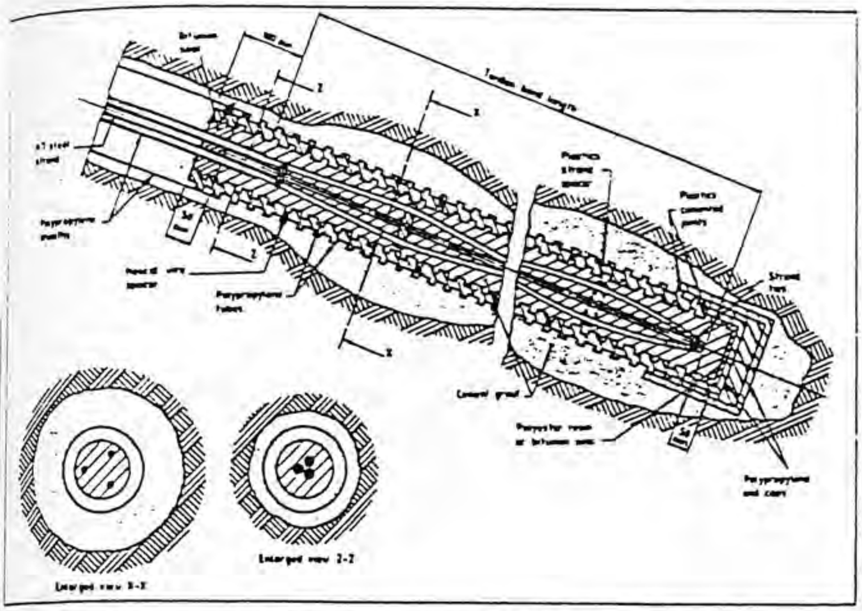


Figure 4: Typical double protection of bond length of strand tendon using a double corrugated sheath and cement grout.

tendons from the structure and the ground is recommended as a routine prudent practice.

In broad terms FIP (1986) highlights that the following, or similar, circumstances warrant consideration of corrosion protection of ground anchorages:

(i) situations exposed to sea water which contains chlorides and sulphates;
(ii) saturated clays with low oxygen content and high sulphate content;
(iii) fill materials;
(iv) evaporite rocks that contain chlorides eg salt lake deposits;
(v) soils in the vicinity of chemical factories that have corrosive effluents;
(vi) where tendons pass through ground with fluctuating water levels;
(vii) where tendons pass through strata of differing character with reference to chemical composition or differences in water or gas content;
(viii) tendons subject to cyclic stress changes.

In summary, exposure to combinations of oxygen and chlorides, anaerobic conditions in the presence of sulphates, or severely fluctuating and high stress levels all enhance the rate of corrosion.

5. Corrosion protection

5.1 General

The object of design against corrosion is to ensure that during the design life of the ground anchorage the probability of unacceptable corrosion occuring is small. In addition, the safety of people and property in the event of anchorage failure should be assessed carefully in relation to the cost of providing protection.

Given that 19 failures described by FIP occurred within 18 months of installation, it is clear that temporary anchorages exposed to aggressive conditions (service life ≯ two years) will demand consideration of corrosion protection as well as permanent anchorages.

Various degrees of protection are possible, but unless there is a positive demonstration that anchorages are not at risk within their working life, protection graded according to the severity of the problem should be provided.

For corrosion resistance, the anchorage should be protected overall as partial protection of the tendon may only induce more severe corrosion on the unprotected part.

Choice of class of protection (see ***Table 2***) should be the responsibility of the designer, and the decision depends on such factors as consequence of failure, aggressivity of the environment and cost of protection.

By definition single protection implies that one physical barrier against corrosion is provided for the tendon prior to installation.

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

Table 2. Proposed classes of protection for ground anchorages.

Double protection implies the supply of two barriers where the purpose of the outer second barrier is to protect the inner barrier against the possibility of damage during tendon handling and placement.

5.2 Properties of a protective system

A protective system should have the following properties:

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

5.3 Principles of protection

To fulfil the above recommendations, protective systems should aim to exclude a moist gaseous atmosphere around the metal by totally enclosing it within an impervious covering or sheath.

Grout injected in situ to bond the tendon to the ground does not consitute a part of a protective system because the grout quality and integrity cannot be assured. Furthermore, fluid materials that become brittle on hardening crack in service as the structure suffers differential strains, the onset of cracking depending upon tensile strength and ductility.

Non-hardening fluid materials such as greases also have limitations

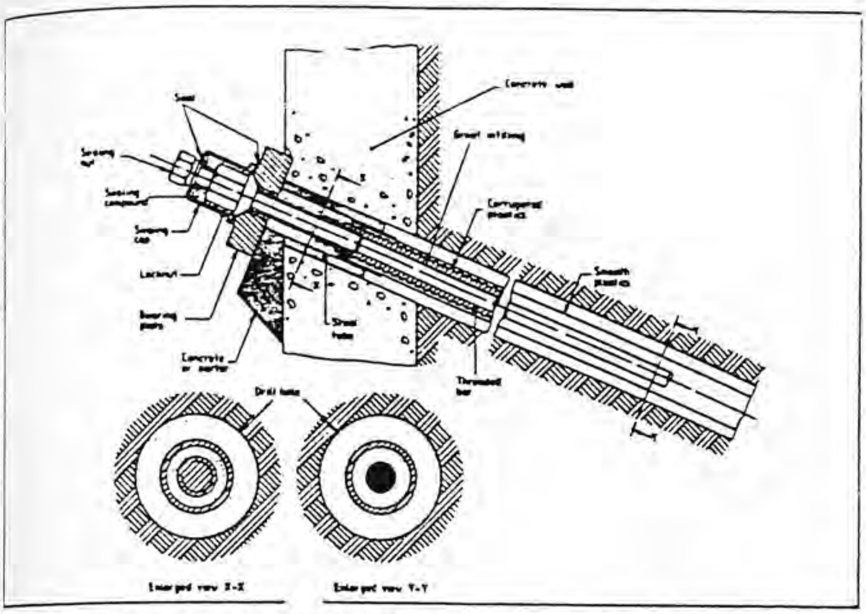


Figure 5: Typical detensionable anchor head detail for double protection of bar tendon.

as corrosion protection media. Reasons include:

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

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

Nevertheless, non-hardening fluids such as grease fulfil an essential role in corrosion protection systems, in that they act as a filler to exclude the atmosphere from the surface of a steel tendon, create the correct electrochemical environment and reduce friction in the free length.

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

Use of thicker metal sections for the tendon, with sacrificial area in lieu of physical barriers, gives no effective protection, as corrosion is rarely uniform and extends most rapidly and preferentially at localised pits or surface irregularities.

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

6. Protective systems

6.1 General

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

Designers should choose systems appropriate to their assessment of service conditions.

6.2 Free length of tendon

Protection is achieved generally by either injection of solidifying fluids to enclose the tendon or by pre-applied coatings, or by a combination of both, depending on circumstances.

Tendon coatings should be applied in factory conditions, either by the manufacturer of the tendon or on site within specially constructed workshops, where air-dry and clean conditions can be assured.

In the free length, the protective system should permit reasonably uninhibited extension of the tendon during stressing, and thereafter, if the anchorage is restressable. Greased and sheathed tendons are a popular solution in such circumstances (see *Figure 2*).

Continuous diffusion impermeable polypropylene or polyethylene sheaths applied in factory conditions are suitable for both temporary and permanent anchorages. Minimum wall thickness should be 0.8mm. Plastics that are susceptible to ultraviolet light may be used, provided that carbon black or ultraviolet inhibitors are incorporated to resist degradation.

Greases should not contain any substance that could provoke corrosion, eg unsaturated fatty acids, and water, and individual contents of sulphides, nitrates and chlorides should not exceed 5×10^{-4}% (m/m). Anti-corrosion compounds should be described. Greases should be stable against water and oxygen and should not separate into soap and oil. Hydrophobic greases are preferable. Other important factors include bacterial and microbiological degradation resistance, low moisture vapour transmission and high electrical resistivity (see FIP, 1986).

Greases should be checked to ensure that they are compatible with wrapping or sheathing materials that may be applied after greasing. Greases should not affect barrier properties of coverings.

Light corrugated metal sheaths are not suitable for corrosion protection, since they are easily perforated by corrosion. Any metal used has to be compatible with the tendon, so as not to induce corrosion potentials between the differing metals.

Sacrificial metallic coatings for high strength steel (> 1040 N/mm^2) should not be used when such coatings can cause part of the steel tendon to act as a cathode in an uncontrolled manner in a galvanitic process.

6.3 Bond length of tendon

The bond length requires the same degree of protection as the free length. In addition the protective elements have all to be capable of transmitting high tendon stresses to the ground. This requires strength and deformability characteristics that have to be checked structurally.

The deformation of individual elements of the corrosion protection system should not be such as to allow continuing creep nor expose the tendon bond length through cracking. The requirements of no creep and no cracking are in conflict and few materials are available that can comply with them under the intensity of stress around the fixed anchor.

Certain materials, notably epoxy or polyester resins, have appropriate strength, ductility and resistance to corrosion. They may be substituted for cementitious grouts but are more expensive.

When used to encapsulate bond lengths of tendon in combination with plastics ducts, compatibility of elastic properties of the anchorage components has to be examined to minimise decoupling or debonding of the resin from the duct.

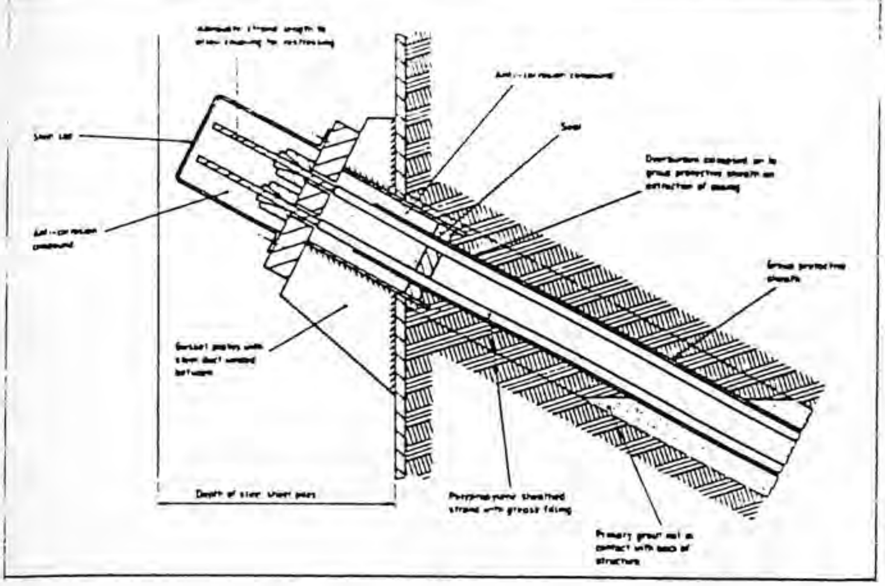


Figure 6: Typical restressable anchor head detail for double protection of strand tendon.

Where cement grout is employed for stressed tendon bonding the total chloride content from all sources should not exceed 0.1% by weight of cement.

The detail of the plastics duct that forms one element of protection is important as the duct has also to transmit stresses from filler to external grout without displacement or distress.

To ensure effective load transfer between duct and grout, ducts are corrugated. The pitch of corrugations should be within six and 12 times the duct wall thickness and amplitude of corrugation not less than three times the wall thickness. The minimum wall thickness is 0.8mm, but consideration of material type, method of installation and service required may demand a greater thickness. Duct material should be impervious to fluids.

Duct joints, whether screwed or not, should be sealed to preclude passage of fluids. Unjointed ducts are preferred.

When forming vertical or inclined grouted duct encapsulations, it is good practice to tremie or otherwise introduce the grout from the bottom of the vertically restrained sheath to ensure complete expulsion of air and to provide good grout contact with the contained wires, strands or bar. This contact is critically important for bond and corrosion protection.

Typical examples of double protection arrangements for the bond length of bar and strand tendons are shown in ***Figure 3 and 4*** (see also FIP, 1986).

Where protection has not been specified, and the conditions are known to be benign then cement grout cover over the fixed length may be deemed appropriate for temporary anchorage proposals, on the basis that nothing more stringent has been required.

6.4 Anchor head

Unlike fixed anchors, anchor heads cannot be wholly prefabricated. Because of the strain in the tendon associated with prestressing, friction grips for strand and locking nuts on bars cannot fix the tendon until extension has been achieved.

All existing locking arrangements require bare wire, strand or bar on which to grip and any preformed corrosion protection of the tendon has to be removed. This leaves two sections of the tendon, above and below the bearing plate (outer head and inner head, respectively), which require separate protective measures in addition to the protection of the bearing plate itself.

If the environment is aggressive, early protection of the anchor head is recommended for both temporary and permanent anchorages.

The essence of the inner head protection is to provide an effective overlap with the free length protection, to protect the short exposed length of tendon below the plate and to isolate the short section of the exposed tendon passing through the plate. In satisfying these recommendations, the protective measures have to allow free movement of the tendon that in certain instances may be solved by the use of a telescopic duct.

Cement grouts are generally considered unsuitable for inner head protection. Primary grout should not be in contact with the structure and where a weak, low bleed secondary grout is required to fill the void above the primary grout, it may be subject to cracking during structural movement.

Grease-based corrosion protection compounds or similar ductile materials immiscible with water may be required. They may be preplaced or injected and should be fully contained within surrounding ducts and retained by an end seal.

Where injection techniques are employed, a lower injection pipe and upper vent pipe should be used to ensure complete filling of the void and displacement of water and air. Preferably, injected material should be conveyed by tremie to the lowest part of the duct, displacing fluids upwards to the vent. Pressures of $150kN/mm^2$ or more are desirable for this operation, subject to limitations arising from structural constraints. In restricted space, simple grease gun techniques may be accepted. Alternatively, the filler inside the duct may be a prepacked grease if there is no access for injection after stressing.

Outer head protection of the bare tendon, the friction grips or the locking nuts above the bearing place generally falls into two categories, controlled by whether the anchorage is restressable or not. Where restressability is called for, both the anchor head cap and the contents should be removable to allow access to an adequate length of tendon for restressing. Clearly these requirements will vary depending on the stressing and locking system employed. Grease is the most commonly used material within plastics or steel caps. Alternatives include corrosion resistant grease impregnated tape and heat shrink sleeving.

A suitable seal and mechanical coupling between the cap and the bearing plate should be provided.

Where restressability is not a requirement of the anchorage, then the cap and its contents are not required to be removable. Thus resins or other setting sealants may be used and a mechanical coupling between the cap and the bearing plate is not essential.

Where the anchor head is to be totally enclosed by the structure, the outer head components may be encased in dense concrete as an alternative protection, given adequate cover.

The bearing plate and other essential exposed steel components at the anchor head should be painted with bitumastic or other protective materials, prior to being brought to site. Steel surfaces should be cleaned of all rust and deleterious matter prior to priming, eg by blast cleaning. The coatings should be compatible with the materials selected for both inner head and outer head protection. Bearing plates on concrete structures may be set in a seating formed of concrete, cement, epoxy or polyester mortar or alternatively may be seated direct on to a cast in steel plate.

Typical examples of double protection arrangements for the anchor head are illustrated in ***Figures 5 and 6.***

6.5 Centralisers and spacers

To ensure proper central positioning and spacing of the tendon in the borehole, appropriate centralisers/spacers should be employed. This ensures a correct thickness of cover of filler or grout around the tendon for efficient load transfer and also guards against the presence of smear from the surrounding ground. For permanent anchorages, centralisers and spacers of the fixed anchor in aggressive ground should be manufactured from non-corrodible materials, eg plastics or plastics coated metal.

7 Quality controls

Corrosion protection systems should be assembled according to an agreed method statement and checks shoud be carried out on each system to ensure the quality and integrity of prefabricated components, adequate overlap of protective barriers at the key interfaces, eg anchor head/free length and free length/tendon bond length, and appropriate grout properties, eg adequate strength and low bleed. It may also be prudent on occasion to cut up a completed protection system into sections, eg tendon bond length, to permit assessment of the quality and integrity of the work.

Electrical resistance measurements offer a simple and convenient method of checking the insulation of the sheath between the steel tendon and the surrounding ground, once the unstressed tendon has been grouted. Based on the recent experience of Swiss ground anchorage companies (see Fischli, 1989), a mimimum resistance of 0.1Mohm should be obtained when a 500V direct current is applied between the steel tendon and the ground (earth). Once the tendon has been stressed but prior to encasing the anchor head, the insulation of the anchor head may be checked by applying a 40V alternating current between the anchor head and earth. In such circumstances experience to date indicates that a minimum resistance of 100Mohm should be obtained. More experience in the use of electrical resistance techniques is required before generally applicable acceptance criteria and tolerances can be proposed together with stage by stage method statements, but these systems could have considerable potential in the future.

8 New developments

Non-metallic fibres with appropriate strength and creep properties may be used for tendons, subject to investigation of their effective life in stressed conditions when exposed to potentially aggressive environments that may differ from those aggressive to steel. In other words, although non-metallic fibres may be resistant to highly acidic conditions, these same materials may deteriorate in a highly alkaline environment such as that provided by a cement grout.

Epoxy coated steel tendon is another recent development (see Cousins, Johnston and Zia, 1990) whereby prestressing strand is coated with an epoxy resin (typically 1mm thick) and impregnated with a crushed glass grit to improve bond. Initial data on load transfer by bond are encouraging and assuming that cost is not an inhibiting factor, the only potential areas of concern which require further investigation relate to the relaxation or creep losses within the anchor head, ie at the wedges, the ongoing need for inner head and outer head protection given the epoxy coating deformations caused by the teeth in the wedges, and the effect of high temperature or fire conditions on service performance. Given a satisfactory outcome from these investigations, it will then be necessary, as with any prefabricated component, to instigate quality controls to ensure the integrity and continuity of the protective epoxy coating.

9 Conclusions

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, some of which were protected only by cement grout cover.

Invariably the corrosion has been localised and failures have occurred after service of only a few weeks to many years.

As a consequence, it is considered that all permanent anchorages, and temporary anchorages exposed to aggressive conditions should be protected, the degree of protection depending primarily on factors such as consequence of failure, aggressivity of the environment and cost of protection.

Acknowledgement

Extracts from BS8081:1989 are reproduced with the permission of BSI. Complete copies can be obtained through national standard bodies.

