Technical Report Documentation Page

1. Report No. FHWA-SA-99-015	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle GEOTECHNICAL ENGINEERING CIRCULAR NO. 4 Ground Anchors and Anchored Systems		4. Report Date June 1999 6. Performing Organization Code:
7. Author(s) P.J. Sabatini, D.G. Pass, R.C. Bachus		8. Performing Organization Report No.
9. Performing Organization Name and Address GeoSyntec Consultants 1100 Lake Hearn Drive Atlanta, Georgia		10. Work Unit No.(TRAIS) 11. Contract or Grant No. DTFH61-94-C-00099
12. Sponsoring Agency Name and Address Office of Bridge Technology Federal Highway Administration HIBT, Room 3203 400 Seventh Street, S.W. Washington D.C. 20590		13 Type of Report and Period Covered Technical Manual 14. Sponsoring Agency Code

15. Supplementary Notes

Contracting Officer's Technical Representative: Chien-Tan Chang (HIBT)
FHWA Technical Consultants: Jerry DiMaggio (HIBT), Richard Cheney (HIBT)

16. Abstract:

This document presents state-of-the-practice information on the design and installation of cement-grouted ground anchors and anchored systems for highway applications. The anchored systems discussed include flexible anchored walls, slopes supported using ground anchors, landslide stabilization systems, and structures that incorporate tiedown anchors.

This document draws extensively from the FHWA-DP-68-IR (1988) design manual in describing issues such as subsurface investigation and laboratory testing, basic anchoring principles, ground anchor load testing, and inspection of construction materials and methods used for anchored systems. This document provides detailed information on design analyses for ground anchored systems. Topics discussed include selection of design earth pressures, ground anchor design, design of corrosion protection system for ground anchors, design of wall components to resist lateral and vertical loads, evaluation of overall anchored system stability, and seismic design of anchored systems. Also included in the document are two detailed design examples and technical specifications for ground anchors and for anchored walls.

17. Key Words Ground anchors, soldier beam and lagging walls, limit equilibrium, earth pressures, axial capacity, tiedowns, seismic design, contracting, specifications	18. Distribution Statement No Restrictions. This document is available to the public from the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report) Unclassified	20. Security Classification (of this page) Unclassified	21. No. of Pages 281	22. Price

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

ACKNOWLEDGEMENTS

The authors would like to express their appreciation to Mr. Richard S. Cheney, P.E., of the U.S. Department of Transportation Federal Highway Administration (FHWA) for providing significant technical assistance and review during preparation of the document. The authors would also like to thank Dr. Donald A. Bruce, C.Eng of ECO GeoSystems Inc. for providing technical assistance. Dr. Richard Jewell, C.Eng, formerly of GeoSyntec Consultants, assisted in preparing sections of the document and provided technical assistance. The authors would also like to thank the following individuals who reviewed the document and served on the Technical Working Group for this project:

- James J. Brennan, P.E. – Kansas Department of Transportation;
- Chien-Tan Chang, P.E. – FHWA;
- Joel Moskowitz, P.E., of Mueser Rutledge Consulting Engineers – Deep Foundations Institute;
- Heinz Nierlich of Dywidag-Systems International – Post Tensioning Institute;
- Monti Singla - FHWA;
- John P. Tiernan – Georgia Department of Transportation;
- David E. Weatherby of Schnabel Foundation Company – International Association of Foundation Drilling; and
- Shan-Tai Yeh – Colorado Department of Transportation.

The authors would also like to acknowledge the following firms and agencies that provided photographs and technical information:

- Dywidag-Systems International;
- Hayward Baker Inc.;
- Lang Tendons Inc.;
- Schnabel Foundation Company; and
- Williams Form Engineering Corp.

Finally, the authors would like to thank Mrs. Ann Taylor and Mr. Michael Harris of GeoSyntec Consultants who drafted the figures and assisted in the layout of the document.

PREFACE

This document presents state-of-the-practice information on the design and installation of cement-grouted ground anchors and anchored systems for highway applications. Anchored systems discussed include flexible anchored walls, slopes supported using ground anchors, landslide stabilization systems, and structures that incorporate tiedown anchors.

This document has been written, in part, to update the design manual titled "Permanent Ground Anchors" (FHWA-DP-68-1R, 1988). This document draws extensively from the FHWA (1988) design manual in describing issues such as subsurface investigation and laboratory testing, basic anchoring principles, ground anchor load testing, and inspection of construction materials and methods used for anchored systems. Since 1988, advances have been made in design methods and from new construction materials, methods, and equipment.

Results of anchored system performance monitoring and research activities conducted since 1989 are also included in this document. Most recently, research was conducted under a FHWA research contract on the design and performance of ground anchors and anchored soldier beam and timber lagging walls. As part of that research project, performance data on model- and full-scale anchored walls were collected and analyzed. Several of the analysis methods and design procedures that were recommended based on the results of the research are adopted herein. This research is described in FHWA-RD-98-065 (1998), FHWA-RD-98-066 (1998), FHWA-RD-98-067 (1998), and FHWA-RD-97-130 (1998).

This document provides detailed information on basic principles and design analyses for ground anchors and anchored systems. Topics discussed include selection of design earth pressures, design of corrosion protection systems for ground anchors, design of wall components to resist lateral and vertical loads, evaluation of overall anchored system stability, and seismic design of anchored systems. Also included in the document are two detailed design examples and technical specifications for ground anchors and for anchored walls.

TABLE OF CONTENTS

CHAPTER 1 INTRODUCTION **1**

1.1 Purpose **1**

1.2 Anchored System Service Life **1**

1.3 Background **2**

CHAPTER 2 GROUND ANCHORS AND ANCHORED SYSTEMS **4**

2.1 INTRODUCTION **4**

2.2 GROUND ANCHORS **4**

2.2.1 General 4

2.2.2 Types of Ground Anchors 6

2.2.2.1 General 6

2.2.2.2 Straight Shaft Gravity-Grouted Ground Anchors 7

2.2.2.3 Straight Shaft Pressure-Grouted Ground Anchors 8

2.2.2.4 Post-grouted Ground Anchors 8

2.2.2.5 Underreamed Anchors 8

2.2.3 Tendon Materials 8

2.2.3.1 Steel Bar and Strand Tendons 8

2.2.3.2 Spacers and Centralizers 9

2.2.3.3 Epoxy-Coated Bar and Epoxy-Coated Filled Strand 10

2.2.3.4 Other Anchor Types and Tendon Materials 10

2.2.4 Cement Grout 11

2.3 ANCHORED WALLS **11**

2.3.1 General 11

2.3.2 Soldier Beam and Lagging Wall 12

2.3.2.1 General 12

2.3.2.2 Soldier Beam 13

2.3.2.3 Lagging 14

2.3.2.4 Construction Sequence 15

2.3.3 Continuous Walls 15

2.4 APPLICATIONS OF GROUND ANCHORS 16

2.4.1 Highway Retaining Walls 16

2.4.2 Slope and Landslide Stabilization 17

2.4.3 Tiedown Structures 17

CHAPTER 3 SITE INVESTIGATION AND TESTING 19

3.1 INTRODUCTION 19

3.2 FIELD RECONNAISSANCE 19

3.3 SUBSURFACE INVESTIGATION 20

3.3.1 General 20

3.3.2 Soil and Rock Stratigraphy 21

3.3.3 Groundwater 22

3.4 LABORATORY SOIL AND ROCK TESTING 22

3.4.1 General 22

3.4.2 Classification and Index Properties 23

3.4.3 Shear Strength 23

3.4.4 Consolidation 23

3.4.5 Electrochemical Criteria 24

3.5 IN SITU SOIL AND ROCK TESTING 24

CHAPTER 4 BASIC PRINCIPLES OF ANCHORED SYSTEM DESIGN 26

4.1 GENERAL DESIGN CONCEPTS FOR ANCHORED WALLS 26

4.2 FAILURE MECHANISMS OF ANCHORED SYSTEMS 28

4.2.1 General 28

4.2.2 Failure Mechanisms of the Ground Anchor 28

4.2.3 Failure of Soldier Beams 31

4.2.4. Failure of Lagging 32

4.3 SELECTION OF SOIL SHEAR STRENGTH PARAMETERS FOR DESIGN 33

4.3.1 General 33

4.3.2 Drained Shear Strength of Granular Soils 33

4.3.3 Undrained Shear Strength of Normally Consolidated Clay 33

4.3.4 Undrained Shear Strength of Overconsolidated Clay....34
4.3.5 Drained Shear Strength of Overconsolidated Clay....34
4.4 EARTH PRESSURES 36
4.4.1 General....36
4.4.2 Active and Passive Earth Pressure....36
4.4.3 Earth Pressure at Rest....41
4.4.4 Influence of Movement on Earth Pressure....41

CHAPTER 5 DESIGN OF ANCHORED SYSTEMS 46

5.1 INTRODUCTION 46
5.2 EVALUATION OF EARTH PRESSURES FOR WALL DESIGN 47
5.2.1 Introduction....47
5.2.2 Background....48
5.2.3 Terzaghi and Peck Apparent Earth Pressure Diagrams....49
5.2.4 Recommended Apparent Earth Pressure Diagram for Sands....50
5.2.5 Recommended Apparent Earth Pressure Diagram for Stiff to Hard Fissured Clays....52
5.2.6 Recommended Apparent Earth Pressure Diagram for Soft to Medium Clays....57
5.2.7 Loading Diagrams for Stratified Soil Profiles....60
5.2.8 Sliding Wedge Analysis Method....60
5.2.9 Water Pressures....62
5.2.10 Earth Pressures Due To Surface Loads....64
5.2.10.1 Uniform Surcharge Loads....64
5.2.10.2 Point Loads, Line Loads, and Strip Loads....64
5.3 GROUND ANCHOR DESIGN 65
5.3.1 Introduction....65
5.3.2 Location of Critical Potential Failure Surface....65
5.3.3 Calculation of Ground Anchor Loads from Apparent Earth Pressure Diagrams....65
5.3.4 Design of the Unbonded Length....67
5.3.5 Compression Anchors....68
5.3.6 Design of the Anchor Bond Length....69

5.3.7 Spacing Requirements for Ground Anchors....75
5.3.8 Selection of Prestressing Steel Element....77
5.4 WALL DESIGN BASED ON LATERAL PRESSURES....78
5.4.1 Design of Soldier Beams and Sheet-Piles....78
5.4.2 Design of Lagging for Temporary Support....81
5.4.3 Design of Wales and Permanent Facing....83
5.5 LATERAL CAPACITY OF EMBEDDED PORTION OF WALL....84
5.5.1 General....84
5.5.2 Evaluation of Ultimate Passive Resistance....84
5.5.2.1 Soldier Beam and Lagging Walls....84
5.5.2.2 Continuous Walls....86
5.5.3 Depth of Penetration below Excavation....86
5.5.4 Comparison of Wang-Reese and Broms Method for Competent Soils....87
5.6 AXIAL CAPACITY OF WALL....88
5.6.1 Introduction....88
5.6.2 Axial Load Evaluation....89
5.6.3 Axial Capacity Design of Driven Soldier Beams....90
5.6.3.1 General....90
5.6.3.2 Effective Stress Analysis for Driven Soldier Beams....90
5.6.3.3 Total Stress Analysis for Driven Soldier Beams in Clays....92
5.6.4 Axial Capacity Design Of Drilled-in Soldier Beams....93
5.6.4.1 General....93
5.6.4.2 Cohesionless Soils....93
5.6.4.3 Cohesive Soils....94
5.6.4.4 Design Issues for Concrete Backfill of Predrilled Soldier Beam Holes....95
5.7 ANCHORED SLOPES AND LANDSLIDE STABILIZATION SYSTEMS....96
5.7.1 General....96
5.7.2 Design Concepts....96
5.7.3 Limit Equilibrium Calculations....97
5.7.3.1 Overall Approach....97

5.7.3.2 Method 1 Analysis 98
5.7.3.3 Method 2 Analysis 100
5.7.4 Modeling Lateral Wall Resistance in Limit Equilibrium Analyses....101
5.7.5 Comparison of Methods to Evaluate Required Earth Load in Homogeneous Soils 102
5.8 GROUND MASS STABILITY....105
5.8.1 Introduction 105
5.8.2 Basal Stability 105
5.8.2.1 General....105
5.8.2.2 Evaluation of Bottom Heave Potential in Soft to Medium Clays 105
5.8.3 External Stability 107
5.8.3.1 Introduction 107
5.8.3.2 Evaluation of External Stability Using Limit Equilibrium....108
5.9 TIEDOWN DESIGN 109
5.9.1 Introduction 109
5.9.2 Uplift Capacity of Rock Tiedown Anchors....109
5.9.3 Uplift Capacity of Soil Tiedown Anchors 110
5.9.4 Design of Tiedown Anchors to Resist Hydrostatic Uplift....112
5.10 SEISMIC DESIGN....113
5.10.1 Introduction 113
5.10.2 Internal Stability Using Pseudo-Static Theory....113
5.10.2.1 Lateral Earth Pressure 113
5.10.2.2 Wall Design Considerations 116
5.10.2.3 Liquefaction....117
5.10.3 External Stability 117
5.10.3.1 Pseudo-Static Analysis 117
5.10.3.2 Seismic Deformation Analysis 118
5.11 OTHER DESIGN ISSUES 119
5.11.1 Wall and Ground Movements....119
5.11.2 Drainage Systems for Anchored Walls and Slopes 120

5.11.3 Wall System Appurtenances 121
5.11.4 Resisting the Upper Anchor Test Load 122
5.11.5 Anchored Walls for Fill Applications 122

CHAPTER 6 CORROSION CONSIDERATIONS IN DESIGN 124

6.1 INTRODUCTION 124
6.2 CORROSION AND EFFECTS ON GROUND ANCHORS 124
6.2.1 Mechanism of Metallic Corrosion 124
6.2.2 Types of Corrosion for Prestressing Steel 124
6.3 CORROSION PROTECTION OF GROUND ANCHORS 126
6.3.1 Requirements of Corrosion Protection Systems 126
6.3.2 Design of Corrosion Protection Systems 126
6.3.2.1 General 126
6.3.2.2 Anchorage Protection 131
6.3.2.3 Unbonded Tendon Length Protection 132
6.3.2.4 Tendon Bond Length Protection 132
6.3.2.5 Protection Against Stray Currents 132
6.3.2.6 Corrosion Protection of Anchors for Structures Subject to Hydrostatic Uplift 133
6.4 SELECTION OF CORROSION PROTECTION LEVEL 133
6.4.1 General 133
6.4.2 Service Life of the Anchored Structure 133
6.4.3 Aggressivity of the Ground Environment 133
6.4.4 Consequences of Failure of the Anchored System 135
6.4.5 Cost for a Higher Level of Protection 135
6.5 CORROSION OF STRUCTURAL STEEL, CEMENT GROUT, AND CONCRETE. 135
6.5.1 Corrosion and Protection of Steel Soldier Beams and Sheet Piles 135
6.5.2 Degradation and Protection of Cement Grout and Concrete 136

CHAPTER 7 LOAD TESTING AND TRANSFER OF LOAD TO THE ANCHORED SYSTEM 137

7.1 INTRODUCTION 137

7.2 CONCEPTS FOR MONITORING ANCHOR BOND ZONE CAPACITY 137

7.3 TESTING AND STRESSING EQUIPMENT 139

7.3.1 General 139

7.3.2 Equipment Used in Load Testing 140

7.3.2.1 Hydraulic Jack and Pump 140

7.3.2.2 Stressing Anchorage 141

7.3.2.3 Pressure Gauges and Load Cells 142

7.3.2.4 Dial Gauge to Measure Movement 142

7.3.2.5 Jack Chair 142

7.4 ANCHOR LOAD TESTING 142

7.4.1 Introduction 142

7.4.2 Performance Tests 143

7.4.2.1 General 143

7.4.2.2 Procedures for Performance Test 143

7.4.2.3 Recording of Performance Test Data 145

7.4.2.4 Analysis of Performance Test Data 146

7.4.3 Proof Tests 147

7.4.3.1 General 147

7.4.3.2 Proof Test Procedures and Recording and Analysis of Proof Test Data 147

7.4.4 Extended Creep Testing 148

7.4.4.1 General 148

7.4.4.2 Procedures for Extended Creep Test 148

7.4.4.3 Recording and Analysis of Extended Creep Test Data 149

7.4.5 Acceptance Criteria 150

7.4.5.1 General 150

7.4.5.2 Creep 150

7.4.5.3 Apparent Free Length 150

7.4.5.4 Ground Anchor Acceptance Decision Tree 152
7.4.5.5 Modification of Design or Installation Procedures 154
7.5 ANCHOR LOCK-OFF LOAD 154
7.6 LIFT-OFF TESTING 155