References

American Society for Testing Materials. Underground corrosion. ASTM Symp on Corrosion of Metals, Williamsburg, Virginia, 1979.
British Standards Institution. Ground anchorages, BS8081. British Standards Institution, London, 1989.
Burdekin, FM & Rothwell, GP. Survey of corrosion and stress corrosion in prestressing components used in concrete structures with particular reference to offshore applications. Cement and Concrete Association, Slough, England, 1981.
Cousins, TE, Johnston, DW & Zia, P. Transfer length of epoxy-coated prestressing strand. ACI Materials Journal (May-June), 193-203, 1990.
Department of Transport, Specification for Highway Works Part 2 Earthworks. Her Majesty's Stationery Office, London, 1986.
Federation Internationale de la Precontrainte. Corrosion and corrosion protection of prestressed ground anchorages. State of the art report. Thomas Telford Ltd, London, 1986.
Fischli, F. Private correspondence related to unpublished recommendations for the design and execution of corrosion protection of permanent soil and rock anchors, 1989.
Graber, F. Excavation of a VSL rock anchor at Tarbela. VSL Silver Jubilee Symposium, Losinger Ltd, Berne, Switzerland, 1981.
King, RA. A review of soil corrosiveness with particular reference to reinforced earth. TRRL Supplementary Report 316. Transport & Road Research Laboratory, Crowthorne, England, 1977.
Meyer, A. report on discussion to Session VI by JM Mitchell, A review of diaphragm walls, Institution of Civil Engineers, London, 1977.
Palmer, JD. Soil resistivity – measurement and analysis. Materials Performance (Jan.), 41-46, 1974.
Phelps, EH. A review of the stress-corrosion behaviour of steels with high yield strength. Conf on fundamental aspects of stress corrosion cracking, Ohio State University, Columbus, Ohio, 1967.
Uhlig, HH. Corrosion and corrosion control. J Wiley & Sons, New York, 1971.

Papers

Routine on-site acceptance tests for ground anchorages

by G S Littlejohn*

Introduction

Following a review of ground anchorage practice it is apparent that there still exists a wide variety of testing procedures and criteria for the acceptance of individual anchorages which are to be incorporated into temporary or permanent works.

The purpose of this paper is to describe the minimum requirements for routine on-site acceptance testing and the associated reasoning, based on the recommendations of the Fédération Internationale de la Précontrainte (FIP, 1991) and the British Standards Institution (BS8081, 1989).

General considerations

To put the subject into perspective there are three major classes of tests for ground anchorages, namely (i) proving tests, (ii) on-site suitability tests and (iii) on-site acceptance tests.

Proving tests are required to demonstrate or investigate in advance of the installation of working anchorages, the quality and adequacy of the design in relation to ground conditions and materials used and the levels of safety that the design provides. The tests may be more rigorous than on-site suitability tests and the results, therefore, cannot always be directly compared, eg where short fixed anchors of different lengths are installed and tested, ideally to failure.

On-site suitability tests are carried out on anchorages constructed under identical conditions to the working anchorages and loaded in the same way to the same level. These may be carried out in advance of the main contract or on selected working anchorages during the course of construction. The period of monitoring should be sufficient to ensure that prestress or creep fluctuations stabilise within tolerable limits. These tests indicate the results that should be obtained from the working anchorages and constitute the models against which the working anchorages can be assessed.

On-site acceptance tests are carried out on all anchorages and demonstrate the short term ability of the anchorage to support a load that is greater than the design working load and the efficiency of load transmission to the fixed anchor zone. A proper comparison of the short term service results with those of the on-site suitability tests provide a guide to longer term behaviour.

As ground is a variable material and anchorage construction is sensitive to workmanship, it is not considered prudent to simply select and test a proportion of the anchorages, say 10%. If all the selected anchorages pass, the writer's field experience, particularly in alluvial deposits, fissured clays and weak mudstones indicates that there is no guarantee that the remaining 90% will follow a similar behaviour. Furthermore, if one or two do fail, unless the reasons can be identified clearly, there is a strong obligation to confirm the 'health' of all the remaining anchorages in order to provide the necessary reassurance that they are fit for their intended purpose.

* Professor of Civil Engineering, University of Bradford, Chairman of BSI Committee on Ground Anchorages, Chairman of FIP Working Group on Ground Anchorages.

Table 1: Minimum safety factors recommended for design of individual working anchorages

Anchorage category	Minimum load safety factor $\gamma = \frac{T_f}{T_w}$	Minimum proof load factor $\frac{T_t}{T_w}$
1 Temporary anchorages where the service life is less than six months and failure would have few serious consequences and would not endanger public safety, eg short term pile test loading using anchorages as a reaction system.	1.4	1.1
2 Temporary anchorages with a service life of up two years, where, although the consequences of local failure are quite serious, there is no danger to public safety without adequate warning, eg retaining wall tie backs.	1.6	1.25
3 Permanent anchorages and also temporary anchorages where the consequences of failure are serious, eg temporary anchorages for main cables of a suspension bridge, or as a reaction for lifting heavy structural members.	2.0	1.5

As a principle, acceptance testing should comprise standard procedures of short duration, and be independent of ground type.

Where specific procedures and durations of testing are specified for different major ground types arguments can arise over the most appropriate geotechnical class for variable ground.

Guidelines on maximum test or proof loads are provided in **Table 1** which is recognised internationally through FIP.

Although the principle of proof loading is now widely accepted, slight variations in the magnitude of the specified proof load may still be encountered in practice from one country to another.

Cyclic loading is also traditional since non-recoverable movements, such as 'bedding in' and wedge 'pull-in' of the anchor head, are encountered during the initial loading phase. These movements are not repeated in subsequent cycles and so the reproducible behaviour of the anchorages can be both confirmed and measured.

Given increased confidence and the improved reliability of ground anchorage technology, the number of cyclic load increments and the minimum periods of observation have gradually been reduced over the years. These reductions have saved time and money, and **Table 2** provides an example of current recommendations. In spite of the simplicity of these test procedures acceptance criteria for ground anchorages remain rigorous when compared with other foundation systems.

At each stage of loading, the displacement should be recorded at the beginning and end of each period, and for proof loads the minimum period of one-minute is extended to at least 15 minutes with an intermediate displacement reading at five minutes. With these procedures any tendency to creep can be monitored.

In some countries, creep displacements at proof load are recorded in greater detail after the proof load is applied, while in other countries engineers prefer to monitor such displacements at the initial lock-off load ($\approx 110\%\ T_W$) in order to predict service behaviour.

Table 2: Recommended load increments and minimum periods of observation for on-site acceptance tests

Temporary anchorages load increment ($\%T_w$)		Permanent anchorages load increment ($\%T_w$)		
1st load cycle* %	2nd load cycle %	1st load cycle* %	2nd load cycle %	Minimum period of observation min
10	10	10	10	1
50	50	50	50	1
100	100	100	100	1
125	125	150	150	15
100	100	100	100	1
50	50	50	50	1
10	10	10	10	1

* For this load cycle, which often includes extraneous non-recoverable movements such as wedge 'pull-in', bearing plate settlement and initial fixed anchor displacement, there is no pause other than that necessary for the recording of displacement data.

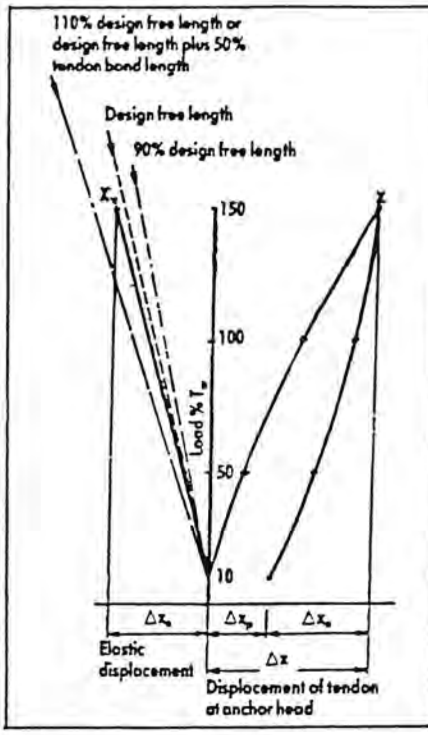


Figure 1: Acceptance criteria for displacement of tendon at anchor head.

With regard to design considerations related to overall stability, it is important to confirm that the post-tensioned load is properly transferred through the free 'decoupled' length of the tendon into the fixed anchor zone.

To establish the actual seat of load transfer within the anchorage, the apparent free length of the tendon should be calculated from the load-elastic displacement curve over the proof loading range using the manufacturer's value of elastic modulus and allowing for such effects as bedding of the anchor head and, in exceptional circumstances, temperature. It is normally adequate simply to record the ambient temperature during the test, unless the monitoring equipment or anchored structure is known or observed to be temperature sensitive.

The free length analysis should be based on the results obtained during the second cycle, otherwise extraneous non-recoverable movements may mask the reproducible behaviour of the anchorage in service (**Figure 1**).

For simplicity in practice the following equation is employed

$$\text{Apparent free tendon length} = \frac{A_t\, E_s\, \Delta X_e}{T}$$

where A_t is the cross section of the tendon, E_s is the manufacturer's elastic modulus for the tendon unit, ΔX_e is the elastic displacement of the tendon (ΔX_e is equated to the displacement monitored at proof load minus the displacement at datum load, ie 10% T_w say) and T is the proof load minus datum load.

On completion of the second cycle, the anchorage should be reloaded in one operation to 110% T_w say, and locked-off, after which the load is re-read to establish the initial residual load. This moment represents zero time for monitoring load or displacement-time behaviour during service.

Where loss of load is monitored accurately using load cells with a relative accuracy of 0.5%, readings can be attempted within the first 50 minutes (**Figure 2**). This development, although demanding more sophisticated instrumentation, permits the on-site acceptance test to be carried out in one operation alongside the routine post-tensioning stage, and the anchorage can be passed or failed, within a period of one to one and a half hours. Where monitoring involves a stressing operation, eg a single lift-off check without load cell, an accuracy of less than 5% is unlikely and longer observation periods of one day and beyond are required. If necessary, the accuracy of lift-off checks can be improved by repeating the test several times.

Where displacement-time data are required, a dial gauge/tripod system is suitable for short duration testing, given that the tripod base should be surveyed accurately for movement (**Figure 3**). In practice dial gauges reading to 0.01mm are commonly used during the test, and where movement of the tripod base is anticipated, its position is checked before and after the test to an accuracy of 1mm.

For the testing procedures outlined above, acceptance criteria based on proof load-time data, apparent free tendon length, and short term service behaviour, should be established for temporary and permanent anchorages. Appropriate criteria, which are well proven on site and judged to be cost effective, are detailed in the following sections (*see also* BS8081, 1989 and FIP, 1991).

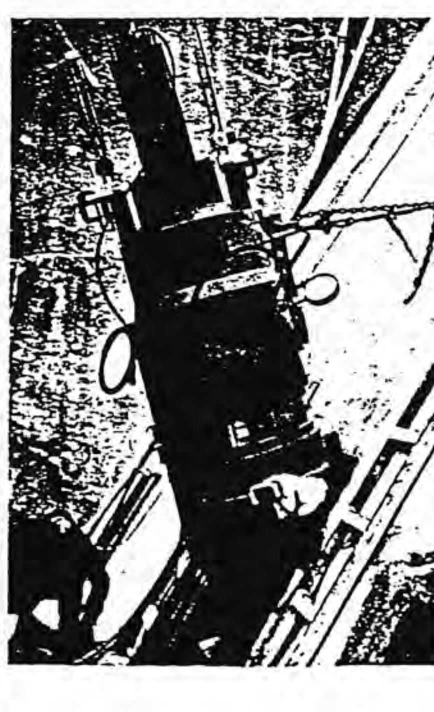


Figure 2: Proof testing of 58 strand tendon at L Lynn dam, West Virginia (courtesy of Nicolson Construction Co of America).

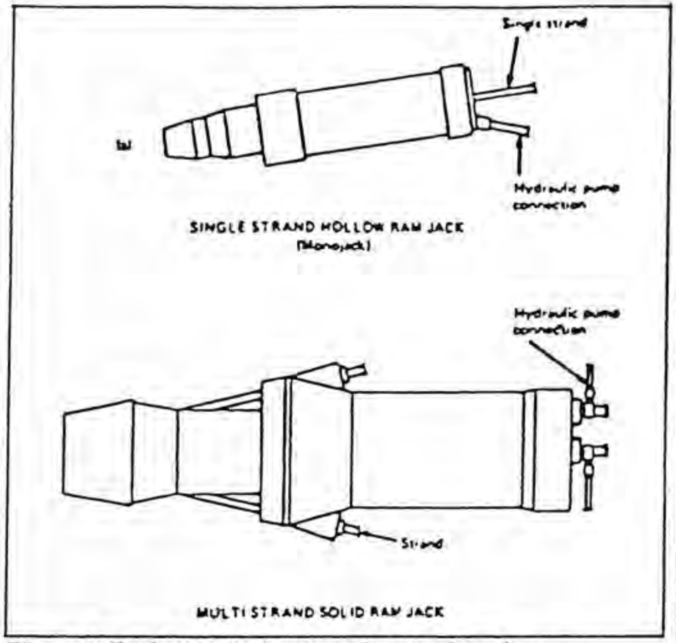


Figure 4: Typical jacks for stressing steel tendons.

Proof load-time acceptance criteria

If the proof load has not reduced during the 15 minute observation period by more than 5% after allowing for any movement of the anchored structure, the anchorage may be deemed satisfactory. If a greater loss of prestress is recorded the anchorage should be subject to two further proof load cycles and the behaviour recorded. If the 5% criterion is not exceeded on both occasions the anchorage may be deemed satisfactory. If the 5% criterion is exceeded on either cycle the proof load should be reduced to a value at which compliance with the 5% criterion can be achieved. Thereafter, the anchorage may be accepted at a derated proof load, if appropriate.

The 5% loss limit merely serves to illustrate that the anchorage will not yield significantly at proof load and no attempt is made to ascertain the proof load-time characteristic of the anchorage at this abnormally high stress level.

As an alternative to these recommendations, the proof load can be maintained by jacking and the anchor head monitored after 15 minutes in which case the creep criterion is 5% ΔX_e, ie the displacement which would cause a 5% loss of proof load.


Figure 3: Displacement-time monitoring at Delli in Switzerland (courtesy of VSL International).

In some countries limiting creep displacements are specified irrespective of free tendon length eg 2mm (0.5 – 5 min.) for the US Department of Transport (1984), 1.5mm (1 – 10 min.) for the Bureau Securitas (1989) and 0.5mm (5 – 15 min.) for the Deutsche Industrie Norm (1974 and 1976). These figures illustrate the ad hoc nature of current creep criteria and at the present time few countries provide correlations to permit either load or creep monitoring to be adopted.

For anchorages that do fail a proof load criterion it is noteworthy that tendon unit stressing (mono jacking) may help to ascertain location of failure (**Figure 4**). For standard bonded tendons the pull-out of individual tendon units by mono jack indicates debonding at the grout-tendon interface, whereas, if all tendon units hold their individual proof loads, attention is directed towards failure of the fixed anchor at the ground-grout interface. In this diagnostic test there is no preferred sequence for stressing individual strands. For long high capacity anchorages, mono jacking may also be useful in establishing a uniform initial loading of strands, prior to cyclic loading by multi tendon unit stressing (multi jack).

Where a multiple encapsulation system or an encapsulation with a multi-unit load transfer mechanism is employed the tendon load is transferred to the adjacent grout though discrete lengths uniformly distributed at intervals along the encapsulation or fixed anchor length. In such circumstances individual strands or groups of strands have different free tendon lengths and care is required to avoid strand overstressing at the tendon proof load.

For routine multi strand stressing of these anchorages all strands are stressed to different levels at a given tendon load or displacement, and the maximum tendon proof load will be reached when the load in the shortest strand attains 80% fpu (fpu = characteristic strength of the strand). If it is necessary to increase further the proof load using a multi jack, then a prestretch procedure must be introduced whereby all strands, which are longer than the shortest strand length, are tensioned to predetermined displacements such that when multistrand stressing of the tendon takes place all strands attain the same stress level at the required proof load. Alternatively, a monojack may be used to load incrementally each strand to the same value for proof loading.

Irrespective of the mode of stressing, which may be more time consuming for distributed stress transfer fixed anchors, the proof load-time acceptance criteria above still apply.

Load transfer acceptance criteria

The apparent free tendon length should be not less than 90% of the free length intended in the design, nor more than the intended free length plus 50% of tendon bond length or 110% of the intended free tendon length (**Figure 1**).

The latter upper limit takes account of relatively short encapsulated tendon bond lengths of 1m to 3m and fully decoupled tendons with an end plate or nut, and the application of this upper limit should be restricted to such circumstances.

The boundary limit of '90% free length' reflects a tightening of tolerances over the years, bearing in mind that a limit of '80% free length' was in common use in the 1960s. Where greased and sheathed tendons are assembled under 'factory-controlled' conditions the more rigorous criterion is attained without difficulty. The key to success is to ensure that the plastics sheathing has a 'loose' fit over the tendon.

If the observed free tendon length does fall outside either of the limits a further two load cycles up to proof load should be carried out in order to gauge reproducibility of the load-displacement data particularly during the third and fourth cycles. For a 'long' apparent free tendon length the comparison of load-displacement behaviour during the second, third and fourth cycles checks for progressive debonding within the tendon's bond length. For a 'short' apparent free tendon length a potential explanation is friction within a greased and sheathed decoupled free tendon length. Such a load transfer mechanism is often visco-elastic in nature and given time, eg 6 to 12 hours, creep within the decoupled system will permit the friction load to be transferred to the tendon bond length. In these circumstances the reproducibility check is only relevant to the load-displacement behaviour during the third and fourth cycles.

Where the anchorage behaves consistently in an elastic manner, the anchorage need not be abandoned, provided the reason can be diagnosed and accepted. In this regard, it is noteworthy that the elastic modulus E of a long strand tendon may be less than the manufacturer's E value for a single strand, which has been measured over a short gauge length between rigid platens (see also Janische, 1968 and Leeming, 1974). A reduction in the manufacturer's E value of up to 10% should be allowed in any field diagnosis of the load-elastic displacement behaviour of single or multi-strand tendons.

In the case of distributed stress transfer fixed anchors the above load transfer criteria apply to an analysis of the apparent free length of each individual strand or group of strands of equal length, bearing in mind the designed decoupled and bonded lengths. To obtain the appropriate strand load-displacement data it is usually necessary to use multijacking with prestretch or monojacking. Analysis of an average apparent free tendon length is not recommended since this parameter masks the true load transfer behaviour of individual strands.

Short term service acceptance criteria

Using accurate load cell and logging equipment, the residual load may be monitored at 5, 15 and 50 minutes. If the rate of load loss reduces to 1% or less per time interval for these specific observation periods after allowing for temperature (where necessary), structural movements and relaxation of the tendon, the anchorage may be deemed satisfactory in relation to this serviceability criterion. If the rate of load loss exceeds 1%, further readings should be taken at observation periods up to 10 days (**Table 3**).

If, after 10 days, the anchorage fails to hold its load as given in Table 3, the anchorage is not satisfactory and following an investigation as to the cause of failure, the anchorage should be (i) abandoned and replaced, (ii) reduced in capacity or (iii) subjected to a remedial stressing programme.

Where prestress gains are recorded, monitoring should continue to ensure stabilisation of prestress within a load increment of 10% T_w. Should the gain exceed 10% T_w, a careful analysis is required and it will be prudent to monitor the overall structure/ground/anchorage system. If, for example, overloading progressively increases due to insufficient anchorage capacity in design or failure of a slope, then additional support is required to stabilise the overall anchorage system. Destressing to working loads should be carried out as prestress values approach proof loads, accepting that movement may continue until additional support is provided.

As an alternative to load monitoring, displacement-time data at the residual load may be obtained at the specific observation periods in Table 3, in which case the rate of displacement should reduce to 1% Δe or less per time interval. To ensure compatibility of the acceptance criteria 1% Δe is the displacement equivalent to the amount of tendon shortening caused by a prestress loss of 1% initial residual load, ie.

$$\Delta e = \frac{\text{initial residual load} \times \text{apparent free tendon length}}{\text{area of tendon} \times \text{elastic modulus of tendon}}$$

If the anchorages are to be used in the work and, on completion of the on-site acceptance test, the cumulative relaxation or creep has exceeded 5% initial residual load or 5% Δe, respectively, the anchorage should be restressed and locked-off at 110% T_w, say. This procedure ensures that a contingency overload is locked into the ground anchorage at the start of its service.

Table 3: Acceptance criteria for service behaviour at residual load

Period of observation	Permissible loss of load (% initial residual load)	Permissible displacement (% of elastic extension Δe of tendon at initial residual load)
min	%	%
5	1	1
15	2	2
50	3	3
150	4	4
500	5	5
1500 (≈ 1 day)	6	6
5000 (≈ 3 days)	7	7
15000 (≈ 10 days)	8	8

As a general guide, either acceptance criterion for short term service, ie rate of prestress loss or rate of displacement may be applied quite independently for the common range of free tendon lengths. For short free tendon lengths (< 5m), loss of prestress becomes the more appropriate criterion, while for long free tendon lengths (> 30m) it is clear that creep displacement may be more important to limit and therefore more appropriate as an acceptance criterion.

Records

Details of all forces, displacements, seating and other losses observed during all stressing operations and the times at which the data were monitored should be recorded in an appropriate form for every anchorage. The completion of the record sheet and graphical plot of load displacement during a stressing operation allows on-going assessment of the anchorage performance and immediate confirmation regarding compliance with the acceptance criteria (load transfer, and percentage load or displacement change).

Safety

During stressing safety precautions are essential and operatives and observers should stand to one side of the tensioning equipment and never pass behind when it is under load. Notices should also be displayed stating 'DANGER – Tensioning in Progress' or similar wording.

Reference should be made to published guidelines eg Concrete Society (1980) and FIP (1989).

References

1 'Ground anchorages' BS8081. British Standards Institution. 2 Park Street, London. (1989).
2 Bureau Securitas 'Recommendations for the design, calculation, construction and monitoring of ground anchorages' AA Balkema, Rotterdam. (1989).
3 RS Cheney 'Permanent ground anchors' US Department of Transport, Federal Highway Administration Report FHWA-DP-68-1R, Washington DC. (1984).
4 'Safety precautions for prestressing operations (post-tensioning) – Notes for guidance'. The Concrete Society, Terminal House, Grosvenor Gardens, London. (1980).
5 'Soil and rock anchors; bonded anchors for temporary use. DIN 4125 Part 1; Permanent anchors, Part 2 (1976), Deutsche Industrie Norm Fachnormen-ausschus Bauwesen. Berlin. (1974).
6 Federation Internationale de la Precontrainte 'Prestressed concrete – safety precautions in post-tensioning'. Thomas Telford Ltd., London. (1989).
7 Federation Internationale de la Precontrainte 'Recommendations for the design and construction of prestressed ground anchorages'. Thomas Telford Ltd, London. (1991 to be published).
8 W Janiche 'Recent improvements in the manufacture and properties of prestressing steels', in FIP Proc. of Symposium 'Steel for Prestressing', Madrid, 1-4. (1968).
9 MB Leeming 'Discussion to prestressing steels by KW Longbottom and CP Mallet. The Structural Engineer, Vol 52 (9), 357-362. (1974).

Acknowledgement

Extracts from BS8081: 1989 are reproduced with the permission of BSI. Complete copies can be obtained through national standard bodies.

GROUND ANCHORAGE TECHNOLOGY - A FORWARD LOOK

by

Stuart Littlejohn[1]

Introduction

Although reliable methods of designing, constructing and testing ground anchorages are now well established in many parts of the world for both temporary and permanent works, the subject remains a fertile field for research and practical innovation.

The purpose of this paper is to highlight those areas where further investigation, improved standards, and in some cases better technical explanation, would enhance understanding of anchorage behaviour, increase confidence, and thereby extend the anchorage market place for the benefit of the construction industry and its clients.

Uplift Capacity

For vertical or downward inclined anchorages subjected to uplift forces, the fixed anchors must be installed at a depth sufficient to resist safely the applied working load without failure developing within the ground mass. Current design assumptions of cone, wedge or block failure mechanisms tend to be conservative as the shear strength of the ground is often ignored and calculations to estimate the uplift resistance are based simply on the weight of overlying ground mobilised at failure. This weight is calculated from the top, mid point or base of the fixed anchor, and for standard bonded tendons the former choice is the most conservative. For less conservative designs, e.g. where the apex of an inverted cone is taken from the mid point of the fixed anchor length, evidence should be available to substantiate that design assumption.

Practical experience indicates that general failure in the ground with accompanying surface heave does not occur for slenderness ratios ($^h/_D$) in excess of 15, where h is the depth to the top of the fixed anchor and D is the diameter of the fixed anchor (Bruce, 1976). In current practice a minimum depth of 5 m to the top of the fixed anchor is also commonly considered prudent.

---

[1]Professor of Civil Engineering, University of Bradford, England.

There is little experimental evidence to substantiate these empirical design methods and yet the calculated minimum depth of embedment can affect significantly the cost of the anchorage solution. Furthermore, routine co-axial loading during acceptance testing (see Figure 1) does not confirm the margin of safety for uplift capacity because the ground immediately surrounding the anchor head is used as a bearing surface for the stressing jack, thus an appropriate ground mass failure mechanism cannot be mobilised.

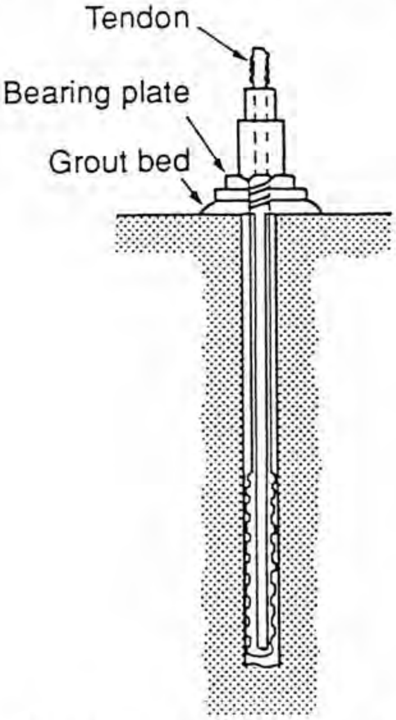


Figure 1. Co-axial loading of an anchorage (after ISRM, 1985)

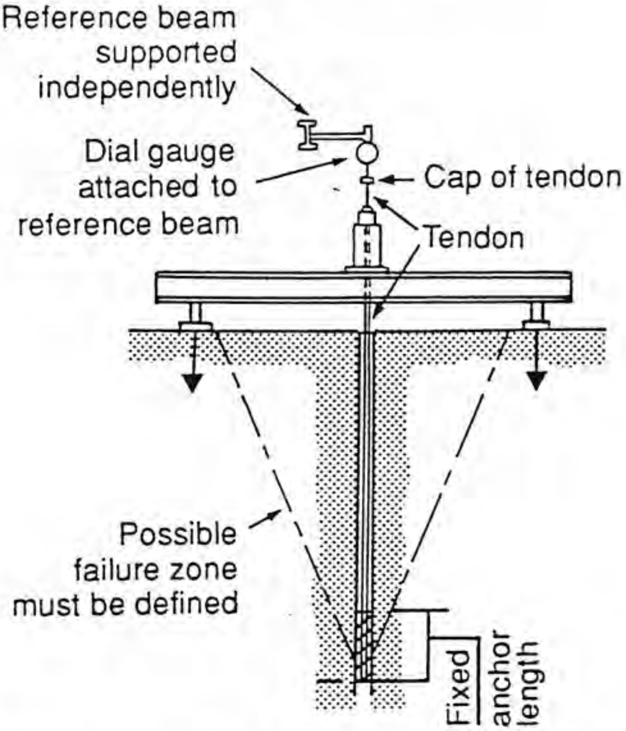


Figure 2. Remote loading of an anchorage (after ISRM, 1985)

Full scale pull-out tests by remote loading (see Figure 2) are therefore recommended for a variety of ground types and grouted fixed anchor geometries, including different load transfer systems. In ground masses which are horizontally bedded the mechanism of laminar failure is of particular interest, including the influence of fracture geometry. The objective of each study should be to establish the mechanism of failure and its relationship to the geotechnical classification of the ground and type of anchorage, coupled with a safe method of estimating uplift capacity which accommodates both the service and limit states.

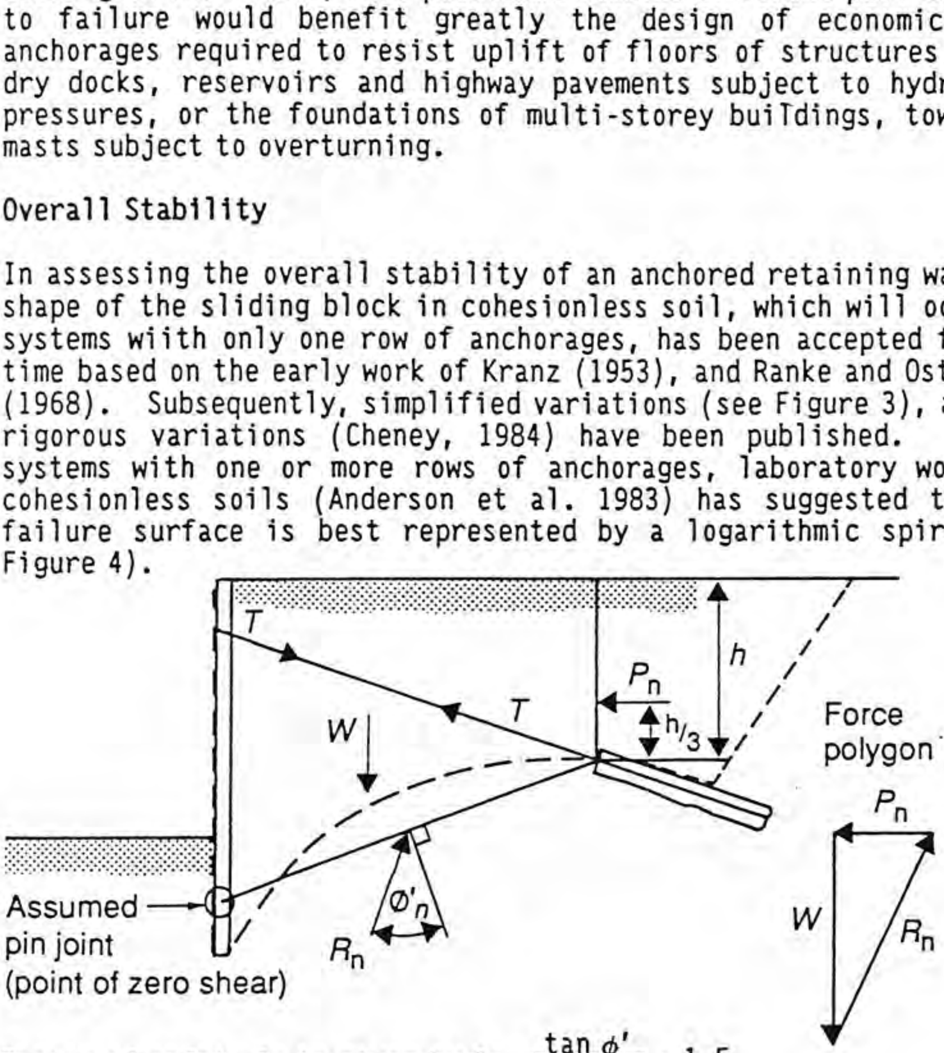
Initially, tests should be carried out on individual anchorages both post-tensioned and untensioned (passive) in order to study the influence of post-tensioning. Subsequently, group effects should be investigated where two or more closely spaced anchorages are pulled out simultaneously. The tendons of individual anchorages could also be instrumented to monitor the effects of stressing and de-stressing (failure simulation) on adjacent post-tensioned anchorages.

Where closely spaced high capacity anchorages are planned a highly stressed fixed anchor zone may be created at a single elevation. In such circumstances it would appear that the designer simply uses intuition to (i) stagger the depth location of alternate fixed anchors (Littlejohn and Truman Davies, 1974) or (ii) spread fixed anchors further apart by choosing different inclinations (Soletanche, 1968), in order to reduce the intensity of stress on any plane. Speaking personally, I cannot always justify the decision by a rigorous calculation but it is like a glass of wine, it makes me feel better.

Looking to the future, more published results of remote pull-out tests to failure would benefit greatly the design of economic ground anchorages required to resist uplift of floors of structures such as dry docks, reservoirs and highway pavements subject to hydrostatic pressures, or the foundations of multi-storey buildings, towers and masts subject to overturning.

Overall Stability

In assessing the overall stability of an anchored retaining wall, the shape of the sliding block in cohesionless soil, which will occur for systems wiith only one row of anchorages, has been accepted for some time based on the early work of Kranz (1953), and Ranke and Ostermayer (1968). Subsequently, simplified variations (see Figure 3), and more rigorous variations (Cheney, 1984) have been published. For systems with one or more rows of anchorages, laboratory work with cohesionless soils (Anderson et al. 1983) has suggested that the failure surface is best represented by a logarithmic spiral (see Figure 4).



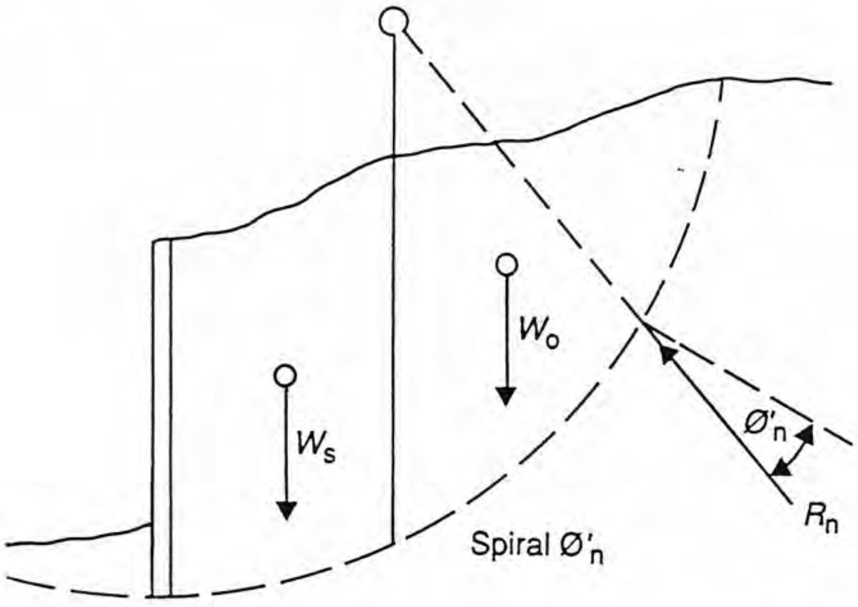
Factor of safety S_f is given by $S_f = \frac{\tan \phi'}{\tan \phi'_n} \geq 1.5$

where ϕ'_n is nominal angle of shearing resistance (in degrees).

NOTE. If ϕ'_n has been correctly assumed, the weight W and the forces R_n and P_n are in equilibrium. If this is not the case ϕ'_n has to be altered.

Figure 3. Sliding block method of analysis (after Littlejohn, 1970)

In the stability analyses described the basic assumption is that anchorage prestress increases the shear strength of the cohesionless soil sufficiently to displace the potential failure plane beyond the proximal end of the fixed anchor. Care should therefore be taken not to apply these methods outside the range of cohesionless soils. Further research is needed to extend the study to cohesive granular soils and all mathematical and physical models should be validated under field or large scale conditions.



$$S_f = \frac{\tan \phi'}{\tan \phi'_n} \qquad \text{For equilibrium } \frac{\text{Moment due to } W_S}{\text{Moment due to } W_0} = 1$$

Figure 4. Stability analysis using a logarithmic spiral (after Littlejohn, 1970)

In cohesive soils it is clear that anchorage prestress will only increase the shear strength of the soil gradually as consolidation occurs. Consequently, in this situation it is considered prudent to carry out a conventional analysis of overall stability neglecting the presence of the soil anchorages. The fixed anchors should then be located some distance, typically 2 to 3 m, beyond the potential slip zone to provide a stable founding material.

As soil nailing technology develops it is important to note that in overall stability analyses, a reinforced soil gravity structure is often assumed which demands that the nails interact with the soil and each other to create a composite structure. Although anchorage spacings for retaining wall tie backs may be of a similar order to nails, the overall stability analyses for anchorages are much more conservative at the present time. To avoid potential conflict in the future more data are required on the load transfer distributions and interactions for both anchorages and nails installed in cohesionless, cohesive and cohesive-granular soils.