CHAPTER 8 CONTRACTING APPROACHES 156

8.1 INTRODUCTION **156**
8.2 METHOD CONTRACTING APPROACH **157**
8.2.1 Introduction 157
8.2.2 Contract Documents for Method Approach 158
8.3 PERFORMANCE CONTRACTING APPROACH **159**
8.3.1 Introduction 159
8.3.2 Implementing Performance Contracting Approach 159
8.3.2.1 Pre-bid Wall Design 159
8.3.2.2 Pre-bid Typical Section Design 160
8.3.2.3 Post-bid Wall Design 160
8.3.3 Contract Documents for Performance Approach 161
8.3.4 Review and Approval 162
8.4 CONTRACTOR DESIGN/BUILD APPROACH **163**
8.5 RECOMMENDATIONS **163**

CHAPTER 9 CONSTRUCTION INSPECTION AND PERFORMANCE MONITORING 164

9.1 INTRODUCTION 164
9.2 INSPECTION ROLES UNDER CERTAIN CONTRACT APPROACHES 164
9.3 PRE-PROJECT PREPARATION 165
9.4 INSPECTION OF CONSTRUCTION MATERIALS 165
9.4.1 Introduction 165
9.4.2 Inspection of Wall Materials 166
9.4.3 Inspection of Ground Anchor Materials 166
9.4.4 Storage and Handling of Construction Materials 167

9.5 INSPECTION OF CONSTRUCTION ACTIVITIES.....167

9.5.1 Surface-Water Control.....167

9.5.2 Vertical Wall Element Installation.....168

9.5.2.1 Drilled-in Soldier Beams.....168

9.5.2.2 Driven Soldier Beams.....168

9.5.2.3 Sheet-Piles.....169

9.5.3 Excavation.....169

9.5.4 Anchor Construction.....169

9.5.4.1 Introduction.....169

9.5.4.2 Anchor Hole Drilling.....170

9.5.4.3 Tendon Insertion.....170

9.5.4.4 Anchor Grouting.....171

9.5.4.5 Anchorage Installation.....172

9.5.5 Ancillary Wall Element Installation.....173

9.5.5.1 Timber Lagging Installation.....173

9.5.5.2 Wall Drainage System Installation.....173

9.5.5.3 Horizontal Drains.....173

9.5.5.4 Permanent Facing Installation.....174

9.6 SHORT-TERM AND LONG-TERM MONITORING.....174

9.6.1 Monitoring of Anchor Load Tests.....174

9.6.2 Short-Term Monitoring of Wall Performance.....175

9.6.3 Long-Term Monitoring.....176

REFERENCES

BIBLIOGRAPHY

APPENDIX

APPENDIX A	Design Examples
APPENDIX B	Development of Wang - Reese Equations
APPENDIX C	Example Calculation of Bending Moment for Wall in Weak Cohesive Soil
APPENDIX D	Predesign Load Testing Procedures to Evaluate Ultimate Ground Anchor Load
APPENDIX E	Specification for Ground Anchors
APPENDIX F	Specifications for Anchored Sheet-pile or Soldier Beam and Lagging Wall

LIST OF TABLES

Table **Page**

1 Soil density/consistency description based on SPT blowcount values (after AASHTO, 1988) 24

2 Summary of common in situ tests for soils 25

3 Typical factors influencing bond stress transfer for small diameter ground anchors 30

4 Typical design steps for an anchored wall (modified after FHWA-RD-81-150, 1982) 47

5 Summary of trapezoidal apparent pressure envelopes for temporary excavations in stiff to hard clays 53

6 Presumptive ultimate values of load transfer for preliminary design of small diameter straight shaft gravity-grouted ground anchors in soil 71

7 Presumptive average ultimate bond stress for ground/grout interface along anchor bond zone (after PTI, 1996) 73

8 Presumptive ultimate values of load transfer for preliminary design of ground anchors in rock 74

9 Properties of prestressing steel bars (ASTM A722) 77

10 Properties of 15-mm diameter prestressing steel strands (ASTM A 416, Grade 270 (metric 1860)) 78

11 Guidance relationship between tendon size and trumpet opening size 78

12 Recommended thickness of temporary timber lagging (after FHWA-RD-75-130, 1976) 82

13 Maximum design bending moments for wales and permanent facing (after AASHTO, 1996) 83

14 Recommended factors of safety for axial capacity of driven and drilled-in soldier beams 90

15 Bearing capacity factors for evaluation of end bearing in drilled shafts in clays 95

16 Procedure to evaluate total lateral earth load using slope stability computer programs 98

17 Procedure to evaluate total lateral earth load for anchored systems constructed in weak cohesive soils.....101

18 Values of K_{REQ} in cohesionless soil using various methods to evaluate earth pressures.....103

19 Horizontal stress coefficient, K, for pressure grouted anchors (after Kulhawy et al., 1983) 111

20 Corrosion protection requirements (modified after PTI, 1996) 131

21 Steps for the performance test.....144

22 Test procedure for ground anchor proof test.....147

23 Load schedule and observation periods for extended creep test for permanent anchor.....149

LIST OF FIGURES

Figure		Page
1	Components of a ground anchor	4
2	Anchorage components for a bar tendon	5
3	Anchorage components for a strand tendon	6
4	Main types of grouted ground anchors (modified after Littlejohn, 1990)	7
5	Cut away view of bar tendon	9
6	Cut away view of strand tendon	10
7	Construction sequence for permanent soldier beam and lagging wall	12
8	Comparison of concrete gravity wall and anchored wall for a depressed roadway	16
9	Applications of ground anchors and anchored systems	18
10	Geotechnical boring layout for permanent anchored wall	20
11	Potential failure conditions to be considered in design of anchored walls	27
12	Contribution of ground anchors to wall stability	28
13	Simplified drained stress-displacement relationship for a stiff clay (modified after CIRIA, 1984)	35
14	Mobilization of Rankine active and passive horizontal pressures for a smooth retaining wall	37
15	Limiting active and passive horizontal pressures	37
16	Active and passive earth pressure coefficients (effect of wall inclination)	39
17	Active and passive earth pressure coefficients (effect of backslope inclination)	40
18	Cross section of model wall (modified after FHWA-RD-98-067, 1998)	42
19	Lateral wall movements and earth pressures with excavation at first anchor level (cantilever stage) (modified after FHWA-RD-98-067, 1998)	42
20	Lateral wall movements and earth pressures during anchor stressing (modified after FHWA-RD-98-067, 1998)	43

LIST OF FIGURES (Continued)

Figure **Page**

21 Lateral wall movements and earth pressures with excavation at lower anchor level (modified after FHWA-RD-98-067, 1998) 44

22 Lateral wall movements and earth pressures with excavation at design grade (modified after FHWA-RD-98-067, 1998) 45

23 Terzaghi and Peck apparent pressure envelopes (after Terzaghi and Peck, 1967) 50

24 Recommended apparent earth pressure diagram for sands....51

25 Measured anchor loads for seven projects (after Ulrich, 1989)....54

26 Wall pressure envelopes (after Winter, 1990) 55

27 Recommended apparent earth pressure envelope for stiff to hard clays 56

28 Henkel's mechanism of base failure 58

29 Values of K_A based on Terzaghi and Peck envelope and Henkel's method....59

30 Force equilibrium method for anchored walls (after FHWA-RD-98-065, 1998)....61

31 Flow net for a retaining wall (after CIRIA, 1984) 62

32 Gross and net water pressures across a retaining wall (modified after CIRIA, 1984) 63

33 Calculation of anchor loads for one-level wall....66

34 Calculation of anchor loads for multi-level wall 67

35 Types of compression anchors....69

36 Mobilization of bond stress for a tension anchor 72

37 Vertical and horizontal spacing requirements for ground anchors....76

38 Calculation of wall bending moments using hinge method 79

39 Calculation of wall bending moments using tributary area method....80

40 Relationship between lateral earth pressure, wall deflection, and depth of wall embedment 85

41 Broms method for evaluating ultimate passive resistance 86

42 Comparison of Broms and Wang-Reese method for wall in sand 88

LIST OF FIGURES (Continued)

Figure **Page**

43 Comparison of Broms and Wang-Reese method for wall in clay.....88

44 Chart for estimating β coefficient versus soil type friction angle (after Fellenius, 1991) 91

45 Chart for estimating N_t coefficients versus soil type friction angle (after Fellenius, 1991) 92

46 Adhesion values for piles in cohesive soils (after Tomlinson, 1980) 93

47 Modeling the ground anchor force in limit equilibrium analysis (after FHWA-RD-97-130, 1998) 99

48 Limit equilibrium analyses used to evaluate total lateral earth load for anchored systems constructed in weak cohesive soils analysis (after FHWA-RD-97-130, 1998).....100

49 Total passive force for example wall in cohesionless soil 102

50 Comparison of limit equilibrium methods for cohesive soils (after FHWA-RD-98-065, 1998)104

51 Analysis of basal stability (modified after Terzaghi et al., 1996).....106

52 Failure surfaces for external stability evaluations 108

53 Inverted cone mechanisms for overall rock mass stability. 110

54 Stability of structure subjected to hydrostatic uplift 112

55 Forces behind a gravity wall 115

56 Effect of seismic coefficients and friction angle on seismic active pressure coefficient (after Lam and Martin, 1986).....116

57 Variation of failure surface inclination with horizontal acceleration coefficient 118

58 Permanent seismic deformation chart (after Hynes and Franklin, 1984).....119

59 Settlement profile behind braced and anchored walls 120

60 Examples of corrosion protection for anchorages 128

61 Examples of corrosion protection classes I and II for strand tendons.....129

62 Examples of corrosion protection classes I and II for bar tendons 130

LIST OF FIGURES (Continued)

Figure		Page
63	Decision tree for selection of corrosion protection level (modified after PTI, 1996)	134
64	Skin friction versus strain diagrams for ground anchors	137
65	Stress propagation in bond length of ground anchor	138
66	Evaluation of critical creep tension	139
67	Typical equipment for load testing of strand ground anchor	140
68	Typical equipment for load testing of bar ground anchor	141
69	Plotting of performance test data (after PTI, 1996)	145
70	Plotting of elastic and residual movement for a performance test (after PTI, 1996)	146
71	Plotting of proof test data (after PTI, 1996)	148
72	Plotting of extended creep test data (after PTI, 1996)	149
73	Ground anchor acceptance decision tree (after PTI, 1996)	153
A-1	Subsurface stratigraphy and design cross section	A-2
A-2	Apparent earth pressure diagram and surcharge pressure diagram	A-4
A-3	Location of unbonded and bond lengths for ground anchors	A-8
A-4	Embedment depth calculations (Wang-Reese method)	A-12
A-5	Subsurface stratigraphy and design cross section	A-18
A-6	Secant residual friction angle (after Stark and Eid, 1994)	A-20
A-7	Slope stability analysis of existing site conditions	A-22
A-8	Apparent earth pressure diagram	A-23
A-9	Calculation of T_{H3} and M_4	A-25
A-10	Location of unbonded and bond lengths for ground anchors	A-27
B-1	Passive wedge failure for a soldier beam in sand (after Reese, . al., 1974)	B-2
B-2	Intersecting failure wedges for soldier beams in sand (after Wang and Reese, 1986)	B-3
B-3	Plastic flow around a soldier beam toe (after Wang and Reese, 1986)	B-4

LIST OF FIGURES (Continued)

Figure **Page**

B-4 Passive wedge failure for a soldier beam in clay (after Reese, 1958) B-6

B-5 Failure wedges for soldier beams in clay (after Wang and Reese, 1986) B-7

D-1 Determination of critical creep tension..... D-2

D-2 Extrapolation of creep curves for determining working tension..... D-4

CHAPTER 1

INTRODUCTION

1.1 PURPOSE

The purpose of this document is to provide state-of-the-practice information on ground anchors and anchored systems for highway applications. Ground anchors discussed in this document are cement grouted, prestressed tendons that are installed in soil or rock. Anchored systems discussed include flexible anchored walls, slopes supported using ground anchors, slope and landslide stabilization systems, and structures that incorporate tiedown anchors. The intended audience includes geotechnical, structural, and highway design and construction specialists involved with the design, construction, contracting, and inspection of these systems.

Ground anchors and anchored systems have become increasingly more cost-effective through improvements in design methods, construction techniques, anchor component materials, and on-site acceptance testing. This has resulted in an increase in the use of both temporary and permanent anchors. The reader should recognize that, as a result of the evolving nature of anchoring practice, the information presented herein is not intended to be prescriptive. Design, construction, and load testing methods are described that are currently used in U.S. practice.

1.2 ANCHORED SYSTEM SERVICE LIFE

The focus of this document is on design methods and procedures for permanent ground anchors and anchored systems. Permanent anchored systems are generally considered to have a service life of 75 to 100 years. However, anchored systems are also commonly used for temporary applications. The service life of temporary earth support systems is based on the time required to support the ground while the permanent systems are installed. This document has adopted the American Association of State Highway and Transportation Officials (AASHTO) guidance which considers temporary systems to be those that are removed or become inoperative upon completion of the permanent systems. The time period for temporary systems is commonly stated to be 18 to 36 months but may be shorter or longer based on actual project conditions.

Furthermore this document has subdivided temporary systems into “support of excavation” (SOE) temporary systems and “critical” temporary systems. In general the owner will determine which temporary systems are to be designated as critical. Often that decision is based on the owner’s need to restrict lateral movement of the support system to minimize ground movements behind the support system. In this document, it is recommended that critical temporary systems be designed to the same criteria used for permanent anchored systems. Conversely, SOE anchored systems are commonly designed to less restrictive criteria than permanent anchored systems. The owner commonly assigns the responsibility for design and performance of SOE anchored systems to the contractor. The design of these SOE anchored systems is often based more on system stability than on minimizing ground movements.

In this document, the basic design recommendations pertain to both permanent anchored systems and critical temporary systems. In this document, the term "permanent anchored systems" or "permanent applications" include critical temporary systems. Whenever appropriate in this document, discussion is provided concerning the differences in design requirements for SOE systems and permanent systems. The following components of an anchored system design are generally less restrictive for temporary SOE systems as compared to permanent systems: (1) selection of timber lagging; (2) allowable stresses in structural components; (3) factors of safety; (4) design for axial load; (5) surcharge loads used to evaluate wall loadings; (6) seismic design criteria; and (7) anchor load testing.

1.3 BACKGROUND

The first use of ground anchors in the U.S. was for temporary support of excavation systems. These systems were typically designed and constructed by specialty contractors. The use of permanent ground anchors for public sector projects in the U.S. did not become common until the late 1970s and today, represent a common technique for earth retention and slope stabilization for highway applications. In certain design and construction conditions, anchored systems offer several advantages over more conventional systems that have resulted in economic and technical benefits. For example, benefits of anchored walls over concrete gravity retaining walls for support of a highway cut include:

- unobstructed workspace for excavations;
- ability to withstand relatively large horizontal wall pressures without requiring a significant increase in wall cross section;
- elimination of the need to provide temporary excavation support since an anchored wall can be incorporated into the permanent structure;
- elimination of need for select backfill;
- elimination of need for deep foundation support;
- reduced construction time; and
- reduced right-of-way (ROW) acquisition.

In 1979, the U.S. Department of Transportation Federal Highway Administration (FHWA) Office of Technology Applications authorized a permanent ground anchor demonstration project. The objective of the project was to provide highway agencies with adequate information to promote routine use of permanent ground anchors and anchored walls. The purpose of the demonstration project was to: (1) study existing ground anchor technology and installation procedures; (2) determine areas where additional work was required; (3) update existing technology; (4) develop a basic design manual; and (5) solicit installations on highway projects. Between 1979 and 1982, two FHWA research reports were completed ("Permanent Ground Anchors" FHWA Report Nos. FHWA-RD- 81-150, 151, and 152 and "Tiebacks" FHWA Report No. FHWA-RD-82-047) and pilot test projects were begun by highway agencies. A design manual was developed by FHWA in 1984,

which was updated in 1988 (FHWA-DP-68-1R, 1988), as part of the demonstration project. During the demonstration project, five U.S. highway projects with permanent anchored systems were instrumented and performance data were gathered (see FHWA-DP-90-068-003, 1990). Today, ground anchors and anchored systems have become an integral component of highway design in the U.S.

This document has been written, in part, to update the FHWA (1988) design manual titled "Permanent Ground Anchors". That document provides an introduction to basic ground anchor concepts and provides the practicing highway engineer with sufficient information to contract for permanent ground anchors and anchored systems. This document draws extensively from FHWA (1988) in describing issues such as subsurface investigation and laboratory testing, basic anchoring principles, ground anchor load testing, and inspection of construction materials and methods used for anchored systems. Since 1988, advances have been made in design methods resulting from anchored system performance data and from new construction materials, methods, and equipment.