In these investigations emphasis should be placed on large and full scale observations, but mathematical modelling and centrifugal testing could also be exploited for comparative studies, with particular reference to the failure mechanisms mobilised by different anchorage and nailing systems.

Resistance to Withdrawal of Fixed Anchor

In general, more full scale pull-out tests to failure in highly weathered rocks and cohesive soils liable to creep are required, to check the validity of current empirical design rules, and to extend our knowledge of anchoring in these poorer quality-materials.

With regard to the resistance to withdrawal of fixed anchors at depth (local shear failure), improved design data and valuable case histories have been published by Barley (1988) for weak rocks. However, highly weathered mudstones, shales and marls continue to be rather unreliable founding materials for straight shafted type A anchorages (see Figure 5), unless very low skin frictions are used in design. There is a need to carry out more full scale proving tests to failure where load versus fixed anchor displacement is monitored (preferably with load distribution along the fixed anchor) and where the geotechnical properties are detailed in order to ascertain which geotechnical parameters dictate anchorage performance. In this way empirical design data may be won relating for example skin friction at the rock/grout interface directly to site investigation data.

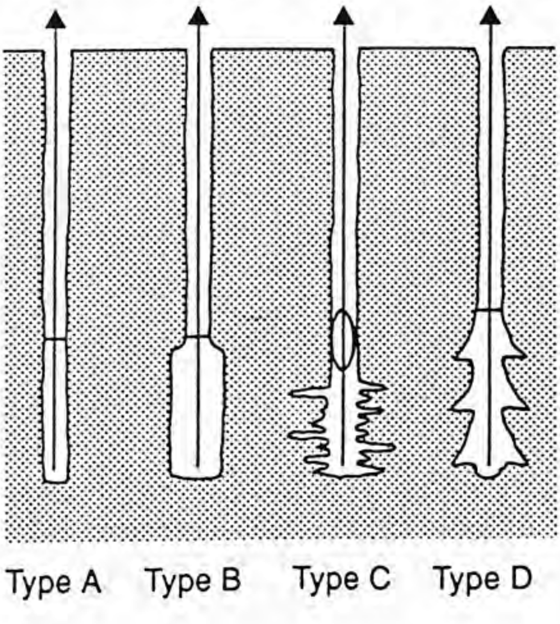


Figure 5. Main types of cement grout injection anchorage (after BS.8081, 1989)

For pressure grouted soil anchorages of types B and C, two distinct design approaches have evolved, namely empirical equations and skin friction envelopes, respectively. Since the main distinction between the two anchorage types relates to magnitude of grout injection pressure, more guidance is required on injection pressure limits that determine if the ground is permeated, compacted or hydrofractured, together with the influence of grout pressure on skin friction and fixed anchor diameter for a variety of soil conditions.

The subject of load transfer in the fixed anchor zone, with particular reference to the major parameters that influence stress distribution, warrants further study. Under failure conditions, the results for standard bonded tendons could indicate an upper limit to fixed anchor length. This seldom exceeds 10 m in current practice. Under service conditions, a knowledge of the stresses imposed on the ground would assist calculation of the magnitude and rate of consolidation, where appropriate, around the fixed anchor and improve our predictive capability concerning loss of prestress and creep displacement with time. Stress/strain contours or pressure bulbs for fixed anchors would be useful in practice. The relative importance of the tendon type, e.g. bar or strand, should also be noted, bearing in mind the greater stiffness of bars that will magnify prestress loss in any comparative study.

To distribute load more uniformly in weak ground, distributed stress transfer fixed anchors have been developed over the past decade, whereby the tendon load is transferred to the adjacent grout through discrete lengths uniformly distributed at intervals along the encapsulation or fixed anchor length. To ensure uniform stresses within the tendon mono-strand stressing is essential. If detailed monitoring of these anchorages confirms enhanced performances in weak ground compared with standard bonded systems no upper limit to fixed anchor length will apply. Some national codes may need amendment to accommodate this practical innovation.

At the grout/tendon interface of fixed anchors, debonding requires study. This is particularly important in anchorages where the service load exceeds 2000 kN and occurs as the ductile tendon transfers stress to the brittle cement grout. The influence of tendon density, centralizers and spacers on load transfer and microcracking should be studied. Similar tests are required for resin grouts used in tendon bond length encapsulations, and the more recently developed resin coated strands (Cousins, Johnston and Zia, 1990).

As a practical improvement centralization should be provided on all tendons to ensure that the tendon is centred in the grout column, with a minimum grout cover of 5 to 10 mm. The possible exception is coarse sands and gravels where cement grout can permeate the soil beyond the borehole. In current practice, tendon centralization receives insufficient attention and grout/tendon bond failures due to ground contamination have been recorded at the acceptance testing stage, particularly in clay and chalk.

Corrosion Performance

Following a worldwide review by the Fédération Internationale de la Précontrainte (FIP), 35 case histories of failure by tendon corrosion were published in 1986, related to permanent anchorages (protected and unprotected), and temporary anchorages with no designed protection other than cement grout cover for the fixed length and on occasion a decoupling sheath over the free length. The findings are important and bear repeating.

Analysis of the results shows that the corrosion is invariably localised and appears to be independent of tendon type in that 9 incidents involved bar, 19 involved wire and 8 involved strand, the period of service before failure ranging from a few weeks to many years for each tendon type. Short term failures (after a few weeks) were due to stress corrosion cracking or hydrogen embrittlement.

These observations invalidate the traditional or intuitive view of some designers that an increase in steel tendon diameter will secure the design service life of post-tensioned anchorages.

In terms of duration of service, 9 failures occurred within six months, 10 in the period of six months to two years, and the remaining 18 beyond two years and up to thirty-one years.

The fact that 19 failures occurred within two years of installation confirms that where the environment is aggressive, temporary anchorages should be given appropriate protection. However, there is no evidence to suggest that the recommended limit of two years for the service period of temporary anchorages should be reduced or extended.

With regard to failure location, 19 incidents occurred at, or within 1 m of the anchor head, 21 incidents in the free length and 2 incidents in the fixed length. Both fixed anchor problems were caused by inadequate grouting of the tendon bond length which exposed the tendon to an aggressive environment.

Failures in the free length were recorded under a variety of individual and combined circumstances such as

(a) tendon overstressing caused by ground movement leading to tendon cracking, sometimes augmented by pitting corrosion or corrosion fatigue,

(b) inadequate or no cement grout cover in the presence of chlorides, e.g. industrial waste fills or organic materials,

(c) breakdown of bitumen cover due to lack of durability,

(d) inappropriate choice of protective material, e.g. chemical grout containing nitrate ions or hygroscopic mastic, and

(e) use of tendon stored on site for a long period in an unprotected state.

In regard to (e), steel tendons showing signs of pitting or transverse defects must not be used for temporary or permanent ground anchorages under any circumstances. On the other hand a film of rust on the tendon is not considered harmful and may improve bond. In practice, a light film of rust also provides reassurance that no lubricant materials, such as grease or soap, are present on the surface of the tendons. For practical guidance on tendon supplied or stored on site, an unacceptable film of rust cannot be removed by wiping with a cloth.

Failures at, or adjacent to the anchor head were due to causes ranging from absence of protection (even for only a few weeks in aggressive environments) to inadequate cover due to incomplete filling initially or slumping of the protective filler during service.

From all the case histories reviewed, it is apparent that corrosion incidents are somewhat random in terms of cause, with the possible exception of choice of steel. Quenched and tempered plain carbon steels and high strength alloy steels are more susceptible to hydrogen embrittlement than other varieties. Accordingly, those named steels should be used with extreme caution where environmental conditions are aggressive.

To provide better guidelines on the maximum safe period of service for unprotected temporary anchorages more fundamental research in the laboratory and field monitoring are needed in relation to steel tendon corrosion. The overall objective should be to create a predictive capacity concerning rates of localised corrosion and limiting acceptable degrees of corrosion for stressed steel tendon, given properly classified aggressivities for the ground.

Corrosion studies in the field should involve the monitoring of prestress with time of full scale anchorages installed in specific aggressive environments. Bar, wire and strand tendons should be investigated where excavation of individual anchorages takes place after regular intervals, e.g. 5 years, or when significant prestress losses are recorded. To facilitate such studies alternative means of monitoring tendon corrosion should also be investigated, e.g. by electrical resistivity, ultrasonic or acoustic monitoring techniques and electrochemical potential measurement.

As for steel tendon corrosion, fundamental research and field monitoring are needed in relation to the durability of cement based grouts in known aggressive ground and ground water conditions. On the multi thousand anchorage contract for the new ring road to Kuwait City, the fixed anchors were installed with at least one underream to provide mechanical interlock and thereby avoid total reliance on skin friction, because of concern over the durability of sulphate resisting cement, when subjected to ground water containing over 5000 ppm each of sulphates and chlorides.

Effort should also be directed towards establishing minimum crack widths in cementitious grouts for no corrosion under aggressive ground water conditions, both static and flowing, in order to check the validity of the 0.1 mm acceptance limit assumed in some national standards. For steel ribbed bar tendons more research is required to confirm that ribs can limit crack widths to 0.1 mm in poor ground with low lateral restraint.

Aggressivity of the Ground

While the mechanisms of corrosion are well understood (ASTM, 1979; FIP, 1986), the aggressivity of the ground and general environment is seldom quantified at the site investigation stage. In the absence of quantified aggressivity data it is unlikely that case histories involving tendon corrosion will provide reliable information for the prediction of corrosion rates in service.

There is no single parameter which can be used to predict the risk of corrosion to an embedded anchorage, bearing in mind that corrosion can be chemical, electrochemical and/or microbiological in nature. The risk of these types of corrosion is currently assessed by resistivity, pH and redox (oxidation reduction) potential.

Corrosion specialists today use a global technique which assigns a value to each parameter measured and then a summation of these values determines the overall aggressivity (see Table 1). Preliminary tests, as part of the site investigation, should include all items listed under Section 1 of Table 1. If the results of these tests are marginal, e.g. total value of -1 to -4 say, then the tests under Section 2 should be undertaken to provide a global assessment of aggressivity (see Table 2). Given a reliable data base in the future it should be possible to relate class of protection directly to a global value of aggressivity, but such a value will have to be sought routinely from site investigations if practice is to be improved.

Item	Measured Characteristic	Value
Section 1		
Soil classification	(i) $\not>$10% passing 63 micron sieve; plasticity index <2 for material passing 425 micron sieve	+2
	(ii) $\not>$75% and $\not>$10% passing 63 and 2 micron sieves, resp.; plasticity index <6 for material passing 425 micron sieve	0
	(iii) all material passing 425 micron sieve; plasticity index <15	-2
	(iv) all material passing 425 micron sieve; plasticity index $\geq$15	-4
	(v) material with organic content $\geq$0.2% by weight	-4
Groundwater	(i) anchorage in well drained area	+1
	(ii) anchorage in poorly drained area	-1
Resistivity (ohm-cm)	$R \geq 10{,}000$	0
	$3{,}000 < R \leq 10{,}000$	-1
	$1{,}000 < R \leq 3{,}000$	-2
	$100 < R \leq 1{,}000$	-3
	$R \leq 100$	-4
Moisture content (by weight)	$m \leq 20\%$	0
	$m > 20\%$	-1
pH	$pH \geq 6$	0
	$pH < 6$	-2
Soluble sulphate (ppm)	$SO_4 \leq 200$	0
	$200 < SO_4 \leq 500$	-1
	$500 < SO_4 \leq 1000$	-2
	$1000 \leq SO_4$	-3
Cinder, coke or made ground	None	0
	Exist	-4
Section 2		
Redox potential (mV)	$RP \geq 400$	+2
	$400 > RP \geq 200$	0
	$200 > RP \geq 0$	-2
	$0 > RP$	-4
Sulphide	None	0
	Trace	-2
	Present	-3
	High	-4
Carbonate	High	+2
	Present	+1
	Trace	0
Chloride ion (ppm)	$C\ell \leq 50$	0
	$50 < C\ell \leq 250$	-1
	$250 < C\ell \leq 500$	-2
	$500 < C\ell$	-4

Table 1. Elements of global assessment of soil aggressivity (after Eyre & Lewis, 1987)

Global Aggressivity Value	Qualitative Classification
0 or higher	Unlikely to be aggressive
-1 to -4	Mildly aggressive
-5 to -10	Aggressive
-11 or less	Highly aggressive

Table 2. Global aggressiviity of soil (after Eyre & Lewis, 1987).

Corrosion Protection

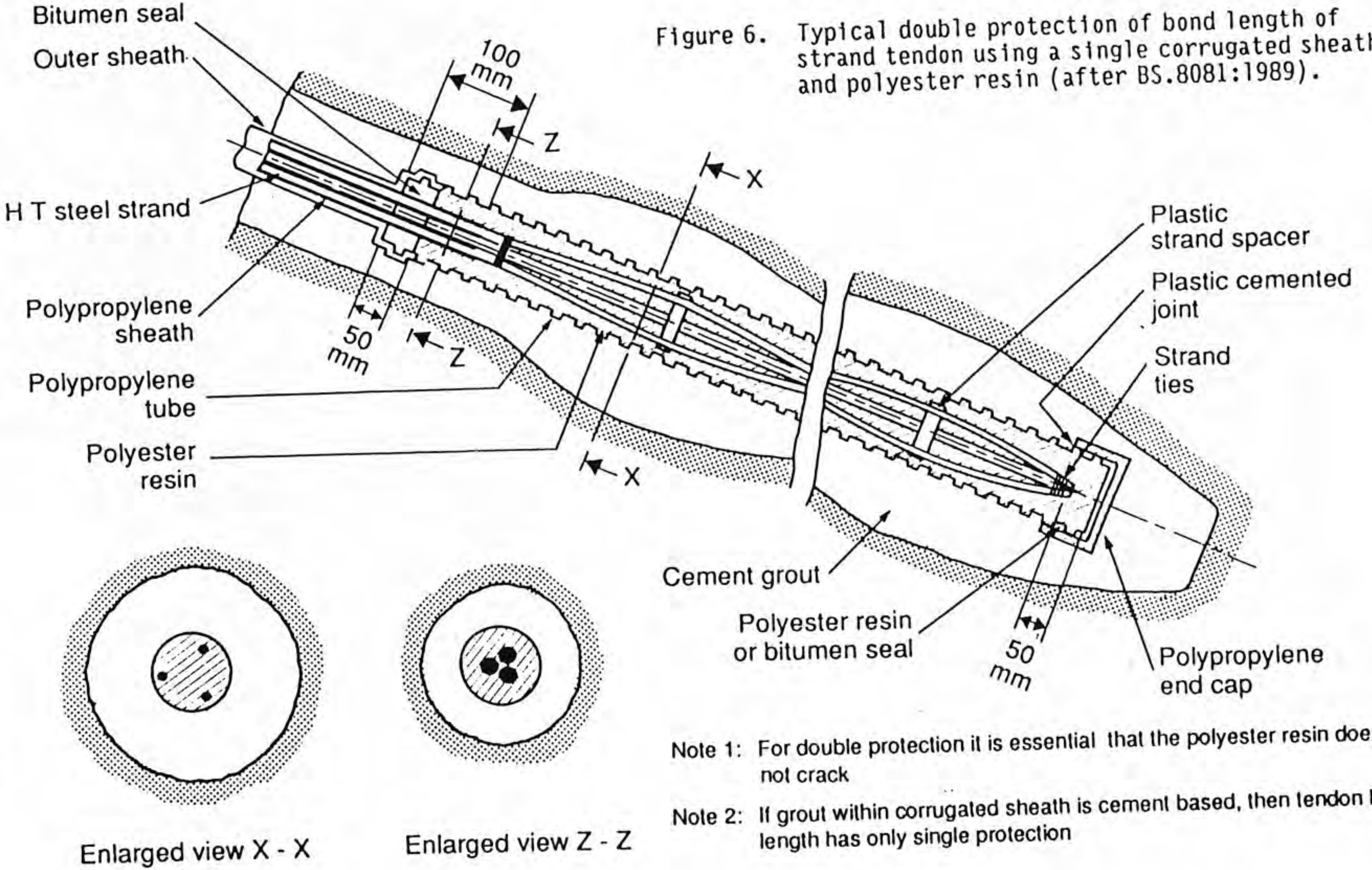
Since at the present time there is no certain way of predicting localised corrosion rates, where aggressivity is recognised, albeit qualitatively, some degree of protection to the steel tendon should be specified by the designer. A typical protection for permanent anchorages comprises a galvanised steel hat over the anchor head, a greased and plastics sheathed free tendon length and a tendon bond length encased in a corrugated plastics duct (see Figure 6).

Provision of adequate tendon protection is not routine practice at the present time, and in at least one national standard simple cement grout cover is still considered acceptable for the protection of permanent fixed anchors.

In view of the number of ground anchorages currently being installed around the world, where cement grout cover is considered to provide adequate protection against corrosion of the steel tendon, it is important to emphasise that when a fluid grout is injected remotely into the ground the quality and integrity of the cured grout as a low permeability barrier cannot be assured. Furthermore, when smooth bar, wire or strand tendons in cement fixed anchor grouts are stressed, cracks tend to occur at about 50 to 100 mm apart and of widths of 1 to 2 mm (Meyer, 1977 and Graber, 1981). In such circumstances, the protective alkaline environment of the cement grout (pH = 11-13) can be depassivated quickly in the presence of aggressive anions, notably chloride (FIP, 1986). As a consequence, grout injected in situ to bond the tendon or its encapsulation to the ground should not be considered as part of the designed protective system in aggressive ground.

Non-hardening fluid materials such as greases also have limitations as corrosion protection media, e.g. liable to leakage or displacement, which means that non-hardening materials must themselves be protected or contained by a moisture proof, robust form of protective sheathing. As a consequence, a layer of grease should not be considered as one of the physical barriers required in the decoupled free length of a double corrosion protection system. On the other hand, grease is acceptable as a protective barrier in a restressable anchor head, since the grease can be replaced or replenished.

Figure 6. Typical double protection of bond length of strand tendon using a single corrugated sheath and polyester resin (after BS.8081:1989).



Note 1: For double protection it is essential that the polyester resin does not crack

Note 2: If grout within corrugated sheath is cement based, then tendon bond length has only single protection

In spite of the above limitations, greases fulfil an essential role in corrosion protection systems, in that they act as a filler to exclude the atmosphere from the surface of a steel tendon, create the correct electrochemical environment and reduce friction in the free length.

For corrosion resistance, the anchorage should be protected overall as partial protection of the tendon may only induce more severe corrosion of the unprotected part. Thus, the least protected zone of a ground anchorage defines the class of protection provided. Junctions between the fixed length, free length and anchor head are particularly vulnerable, as are joints and couplers.

Choice of class of protection (see Table 3) depends on such factors as consequence of failure, aggressivity of the environment and cost of protection. By definition single protection implies that one physical barrier against corrosion is provided for the tendon prior to installation. Double protection implies the supply of two barriers where the purpose of the outer second barrier is to protect the inner barrier against the possibility of damage during final tendon handling and placement.

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

Table 3. Proposed classes of protection for ground anchorages. (after FIP, 1986)

It is considered that anchorage designers should explain the design philosophy behind each proposed protection system, highlighting, for example, the specific roles of materials such as greases, plastics and cementitious grouts. Detailed guidelines on the principles of protection are provided by FIP (1986), and if the conservative assumptions and associated tendon protection systems were communicated more widely in future, confidence in the use of permanent anchorages would be enhanced.

To improve standards further corrosion protection systems should be assembled according to an agreed method statement including checks on each system to ensure the quality and integrity of prefabricated components, adequate overlap of protective barriers at the key interfaces, e.g. anchor head/free length and free length/tendon bond length, and appropriate grout properties, e.g. adequate strength and low bleed. It may also be prudent on occasion to cut up a completed protection system into sections to permit assessment of the quality and integrity of the work.

In future, non-destructive quality controls should be exploited for protected tendons installed in the ground. Electrical resistance measurements offer a simple and convenient method of checking the insulation of the sheath between the steel tendon and the surrounding ground, once the unstressed tendon has been grouted. Based on the recent experience of Swiss ground anchorage companies, a minimum resistance of 0.1 Mohm should be obtained when a 500 V direct current is applied between the steel tendon and the ground (earth). Once the tendon has been stressed but prior to encasing the anchor head, the insulation of the anchor head may be checked by applying a 40 V alternating current between the anchor head and earth. In such circumstances experience to date indicates that a minimum resistance of 100 Mohm should be obtained. More experience in the use of electrical resistance techniques is required before generally applicable acceptance criteria and tolerances can be proposed, but these systems could have considerable potential.

Looking further ahead, non-metallic fibres with appropriate strength and creep properties may offer an alternative to steel tendons, subject to investigation of the effective life of fibres in stressed conditions, when exposed to potentially aggressive environments that may differ from those aggressive to steel. In other words, although non-metallic fibres may be resistant to highly acidic conditions, these same materials may deteriorate in a highly alkaline environment such as that provided by a cement grout. Adoption of non-metallic fibre tendons is only likely if they provide a cheaper alternative to protected steel tendons, including considerations related to different anchor head and stressing details.

Acceptance Testing

In ground anchorage practice there still exists a wide variety of testing procedures and criteria for the acceptance of individual anchorages which are to be incorporated into temporary or permanent works.

In those countries where specific procedures and durations of testing are specified for different major ground types, arguments can arise over the most appropriate geotechnical class for variable ground. For example, if the ground is cohesive and judged to be liable to creep the specified test duration may increase from 2 to 24 hours, with significant cost implications. As a consequence, it is considered that acceptance testing procedures should be independent of ground type, and comprise standard time intervals initially of short duration, where the overall duration of the test is only extended when the anchorage fails to demonstrate a stabilising trend within specified limits. In other words, the observational method is exploited to direct decisions.

As ground is a variable material and anchorage construction is sensitive to workmanship, the selection of a proportion of the anchorages for testing, say 10%, is also not considered appropriate or prudent. If all the selected anchorages pass there is no guarantee that the remaining 90% will follow a similar behaviour, particularly in alluvial deposits, fissured clays and highly weathered rocks. Furthermore, if one or two do fail, unless the reasons can be identified and accepted as specific to the failed installations, there is a strong obligation to confirm the 'health' of all the remaining anchorages, in order to provide the necessary reassurance that they are all fit for their intended purpose.

Looking to the future, it is the writer's view that on-site acceptance tests should be carried out on all anchorages to demonstrate (i) the short term ability of the anchorage to support a proof load that is greater than the design working load, and (ii) the efficiency of load transfer to the fixed anchor zone. Thereafter, the short term service behaviour should be monitored to ensure that prestress or creep fluctuations follow a stabilising trend within tolerable limits.

The principle of proof loading is now widely accepted within a range of 1.25 to 1.5 times the design working load. This practice is encouraging but with the current trend towards the use of high capacity permanent anchorages (>5000 kN) in dam strengthening, the standard proof load multipliers should not be used arbitrarily. Our concept of safety needs to be reviewed and consideration should be given to reductions in both the load safety factor for the tendon and associated proof load factor, bearing in mind that significant overloads can still be applied during acceptance testing (see Table 4). Adoption of a load safety factor of 1.5 for high capacity tendon design under appropriate conditions could provide substantial savings.

Working Load (kN)	Load Safety Factor	Ultimate Load (kN)	Proof Load Factor	Proof Load (kN)	Overload Margin (kN)	Working Stress (%)*
750	2.0	1500	1.5	1125	375	50.0
7500	2.0	15000	1.5	11250	3750	50.0
7500	1.5	11250	1.2	9000	1500	66.7

Table 4. Influence of load safety factor on proof load factor and overload margin.

* % of characteristic strength (f_{pu}) in Europe or % of guaranteed ultimate tensile strength (GUTS) in North America.

It is encouraging to observe that cyclic loading is now becoming routine practice bearing in mind that non-recoverable movements, such as "bedding-in" and wedge "pull-in" of the anchor head, are encountered during the initial loading phase. These movements are not repeated in subsequent cycles and so the reproducible behaviour of the anchorages can be both confirmed and measured.

Given increased confidence and the improved reliability of ground anchorage technology, the number of cyclic load increments and the minimum periods of observation have gradually been reduced over the years. These reductions have saved time and money, and Table 5 provides an example of international recommendations to be published shortly (FIP, 1992).

Temporary anchorages load increment (% T_W)		Permanent anchorages load increment (% T_W)		
1st load cycle* %	2nd load cycle %	1st load cycle* %	2nd load cycle %	Minimum period of observation min
10	10	10	10	1
50	50	50	50	1
100	100	100	100	1
125	125	150	150	15
100	100	100	100	1
50	50	50	50	1
10	10	10	10	1

*For this load cycle, which often includes extraneous non-recoverable movements such as wedge 'pull-in', bearing plate settlement and initial fixed anchor displacement, there is no pause other than that necessary for the recording of displacement data.

Table 5. Recommended load increments and minimum periods of observation for on-site acceptance tests (after FIP, 1992).

At each stage of loading the displacement should be recorded at the beginning and end of each period, and for proof loads the minimum period of one minute is extended to at least 15 minutes with an intermediate displacement reading at five minutes. With these procedures any tendency to creep can be monitored.

With reference to uplift capacity or overall stability, it is important to check that the post-tensioned load is properly transferred through the free decoupled length of the tendon into the fixed anchor zone, otherwise the design assumptions may be invalidated.

To establish the actual seat of load transfer within the anchorage the apparent free length of the tendon should be calculated from the load-elastic displacement curve over the proof loading range using the manufacturer's value of elastic modulus and allowing for such effects as bedding of the anchor head and, in exceptional circumstances, temperature. The analysis should be based on the results obtained during the second cycle to avoid extraneous non-recoverable movements.

For standard bonded anchorages the apparent free tendon length should be not less than 90% of the free length intended in the design, nor more than the intended free length plus 50% of tendon bond length.

This load transfer acceptance criterion is only applied in a few countries and yet the information is vital to confirm that (i) the anchorage load is transferred into stable ground beyond any potential slip plane and (ii) debonding is not excessive.

With reference to distributed stress transfer fixed anchors in weak ground, the above load transfer criterion still applies, but the analysis of apparent free length must be applied to each individual strand or group of strands of equal length. To obtain the appropriate strand load-displacement data it is necessary to use multijacking with prestretch or monojacking. Analysis of an average apparent free tendon length is not appropriate since this parameter masks the true load transfer behaviour of individual strands.

To improve the accuracy of estimating the free tendon length in practice, a comparison of load-extension graphs and elastic modulus values is recommended between the standard 610 mm test length often used by tendon manufacturers and the longer lengths, i.e. 10 m, 20 m and 30 m, that are applicable in ground anchorage practice. Reductions in elastic modulus of up to 9.2% have been observed in long strand tendons (BS.8081, 1989). Phase one of the test work should concentrate on single-unit tendons, and phase two should accommodate multi-unit tendons, where load distribution between tendon units is a further variable.

To monitor the short term service behaviour of a works anchorage, on completion of the second cycle (see Table 5) the anchorage should be reloaded in one operation to 110% design working load say, and locked-off, after which the load is re-read to establish the initial residual load. This moment represents zero time for monitoring load or displacement-time behaviour during service.

Using accurate load cell and logging equipment, the residual load may be monitored at 5, 15 and 50 minutes. If the rate of load loss reduces to 1% or less per time interval for these specific observation periods after allowing for temperature (where necessary), structural movements and relaxation of the tendon, the anchorage may be deemed satisfactory in relation to this serviceability criterion. If the rate of loss exceeds 1%, further readings should be taken at observation periods up to 10 days (see Table 6).

As an alternative to load monitoring, displacement-time data at the residual load may be obtained at the same observations periods, in which case the rate of displacement should reduce to 1% Δ_e or less per time interval (see Table 6).

To ensure compatibility of the acceptance criteria 1% Δ_e is the displacement equivalent to the amount of tendon shortening caused by a prestress loss of 1% initial load, i.e.

$$\Delta_e = \frac{\text{initial residual load} \times \text{apparent free tendon length}}{\text{area of tendon} \times \text{elastic modulus of tendon}}$$

Period of observation	Permissible loss of load (% initial residual load)	Permissible displacement (% of elastic extension Δ_e of tendon at initial residual load)
min	%	%
5	1	1
15	2	2
50	3	3
150	4	4
500	5	5
1500 (~ 1 day)	6	6
5000 (~ 3 days)	7	7
15000 (~ 10 days)	8	8

Table 6. Acceptance criteria for service behaviour at residual load (after FIP, 1992).

If on completion of the acceptance test, the cumulative relaxation or creep has exceeded 5% initial residual load or 5% Δ_e, respectively, the anchorage should be restressed and locked-off at 110% design working load. This procedure ensures that a contingency overload is locked into the ground anchorage at the start of its service.

As a general guide, either acceptance criterion for short term service, i.e. rate of prestress loss or rate of displacement may be applied quite independently for the common range of free tendon lengths. For long free tendon lengths (>30 m) it is clear that creep displacement may be more important to limit and therefore more appropriate as an acceptance criterion, while for shorter free tendon lengths (<10 m) loss of prestress becomes the more appropriate criterion.

At the present time few national standards provide correlations to permit either load or creep monitoring to be adopted, but it is encouraging to know that international organisations such as FIP and ISRM are addressing this issue.

Bearing in mind that some engineers may specify a design life of say 100 years, it is important to note that Table 6 provides load loss or displacement limits against log time. The acceptance criterion for load loss is 2% per log cycle, so that the maximum predicted loss after 100 years (~ 7.5 log cycles) would be 15% approximately, leaving a residual load equivalent to 93% design working load. Variations of up to 10% of working load do not generally cause concern in practice.

Where the designer wishes to maintain the load in service above the design working load, any prestress losses accumulated over the first day, equivalent to three log cycles, can be eliminated by restressing up to 110% design working load.

It is considered that the unique nature of the three elements of on-site acceptance testing proposed, namely proof loading, load transfer analysis and serviceability check, should be advanced and explained at every opportunity, in order to provide greater assurance to engineers and clients. Moreover, if compatible on-site acceptance testing procedures could be developed between countries in the future, a wealth of useful short term performance data could be pooled and long term predictions could be attempted for anchorage types A to D in a range of classified ground conditions.

Service Behaviour

There is a dearth of published data on long term monitoring, both for individual anchorages and complete structure/ground/anchorage systems. In spite of an absence of problems one important consequence is that some engineers lack the confidence to accept ground anchorages for permanent works. Another consequence for acceptance testing is that further optimisation of procedures is inhibited. In order to establish the optimum short term acceptance test for satisfactory performance over a service period of 50 years or more, more results should be published on long term service behaviour where short term acceptance criteria have been met.

The advantages of monitoring should be explained to clients. They include (i) the engineer being able to feed back performance observations into future designs and thereby optimise such parameters as overload allowances and load safety factors; and (ii) the client being accurately and confidently informed of how anchorages installed at his expense will perform after installation. Furthermore, the data collection permits all parties to judge at the earliest possible stage whether anchorages being monitored are, in fact, acting satisfactorily. On a broader front, this form of monitoring may permit correlation of anchorage load and structural movement, and thereby lead to a better understanding of anchorage/ground/structure interaction.

Generally speaking, short term monitoring over 3 to 6 months of anchorages installed in cohesionless soils has shown a rapid stabilisation of load after initial post-tensioning (Littlejohn, 1970). Where overall ground movements are mobilised during excavation, such stabilisation of anchorage loads usually occurs shortly after completion of the excavation. No long term prestress losses due to creep have been noted in cohesionless soils and there appears to be little concern in practice over the ability of these anchorages to maintain their load holding capacity in the long term, given adequate corrosion protection. Early examples of permanent anchorages installed in sands and gravels in the UK are included in Table 7, simply to give some historical perspective for this particular country. It would be useful if engineers in other countries could cite early examples.

Location	Number of Anchorages	Working Load (kN)	Type of Soil	Date of Installation
Tilbury, Essex	52	300	gravels	1968
Grosvenor Road, London	44	300 & 360	gravels	1969
Ponders End	18	300 & 400	gravels	1969
Bromley Theatre, Kent	10	630	sands & gravels	1970
Solihull, Birmingham	14	250	gravels	1970

Table 7. Early examples of permanent anchorages installed in cohesionless soils in the UK.

In cohesive soils such as clays that are known to be susceptible to creep, the dearth of monitoring has left some engineers concerned about long term behaviour. Again there are no incidents of adverse service behaviour due to creep known to the author, and most modern national codes now demand a stabilising trend for prestress loss or displacement with time, coupled with an appropriate overload allowance in routine on-site acceptance testing. Early examples of permanent anchorages installed in cohesive soils in the UK are included in Table 8.

Location	Number of Anchorages	Working Load (kN)	Type of Soil	Date of Installation
New Pithay, Bristol	26	500	marl	1964
Kilburn Square, London	18	300	London clay	1968
Coventry	102	450 & 900	marl	1969
Scarborough	23	400	sandy clay	1969
Neasden Underpass, London	580	100 - 500	London clay	1969
Derby Underpass	40	650	marl	1970

Table 8. Early examples of permanent anchorages installed in cohesive soils in the UK.

In the case of Kilburn Square, although the anchorages were not subjected to rigorous serviceability tests, which are now imposed by the British code (BS.8081, 1989) satisfactory behaviour for all 18 anchorages has been confirmed by lift-off tests after 11 years of service, when residual loads ranged from 108% to 93% of the initial residual load of 312 kN.

To offset the concerns related to permanent anchorages installed in clays, more service monitoring is recommended where details of the clay mineralogy and plasticity are included in the site investigation. Given adequate case histories it should be possible to relate service performance and design assumptions to the material and mass properties of clays.

As in the case of soils, there is a dearth of monitored performance data for rocks where the case histories have been properly documented in terms of rock classification, type and location of anchorages, including design loading, and prestress fluctuations or creep displacement with time. Again there is an absence of problems, and Table 9 simply lists some early examples of permanent rock anchorages to illustrate up to 56 years of successful experience. Table 10 indicates the nominal load losses over the first 18 years of service for the 10,000 kN anchorages at Cheurfas Dam.

Location	Number of Anchorages	Working Load (kN)	Type of Rock	Date of Installation
Cheurfas dam, Algeria	37	10000	sandstone	1934
Steenbras dam, South Africa	326	700	sandstone	1953-54
Tansa dam, India	2399	700	basalt	1953-55
Swallow Falls dam, South Africa	70	2000	granite	1956-58
Witbank dam, South Africa	236	2000	felsite	1957-59
Catagunya dam, Tasmania	412	2000	dolerite	1959-61
Forth Road Bridge, Scotland	80	1200	whinstone & coal measures	1960

Table 9. Early examples of permanent anchorages installed in rocks.

Period of Service (years)	Loss (kN)	Loss (%)
3	408	4
6	449	4.4
9	459	4.5
18	561	5.5

Table 10. Record of prestress loss at Cheurfas Dam.

Since there is a shortage of published case histories on successful long term performance or detailed service behaviour, it would be beneficial to collect simple records of the type listed in Tables 7 to 9 for each country. This would illustrate just how long and widely established the permanent anchorage market place is throughout the world. Such cases should also be augmented by more recent records where anchorage behaviour (load fluctuation or displacement) has been monitored. The writer would welcome any service records and through the Fédération Internationale de la Précontrainte an international data base could be published for the ultimate benefit of both anchorage specialists and clients.

Aside from establishing a credible data base, field monitoring of anchorage loads and the movements of the structure/ground/anchorage system should be organized to study overall service behaviour and, in particular, the effect of prestress on deformations. The distribution of load in walings also warrants study, together with the effect of anchorage detensioning (failure simulation) on redistribution of anchorage loads and bending moments in the walings. An excellent example of this type of work is the treatise by Stille on the behaviour of anchored sheet pile wall in clays in Sweden which was published in 1976. These observations and an understanding of the overall behaviour of the structure/ground/anchorage system should be communicated more since the data can influence the conceptual thinking of a designer in terms of risk and dictate the load safety factor for the tendon and/or the required proof load factor.

In regard to dynamic loading, guidance on the influence of cyclic loading on untensioned model plate and model short cylinder fixed anchors in cohesionless soil is available through the work of Hanna et al. (1978) and Maddocks (1978), which generally indicates that once the fixed anchor begins to yield it will do so at an increasing rate until failure. However, many structures, e.g. transmission towers and quays, subject to such loads and restrained by post-tensioned anchorages, have performed satisfactorily.

In regard to seismic effects, a major tied back excavation at the Atlantic-Richfield Plaza in Los Angeles was at a depth of 20 m when struck by the San Fernando earthquake of 9th February 1971 and survived without incident. The 468 kN design working load anchorages were installed in a soft grey siltstone, almost a preconsolidated clay consistency with some limestone layers (Feld and White, 1974).