Results of research activities conducted since 1989 are also included in this document. Most recently, research was conducted under a FHWA research contract on the design and performance of ground anchors and anchored soldier beam and timber lagging walls. As part of that research project, performance data on two full-scale anchored walls and four large-scale model anchored walls were collected and analyzed. The settlement, axial load, and downdrag force on soldier beams, and lateral wall movements of the wall systems were evaluated (see FHWA-RD-98-066, 1998 and FHWA-RD-98-067, 1998). Several of the analysis methods and design procedures that were recommended based on the results of the research (see FHWA-RD-97-130, 1998) are adopted herein.

Procedures used for ground anchor acceptance testing have also been improved since FHWA (1988) was published. The AASHTO Task Force 27 report "In-Situ Soil Improvement Techniques" (1990) included both a generic construction specification for permanent ground anchors and a ground anchor inspection manual. Those documents form the basis for the construction standards developed by many highway agencies. The Post-Tensioning Institute (PTI) document titled "Recommendations for Prestressed Rock and Soil Anchors" (PTI, 1996) is a document commonly referenced that was developed collectively by owners, design consultants, specialty contractors, and material suppliers. AASHTO Task Force 27 (1990) and PTI (1996) were used as the basis for the chapters of this document on ground anchor acceptance testing and ground anchor corrosion protection. Information from those documents was also used to develop the generic ground anchor specification provided in appendix E.

CHAPTER 2

GROUND ANCHORS AND ANCHORED SYSTEMS

2.1 INTRODUCTION

The previously referenced AASHTO Task Force 27 (1990) and PTI (1996) documents introduced standardized terminology and definitions of ground anchor components. The terminology presented in those documents is adopted and used throughout this document. Ground anchor materials, anchored system construction, and anchored system applications are presented in this chapter.

2.2 GROUND ANCHORS

2.2.1 General

A prestressed grouted ground anchor is a structural element installed in soil or rock that is used to transmit an applied tensile load into the ground. Grouted ground anchors, referenced simply as ground anchors, are installed in grout filled drill holes. Grouted ground anchors are also referred to as "tiebacks". The basic components of a grouted ground anchor include the: (1) anchorage; (2) free stressing (unbonded) length; and (3) bond length. These and other components of a ground anchor are shown schematically in figure 1. The anchorage is the combined system of anchor head, bearing plate, and trumpet that is capable of transmitting the prestressing force from the prestressing

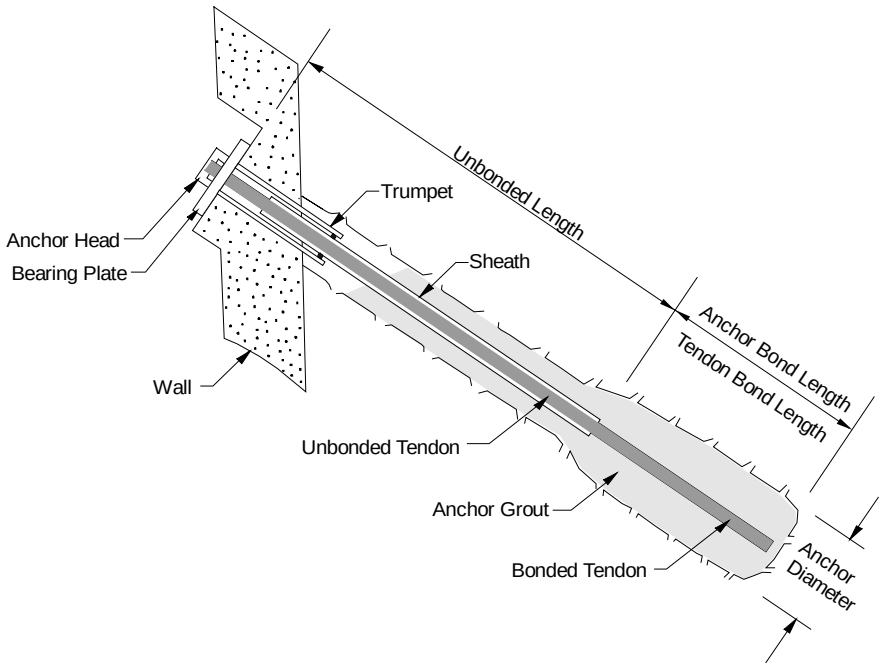


Figure 1. Components of a ground anchor.

steel (bar or strand) to the ground surface or the supported structure. Anchorage components for a bar tendon and a strand tendon are shown in figure 2 and figure 3, respectively. The unbonded length is that portion of the prestressing steel that is free to elongate elastically and transfer the resisting force from the bond length to the structure. A bondbreaker is a smooth plastic sleeve that is placed over the tendon in the unbonded length to prevent the prestressing steel from bonding to the surrounding grout. It enables the prestressing steel in the unbonded length to elongate without obstruction during testing and stressing and leaves the prestressing steel unbonded after lock-off. The tendon bond length is that length of the prestressing steel that is bonded to the grout and is capable of transmitting the applied tensile load into the ground. The anchor bond length should be located behind the critical failure surface.

A portion of the complete ground anchor assembly is referred to as the tendon. The tendon includes the prestressing steel element (strands or bars), corrosion protection, sheaths (also referred to as sheathings), centralizers, and spacers, but specifically excludes the grout. The definition of a tendon, as described in PTI (1996), also includes the anchorage; however, it is assumed herein that the tendon does not include the anchorage. The sheath is a smooth or corrugated pipe or tube that protects the prestressing steel in the unbonded length from corrosion. Centralizers position the tendon in the drill hole such that the specified minimum grout cover is achieved around the tendon. For multiple element tendons, spacers are used to separate the strands or bars of the tendons so that each element is adequately bonded to the anchor grout. The grout is a Portland cement based mixture that provides load transfer from the tendon to the ground and provides corrosion protection for the tendon.

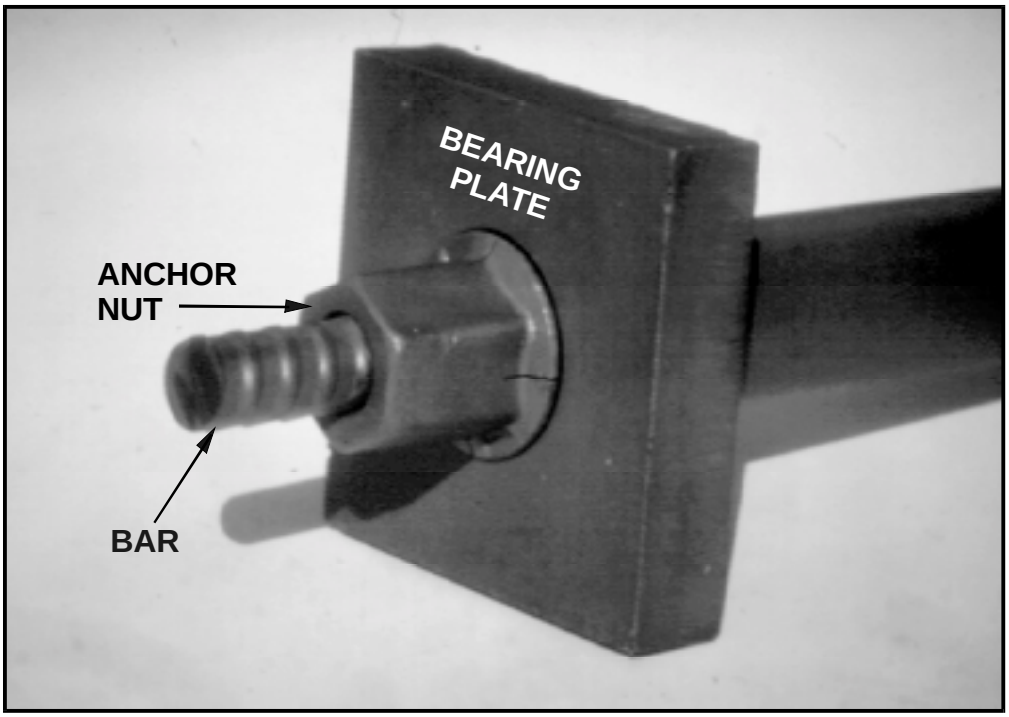


Figure 2. Anchorage components for a bar tendon.

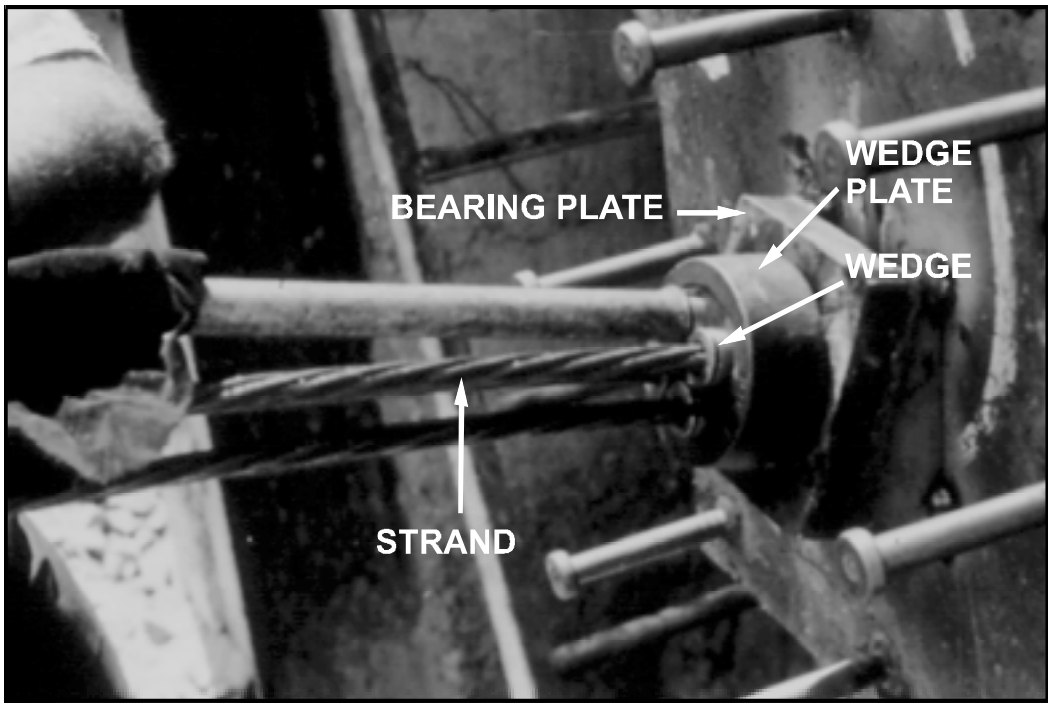


Figure 3. Anchorage components for a strand tendon.

2.2.2 Types of Ground Anchors

2.2.2.1 General

There are three main ground anchor types that are currently used in U.S. practice: (1) straight shaft gravity-grouted ground anchors (Type A); (2) straight shaft pressure-grouted ground anchors (Type B); and (3) post-grouted ground anchors (Type C). Although not commonly used today in U.S. practice, another type of anchor is the underreamed anchor (Type D). These ground anchor types are illustrated schematically in figure 4 and are briefly described in the following sections.

Drilling methods for each of the three main soil and rock ground anchors include rotary, percussion, rotary/percussive, or auger drilling. Detailed information on these drilling techniques may be found in Bruce (1989). The procedures and methods used to drill holes for ground anchors are usually selected by the contractor. The choice of a particular drilling method must also consider the overall site conditions and it is for this reason that the engineer may place limitations on the drilling method.

The drilling method must not adversely affect the integrity of structures near the ground anchor locations or on the ground surface. With respect to drilling, excessive ground loss into the drill hole and ground surface heave are the primary causes of damage to these structures. For example, the use of large diameter hollow stem augered anchors should be discouraged in sands and gravels since the auger will tend to remove larger quantities of soil from the drill hole as compared to the net volume of the auger. This may result in loss of support of the drill hole. In unstable soil or rock, drill casing is used. Water or air is used to flush the drill cuttings out of the cased hole. Caution should be exercised when using air flushing to clean the hole. Excess air pressures may result in unwanted

removal of groundwater and fines from the drill hole leading to potential hole collapse or these excess pressures may result in ground heave.

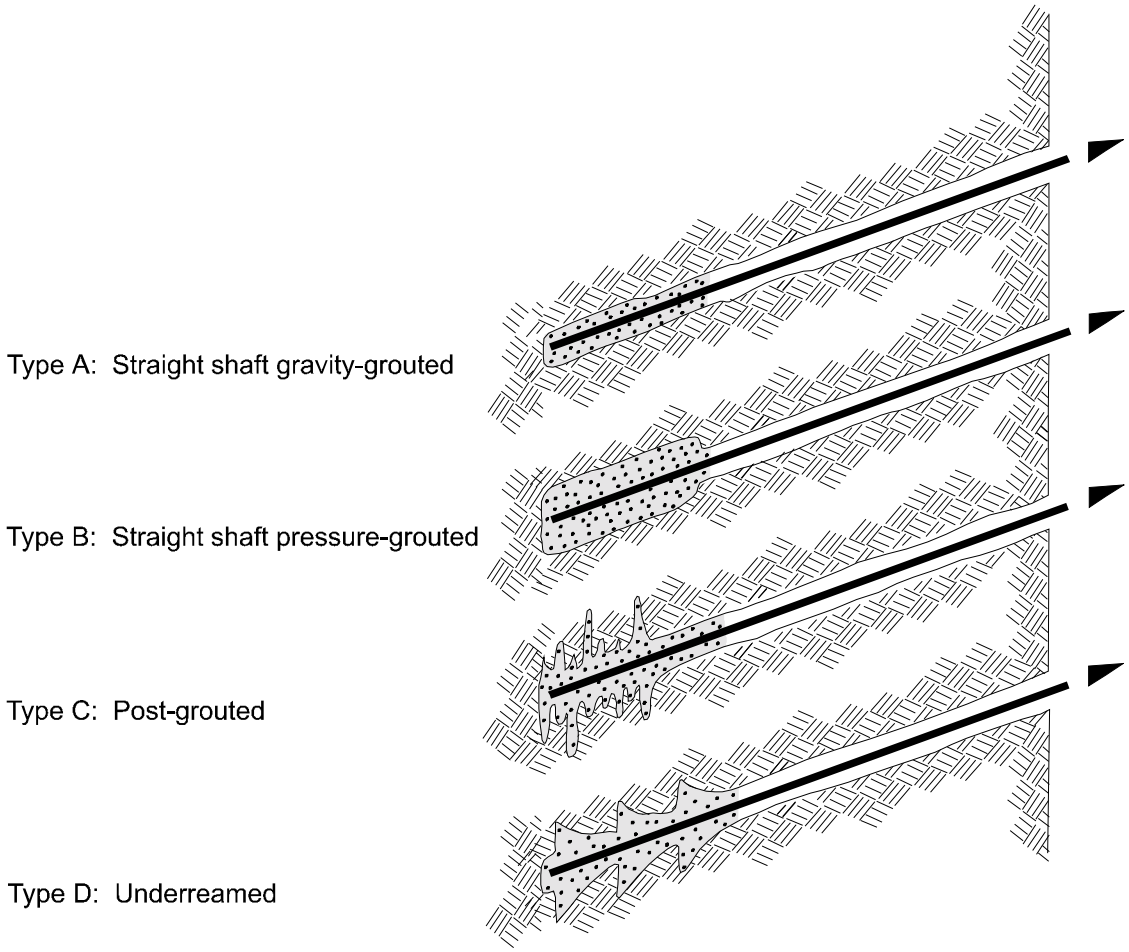


Figure 4. Main types of grouted ground anchors (modified after Littlejohn, 1990, "Ground Anchorage Practice", Design and Performance of Earth Retaining Structures, Geotechnical Special Publication No. 25, Reprinted by permission of ASCE).

2.2.2.2 Straight Shaft Gravity-Grouted Ground Anchors

Straight shaft gravity-grouted ground anchors are typically installed in rock and very stiff to hard cohesive soil deposits using either rotary drilling or hollow-stem auger methods. Tremie (gravity displacement) methods are used to grout the anchor in a straight shaft borehole. The borehole may be cased or uncased depending on the stability of the borehole. Anchor resistance to pullout of the grouted anchor depends on the shear resistance that is mobilized at the grout/ground interface.