Anchorages have also been found to be resilient to the effects of close proximity blasting. At Westfield open pit in Scotland, 1600 kN design working load anchorages were located in coal bearing strata comprising the normal sequence of sandstones, siltstones, mudstones and seatearths, with occasional clay mylonite bands having low shear strength properties ($c_r = 0$; $\phi_r = 10^0$-15^0). The monitored anchorages had a free length of 12 metres and a fixed anchor length ranging from 4 to 6 metres. Over 1200 kg of explosives were detonated within 5 metres of the anchor heads and the greatest increase in load recorded was 110 kN (7% of service load) within one second of detonation, a residual increase of 64 kN (4% of service load) being noted after 10 seconds. A peak particle velocity of 40 mm/sec was measured 60 metres from the blast. These results are very encouraging bearing in mind that post-tensioned permanent anchorages can accommodate much higher overloads without distress (Littlejohn, Norton and Turner, 1977).

Looking ahead, more field records of dynamic behaviour are required and monitoring of special full scale anchorages is recommended, to investigate the influence of cyclic loading at different amplitudes, e.g. $\pm10\%$ T_W, $\pm20\%$ T_W and $\pm30\%$ T_W, on the service performance of post-tensioned anchorages, locked-off at 110% T_W. Comparative studies could also be run on untensioned anchorages. This information covering bar, wire and strand tendons would provide further guidance on the sensitivity of anchorages to dynamic loading, including seismic effects.

Final Remarks

Given the specialised nature of ground anchorage work and the wide variety of anchorage types and construction procedures, coupled with the variability of ground, more reliance in future should be placed on performance specifications related to choice of materials and acceptance testing of all anchorages, compared with attempts to supervise and control the construction phase.

Aside from load holding considerations which can be confirmed from precedent practice or proving tests, the choice of materials will be primarily related to aggressivity which in future should be assessed using a global technique.

Routine testing of all anchorages should involve proof loading to provide a margin of safety, load-displacement analysis to confirm that the resistance to withdrawal is mobilised correctly in the fixed anchor zone, and short term monitoring of the service behaviour to ensure a stabilising trend within tolerable limits. In this way reliable performance should be assured in the long term.

Systematic full scale testing remains the finest source of information on the behaviour of anchorages and more research should be directed towards investigations of the performance characteristics of full scale anchorages and structure/ground/anchorage systems, with particular reference to the long term behaviour of permanent works.

These recommendations should in no sense be taken as evidence of areas of outstanding uncertainty and therefore doubt. With millions of anchorages successfully exploited or performing throughout the world, the market place is well established and quality controls in terms of on-site acceptance testing are second to none in the field of geotechnical processes. The purpose of this paper is simply to highlight those areas which are worthy of further investigation and development in order to maintain ground anchorage technology at the forefront in the field of ground improvement.

Acknowledgment

Extracts from BS.8081:1989 are reproduced with the permission of BSI. Complete copies can be obtained through national standards bodies.

References

American Society for Testing Materials (1979). "Underground Corrosion", ASTM Symp. on Corrosion of Metals, Williamsburg, Virginia, USA.

Anderson, W.F., Hanna, T.H. and Abdel-Malek, M.N. (1983). "Overall Stability of Anchored Retaining Walls", Proc. ASCE, 109(11), 1416-1433, (12), 1817-1818.

Barley, A.D. (1988). "Ten Thousand Anchorages in Rock", Ground Engineering, 21(6), 20-21, 23, 25-29, (7), 24-25, 27-35, (8), 35-37 and 39.

British Standards Institution (1989). Ground Anchorages, BS.8081, British Standards Institution, 2 Park Street, London, 176 pp.
Bruce, D.A. (1976). "The Design and Performance of Prestressed Rock Anchors with Particular Reference to Load Transfer Mechanisms", Ph.D. Thesis, Dept. of Engng., University of Aberdeen, Scotland.
Cheney, R.S. (1984). "Permanent Ground Anchors", US Dept. of Trans. Federal Highway Admin. Report FHWA-DP-68-IR, 132 pp., Nov.
Cousins, T.E., Johnston, D.W. and Zia, P. (1990). "Transfer Length of Epoxy-Coated Prestressing Strand", ACI Materials Journal, 192-203, May-June.
Fédération Internationale de la Précontrainte (1986). "Corrosion and Corrosion Protection of Prestressed Ground Anchorages", Thomas Telford Ltd., London.
Fédération Internationale de la Précontrainte (1992). "Recommendations for the Design and Construction of Prestressed Ground Anchorages", Thomas Telford Ltd., London, (to be published).
Feld, J. and White, R.E. (1974). "Prestressed Tendons in Foundation Construction", Proc. 7th FIP Congress, Tech. Session on Prestressed Concrete Foundations and Ground Anchors, New York, 25-32.
Graber, F. (1981). "Excavation of a VSL Rock Anchor at Tarbela", VSL Silver Jubilee Symposium, Losinger Ltd., Berne, Switzerland.
Hanna, T.H., Sivapalan, E. and Senturk, A. (1978). "The Behaviour of Dead Anchors Subjected to Repeated and Alternating Loads", Ground Engineering, 11(3), 28-32, 34 and 40.
International Society for Rock Mechanics(1985). "Suggested Method for Rock Anchorage Testing", Int. J. Rock Mech. Min. Sci. & Geomech. Abstr., 22(2), 71-83.
Kranz, F. (1953). "Uber die Verankerung von Spundwanden", Berlin, Veriag von Wilhelm Ernst and Sohn, 1-53.
Littlejohn, G.S. (1970). "Soil Anchors", ICE Conference on Ground Engineering, London, 33-44 and discussion 115-120.
Littlejohn, G.S. and Truman Davies, C. (1974). "Ground Anchors at Devonport Nuclear Complex", Ground Engineering, 7(6), 19-24.
Littlejohn, G.S., Norton, P.J. and Turner, M.J. (1977). "A Study of Rock Slope Reinforcement at Westfield Open Pit and the Effect of Blasting on Prestressed Tendons", Proc. of Conf. on Rock Engineering, University of Newcastle-upon-Tyne, England, 297-310.
Maddocks, D.V. (1978). "The Behaviour of Model Ground Anchors Installed in Sand and Subjected to Pull-Out and Repeated Loading", Ph.D. Thesis, Department of Civil Engineering, University of Bristol, England.
Meyer, A. (1977). Report on Discussion to Session VI by J.M. Mitchell, "A Review of Diaphragm Walls", ICE, London.
Ranke, A. and Ostermayer, H. (1968). "Beitrag zur Stabilitatsuntersuchung mehrfach verankerter Bangrubenumschliessugen", Die Bautechnik, 45, (10), 341-350.
Soletanche Enterprise (1968). La Surrelevation du Barrage des Zardezas sur L'Oued Saf-Saf, Paris (unpublished report).

Options for Tunnelling 1993 edited by H. Burger
(Developments in Geotechnical Engineering, 74)
© 1993 Elsevier Science Publishers B.V. All rights reserved.

Dynamic Response of Rock Bolt Systems at Pen y Clip Tunnel in North Wales

Dr. A.A. Rodger*, Professor G.S. Littlejohn**, Mr. D.C. Holland*, Mr. H. Xu**

*University of Aberdeen, **University of Bradford, U.K.

Synopsis
This paper describes a research programme devised to examine the dynamic response of rock bolt systems to blast loading. The research work has comprised field monitoring of bolt response at two active tunnel construction sites in North Wales. In addition finite element and laboratory model investigations have been conducted. The paper presents the results which have arisen to date from the field investigations. The conservative nature of current design practice is confirmed as is the resilience of resin bonded rock bolt systems to blast loading. Attenuation relationships for the two tunnels are presented. The influence of prestress load on the dynamic response of a rock bolt is discussed as is the nature of the dynamic stress distribution along the length of the rock bolt.

1. INTRODUCTION

In many hard rock tunnels, constructed by drill and blast methods, rock bolts are the primary form of support. Due to a dearth of published information on research into this complex problem, current methods for assessing the safe distance for the installation of permanent bolts are very conservative. Design practice either involves safe distances derived from case histories, or trial blasting combined with some limiting dynamic parameter, normally peak particle velocity. If the rock bolts need to be placed closer to the blast face than the specified safe distance the bolts are deemed to be temporary and are replaced by permanent bolts after blasting. This duplication of bolts is both costly and time consuming and may be unnecessary. Whereas trial blasting provides an indication of the level of vibration transmitted from a blast source to a bolt, there is no current design procedure for assessing the effect of vibration on rock bolt performance.

To enable the development of an improved design procedure a research programme has been conducted by the two universities in two phases over the period 1986-1992. The first phase was conducted in three main parts and involved:

1. A full scale field investigation of the dynamic response of resin bonded rock bolt systems. This work was conducted on an active tunnel construction site at Penmaenbach in North Wales. At this site axial load and acceleration were

measured at the head of resin bonded bolts positioned at distances varying from 20 m down to only 0.7 m from a substantial blast face. The influence of prestress load on bolt performance was also studied as was the difference in bolt response resulting from the use of single speed rather than two speed resin bonding.

2. Laboratory model tests to examine the stress distribution within dynamically loaded rock bolts.

3. Finite element simulation of the response of rock bolts to transient dynamic loading to assist with the interpretation and generalisation of field and laboratory experimental work. This work was a development of previous finite element studies of the static behaviour of rock anchorage systems (Yap & Rodger (1981)).

The rock bolts monitored at Penmaenbach Tunnel were 6 m long and were embedded in a slightly weathered, fine grained, very strong rhyolite with a fracture spacing of 0.5 to 1 m. The results of the field monitoring tests showed that no significant load loss or resin/bolt debonding was registered for any bolts, even those positioned only 0.7 m from the blast face. In the tunnel design 5 m had been specified as the safe distance for permanent rock bolt installation. Consequently many bolts were scheduled to be replaced that were in fact undamaged. Reducing the safe distance to 3 m would have led to a 38% saving in rock bolting costs. The field monitoring trials also provided a corroboration of an important result found from laboratory and finite element studies, namely that prestressing the rock bolts decreases the effect of vibrational loading on the bolts. In addition it was found that single speed resin bonded bolts experienced twice the dynamic loading of the equivalent two speed bolts due to their shorter decoupled length. (Further details of the first phase of the research programme were contained in references 2-4.)

Aside from the findings of immediate practical importance, the first phase of the research provided fundamental information on the character of the blast induced waveforms within a rock mass and on how these dynamic signatures affected the overall rock bolt system used at Penmaenbach. The finite element simulation provided a mechanism for investigating the stress distribution along the fixed anchor length. This distribution was also investigated using the laboratory model. Field verification of the load transfer results could not, however, be obtained with the instrumentation employed at Penmaenbach Tunnel.

The second phase of the research programme, which is the subject of this paper, was conducted over the period 1989-1992 at the construction site of the Pen y Clip Tunnel in North Wales and was devised to examine the validity of the results obtained in the first phase across a wider range of rock conditions. In addition, it was hoped that the nature of the stress distribution along the fixed anchor length resulting from dynamic loading conditions could be investigated in more detail. This paper presents preliminary results arising from this phase of the programme.

2. FIELD TRIALS AT PEN Y CLIP TUNNEL

2.1 Site conditions

Construction of the tunnel is part of a scheme for dualling of a 1.9 km section of the A55 North Wales Coast Road between Penmaenmawr and Llanfairfechan.

The tunnel was designed to provide for a 930 m long, 7.3 m wide, two lane carriageway with a minimum vertical clearance of 5.1 m. The tunnel passes through the Pen y Clip Headland which is a steeply rising microdiorite intrusion. The rock is overlain by sandy scree, coarse scree and quarry related debris. Prior to commencement of work the tunnelling conditions were envisaged as ranging from good to extremely poor (fracture spacing 50 to100 mm) with structural support being required throughout, primarily by shotcrete and rock bolting. In the zones near the portals, where rock cover is shallow or the quality very poor, steel ribs were specified. A secondary concrete lining was specified throughout.

2.2 Research instrumentation

A two part instrumentation system was designed for the field trials at Pen y Clip. The first part of the system was developed to monitor the stress distribution along the length of the rock bolts. This consisted of special rock bolts, 6 m long, with load sensing inserts introduced at various positions along the bolt length. Associated with these instrumented bolts was a signal conditioning and amplification system designed to transmit signals 500 m to a remote instrumentation cabin. At the cabin dynamic signals were monitored using FM magnetic tape recorders, and static response was recorded over a long period using a computer controlled data logging system. The second part of the instrumentation system enabled measurements to be effected of changes in prestress load and the corresponding dynamic movement of the bolt head. Figure 1 shows diagrammatically both parts of the instrumentation system.

2.3 Summary of results of field monitoring

2.3.1 Introduction

The rock bolts involved were two speed resin bolts, chosen because they have been found to be the most resilient to blasting. This form of bolt uses a fast setting resin for a 2 m length of bolt furthest from the rock surface. The remaining 4 m length of the bolt is grouted with a slow setting resin (except for a 700 mm decoupled length at the bolt head). The fast setting resin is used to create the fixed anchor which is tensioned before the slow setting resin bonds the remaining length of bolt to the surrounding rock.

The programme of work involved monitoring 24 instrumented rock bolts, subjected to full face burn cut blasting to assess the influence of distance from the face, prestress load and rock class on the load carrying characteristics of the bolts. Eight bolts with load sensing inserts were installed to assess the stress distribution along the bolt length resulting from static and dynamic loading. Four of these bolts had the inserts installed in the slow setting resin and the other four had them in the fast setting resin zone.

At each instrumented rock bolt position, 30 to 40 response waveforms were recorded for each 6 second blast sequence. Vibrational acceleration and dynamic load at the proximal end of the bolts were recorded to give typically 35 values each of peak particle acceleration (ppa) and peak dynamic load (pdl) for each rock bolt responding to a blast sequence (4500 results in all). Values of peak particle velocity (ppv) were obtained by integration of acceleration signals.

As expected measurement of ppa, ppv and pdl indicated that values attenuate rapidly with distance from the source. A major finding was confirmation of the Penmaenbach result that safe distances to the blast face could be reduced to 1 m without perceptible damage to the bolt - even though the ground at Pen y Clip was much weaker as a rock mass than that of Penmaenbach. This has major implications for bolt installation practice.

2.3.2 Blast characteristics

The characteristics of the blast source are assumed to be affected by the following parameters: charge mass per delay, form of charge confinement (as influenced in particular by surrounding rock mass properties), spatial distribution of blast holes throughout the face and the scatter of detonation times. Figure 2a shows the relationship between ppa and charge mass for one rock bolt responding to a blast sequence. The graph indicates no clear relationship between charge mass per delay and ppa for the rock mass structure at Pen y Clip. The results obtained at Penmaenbach Tunnel (Figure 2b) indicated a greater reliance of ppa on charge mass. It seems possible therefore that the effect of varying charge mass on vibration induced in nearby rock bolts increases with increasing rock mass quality, as a result of improved charge confinement.

2.3.3 Attenuation

By examining how a group of rock bolts responds to the same blast sequence, the effect of blast characteristics is removed. Using this method an attenuation relationship for Pen y Clip is:

$$ppa = 1137\ r^{-1.09}$$

where r is the distance from the blast face.

Back analysis of the Penmaenbach data using the same method gives:

$$ppa = 1200\ r^{-0.87}$$

In all relationships in this paper the units of acceleration are m/s/s, force kN, velocity mm/s and distance is in metres. The higher attenuation of blast energy at Pen y Clip was expected due to the more highly fractured rock mass.

2.3.4 Relationship between dynamic load and acceleration of the bolt head

For each of the 45 blast sequence responses at Pen y Clip a linear relationship was found in all plots of peak dynamic load against peak acceleration. Figure 3a shows a typical example.

This form of relationship was also found at Penmaenbach, an example of which is shown in Figure 3b. The availability of such graphs, along with trial blasting results provide a means for assessing rock anchorage response to dynamic loading.

It was also found that there is a tendency for the gradient of the pdl/ppa line to increase with increasing prestress load for results relating to similar distances from the face. Figure 4 shows the relationship between the pdl/ppa gradient and prestress load for distances less than 4 m from the face for both the Pen y Clip and Penmaenbach Tunnels. The difference in the pdl/ppa gradient between the two tunnels is assumed to be due primarily to the differences in rock mass quality.

2.3.5 Stress distribution within the bolts

From laboratory model and finite element studies it was found that the dynamic stress distribution along the fixed anchor of the rock bolts took the form of an exponential decay from the proximal end of the resin/bolt interface in response to an impulse load applied axially at the head of the bolt. In the tunnel however the loading condition is considerably more complex as the blast creates both surface and body waves. If the bolts installed normal to the tunnel walls are subjected primarily to body waves the loading along the length of the fixed anchor would be expected to be relatively uniform. If, however, a surface wave predominates, the component normal to the tunnel wall would contain a significant proportion of the total energy of the wavefront with the amplitude attenuating rapidly with distance from the tunnel perimeter. Consequently in this situation a result similar to that found in the finite element and laboratory tests may be possible. The results obtained at Pen y Clip are still under analysis. However, early results suggest that body waves were influential in determining the bolt response. A new finite element model is being used to simulate this complex loading on the bolts and good progress has been achieved.

3. CONCLUSIONS

A full scale investigation has been conducted into the dynamic response of rock bolt systems at the construction site of Pen y Clip Tunnel in North Wales. Associated finite element and laboratory model studies have also been conducted. Based on this work, and the earlier research results obtained at Penmaenbach Tunnel, the following conclusions have been reached:

1. For both the very strong rhyolite at Penmaenbach Tunnel and the weaker microdiorite mass at Pen y Clip Tunnel, no significant load loss or resin/bolt debonding was registered for any bolts, even those positioned only 1.0 m from the face. This confirms the conservative nature of current design practice and the resilience of resin grouted rock bolt systems to dynamic loads.
2. Attenuation relationships for peak particle velocity and peak dynamic load have been established for both the Penmaenbach and Pen y Clip Tunnels. It is postulated that the effect of varying charge mass per delay on vibration induced in nearby rock bolts increases with increasing rock mass quality.
3. A linear relationship has been established between peak dynamic load and acceleration of the bolt head for both the Penmaenbach and Pen y Clip Tunnels. The gradient of this relationship depends on the prestress level applied to the bolt. The results from the Penmaenbach Tunnel indicated that prestressing the bolt serves to decrease the effect of vibrational loading on the bolt. This has been corroborated by both finite element and laboratory model tests.