2.2.2.3 Straight Shaft Pressure-Grouted Ground Anchors

Straight shaft pressure-grouted ground anchors are most suitable for coarse granular soils and weak fissured rock. This anchor type is also used in fine grained cohesionless soils. With this type of anchor, grout is injected into the bond zone under pressures greater than 0.35 MPa. The borehole is typically drilled using a hollow stem auger or using rotary techniques with drill casings. As the auger or casing is withdrawn, the grout is injected into the hole under pressure until the entire anchor bond length is grouted. This grouting procedure increases resistance to pullout relative to tremie grouting methods by: (1) increasing the normal stress (i.e., confining pressure) on the grout bulb resulting from compaction of the surrounding material locally around the grout bulb; and (2) increasing the effective diameter of the grout bulb.

2.2.2.4 Post-grouted Ground Anchors

Post-grouted ground anchors use delayed multiple grout injections to enlarge the grout body of straight shafted gravity grouted ground anchors. Each injection is separated by one or two days. Postgrouting is accomplished through a sealed grout tube installed with the tendon. The tube is equipped with check valves in the bond zone. The check valves allow additional grout to be injected under high pressure into the initial grout which has set. The high pressure grout fractures the initial grout and wedges it outward into the soil enlarging the grout body. Two fundamental types of post-grouted anchors are used. One system uses a packer to isolate each valve. The other system pumps the grout down the post-grout tube without controlling which valves are opened.

2.2.2.5 Underreamed Anchors

Underreamed anchors consist of tremie grouted boreholes that include a series of enlargement bells or underreams. This type of anchor may be used in firm to hard cohesive deposits. In addition to resistance through side shear, as is the principal load transfer mechanism for other anchors, resistance may also be mobilized through end bearing. Care must be taken to form and clean the underreams.

2.2.3 Tendon Materials

2.2.3.1 Steel Bar and Strand Tendons

Both bar and strand tendons are commonly used for soil and rock anchors for highway applications in the U.S. Material specifications for bar and strand tendons are codified in American Society for Testing and Materials (ASTM) A722 and ASTM A416, respectively. Indented strand is codified in ASTM A886. Bar tendons are commonly available in 26 mm, 32 mm, 36 mm, 45 mm, and 64 mm diameters in uncoupled lengths up to approximately 18 m. Anchor design loads up to approximately 2,077 kN can be resisted by a single 64-mm diameter bar tendon. For lengths greater than 18 m and where space constraints limit bar tendon lengths, couplers may be used to extend the tendon length. As compared to strand tendons, bars are easier to stress and their load can be adjusted after lock-off.

Strand tendons comprise multiple seven-wire strands. The common strand in U.S. practice is 15 mm in diameter. Anchors using multiple strands have no practical load or anchor length limitations. Tendon steels have sufficiently low relaxation properties to minimize long-term anchor load losses. Couplers are available for individual seven-wire strands but are rarely used since strand tendons can be manufactured in any length. Strand couplers are not recommended for routine anchor projects as the diameter of the coupler is much larger than the strand diameter, but strand couplers may be used to repair damaged tendons. Where couplers are used, corrosion protection of the tendon at the location of the coupler must be verified.

2.2.3.2 Spacers and Centralizers

Spacer/centralizer units are placed at regular intervals (e.g., typically 3 m) along the anchor bond zone. For strand tendons, spacers usually provide a minimum interstrand spacing of 6 to 13 mm and a minimum outer grout cover of 13 mm. Both spacers and centralizers should be made of non-corrosive materials and be designed to permit free flow of grout. Figure 5 and figure 6 show a cut away section of a bar and a strand tendon, respectively.

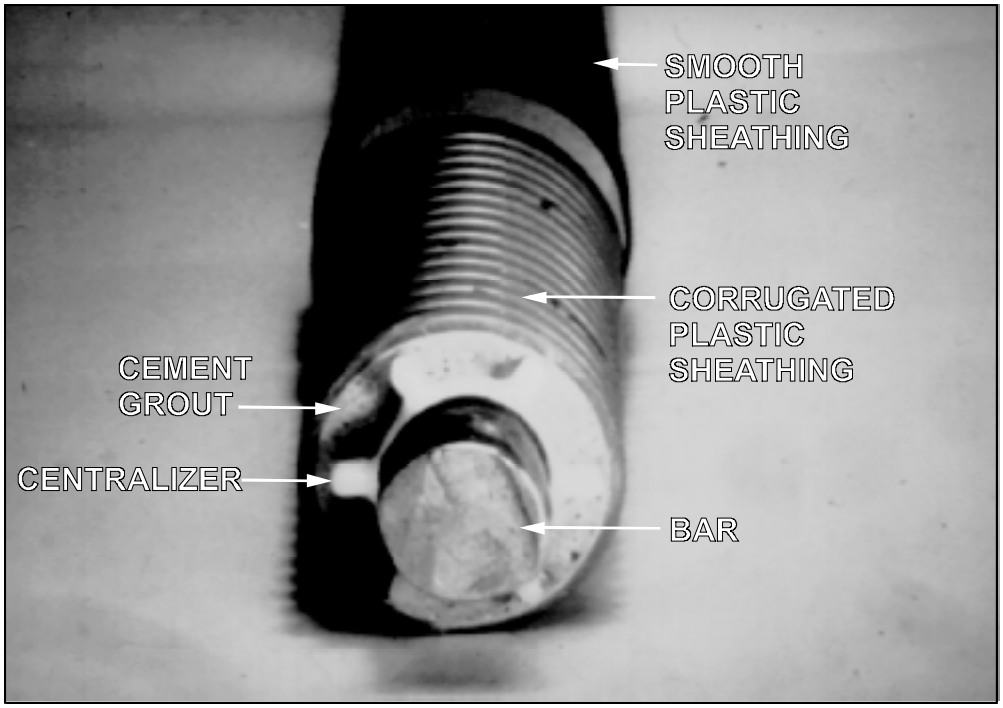


Figure 5. Cut away view of bar tendon.

2.2.3.3 Epoxy-Coated Bar and Epoxy-Coated Filled Strand

Epoxy-coated bar (AASHTO M284) and epoxy-coated filled strand (supplement to ASTM A882), while not used extensively for highway applications, are becoming more widely used for dam tiedown projects. The epoxy coating provides an additional layer of corrosion protection in the unbonded and bond length as compared to bare prestressing steel.

For epoxy-coated filled strand, in addition to the epoxy around the outside of the strand, the center wire of the seven-wire strand is coated with epoxy. Unfilled epoxy-coated strand is not recommended because water may enter the gaps around the center wire and lead to corrosion. Unlike bare strand, creep deformations of epoxy-coated filled strands themselves are relatively significant during anchor testing. When evaluating anchor acceptance with respect to creep, the creep of the epoxy-coated filled strands themselves must be deducted from the total creep movements to obtain a reliable measurement of the movements in the bond zone. Estimates of intrinsic creep movements of epoxy-coated filled strand are provided in PTI (1996).

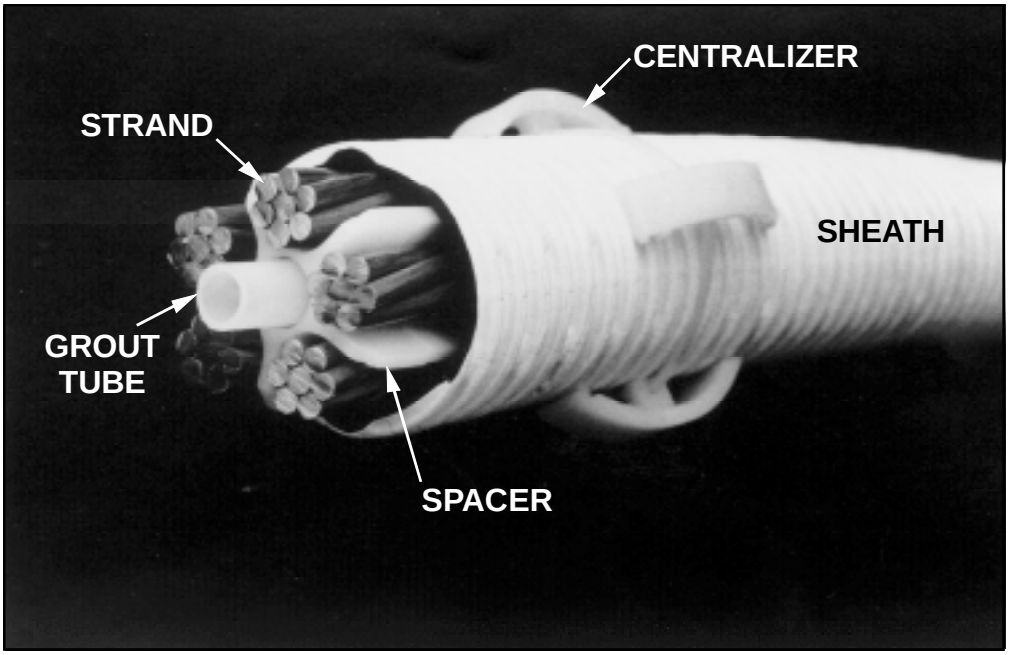


Figure 6. Cut away view of strand tendon.

2.2.3.4 Other Anchor Types and Tendon Materials

In addition to cement grouted anchors incorporating high strength prestressing steels, alternative anchor types and tendon materials are used in the U.S. Examples include Grade 60 and Grade 75 grouted steel bars, helical anchors, plate anchors, and mechanical rock anchors. The design and testing methods described in this document are used for cement grouted anchors that use high strength prestressing steels. These methods may not be appropriate for use with the alternative anchor types mentioned above.

Research on the use of fiber reinforced plastic (FRP) prestressing tendons is currently being performed (e.g., Schmidt et al., 1994). FRP tendons have high tensile strength, are corrosion resistant, and are lightweight. These products, however, are not used in current U.S. construction practice. Other materials such as fiberglass and stainless steel have been used experimentally but cost and/or construction concerns have restricted widespread use.

2.2.4 Cement Grout

Anchor grout for soil and rock anchors is typically a neat cement grout (i.e., grout containing no aggregate) conforming to ASTM C150 although sand-cement grout may also be used for large diameter drill holes. Pea gravel-sand-cement grout may be used for anchor grout outside the tendon encapsulation. High speed cement grout mixers are commonly used which can reasonably ensure uniform mixing between grout and water. A water/cement (w/c) ratio of 0.4 to 0.55 by weight and Type I cement will normally provide a minimum compressive strength of 21 MPa at the time of anchor stressing. For some projects, special additives may be required to improve the fluid flow characteristics of the grout. Admixtures are not typically required for most applications, but plasticizers may be beneficial for applications in high temperature and for long grout pumping distances.

2.3 ANCHORED WALLS

2.3.1 General

A common application of ground anchors for highway projects is for the construction of anchored walls used to stabilize excavations and slopes. These anchored walls consist of nongravity cantilevered walls with one or more levels of ground anchors. Nongravity cantilevered walls employ either discrete (e.g., soldier beam) or continuous (e.g., sheet-pile) vertical elements that are either driven or drilled to depths below the finished excavation grade. For nongravity cantilevered walls, support is provided through the shear and bending stiffness of the vertical wall elements and passive resistance from the soil below the finished excavation grade. Anchored wall support relies on these components as well as lateral resistance provided by the ground anchors to resist horizontal pressures (e.g., earth, water, seismic, etc.) acting on the wall.

Various construction materials and methods are used for the wall elements of an anchored wall. Discrete vertical wall elements often consist of steel piles or drilled shafts that are spanned by a structural facing. Permanent facings are usually cast-in-place (CIP) concrete although timber lagging or precast concrete panels have been used. Continuous wall elements do not require separate structural facing and include steel sheet-piles, CIP or precast concrete wall panels constructed in slurry trenches (i.e., slurry (diaphragm) walls), tangent/secant piles, soil-cement columns, and jet grouted columns.

2.3.2 Soldier Beam and Lagging Wall

2.3.2.1 General

Soldier beam and lagging walls are the most commonly used type of anchored wall system in the U.S. This wall system uses discrete vertical wall elements spanned by lagging which is typically timber, but which may also be reinforced shotcrete. These wall systems can be constructed in most ground types, however, care must be exercised in grounds such as cohesionless soils and soft clays that may have limited "stand-up" time for lagging installation. These wall systems are also highly pervious. The construction sequence for a permanent soldier beam and lagging wall is illustrated in figure 7 and is described below.

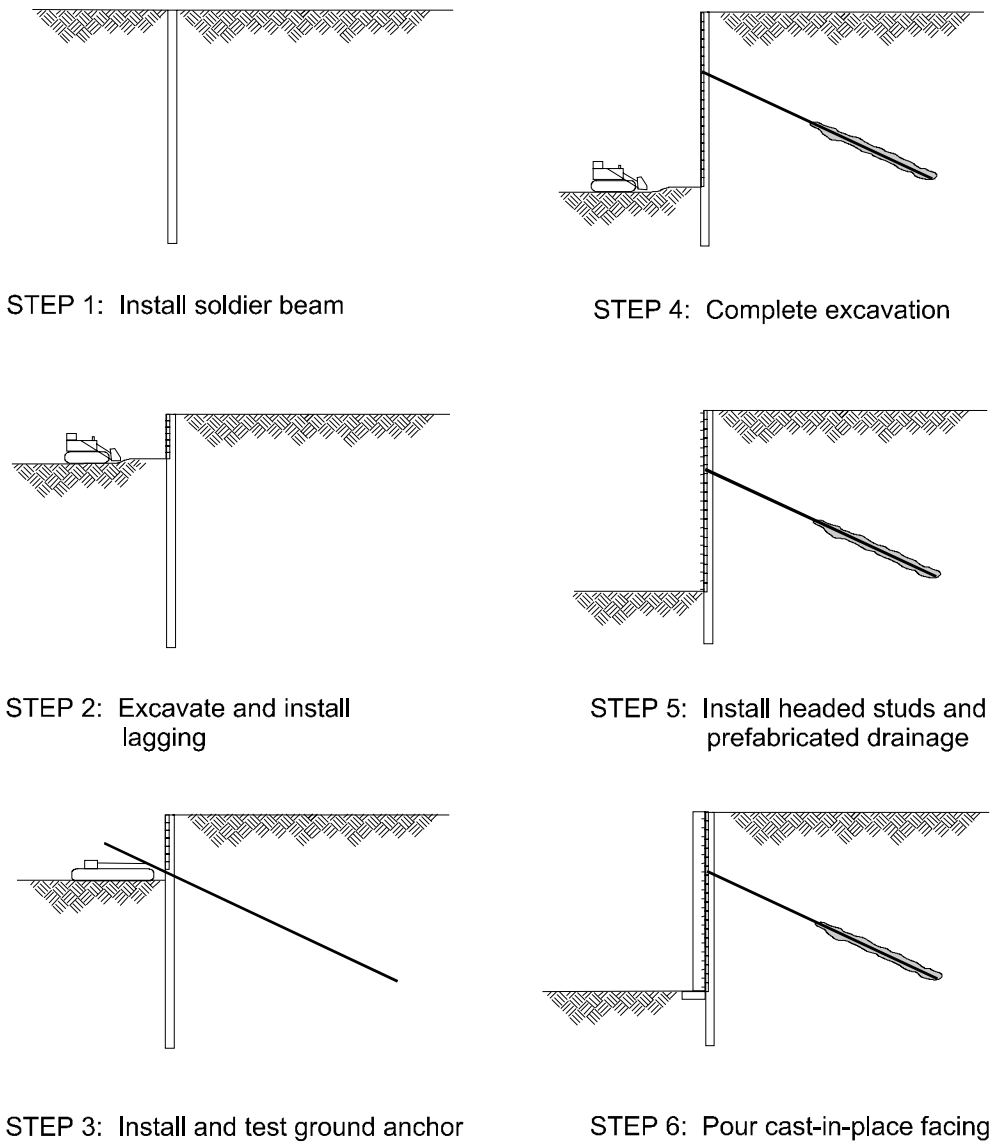


Figure 7. Construction sequence for permanent soldier beam and lagging wall.

2.3.2.2 Soldier Beam

The initial step of construction for a soldier beam and lagging wall consists of installing the soldier beams from the ground surface to their final design elevation. Horizontal spacing of the soldier beams typically varies from 1.5 to 3 m. The soldier beams may be steel beams or drilled shafts, although drilled shafts are seldom used in combination with timber lagging.

Drilled-in Soldier Beams

Steel beams such as wide flange (WF) sections or double channel sections may be placed in excavated holes that are subsequently backfilled with concrete. It is recommended that the excavated hole be backfilled with either structural or lean-mix concrete from the bottom of the hole to the level of the excavation subgrade. The selection of lean-mix or structural concrete is based on lateral and vertical capacity requirements of the embedded portion of the wall and is discussed in chapter 5. From the excavation subgrade to the ground surface, the hole should be backfilled with lean-mix concrete that is subsequently scraped off during lagging and anchor installation. Structural concrete is not recommended to be placed in this zone because structural concrete is extremely difficult to scrape off for lagging installation. Lean-mix concrete typically consists of one 94 lb bag of Portland cement per cubic yard of concrete and has a compressive strength that does not typically exceed approximately 1 MPa. As an alternative to lean-mix concrete backfill, controlled low strength material (CLSM) or “flowable fill” may be used. This material, in addition to cement, contains fine aggregate and fly ash. When allowing lean-mix concrete or CLSM for backfilling soldier beam holes, contract specifications should require a minimum compressive strength of 0.35 MPa. Like lean-mix concrete, CLSM should be weak enough to enable it to be easily removed for lagging installation.

Ground anchors are installed between the structural steel sections and the distance between the sections depends upon the type of ground anchor used. Drill hole diameters for the soldier beams depend upon the structural shape and the diameter of the anchor. Replacement anchors can be installed between the structural sections at any location along the soldier beam. The ground anchor to soldier beam connection for drilled-in soldier beams can be installed on the front face of the structural sections or between the sections. For small diameter ground anchors, the connection may be prefabricated before the soldier beams are installed. The connections for large-diameter anchors are made after the anchors have been installed.

Driven Soldier Beams

Steel beams such as HP shapes or steel sheet piles are used for driven soldier beams. Driven soldier beams must penetrate to the desired final embedment depth without significant damage. Drive shoes or “points” may be used to improve the ability of the soldier beams to penetrate a hard stratum. High strength steels also improve the ability of the soldier beams to withstand hard driving. If the soldier beams cannot penetrate to the desired depth, then the beams should be drilled-in. Thru-beam connections or horizontal wales are used to connect ground anchors to driven soldier beams.

A thru-beam connection is a connection cut in the beam for a small diameter ground anchor. Thru-beam connections are usually fabricated before the beam is driven. This type of connection is designed so the ground anchor load is applied at the center of the soldier beam in line with the web of the soldier beam. Large-diameter (i.e., greater than approximately 150 mm) ground anchors cannot be used with thru-beam connections. Thru-beam connections are used when few ground anchor failures are anticipated because when a ground anchor fails, the failed anchor has to be removed from the connection or a new connection has to be fabricated. A "sidewinder connection" may be used with a replacement anchor for a temporary support of excavation wall, but it is not recommended for a permanent wall. A sidewinder connection is offset from the center of the soldier beam, and the ground anchor load is applied to the flange some distance from the web. Sidewinder connections subject the soldier beams to bending and torsion.

Horizontal wales may be used to connect the ground anchors to the driven soldier beams. Horizontal wales can be installed on the face of the soldier beams, or they can be recessed behind the front flange. When the wales are placed on the front flange, they can be exposed or embedded in the concrete facing. If the wales remain exposed, then the ground anchor tendon corrosion protection may be exposed to the atmosphere and it is therefore necessary that the corrosion protection for the anchorage be well designed and constructed. However, since exposed wales are unattractive and must be protected from corrosion, they are not recommended for permanent anchored walls. Wales placed on the front face of the soldier beams require a thick cast-in-place concrete facing. Wales can be recessed to allow a normal thickness concrete facing to be poured. Recessed wales must be individually fabricated and the welding required to install them is difficult and expensive. If a wale is added during construction, the horizontal clear distance to the travel lanes should be checked before approval of the change.

2.3.2.3 Lagging

After installation of the soldier beams, the soil in front of the wall is excavated in lifts, followed by installation of lagging. Excavation for lagging installation is commonly performed in 1.2 to 1.5 m lifts, however, smaller lift thicknesses may be required in ground that has limited "stand-up" time. Lagging should be placed from the top-down as soon as possible after excavation to minimize erosion of materials into the excavation. Prior to lagging installation, the soil face should be excavated to create a reasonably smooth contact surface for the lagging. Lagging may be placed either behind the front flange of the soldier beam or on the soldier beam. Lagging placed behind soldier beam flanges is cut to approximate length, placed in-between the flanges of adjacent soldier beams, and secured against the soldier beam webs by driving wood wedges or shims. Lagging can also be attached to the front flange of soldier beams with clips or welded studs. In rare circumstances, lagging can be placed behind the back flange of the soldier beam. With either lagging installation method, gaps between the lagging and the retained ground must be backpacked to ensure good contact. Prior to placing subsequent lagging a spacer, termed a "louver", is nailed to the top of the lagging board at each end of the lagging. This louver creates a gap for drainage between vertically adjacent lagging boards. The size of the gap must be sufficiently wide to permit drainage, while at the same time disallowing the retained soil to fall out from behind the boards. Typically, placing vertically adjacent lagging boards in close contact is considered unacceptable, however, some waterproofing methods may require that the gap between the lagging boards be eliminated. In this case, the contractor must provide an alternate means to provide drainage.

Concrete lagging has been used, but its use may be problematic due to difficulties in handling and very tight tolerances on the horizontal and vertical positioning of the soldier beam to ensure easy installation of standard length concrete lagging. Trimming of concrete lagging is very difficult and field splicing is not possible. Also, the concrete lagging near the anchor location may crack during anchor testing or stressing.

2.3.2.4 Construction Sequence

Top-down installation of lagging continues until the excavation reaches a level of approximately 0.6 m below the design elevation of a ground anchor. At this point, the excavation is halted and the ground anchor is installed. Deeper excavation (i.e., greater than 0.6 m) below the level of a ground anchor may be required to allow the anchor connection to be fabricated or to provide equipment access. The wall must be designed to withstand stresses associated with a deeper excavation. The anchor is installed using appropriate drilling and grouting procedures, as previously described. When the grout has reached an appropriate minimum strength, the anchor is load tested and then locked-off at an appropriate load. Excavation and lagging installation then continues until the elevation of the next anchor is reached and the next anchor is installed. This cycle of excavation, lagging installation, and ground anchor installation is continued until the final excavation depth is reached.

When the excavation and lagging reach the final depth, prefabricated drainage elements may be placed at designed spacings and connected to a collector at the base of the wall. The use of shotcrete in lieu of timber lagging can be effective in certain situations. However, since the shotcrete is of low permeability, drainage must be installed behind the shotcrete. Drainage systems for anchored walls are discussed further in chapter 5. For permanent walls, a concrete facing is typically installed. The facing is either precast or CIP concrete.

2.3.3 Continuous Walls

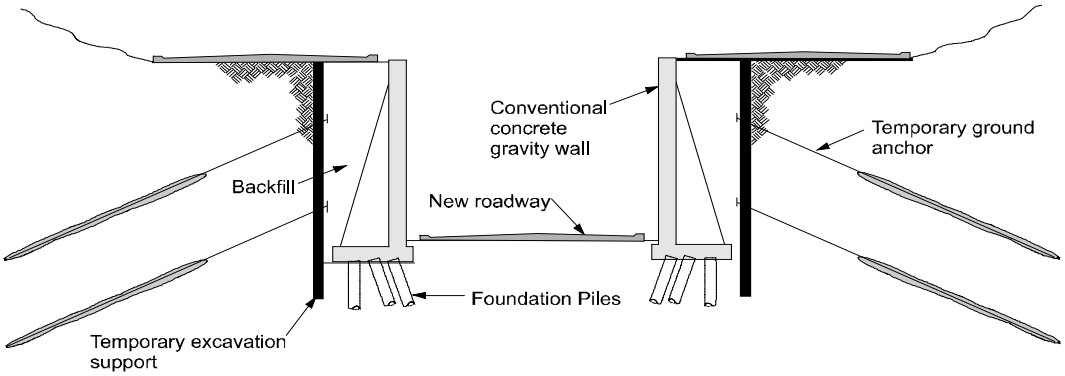
Ground anchors are also used in continuous wall systems such as sheet-pile walls, tangent or secant pile walls, slurry walls, or soil mixed walls. Continuous walls are commonly used for temporary excavation support systems. Sheet-pile walls are constructed in one phase in which interlocking sheet-piles are driven to the final design elevation. Where difficult driving conditions are encountered, a template is often utilized to achieve proper alignment of the sheet-piles, however, it should be recognized that these wall systems may not be feasible for construction in hard ground conditions or where obstructions exist. Interlocking sheet-piles may be either steel or precast concrete, however, steel sheet-piles are normally used due to availability and higher strength than precast concrete sheet-piles. Additional information on wall construction procedures, materials, and equipment for other continuous wall systems is presented in FHWA-HI-99-007 (1999).

Unlike soldier beam and lagging walls, continuous walls act as both vertical and horizontal wall elements. Cycles of excavation and anchor installation proceed from the top of the excavation and then between the level of each anchor. Because of the relative continuity of these wall systems, water pressure behind continuous walls must be considered in design. In cases where the continuous wall must resist permanent hydrostatic forces, a watertight connection must be provided at the ground anchor/wall connection.

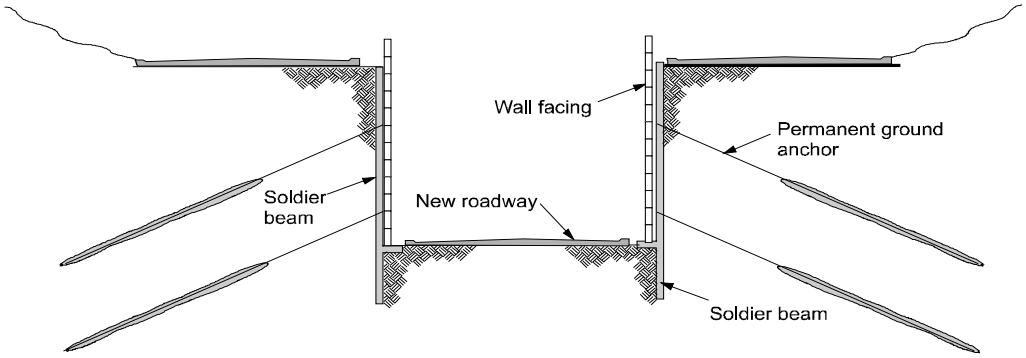
2.4 APPLICATIONS OF GROUND ANCHORS

2.4.1 Highway Retaining Walls

Anchored walls are commonly used for grade separations to construct depressed roadways, roadway widenings, and roadway realignments. The advantages of anchored walls over conventional concrete gravity walls have been described in section 1.2. Figure 8 provides a comparative illustration of a conventional concrete gravity wall and a permanent anchored wall for the construction of a depressed roadway. The conventional gravity wall is more expensive than a permanent anchored wall because it requires temporary excavation support, select backfill, and possibly deep foundation support. Anchored walls may also be used for new bridge abutment construction and end slope removal for existing bridge abutments (see FHWA-RD-97-130, 1998).



(a) Conventional Concrete Gravity Wall



(b) Permanent Anchored Soldier Beam and Lagging Wall

Figure 8. Comparison of concrete gravity wall and anchored wall for a depressed roadway.

2.4.2 Slope and Landslide Stabilization

Ground anchors are often used in combination with walls, horizontal beams, or concrete blocks to stabilize slopes and landslides. Soil and rock anchors permit relatively deep cuts to be made for the construction of new highways (figure 9a). Ground anchors can be used to provide a sufficiently large force to stabilize the mass of ground above the landslide or slip surface (figure 9b). This force may be considerably greater than that required to stabilize a vertical excavation for a typical highway retaining wall. Horizontal beams or concrete blocks may be used to transfer the ground anchor loads to the ground at the slope surface provided the ground does not "run" or compress and is able to resist the anchor reaction forces at the excavated face. Cost, aesthetics, and long-term maintenance of the exposed face will affect the selection of horizontal beams or blocks.

2.4.3 Tiedown Structures

Permanent ground anchors may be used to provide resistance to vertical uplift forces. Vertical uplift forces may be generated by hydrostatic or overturning forces. The method is used in underwater applications where the structure has insufficient dead weight to counteract the hydrostatic uplift forces. An example application of ground anchors to resist uplift forces is shown in figure 9c. The advantage of ground anchors for tiedown structures include: (1) the volume of concrete in the slab is reduced compared to a dead weight slab; and (2) excavation and/or dewatering is reduced. Disadvantages of ground anchors for tiedowns include: (1) potentially large variations in ground anchor load resulting from settlement and heave of the structure; and (2) difficulty in constructing watertight connections at the anchor-structural slab interface, which is particularly important for hydrostatic applications; and (3) variations in stresses in the slab. A major uplift slab that incorporated tiedowns was constructed for the Central Artery Project in Boston, Massachusetts (see Druss, 1994).

Although not a highway application, permanent rock anchor tiedowns may be used to stabilize concrete dams (figure 9d). Existing dams may require additional stabilization to meet current safety standards with respect to maximum flood and earthquake requirements. Anchors provide additional resistance to overturning, sliding, and earthquake loadings.

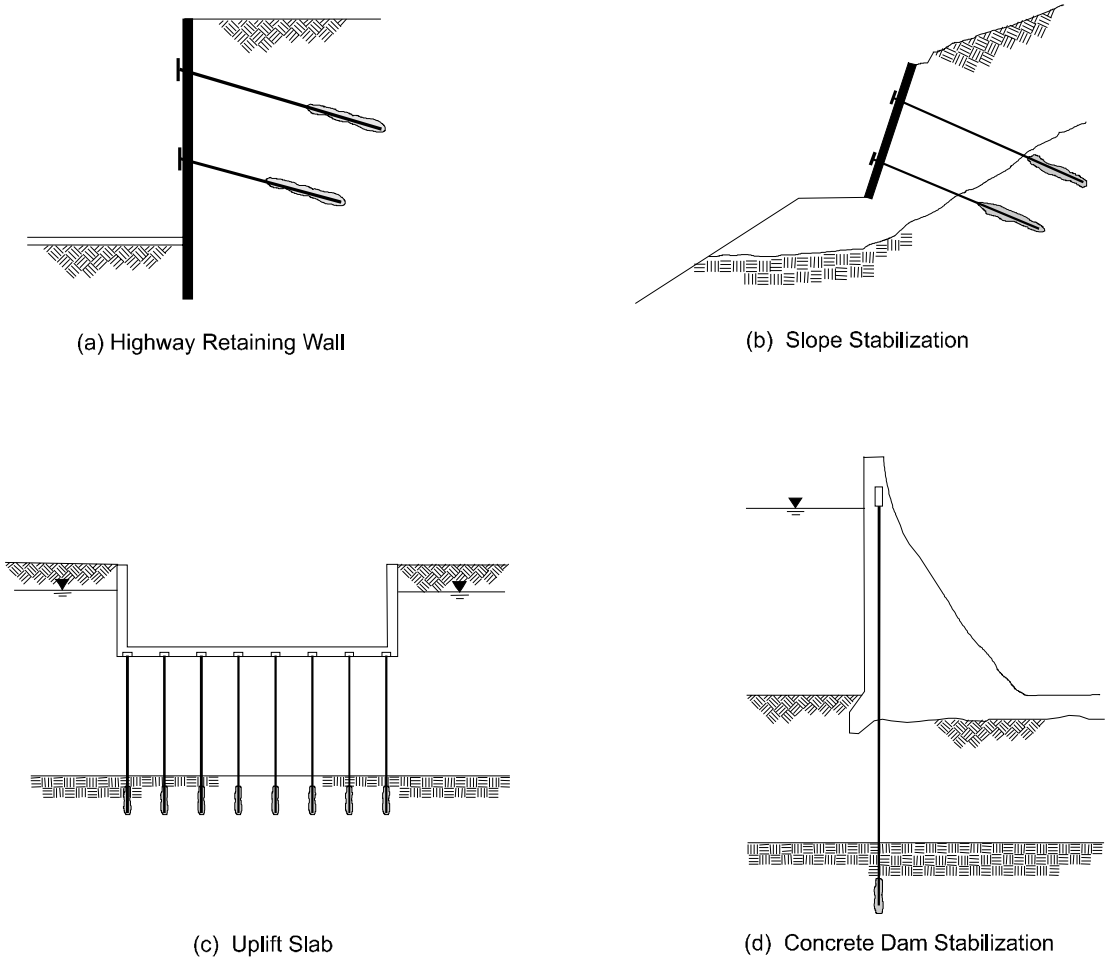


Figure 9. Applications of ground anchors and anchored systems.

CHAPTER 3

SITE INVESTIGATION AND TESTING

3.1 INTRODUCTION

The purpose of this chapter is to describe basic site characterization and soil and rock property evaluation for ground anchor and anchored system design. These activities generally include field reconnaissance, subsurface investigation, in situ testing, and laboratory testing. The engineering properties and behavior of soil and rock material must be evaluated because these materials provide both loading and support for an anchored system.