4. Model and finite element studies have indicated that the stress distribution along the length of a bolt subject to a dynamic load applied along the line of action of the bolt takes the form of an exponential decay from the proximal end of the bolt. The dynamic loading in a tunnel resulting from blast loading is more complex due to the influence of both surface and body waves and depends on the orientation of the rock bolts to the blast source.
5. A refined finite element simulation is being developed that models anchorage behaviour in the tunnel in order to act as a basis for a design predictive capacity.

4. REFERENCES

1 L.P. Yap and A.A. Rodger, A study of the behaviour of vertical rock anchors using the finite element method, Int. J. Rock Mech., Min. Sci. & Geomech. Abstr., Vol. 21, No. 2, pp.47-61, 1984.
2 G.S. Littlejohn et al, Monitoring the influence of blasting on the performance of rock bolts at Penmaenbach Tunnel, Proc. 1st Int. Conf. on Foundations & Tunnels, Vol. 2, pp.99-106, 1987, Engineering Technics Press.
3 G.S. Littlejohn et al, Dynamic response of rock bolts, Proc. 2nd Int. Conf. on Foundations & Tunnels, Vol. 2, pp.57-64, 1989, Engineering Technics Press.
4 A.A. Rodger et al, Instrumentation used to monitor the influence of blasting on the performance of rock bolts at Penmaenbach Tunnel, Proc. Int. Conf. on Instrumentation in Geotechnical Eng., pp.267-279, 1988, TTL, London.

5. ACKNOWLEDGEMENTS

The authors wish to acknowledge the considerable assistance received from the Science & Engineering Research Council, the Welsh Office, Travers Morgan & Partners, Howard Humphreys & Partners, John Laing Construction Ltd. and Cementation Mining Ltd.

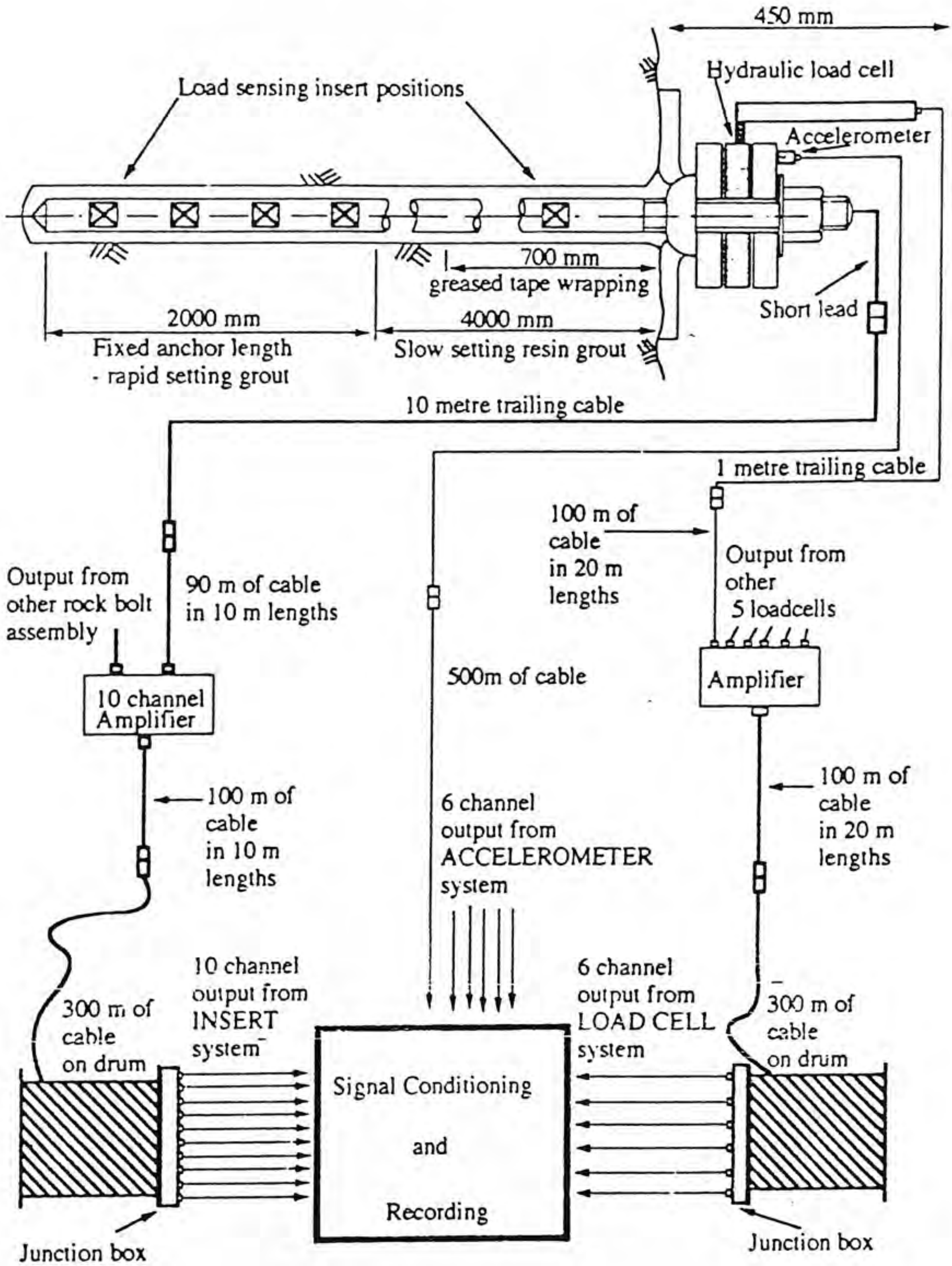


Figure 1 Instrumentation System

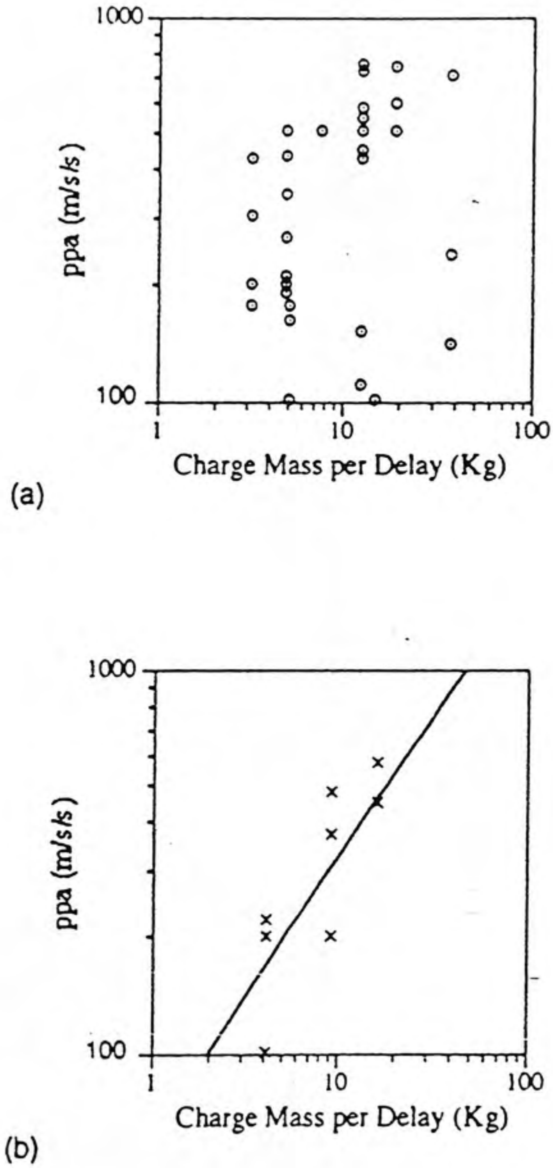
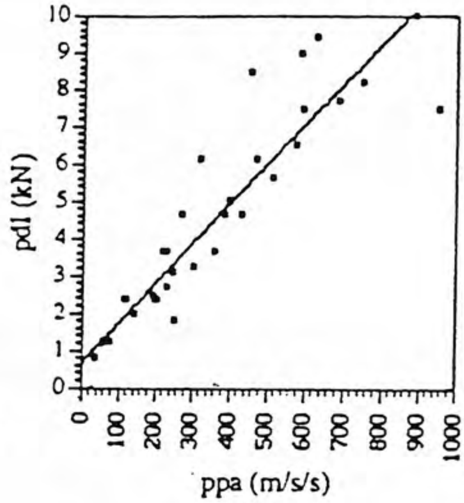
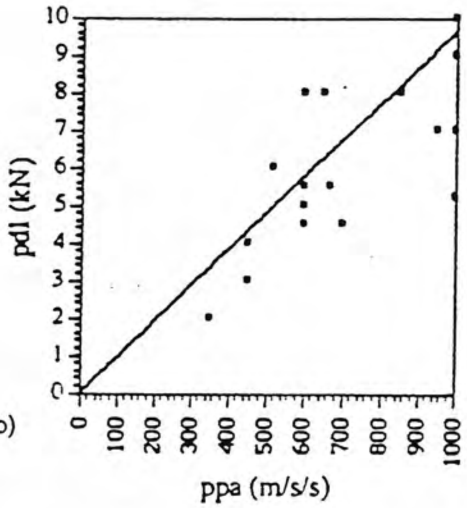


Figure 2 Examples from Pen y clip (a) and Penmaenbach (b) showing relationships between ppa and charge mass per delay for one rock bolt responding to a blast sequence

Figure 3 Examples of pdl/ppa relationships for (a) Pen y clip and (b) Penmaenbach

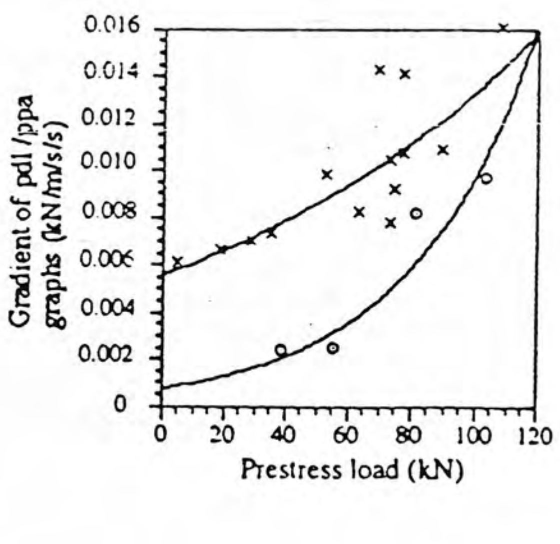


(a)



(b)

Figure 4 Relationship between pdl/ppa gradient and prestress load for distances less than 4m from the face for Pen y clip (×) and Penmaenbach (⊙)



A SIMPLE DEVICE TO CONTROL ROCK BOLT TENSIONING

by

G.S. Littlejohn and J.J. Conway

1. Introduction

When current rock bolt tensioning procedures are employed to provide a predetermined restraint for tunnel rock support, problems are encountered routinely in ensuring that the correct tension is locked into each bolt. To eliminate these problems a minor conversion of the bolt involving the incorporation of a tension control sensor has been investigated. The design principle and conversion technique are outlined and the results of a full scale trial at Pen y Clip tunnel in North Wales are described to demonstrate the practical application of the system for improved control of lock-off loads.

2. Current Tensioning Procedures

Two methods are frequently used for tensioning rock bolts, namely direct axial tensioning using an hydraulic jack, and indirect tensioning by applying torque to the locking nut. In the first method the hydraulic jack is used to apply an axial load to the free threaded end of the bolt (see Figure 1). The load is increased to the prescribed level, which is monitored using a pressure gauge in the jack's hydraulic circuit, after which the bolt is 'locked-off' by tightening the nut against the face plate usually with the aid of a small wrench, and occasionally by hand. In the second method the nut is simply tensioned against the face plate until a prescribed torque is achieved, which is measured using a calibrated torque wrench. There are significant disadvantages associated with both methods.

In direct tensioning the hydraulic jack can be a cumbersome piece of equipment, requiring two persons to handle it. In many tunnelling or mining situations this problem is compounded since the two persons must work in the basket of a jumbo or other lifting equipment and for safety reasons the jack must be secured to the basket. To fit the jack over the bolt shank one person must support the jack while the other fits and tightens a threaded coupler, which can be an awkward and time-consuming operation. Once the bolt has been tensioned, the nut must be tightened against the face plate to lock-in the load when the jack is subsequently released. Again, this can be an awkward operation requiring a wrench to be inserted between the legs of the stressing stool, allowing the nut to be tightened by one-third of a turn before the wrench must be repositioned and the rotation repeated. In principle it should be possible to achieve accurate bolt tensions using this method but recent studies (Xu, 1993) have shown unacceptable and erratic load-loss during lock-off (see Table 1). There is now considerable uncertainty as to whether design loads are actually applied in many production bolting situations. This means that there may be less support immediately available to resist applied loads, and an increase in bolt restraint can only be mobilised as rock convergence develops within the tunnels.

Observations of site practice indicate that serious discrepancies can result from i) inadequate tightening of locking nut by procedure, ii) yielding of face plate by excessive bending, punching or rock deformation, iii) infrequent calibration of jacks, iv) inadequate instruction of bolting crews as to the correct pressure gauge reading for lock-off, and v) absence of a check lift stage.

The indirect tensioning method is simpler, and can readily be carried out by one person. The main disadvantage of this method is the uncertainty associated with the correlation between torque and tension. Under carefully controlled conditions reproducible results may be achieved, however these may not be realised in a production situation. The main source of error is variation in friction between the nut and bolt threads, and between the nut, washer and face plate resulting, for example, from the presence of rust, grease, grout or other contaminants, or mechnical damage to the threads. In addition, the torque wrench can be incorrectly set. In view of these potential problems, direct tensioning is generally specified on British tunnelling contracts in the UK.

BS.8081 Ground Anchorages recommends that a representative sample (1% to 5%) of all rock bolts should be subject to acceptance tests by direct tensioning, except where rock bolts are used as the principal or only means of support in which case a higher proportion (50% to 100%) should be subject to such tests. In many overseas countries torque tensioning is still widely used without any form of direct tensioning check.

3. The Rotabolt System

In the Rotabolt system a tension control sensor is incorporated into the threaded end of standard bolts. The conversion process involves drilling a small 5 mm diameter hole along the central axis of the bolt and inserting a smooth pin with a threaded end to provide a mechanical anchorage at the base of the hole. This gauge pin is made in material which is compatible to the parent bolt, for example a matching coefficient of thermal expansion. At the other end of the pin a head and stainless steel washer protrude just above the threaded end of the bolt (see Figure 2). The washer is attached to an outer control cap which sits on the end of the bolt, and the cap is free to spin in a preset air gap between the washer and the end face of the rock bolt. When the correct tension is applied, the bolt stretches by the set amount, the air gap closes and the cap locks to touch. The air gap is determined by a physical load calibration test as part of the conversion process, and an accuracy of ± 5% can be attained.

The range of tension control can be extended if required by providing two tension settings via a dual indicating cap on a single pin (see Figure 3). If the rock support designer wishes to monitor for overload due to convergence, the outer cap can be set at an overload level whilst the inner cap can be set at the specified design prestress. Alternatively, if loss of prestress during service is the concern, the outer cap can be set at the design prestress with the inner cap set at a lower level, indicating a loss of load where action should be taken. In either case, the operational range of tension can be checked by a simple finger and thumb test.

Since the Rotabolt system provides a direct internal measure of bolt tension, it avoids the tensioning problems outlined earlier and offers a significant improvement in the quality control of both tensioning procedures. As a consequence, the simplicity of torque tensioning can be restored to practice without the attendant disadvantages. Tensioning can be carried out quickly by a single operative resulting in cost and time savings, and because the rockbolts are 'converted' under carefully controlled conditions the problems associated with calibration of field instruments, and the potential errors associated with misreading gauges or incorrectly setting torque wrenches are avoided. Tensioning consists simply of tightening the nut against the face plate until the control cap locks, thereby ensuring a tension within a reasonable tolerance of the specified value. For direct tensioning, the control sensor ensures that the lock-off operation is only completed when the control cap locks and an appropriate and calculable load has been attained, thereby eliminating the need for a check lift stage.

Irrespective of the stressing procedure, it may be observed that the Rotabolt tension control system offers considerable advantages for works involving post-tensioned rock bolts in terms of quality control, consistency of result, time and money.

4. Practical Implementation

Application of the Rotabolt system to production bolting operations requires a minor conversion of each rock bolt to incorporate the tension control sensor. The smallest bolt thread diameter which can be converted is 20 mm whilst there is no upper limit. For both small and large bolt quantity projects, conversions are carried out in the factory. It may be feasible however to set up an on-site conversion if the market develops in that direction.

All Rotabolt system contracts are processed and controlled under a registered BS.5750 quality programme, and as part of the conversion procedure each individual finished product is load tested. For bolt placement in the drill hole a suitable coupler is required for the drilling machine to allow bolts to be spun-in without risk of damage to the control cap. Training of installation crews is required to ensure correct drill hole lengths and that bolts are installed with an appropriate length of protruding thread beyond the face plate i.e. an appropriate stand-off (see Figure 2). For the bolt installation and tensioning phases the extension coupler is externally similar to the standard couplers used on site, but the overall dimensions and internal detail are chosen to prevent damage to the control caps and to ensure a satisfactory nut stand-off position (Figure 4).

5. Field Trial

In order to investigate these practical aspects a field trial was undertaken in the Pen y Clip tunnel in North Wales. The trials were initiated by the University of Bradford and carried out by consultants Travers Morgan and tunnelling contractor Trafalgar House Construction (Tunnelling), working with the supplier Rotabolt. On-site monitoring and assessment of the results were the responsibility of the University researchers.

Twenty rock bolts were fitted with tension control sensors, installed in the tunnel and tested under production conditions. The two-speed resin rock bolts were spun-in by a Tamrock rig as complete assemblies , i.e. bolt, face plate, hemispherical washer and nut, using the protective coupler.

All twenty bolts were installed satisfactorily using routine site procedures. Five of the rock bolts were subject to direct tensioning conventionally using an hydraulic jack, but without the protective coupler. As a result three of the sensors were badly damaged and rendered ineffective. The remaining fifteen bolts were tensioned indirectly using a pneumatic torque wrench and suffered no damage.

Following tensioning, dimensions X and Y (see Figure 5) were measured using a scale and the dual indicators were checked for tightness. The dimension X indicates the nut stand-off and Y checks whether or not the locking nut is located within the operating distance of the sensor, and, if so, the operational length of the gauge pin.