Site investigation and testing programs are necessary to evaluate the technical and economical feasibility of an anchored system for a project application. The extent of the site investigation and testing components for a project should be consistent with the project scope (i.e., location, size, critical nature of the structure, and budget), the project objectives (i.e., temporary or permanent structures), and the project constraints (i.e., geometry, constructability, performance, and environmental impact). Typical elements of a site investigation and testing program are described herein.

3.2 FIELD RECONNAISSANCE

Field reconnaissance involves visual inspection of the site and examination of available documents regarding site conditions. Information collected during field reconnaissance should include the following:

- surface topography and adjacent land use;
- surface drainage patterns, and surface geologic patterns including rock outcrops, landforms, existing excavations, and evidence of surface settlement;
- site access conditions and traffic control requirements for both investigation and construction activities;
- areas of potential instability such as deposits of organic or weak soils, steep terrain slide debris, unfavorably jointed or dipping rock, and areas with a high ground-water table;
- extent and condition (e.g., visible damage, corrosion) of existing above and below ground utilities and structures; and
- available right-of-way (ROW) and easements required for the installation of ground anchors and anchored systems.

3.3 SUBSURFACE INVESTIGATION

3.3.1 General

Subsurface investigation activities for anchored systems typically involve soil borings and rock coring. Figure 10 illustrates guideline recommendations for locations of subsurface borings for a permanent anchored wall or slope. Information on the subsurface soil and rock stratigraphy and ground-water conditions are typically obtained from subsurface investigation activities. Subsurface investigation may also involve conducting in situ soil or rock tests and obtaining disturbed and undisturbed samples for laboratory testing. Detailed information and guidance on subsurface investigation are provided in AASHTO (1988) and FHWA-HI-97-021 (1997).

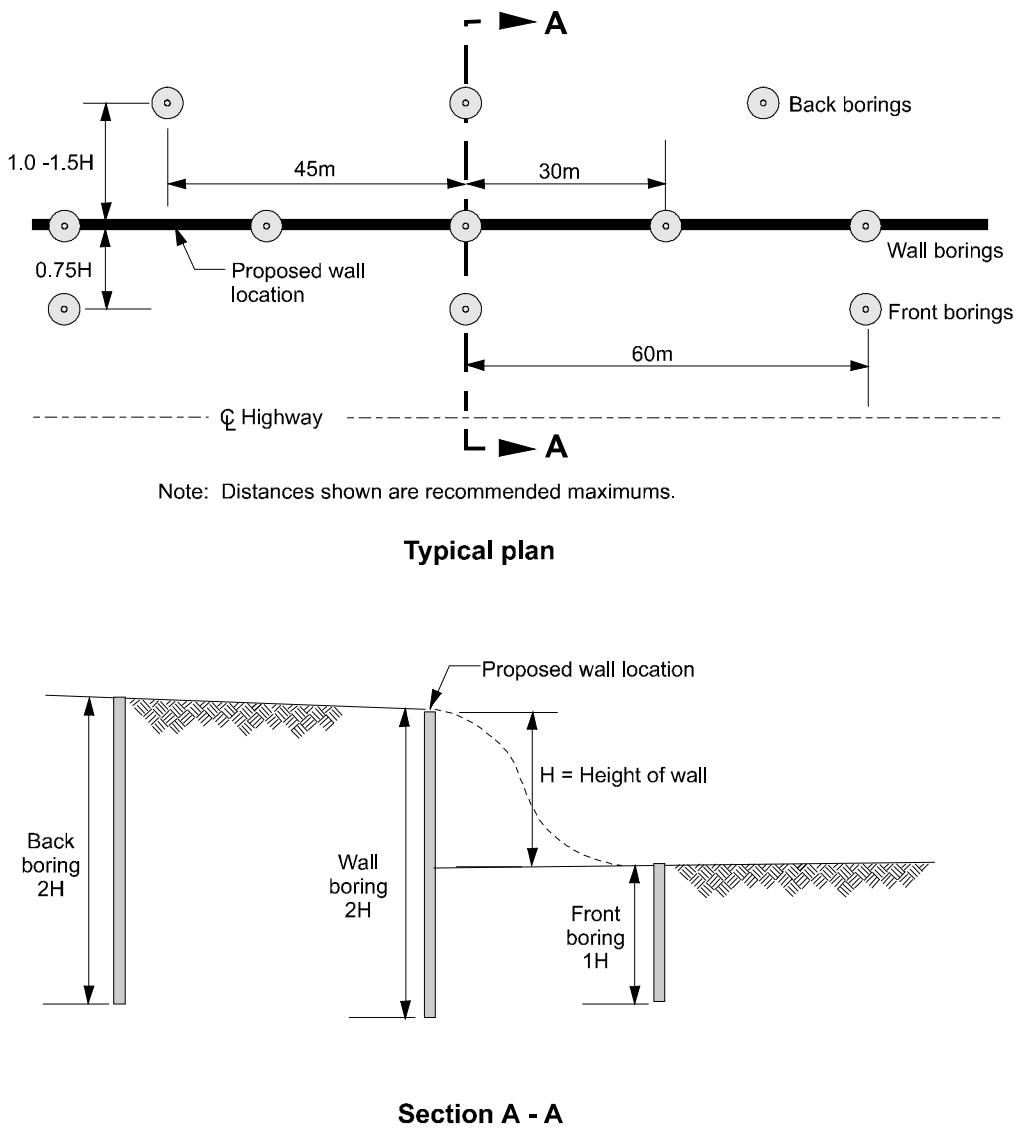


Figure 10. Geotechnical boring layout for permanent anchored wall.

3.3.2 Soil and Rock Stratigraphy

The soil and rock stratigraphy at the project site, including the thickness, elevation, and lateral extent of various layers, should be evaluated through implementation of a project-specific subsurface investigation. The following potentially problematic soils and rock should also be identified during the subsurface investigation which may significantly affect the design and construction of the anchored system:

- cohesionless sands and silts which tend to ravel (i.e., cave-in) when exposed, particularly when water is encountered, and which may be susceptible to liquefaction or vibration-induced densification;

- weak soil or rock layers which are susceptible to sliding instability;

- highly compressible materials such as high plasticity clays and organic soils which are susceptible to long-term (i.e., creep) deformations; and

- obstructions, boulders, and cemented layers which adversely affect anchor hole drilling, grouting, and wall element installation.

As shown in figure 10, subsurface borings should be advanced at regular intervals along, behind, and in front of the wall alignment or slope face. Borings should be located at the site extremities along the wall alignment so that stratigraphy information can be interpolated from the boring information. Typical boring spacing is 15 to 30 m for soil anchors and 30 to 60 m for rock anchors. The back borings are located such that the borings are advanced within the anchor bond zone so that potentially weak or unsuitable soil or rock layers can be identified. Back and wall boring depths should be controlled by the general subsurface conditions, but should penetrate to a depth below the ground surface of at least twice the wall or slope height. Front borings may be terminated at a depth below the proposed wall base equal to the wall height. Borings should be advanced deeper if there is a potential for soft, weak, collapsible, or liquefiable soils at depth. For very steeply inclined ground anchors or for vertical anchors, borings should also, at a minimum, penetrate through to the depth of the anchor bond zone. Additional borings may be required to characterize the geometry of a landslide slip surface.

As a general recommendation, soil samples should be obtained at regular, approximately 1.5 m deep, intervals and at all changes in the underlying soil strata for visual identification and laboratory testing. Methods of soil sampling include the Standard Penetration Test (SPT) (ASTM D1586) and, for cohesive soils, the use of thin-wall tubes (ASTM D1587). The cone penetration test (CPT) (ASTM D3441) may be used, if necessary, to develop a continuous subsurface soil profile.

A minimum rock core of 3 m should be recovered for subsurface conditions in which bedrock is encountered within the previously recommended investigation depths and for all designs that include rock anchors. A description of rock type, mineral composition, texture (i.e., stratification, foliation), degree of weathering, and discontinuities is generally obtained. An estimate of intact rock strength can be evaluated using percentage of core recovery and rock quality designation (RQD). The orientations (i.e., strike and dip) of discontinuities and fractures should be included whenever possible in the rock description so that the potential for sliding instability can be evaluated. This latter information may be available from rock outcrop exposures at or near the site. For jointed rock which has been infilled with soil, the joint fill material should be sampled for laboratory shear

strength testing. Soil samples and rock cores collected during the site investigation should be preserved and made available to the designer and the contractor during the design and bidding phase of a project, respectively.

3.3.3 Groundwater

The groundwater table and any perched groundwater zones must be evaluated as part of a subsurface investigation program. The presence of ground water affects overall stability of the system, lateral pressures applied to the wall facing, vertical uplift forces on structures, drainage system design, watertight requirements at anchor connections, corrosion protection requirements, and construction procedures. At a minimum, the following items need to be considered for anchored systems that will be constructed within or near the groundwater table:

- average high and low groundwater levels;
- corrosion potential of ground anchors based on the aggressivity of the ground water;
- soil and/or rock slope instability resulting from seepage forces;
- necessity for excavation dewatering and specialized drilling and grouting procedures; and
- liquefaction potential of cohesionless soils.

For tiedown structures designed to resist uplift forces, unanticipated changes in groundwater levels can result in excessive consolidation settlement and a resulting decrease in ground anchor loads for cases in which the groundwater level decreases. For cases in which the groundwater level increases, ground anchor loads may increase above design loads.

Groundwater level information is often obtained by observation of the depth to which water accumulates in an open borehole at the time of, or shortly after, exploration. It is important to allow sufficient time to pass after borehole excavation so that water levels can reach equilibrium. Water levels in the subsurface may be measured more accurately using piezometers or observation wells. Water level measurements can be made over a duration of time to obtain an indication of potential water level fluctuations.

3.4 LABORATORY SOIL AND ROCK TESTING

3.4.1 General

Laboratory testing of soil and rock samples recovered during subsurface exploration is often performed to evaluate specific properties necessary for the design of an anchored system. In this section, laboratory tests typically performed to evaluate properties of soil and rock materials are presented along with appropriate ASTM and AASHTO testing specifications.

3.4.2 Classification and Index Properties

All soil samples taken from borings and rock core samples should be visually identified in the laboratory and classified according to ASTM D2488 and ASTM D2487 or the Unified Rock Classification System (URCS). Index soil properties used in the analysis and design of anchored systems include unit weight, moisture content, gradation, and Atterberg limits. Unit weights of foundation material and retained soil are used in evaluating earth pressures and in evaluating the external stability of the anchored system. Moisture content (ASTM D2216) information and Atterberg limits (ASTM D4318; AASHTO T89, T90) may be used with existing correlations to estimate compressibility and shear strength of in situ clayey soils and to evaluate the suitability of ground anchors in cohesive soils. In addition, the presence of organic materials should be determined by either visual description or according to ASTM D2974. The results of soil grain size distribution testing (ASTM D422; AASHTO T88) can be used to develop appropriate drilling and grouting procedures for ground anchors and to identify potentially liquefiable soils.

3.4.3 Shear Strength

Unconfined compression (ASTM D2166; AASHTO T208), direct shear (ASTM D3080; AASHTO T236), or triaxial compression (ASTM D4767; AASHTO T234) testing are typically performed to evaluate soil shear strength. Total stress and effective stress strength parameters of cohesive soils are typically evaluated from the results of undrained triaxial tests with pore pressure measurements. For permanent anchor applications involving cohesive soils, both undrained and drained strength parameters should be obtained, and the design of the anchored system should consider both short-term and long-term conditions. For critical applications involving cohesionless soils, direct shear or triaxial compression testing can be used to evaluate drained shear strength. Typically, however, drained shear strength of cohesionless soil is usually evaluated based on correlations with in situ test results (e.g., SPT and CPT). The selection of design soil shear strengths for anchored systems is described in chapter 4.

Laboratory strength testing of intact rock samples is not often performed for anchored system applications. For the actual field conditions, the strength of the rock mass is typically controlled by discontinuities. If, however, no adverse planes of weakness exist, the compressive strength of the intact rock, evaluated using unconfined compression (ASTM D2938), direct shear (ASTM D5607), or triaxial compression (ASTM D2664; AASHTO T226) testing, may be used to estimate ultimate bond stress (see PTI 1996).

3.4.4 Consolidation

Settlement analyses are not commonly performed for anchored systems constructed in stiff soils and cohesionless soils, but should be performed for structures subjected to groundwater drawdown (both during construction and for long-term conditions) that are constructed in compressible soils. Excessive settlement in these applications may be detrimental to nearby structures and these settlements may result in long-term lateral movements of anchored systems that exceed tolerable limits. The results of index tests including moisture content and Atterberg limits can be used for initial evaluation of settlement parameters. Results of one-dimensional consolidation (ASTM D2435; AASHTO T216) tests are used to evaluate the parameters necessary for a settlement analysis.

3.4.5 Electrochemical Criteria

For permanent anchored systems, the aggressiveness of the ground must be evaluated. Aggressive ground conditions usually do not preclude use of anchored systems if proper corrosion protection for the anchored system is provided. Corrosion potential is of primary concern in aggressive soil applications and is evaluated based on results of tests to measure the following properties: (1) pH (ASTM G51; AASHTO T289); (2) electrical resistivity (ASTM G57; AASHTO T288); (3) chloride content (ASTM D512; AASHTO T291); and (4) sulfate content (ASTM D516; AASHTO T290). Detailed information on ground anchor corrosion and corrosion protection measures is described in chapter 6.

3.5 IN SITU SOIL AND ROCK TESTING

In situ testing techniques are often used to estimate several of the soil properties previously introduced in section 3.4. There are in situ testing techniques which can be used to estimate rock properties, although the use of in situ testing in rocks is not as widespread as the use in soils.

The SPT is the most common in situ geotechnical test used in evaluating the suitability of ground anchors in cohesionless soils. The SPT blowcount value N can be used to estimate the relative density (see table 1) and shear strength of sandy soils. The advantage of the SPT over other in situ tests is that its use is widespread throughout the U.S. and a disturbed sample can be obtained for visual identification and laboratory index testing. For cohesionless soils, SPT N<10 may indicate that the ground is not suitable for ground anchors. SPT blowcounts may be used to evaluate the consistency of cohesive soil strata (see table 1), but not as a reliable indication of shear strength.

Table 1. Soil density/ consistency description based on SPT blowcount values (after AASHTO, 1988).

Cohesionless Soils		Cohesive Soils	
Relative Density	SPT N (blows/300 mm)	Consistency	SPT N (blows/300 mm)
Very loose	0-4	Very soft	0-1
Loose	5-10	Soft	2-4
Medium dense	11-24	Medium stiff	5-8
Dense	25-50	Stiff	9-15
Very dense	> 51	Very stiff	16-30
		Hard	31-60
		Very hard	>61

Other in situ testing procedures may be used to evaluate the suitability of ground anchors for a particular type of ground. These include: (1) CPT; (2) vane shear test (FVT) (ASTM D2573); (3) pressuremeter test (PMT) (ASTM D4719); and (4) flat plate dilatometer test (DMT). The following studies and reports by FHWA have been devoted to the use of in situ testing techniques in soil:

- Cone Penetration Test (FHWA-SA-91-043, 1992);
- Pressuremeter Test (FHWA-IP-89-008, 1989); and

- Flat Plate Dilatometer Test (FHWA-SA-91-044, 1992).

Basic information on these tests is summarized in table 2. Empirical correlations have been developed and may be used to obtain a preliminary estimate of property values. These correlations are published elsewhere (e.g., Kulhawy and Mayne, 1990). In many parts of the country, correlations have been developed for these tests in recognition of local soils and local conditions.

Table 2. Summary of common in situ tests for soils.

Type of Test	Suitable for	Not suitable for	Properties that can be estimated
SPT	sand	soft to firm clays, gravels	Stratigraphy, strength, relative density
CPT	sand, silt, and clay	gravel	Continuous evaluation of stratigraphy, strength of sand, undrained shear strength of clay, relative density, in situ stress, pore pressures
FVT	soft to medium clay	sand and gravel	undrained shear strength
PMT	soft rock, dense sand, nonsensitive clay, gravel, and till	soft, sensitive clays, loose silts and sands	strength, K_o, OCR, in situ stress, compressibility, hydraulic conductivity, elastic shear modulus
DMT	sand and clay	gravel	soil type, K_o, OCR, undrained shear strength, and elastic modulus

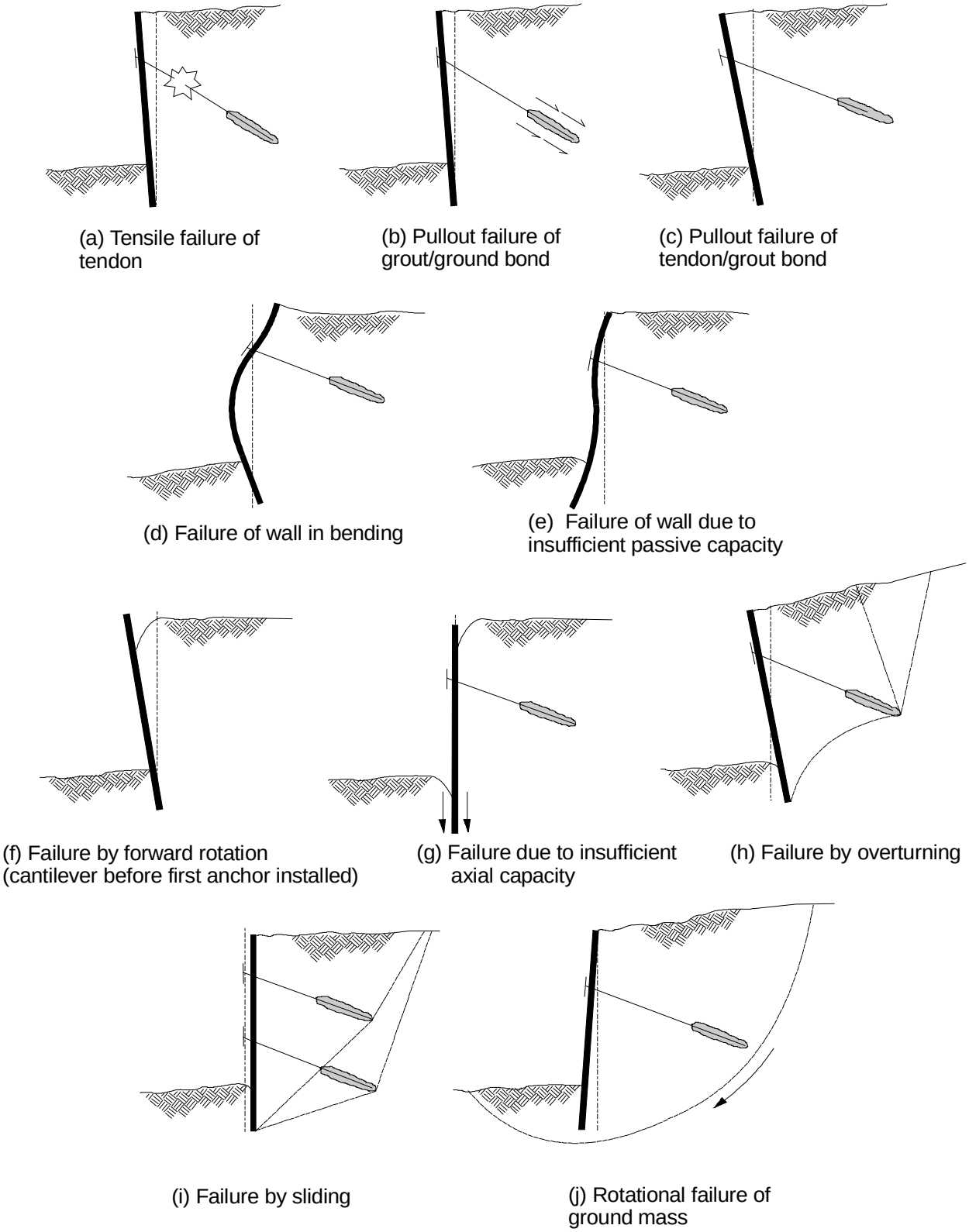


Figure 11. Potential failure conditions to be considered in design of anchored walls.

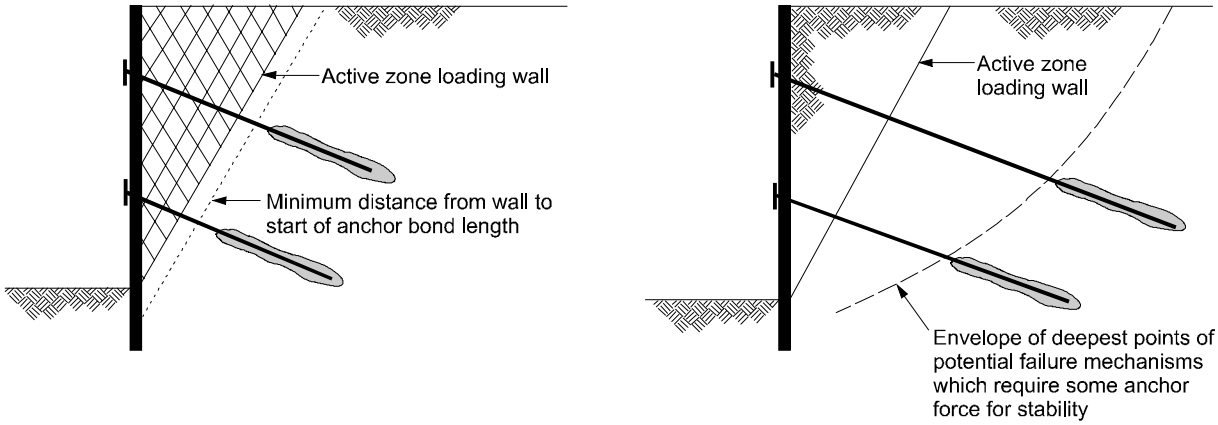


Figure 12. Contribution of ground anchors to wall stability.

4.2 FAILURE MECHANISMS OF ANCHORED SYSTEMS

4.2.1 General

Many different types of anchored systems can usually fulfill the needs of a particular project. To achieve maximum economy, the objective of the designer is to specify only those parameters that are necessary for long-term stability of the anchored system and to leave the final selection of the anchor details to the contractor. Anchor system performance is evaluated by testing each installed anchor at loads that exceed the design load. To determine the parameters that should be specified, the designer must consider various possible failure mechanisms.

4.2.2 Failure Mechanisms of the Ground Anchor

There are several possible failure mechanisms of ground anchors. These are usually caused by excessive static loading of an anchor. Excessive loads can be related to: (1) tension placed in the anchor during load testing or at lock-off; (2) excavation sequence; (3) surcharge by construction materials or equipment; (4) construction of adjacent structures; or (5) a combination of these causes. Ground anchor failure mechanisms may involve the steel tendon, the ground mass, the ground-grout zone, and the grout-tendon zone, as described subsequently.

Failure of the Steel Tendon

As the anchor is loaded, the steel tendon component of the anchor is stressed in tension. If the applied load is greater than the structural capacity of the tendon, failure is inevitable. Therefore, a factor of safety must be used with respect to structural failure of the steel. It is recommended that the tendon load not exceed 60 percent of the specified minimum tensile strength (SMTS) for final design and 80 percent of SMTS for temporary loading conditions (e.g., loading during testing).

Failure of the Ground Mass

Failure of the soil mass, as referred to herein, involves failure resulting from anchor loads, not external forces such as landslides which potentially introduce excessive static loading to the anchor. For shallow soil anchors, failure of the ground mass is characterized by uplift of a mass of soil in front of the anchor bond zone followed by pullout of the bond zone. A shear surface develops in the soil mass ahead of the anchor as increasing stresses cause complete mobilization of resistance in the anchor bond zone. The failure surface simulates a passive earth pressure failure. Practically, failure of the soil mass is not a factor for anchors embedded more than 4.5 m below the ground surface.

For rock anchors, the likely plane of failure for shallow installations in sound bedrock is along a cone generated at approximately a 45 degree angle from the anchorage. In fractured or bedded rock, the cone shape and size varies with the distribution of bedding and cleavage planes and the grout take in fissures. Even in fractured rock, rock mass failure seldom occurs in anchors embedded more than 4.5 m below ground because the bond strength between the rock and grout or the grout and tendon is much less than the rock strength.

Failure of the Ground-Grout Bond

Ground anchors mobilize skin friction between the anchor bond zone and the ground. In general, this bond is dependent on the normal stress acting on the bond zone grout and the adhesion and friction mobilized between the ground and the grout. Anchors which are underreamed may also develop the base resistance of the increased annular area.

In general, the ground-grout bond is mobilized progressively in uniform soil or rock as the stress is transferred along the bond length. Initially, as the anchor is stressed, the portion of the bond length nearest the load application elongates and transfers load to the ground. As the resistance in this portion of the bond length is mobilized, stress is transferred farther down. During this process, the anchor continues to elongate to mobilize deeper bond zones. Once the stress is transferred to the end of the bond zone and the ultimate ground-grout bond is exceeded, anchor failure by pullout occurs. Anchors which have been improperly grouted such that a column of grout exists between the bearing plate or wall and the top of the bond zone will show no load transfer into the bond length when the load is increased. Factors influencing stress transfer for small diameter ground anchors with bond lengths in a uniform soil are summarized in table 3.

Experience has shown that increasing the bond length for typical soil anchors beyond 9 to 12 m does not result in significant increases in resistance. A possible reason for this observation is that after the load has been transferred that distance down the bond zone, sufficient movement at the ground-grout interface has occurred in the upper bond length to decrease the upper ground-grout interface resistance to residual strength levels. Bond lengths greater than 12 m may be used effectively provided special procedures are used to bond the tendon to the grout such that capacity can be mobilized along the longer length.

Table 3. Typical factors influencing bond stress transfer for small diameter ground anchors.

	Soil Type	
Factor	Cohesionless	Cohesive
Soil Properties	Friction angle and grain size distribution.	Adhesion and plasticity index.
Drilling Method	Driven casing increases normal stress and friction.	Drilling without casing or with fluids decreases capacity.
Bond Length	Steady increase in anchor capacity to 6 m with moderating increases to 12 m.	Steady increase in anchor capacity for soils with undrained strength less than 96 kPa.
Hole Diameter	Slight increase in anchor capacity to 100 mm.	Anchor capacity increases to 300 mm.
Grout Pressure	Anchor capacity increases with increasing pressure.	Anchor capacity increases only with stage grouting. High initial pressures should be avoided.

Note: To ensure ground-grout bond, the drill hole should be cleaned and the grout should be placed as quickly as possible after the hole has been drilled.

Failure at the ground-grout interface may also be characterized by excessive deformations under sustained loading (i.e., creep). Soil deposits that are potentially susceptible to excessive creep deformations include: (1) organic soils; (2) clay soils with an average liquidity index (LI) greater than 0.2; (3) clay soils with an average liquid limit (LL) greater than 50; and (4) clay soils with an average plasticity index (PI) greater than 20. Conservative anchor design loads and working bond stress values are recommended for design involving permanent anchor installations in such soils, unless based on results from a predesign or preproduction test program. Predesign and preproduction test programs are described in section 5.3.6.

The LL, plastic limit (PL) and moisture content (w_n) of a clay soil are commonly measured clayey soil index properties. The LI indicates where the moisture content of the clay falls within the range between the plastic and liquid limits. Liquidity index for a soil is defined as:

$$LI = \frac{w_n - PL}{PI} \qquad \text{(Equation 1)}$$

A low LI indicates that the moisture content is relatively close to the PL of the soil, indicating a potentially overconsolidated or stiff soil. A LI close to 1.0 indicates that the moisture content is relatively close to the LL for the soil, indicating a potentially normally consolidated or soft soil.

Failure of Grout-Tendon Bond

The bond between the grout and steel tendon must not be exceeded if the full strength of the supporting ground is to be mobilized. The failure mechanism of the grout-tendon bond involves three components: (1) adhesion; (2) friction; and (3) mechanical interlock. Adhesion is the physical coalescence of the microscopically rough steel and the surrounding grout. This initial bond is replaced by friction after movement occurs. The friction depends on the roughness of the steel surface, the normal stress, and the magnitude of the slip. Mechanical interlock consists of the grout mobilizing its shear strength against major tendon irregularities such as ribs or twists. This interlock is the dominant bond mechanism for threadbars where the ultimate strength of the bar may be developed in a short embedment in the grout. The grout-tendon bond on smooth steel tendons is mobilized progressively in a fashion similar to the ground-grout bond. "Slip" occurs only after the maximum intensity of grout-tendon bond resistance has been mobilized over nearly the total bond length. After this slip, the tendon will only offer frictional resistance (amounting to about half the maximum total resistance obtained) to further elongation. Experience has shown that:

- Bond resistance of the grout to the tendon is not linearly proportional to the compressive strength of the grout. Although the bond strength usually increases as the compressive strength of the grout increases, the ratio of bond to ultimate strength decreases with increasing grout strengths. For example, a 17.2 MPa bond strength for 27.6 MPa grout may only increase by 12 percent to 19.3 MPa when the grout strength is increased by 25 percent to 34.5 MPa.

- Bond resistance developed by added embedment increases as the tendon length increases, but at reduced unit values.

- Flaky rust on bars lowers the bond, but wiping off the loosest rust produces a rougher surface which develops a bond equal to or greater than an unrusted bar. Obviously pitted bars cannot be accepted even though the grout tendon bond may be adequate.

- The loose powdery rust appearing on bars after short exposures does not have a significant effect on grout-tendon bond.

Mill test reports should be requested by the owner for each lot used to fabricate the tendons. Test reports should include the results of bond capacity tests performed in accordance with the prestressing strand bond capacity test described in ASTM A981. ASTM A981 provides a standard test method to evaluate the bond strength between prestressing strand and cement grout. This specification was developed in 1997 in response to an industry initiative concerning the effects of certain residues from the manufacturing process that appeared to reduce the bond between the strand and the cement grout.

4.2.3 Failure of Soldier Beams

Soldier beams are subject to both lateral and vertical loads from the retained soil mass and the forces imparted from prestressing the anchors. The lateral resistance of the soldier beam is most critical during stressing and testing of the first anchor level, and for the final excavation condition when all wall loads have been applied. In the former case, stressing of the upper anchor to the test load is

often done at shallow depths where the available passive resistance behind the soldier beam is low. Soldier beam deflections can be minimized in design by applying a safety factor of 1.5 to the passive resistance and in construction by ensuring that the upper lagging is tight against the soil and that the soil behind the soldier beam has not been removed. For the final excavation condition, the passive resistance in front of the wall must be adequate to restrain the toe of the soldier beam for long term wall loadings and for any future undercuts of the area in front of the wall.

Load transfer of the vertical loads on the soldier beams is more complex than for simple deep foundation elements. As the excavation for the wall deepens, vertical load is transferred above grade to the soil behind the back face of the soldier beam, but the magnitude of the load that is transferred is difficult to estimate. Theoretically, if adequate downward movement of the soldier beam (relative to the soil) occurs, load will be transferred to the soil mass behind the wall. However, this load transfer also results in the development of a negative interface wall friction angle for the active block of soil behind the wall resulting in an increase in the earth pressures behind the wall. In this document, it is assumed that no load transfer (i.e., interface wall friction angle = 0°) occurs above the excavation base since: (1) relative movements between the soldier beam and soil are small; (2) removal of soil from the excavation face may reduce the "bond" between the soldier beam and soil; and (3) the actual amount of load transferred is usually small. Other design procedures which utilize load transfer above the excavation base can be used if appropriate documentation can be provided on relative movements required to develop load transfer.

Vertical load capacity below the excavation base is calculated using common procedures for deep foundations (i.e., driven piles or drilled shafts). Two issues, however, are unique to evaluating axial capacity for soldier beam walls and must be considered. These issues are described below.

- Stress relief in front of the wall caused by excavation will reduce the effective stresses acting on the embedded portion of the soldier beam. This reduction in stress may vary with depth based on the width of the excavation. Common practice is to assume the effective stress is equal to the average of the effective stress imparted by the retained soil height behind the wall and by the depth of the soil in front of the wall.

- Structural sections are commonly placed in predrilled holes which are filled with concrete. In the case of a structural concrete filling, it is usually assumed that axial and lateral load are shared by the steel and the concrete and lateral capacity computations may be performed on the basis of the hole diameter. However, in the case of nonstructural (i.e., "lean-mix") concrete, the shear capacity between the structural section and the lean-mix concrete fill may not be adequate to provide load sharing between the steel and the concrete. This shear capacity should therefore be checked as part of the determination of axial and lateral soldier beam capacity.

4.2.4. Failure of Lagging

In general, the timber lagging is only used for support of temporary loads applied during excavation, however, pressure-treated timber lagging has been used to support permanent loads. The contribution of the temporary lagging is not included in the structural design of the final wall face. Temporary timber lagging is not designed by traditional methods, rather lagging is sized from charts

developed based on previous project experience which accounts for soil arching between adjacent soldier beams (FHWA-RD-75-130, 1976).

4.3 SELECTION OF SOIL SHEAR STRENGTH PARAMETERS FOR DESIGN

4.3.1 General

The purpose of this section is to provide guidance on selection of soil shear strength parameters for anchored system design. Shear strength parameters of the retained soil are required to evaluate earth pressures acting on a wall, axial and lateral capacity of the embedded portion of a wall, and external stability of an anchored system. The evaluation of shear strength parameters for temporary walls constructed in normally to lightly overconsolidated soft to medium clays and for temporary and permanent walls constructed in heavily overconsolidated stiff to hard clays is emphasized herein.