The factory set gauge length L_f was 98.5 mm (including 50%of nut length) and for the low load setting (P) of 80 kN, the factory set air gap (ΔL) was calculated from equation 1.

$$\frac{\Delta L}{L_f} = \frac{P}{AE} \qquad ...1$$

where A is the area of the steel gauge pin
E is the elastic modulus of the steel gauge pin.

Any change in stand-off which creates a new site gauge length (L_S), affects the new low load setting for the pre set air gap, as shown in equation 2.

$$\frac{\Delta L}{L_S} = \frac{\text{new load setting}}{AE} \qquad ...2$$

Using equations 1 and 2,

$$\text{New load setting} = \frac{PL_f}{L_s} = \frac{80x98.5}{L_s}\text{kN}$$

The estimated revised load settings for the stand- off distances measured on site are shown in Table 2.

6. Discussion of Results

The two-speed resin rock bolts used at Pen y Clip were typical of those used in many tunnelling and mining operations. Conventional installation methods can be problematic however in terms of potential damage to the tension control sensors and achieving the correct stand-off. The field tests which made use of the specially designed coupler eliminated the damage problem. None of the bolts tensioned using this coupler or the pneumatic torque wrench suffered damage.

Although use of the incorrect coupler negated the direct tensioning element of the trial, direct tensioning procedures have been employed successfully with the identical Rotabolt system in many other applications where correct bolt tension is critical. These applications give considerable confidence and include crane slewing rings, high pressure flanges, power station valves, subsea riser clamps, coal face shearers, wind turbines and high speed locomotives.

Some problems with variable nut stand-off were observed however, due to a combination of movements during tensioning including bolt displacement, face plate deflection and spherical washer distortion. In spite of these problems 5 of the 14 bolts tensioned with the torque wrench achieved tensions within Rotabolt's normal ± 5% accuracy assurance. The stand-off accuracy can be improved significantly if the following steps are taken.

i) The gauge pin length should be increased to 300 mm to accommodate variations in stand-off. In such circumstances, the readings at Pen y Clip would have been in the range 77.5 to 93.6 kN (see Table 3).

ii) Good quality hardened and tempered 'T' or 'V' grade steel face plates should be used in place of the mild steel plates employed in the trials. The full plate dimensions should be made compatible with the spherical washers used. Both these measures will reduce the degree of plate distortion and punching of the washers, thus reducing the variations in stand-off and improving further the accuracy of the control system.

iii) The bond length resins should be correctly cured prior to tensioning to prevent excessive bolt displacement during tensioning.

Conclusions

The trials carried out at Pen y Clip have demonstrated that tension control sensors can be used successfully in a production bolting situation, with only minor changes to current procedures and equipment. Although the Rotabolt system can be used in conjunction with either direct or indirect tensioning methods, it is with the latter that the main advantages, of simplicity, speed and lower resource requirements, are to be realised.

Implementation of tension sensors would require only minimal training of bolting crews. No modifications are required to the drilling equipment. To install and tension the bolt a special coupler is required. Externally the modified coupler differs little from those with which the crews are already familiar, and tensioning can be achieved using a standard pneumatic or manual wrench. The correct tension is attained simply by tightening the nut until the cap locks.

The cost of converting rock bolts to incorporate the Rotabolt mechanism is estimated to be about £15 to 30 per bolt for single and dual control indicators, respectively, with discounts for large projects. This is not expensive, given the savings in time and personnel offered by torque tensioning.

It is considered that the use of tension sensors offers control of lock-off loads in rock bolting applications, and permits a simple check of all post-tensioned bolts to ensure that they are fit for their intended purpose.

Acknowledgements

This paper is published by kind permission of the Welsh Office. The field trial was carried out in collaboration with consultants Travers Morgan, Howard Humphreys, Brown and Root Civil, contractors John Laing Construction Ltd. and Trafalgar House Construction (Tunnelling) Ltd. and tension control supplier Rotabolt Ltd. The authors wish to thank all parties concerned for their co-operation and support.

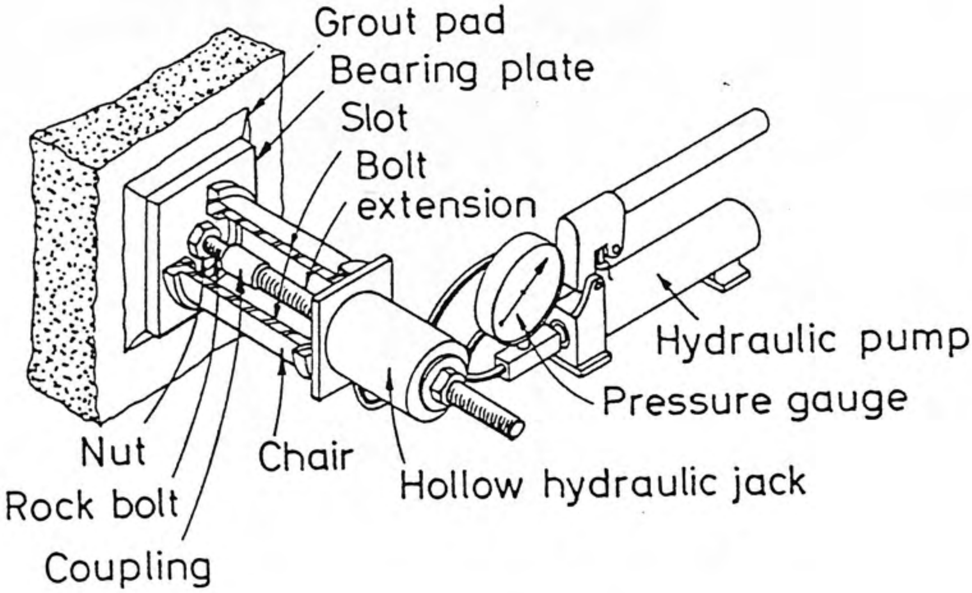


Figure 1. Use of hollow ram jack for direct tensioning of rock bolts

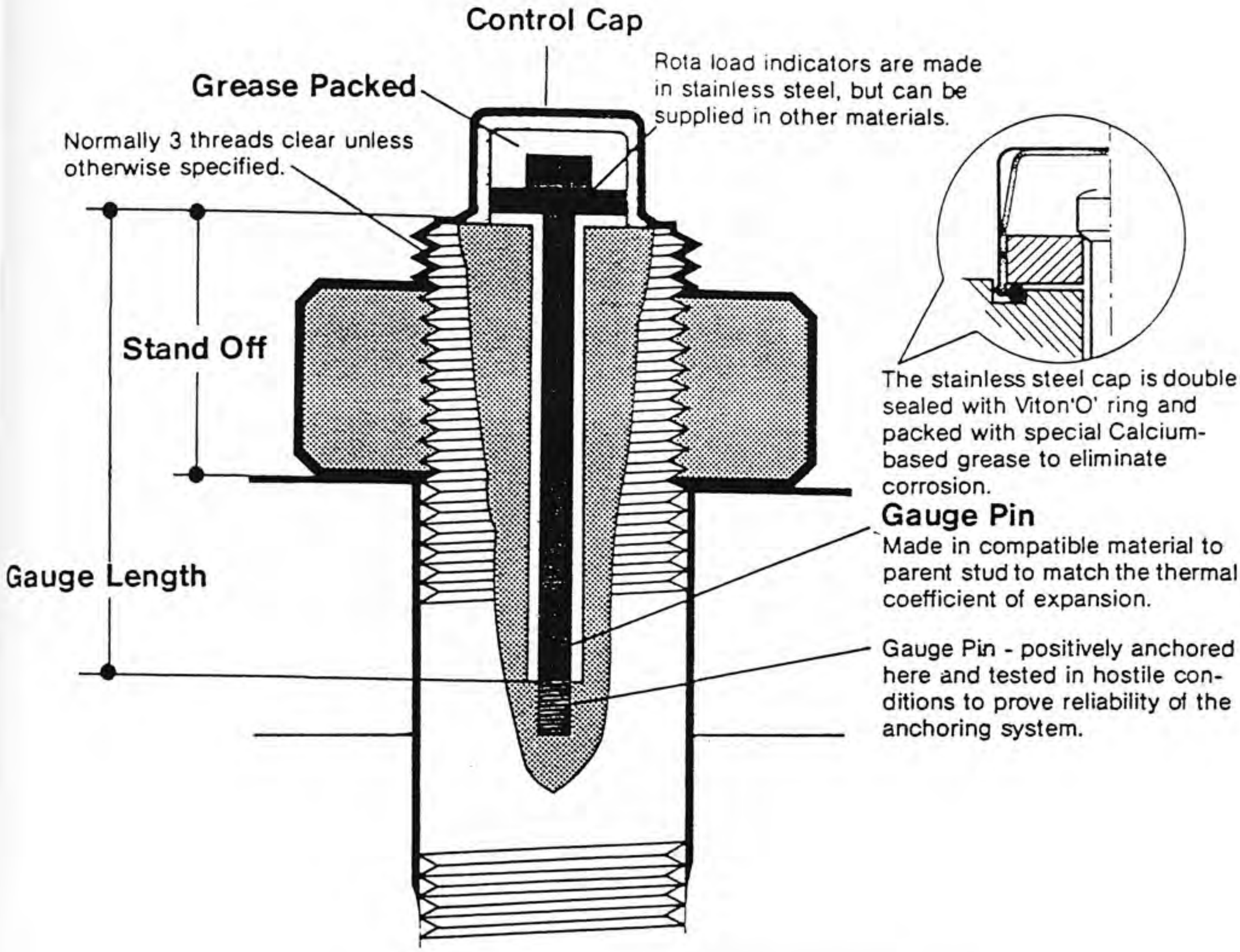


Figure 2. The Rotabolt tension control sensor for a standard bolt.

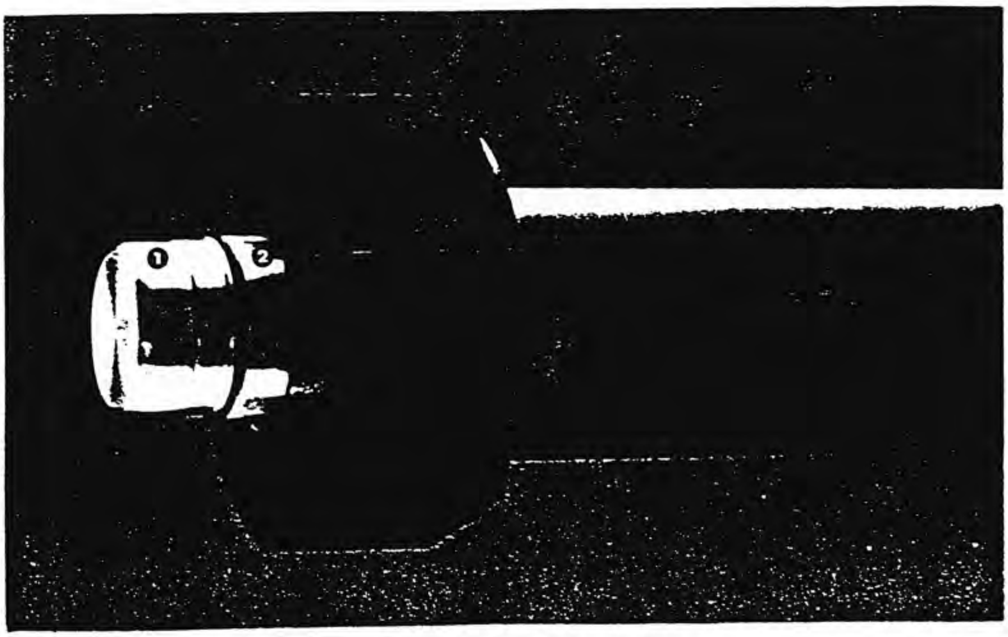


Figure 3. Dual indicating sensor
1 Outer cap - high tension setting. 2 Inner cap - low tension setting

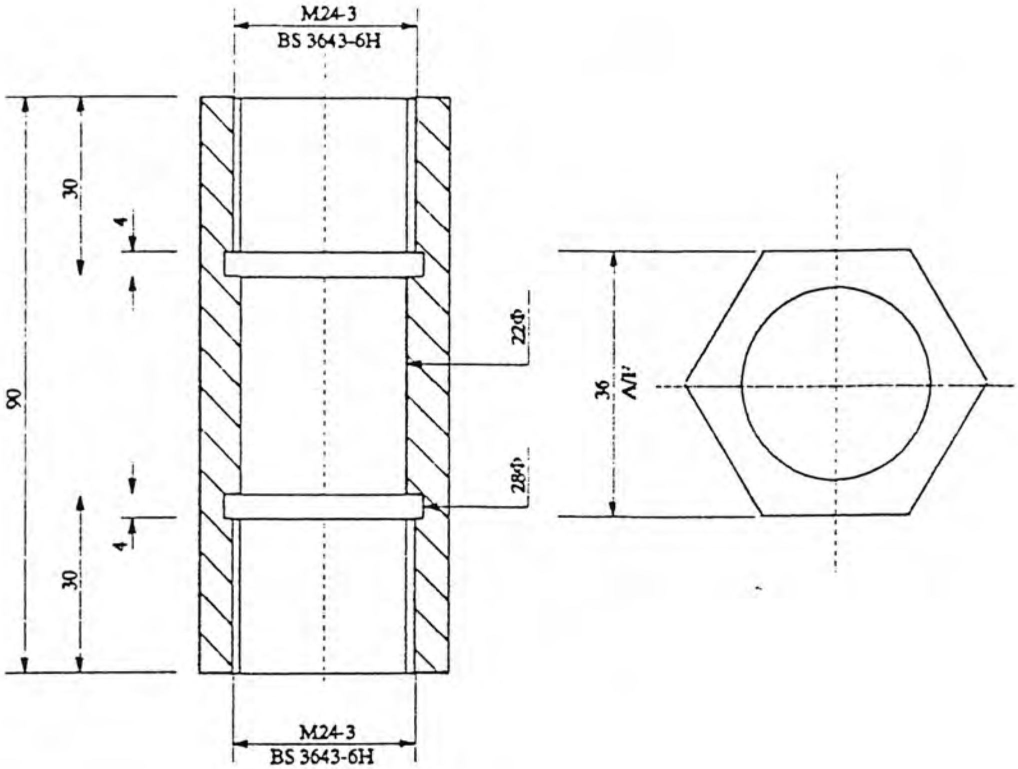


Figure 4. Protection coupler

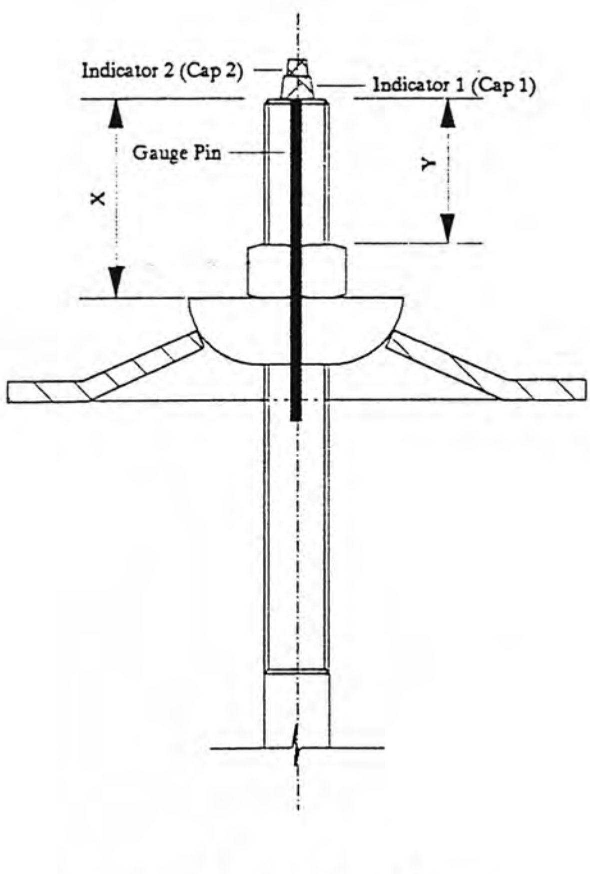


Figure 5. Anchor head detail

Bolt Number	Load (kN)	Bolt Number	Load (kN)
1	19	13	92
2	29	14	46
3	3	15	78
4	65	16	36
5	78	17	70
6	91	18	108
7	5	19	74
8	73	20	90
9	75	21	108
10	74	22	64
11	29	23	13
12	66	24	66

Table 1 Lock-off loads monitored during production rock bolting where specified load was 100 kN.

Bolt Number	X mm	Y mm	Tension Setting kN	Mode of Tensioning	Remarks
1	90	72	-	Direct	Bolt tensioned to 100 kN but gauge pin broke due to jack coupler.
2	77	59	-	Direct	Bolt tensioned to 100 kN. Both caps locked.
3	115	97	-	Direct	Bolt tensioned to 100 kN but gauge pin broke due to jack coupler.
4	-	-	-	Direct	Cap 1 locked. Cap 2 damaged by jack coupler.
5	-	-	-	Direct	Resin did not set.
6	60	42	79	Torque	Both caps locked.
7	65	47	83	Torque	Cap 1 locked. Cap 2 loose.
8	53	35	74	Torque	Both caps locked.
9	58	40	78	Torque	Cap 1 locked. Cap 2 loose.
10	65	47	83	Torque	Both caps locked.
11	52	34	-	Torque/ Direct	Torque tensioning left both caps loose. When jack employed the sensor broke due to jack coupler.
12	75	57	93	Torque	Both caps locked.
13	80	62	99	Torque	Both caps locked.
14	78	60	97	Torque	Both caps loose.
15	73	55	91	Torque	Both caps locked.
16	85	67	106	Torque	Both caps locked.
17	70	52	88	Torque	Cap 1 locked. Cap 2 loose.
18	97	79	126	Torque	Cap 1 locked.
19	127	109	242	Torque	Cap 1 locked. Cap 2 loose. Bolt yield.
20	65	47	83	Torque	Both caps locked.

Table 2 Revised tension settings at recorded X and Y values

Bolt Number	Tension Setting (kN)	Bolt Number	Tension Setting (kN)
6	80	14	86
7	81	15	84
8	78	16	89
9	79	17	83
10	81	18	94
12	85	19	-
13	87	20	81

Table 3 Estimated tension settings assuming a 300 mm long gauge pin at recorded X and Y values