4.3.2 Drained Shear Strength of Granular Soils

The drained shear strength of granular soil is conventionally represented by a drained effective stress friction angle, ϕ'. Because the undisturbed sampling of granular soil deposits is difficult, the representative friction angle to be used for wall design may be estimated using the results of in situ penetration tests such as the SPT and the CPT.

4.3.3 Undrained Shear Strength of Normally Consolidated Clay

Instability under undrained conditions develops mainly under the condition of contractive shear, i.e., the mechanism of deformation which attempts to mobilize frictional shearing resistance also causes the soil to want to contract under the prevailing confining stresses. This tendency to contract during shear is typical for normally to lightly overconsolidated soft to medium clay soils. Since this tendency cannot be realized, due to the clay soil permeability in relation to the rate of shearing, positive porewater pressures are generated in the soil which reduce the effective stress and hence the mobilized frictional shearing resistance. In such cases the short term undrained shearing resistance in the soil is less than would have been the case if drainage (contraction of the soil volume) could have occurred. The short term condition is critical for temporary anchored walls constructed in normally to lightly overconsolidated clay soils.

The undrained shear strength, S_u, may be determined by in situ (e.g., CPT, FVT) and laboratory testing methods. A detailed discussion of the methods used to evaluate S_u is beyond the scope of this document, but this information may be found elsewhere (e.g., Kulhawy and Mayne, 1990). Typically S_u is evaluated using laboratory triaxial tests on nominally undisturbed cohesive soil samples at the natural water content of the soil. The preferred method to evaluate the undrained strength in the laboratory is through consolidated undrained triaxial testing with pore pressure measurements. The use of unconfined compression tests and/or unconsolidated undrained triaxial tests may lead to erroneous measured strengths due to sampling disturbance and the omission of a reconsolidation phase.

The undrained shear strength is not a fundamental property of a soil and is affected by the mode of testing, boundary conditions, rate of loading, initial stress state, and other variables. Consequently, the measured undrained shear strength should be different depending on the type of test performed. The designer should consider how the actual undrained shear strength mobilized under field loading conditions differs from that measured using laboratory or in situ testing methods. For example, for a temporary anchored wall in soft to medium clay, the undrained shear strength used to evaluate the earth pressures acting on the wall may be determined from a triaxial compression test. The lateral capacity of the wall toe, however, is more appropriately evaluated using the undrained strength from a triaxial extension test. The extension loading path more accurately approximates the unloading caused by soil excavation as compared to a compression loading path and, more importantly, experience has shown that strength in the passive zone (inside the excavation) can be less than that in the active zone (in the retained ground) for certain clay soils. Alternatively, correlations may be used to "convert" the undrained strength measured in a conventional triaxial compression test into an undrained strength for a different loading path (see Kulhawy and Mayne, 1990).

4.3.4 Undrained Shear Strength of Overconsolidated Clay

In clay soils subjected to unloading conditions that result from excavation to form an anchored wall, the soil attempts to expand as it mobilizes frictional shearing resistance. This is resisted causing negative porewater pressure to be developed which increases the effective stress in the soil and hence increases the mobilized frictional shearing resistance. Thus, in an overconsolidated clay subject to excavation, the short-term (undrained) strength and stability potentially exceeds that which would apply once drainage has occurred.

Temporary and permanent anchored walls are commonly constructed in stiff to hard overconsolidated clays. Heavily overconsolidated clay soils are often fissured. Due to the fissured nature of overconsolidated clays, which can permit relatively rapid local drainage at the level of the discontinuities in the clay, it is generally difficult to define with any certainty the period of time during which the enhanced undrained shear strength of the clay may reliably be assumed to apply. Therefore, in overconsolidated clays, design analyses should be performed in terms of drained, effective stress parameters. Drained strength parameters for overconsolidated clays are discussed subsequently.

4.3.5 Drained Shear Strength of Overconsolidated Clay

The behavior of an overconsolidated stiff clay can be illustrated as shown in figure 13. As the sample is sheared under drained conditions, the displacement of the soil sample is relatively uniform until the peak stress, τ_p, is reached. After the peak, displacements begin to concentrate on the newly formed failure plane or discontinuity, and the shear stress reduces to τ_d. The shear strength, τ_d, of the newly formed discontinuity is approximately equal to the shear strength of the same clay constituents in a normally consolidated state (i.e., the fully softened strength), such as that produced by laboratory consolidation from a slurry. For relatively high plasticity clays, further displacement beyond that corresponding to the fully softened strength results in a continued reduction in shear stress, and, eventually, at very large displacements along a major discontinuity, the residual strength of the clay soil, τ_r, is reached.

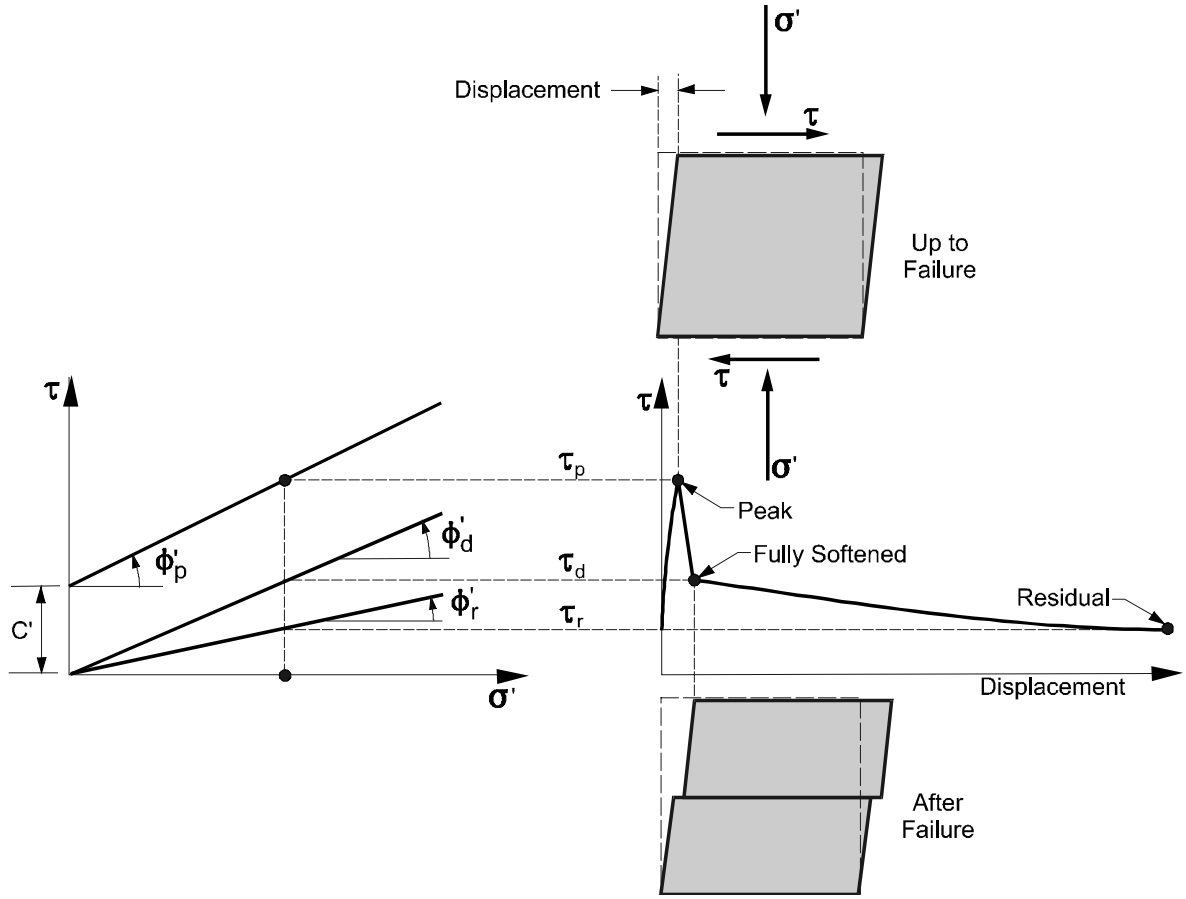


Figure 13. Simplified drained stress-displacement relationship for a stiff clay (modified after Construction Industry Research and Information Association (CIRIA), 1984).

For anchored systems in stiff to hard overconsolidated clays, the designer must decide as to which strength, i.e., peak, fully softened, or residual, should be used for design. Since the additional strength at peak resulting from cohesion (c' in figure 13) tends to reduce relatively rapidly with increasing strain beyond peak, soil deformations associated with flexible anchored walls may be sufficient to appreciably reduce this cohesion. Therefore, unless local experience indicates that a particular value of cohesion can be reliably accounted for, zero cohesion should be used in the analyses of anchored walls in stiff to hard fissured clays for long-term (drained) conditions. Conservative drained shear strength for analysis of anchored walls is therefore the fully softened strength. This strength may be evaluated using triaxial compression testing with pore pressure measurements.

Residual strengths should be used for anchored systems that are designed for a location in which there is evidence of an existing failure surface within the clay (e.g., an anchored system used to stabilize an active landslide). For these conditions, assume that sufficiently large deformations have occurred to reduce the strength to a residual value. A study by Stark and Eid (1994) presents a correlation between residual friction angle and the clay size fraction and liquid limit for clay soils.

4.4 EARTH PRESSURES

4.4.1 General

A wall system is designed to resist the lateral earth pressures and water pressures that develop behind the wall. Earth pressures develop primarily as a result of loads induced by weight of the retained soil, earthquake ground motions, and various surcharge loads. For purposes of anchored wall system design, three different lateral earth pressure conditions are considered: (1) active earth pressure; (2) passive earth pressure; and (3) at-rest earth pressure.

The distinction between actual ground behavior and conventional design assumptions is particularly important when considering earth pressures. The simple linear assumptions about active and passive pressures based on theoretical analyses are a considerable simplification of some very complex processes which depend on the following factors: (1) the mode of wall movement (rotation, translation); (2) wall flexibility; (3) soil stiffness and strength properties; (4) horizontal prestress in the ground; and (5) and wall/soil interface friction. For anchored wall systems with flexible wall elements, semi-empirical "apparent earth pressure envelopes" are commonly used.

4.4.2 Active and Passive Earth Pressure

Active and passive horizontal earth pressures may be considered in terms of limiting horizontal stresses within the soil mass, and, for purposes of this discussion, a smooth (i.e., zero wall friction) wall retaining ground with a horizontal backslope is considered (figure 14); this case defines Rankine conditions. Consider an element of soil in the ground under a vertical effective stress, σ_v' (figure 15). In considering the potential movements of a retaining wall, the element may be brought to failure in two distinct ways that are fundamentally important in the context of retaining wall design. The horizontal soil stress may be increased until the soil element fails at B, when the stress reaches its maximum value $\sigma'_{h\,(max)}$. This scenario will occur when significant outward movement of the wall increases the lateral earth pressure in the soil at the base of the wall (see figure 14). Similarly, the horizontal stress may be reduced until failure at A, when the stress reaches its minimum value $\sigma'_{h\,(min)}$. This scenario models the outward movement which reduces the lateral earth pressures behind the wall (see figure 14).

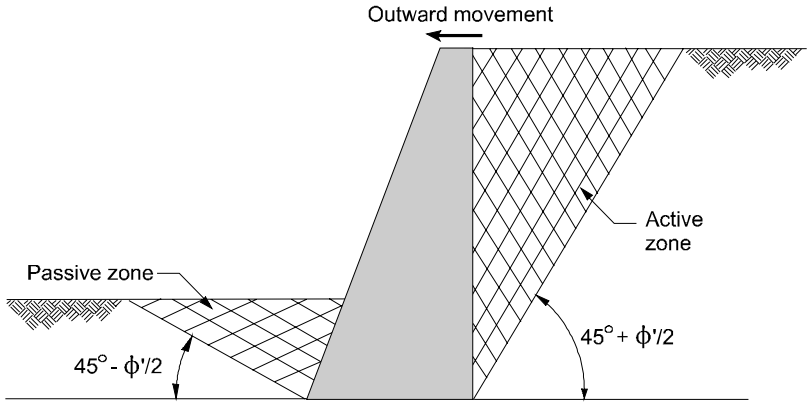


Figure 14. Mobilization of Rankine active and passive horizontal pressures for a smooth retaining wall.

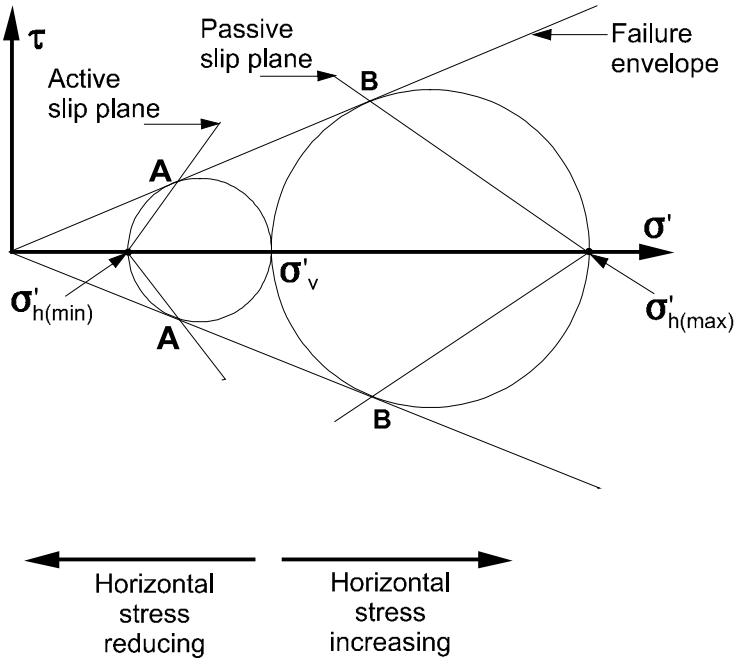


Figure 15. Limiting active and passive horizontal pressures.

The geometry of figure 15 gives the following two relationships:

$$\frac{\sigma'_{h\,(min)}}{\sigma'_v} = K_A = \frac{1-\sin\phi'}{1+\sin\phi'} = \tan^2(45-\phi'/2) \qquad \text{(Equation 2)}$$

$$\frac{\sigma'_{h\,(max)}}{\sigma'_v} = K_P = \frac{1+\sin\phi'}{1-\sin\phi'} = \tan^2(45+\phi'/2) \qquad \text{(Equation 3)}$$

where K_A is the active earth pressure coefficient and K_P is the passive earth pressure coefficient. The definitions of K_A and K_P, based on equations 2 and 3, are consistent with a Rankine analysis for a cohesionless (i.e., c=0) retained soil.

For a cohesive soil defined by effective stress strength parameters ϕ' and c', the active and passive earth pressure coefficients are:

$$K_A = \tan^2(45 - \phi'/2) - \frac{2c'}{\sigma'_v}\tan(45 - \phi'/2) \qquad \text{(Equation 4)}$$

$$K_P = \tan^2(45 + \phi'/2) + \frac{2c'}{\sigma'_v}\tan(45 + \phi'/2) \qquad \text{(Equation 5)}$$

For the undrained case with $\phi = 0$ and $c = S_u$, the total stress active and passive earth pressure coefficients are:

$$K_{AT} = 1 - \frac{2S_u}{\sigma_v} \qquad \text{(Equation 6)}$$

$$K_{PT} = 1 + \frac{2S_u}{\sigma_v} \qquad \text{(Equation 7)}$$

where σ_v is the total vertical stress.

For most anchored wall applications, the effect of wall friction on active earth pressures is relatively small and is often ignored. The active earth pressure coefficient, K_A, may be evaluated using the appropriate equations from above or, for more general cases, from the lower part of figure 16 or figure 17. The earth pressure coefficients depicted in figure 16 and figure 17 are based on the assumption of log-spiral shaped failure surfaces for the active and passive sides of the wall. To evaluate the passive earth pressure coefficient, K_P, the upper part of figure 16 or 17 should be used.

It is acknowledged that in addition to the Rankine equations and the log-spiral method, a third closed-form technique, herein referred to as the Coulomb method, is often used to calculate lateral earth pressures. For this method, equations are available to calculate K_A and K_P (NAVFAC, 1982). While calculations of K_A are considered to be reasonable, the Coulomb method is unreliable for evaluating passive earth pressures since the planar shape of the assumed Coulomb failure surface is

Figure 16. Active and passive earth pressure coefficients (effect of wall inclination).

Figure 17. Active and passive earth pressure coefficients (effect of backslope inclination).