Structural Foundation Designers' Manual

Curtins consulting

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Second Edition revised by

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Dedication

This book is dedicated to Bill Curtin who died suddenly in November 1991 following a short illness.

Bill's contribution to the book at that time was all but complete and certainly well ahead of his co-authors. It is a source of sadness that Bill did not have the pleasure and satisfaction of seeing the completed publication but his input and enthusiasm gave his co-authors the will to complete their input and progress the book to completion.

Contents

Preface xi

Preface to First Edition xii

The Book's Structure and What It Is About xiii

Acknowledgements xiv

Authors' Biographies xv

Notation xvi

PART 1: APPROACH AND FIRST CONSIDERATIONS 1

1 Principles of Foundation Design 3
- 1.1 Introduction 3
- 1.2 Foundation safety criteria 3
- 1.3 Bearing capacity 4
  - 1.3.1 Introduction 4
  - 1.3.2 Bearing capacity 4
  - 1.3.3 Presumed bearing value 4
  - 1.3.4 Allowable bearing pressure 5
  - 1.3.5 Non-vertical loading 5
- 1.4 Settlement 6
- 1.5 Limit state philosophy 7
  - 1.5.1 Working stress design 7
  - 1.5.2 Limit state design 7
- 1.6 Interaction of superstructure and soil 8
  - 1.6.1 Example 1: Three pinned arch 8
  - 1.6.2 Example 2: Vierendeel superstructure 8
  - 1.6.3 Example 3: Prestressed brick diaphragm wall 8
  - 1.6.4 Example 4: Composite deep beams 9
  - 1.6.5 Example 5: Buoyancy raft 9
- 1.7 Foundation types 9
  - 1.7.1 Pad foundations 10
  - 1.7.2 Strip footings 10
  - 1.7.3 Raft foundations 10
  - 1.7.4 Piled foundations 11
- 1.8 Ground treatment (geotechnical processes) 11
- 1.9 Changes of soil properties during excavation 12
- 1.10 Post-construction foundation failure 12
- 1.11 Practical considerations 13
  - 1.11.1 Example 6: Excavation in waterlogged ground 13
  - 1.11.2 Example 7: Variability of ground conditions 13
  - 1.11.3 Example 8: Reliability of the soils investigation 13
  - 1.11.4 Example 9: Deterioration of ground exposed by excavation 13
  - 1.11.5 Example 10: Effect of new foundation on existing structure 14
- 1.12 Design procedures 14
- 1.13 References 14

2 Soil Mechanics, Lab Testing and Geology 15

A: Soil mechanics 15
- 2.1 Introduction to soil mechanics 15
- 2.2 Pressure distribution through ground 15
- 2.3 Bearing capacity 17
  - 2.3.1 Introduction to bearing capacity 17
  - 2.3.2 Main variables affecting bearing capacity 19
  - 2.3.3 Bearing capacity and bearing pressure 19
  - 2.3.4 Determination of ultimate bearing capacity 20
  - 2.3.5 Safe bearing capacity – cohesionless soils 21
  - 2.3.6 Safe bearing capacity – cohesive soils 22
  - 2.3.7 Safe bearing capacity – combined soils 22
- 2.4 Settlement 22
  - 2.4.1 Introduction to settlement 22
  - 2.4.2 Void ratio 23
  - 2.4.3 Consolidation test 23
  - 2.4.4 Coefficient of volume compressibility 24
  - 2.4.5 Magnitude and rate of settlement 25
  - 2.4.6 Settlement calculations 25
- 2.5 Allowable bearing pressure 26
- 2.6 Conclusions 26

B: Laboratory testing 26
- 2.7 Introduction to laboratory testing 26
- 2.8 Classification (disturbed sample tests) 26
  - 2.8.1 Particle size and distribution 26
  - 2.8.2 Density 27
  - 2.8.3 Liquidity and plasticity 29
  - 2.8.4 General 29
- 2.9 Undisturbed sample testing 29
  - 2.9.1 Moisture content 29
  - 2.9.2 Shear strength 29
  - 2.9.3 Consolidation tests (oedometer apparatus) 29

2.9.4 Permeability tests 32
2.9.5 Chemical tests 32
2.10 Summary of tests 32
2.11 Analysis of results 37
2.12 Final observations on testing 37

C: Geology 37
2.13 Introduction to geology 37
2.14 Formation of rock types 38
2.15 Weathering of rocks 38
2.16 Agents of weathering 38
2.16.1 Temperature 38
2.16.2 Water 38
2.16.3 Wind 38
2.16.4 Glaciation 38
2.17 Earth movement 38
2.17.1 Folds, fractures and faults 38
2.17.2 Dip and strike 39
2.17.3 Jointing 39
2.17.4 Drift 39
2.18 Errors in borehole interpretation 40
2.19 Geophysical investigation 42
2.20 Expert knowledge and advice 42
2.21 References 42

3 Ground Investigation 43
3.1 Introduction 43
3.2 The need for investigation 44
3.2.1 The designer's need 44
3.2.2 The contractor's need 45
3.2.3 The client's need 45
3.2.4 Site investigation for failed, or failing, existing foundations 45
3.3 Procedure 45
3.3.1 Site survey plan 47
3.3.2 Study of existing information 47
3.3.3 Preliminary site reconnaissance and site *walkabout* 47
3.4 Soil investigation 48
3.4.1 Borehole layout 48
3.4.2 Trial pits layout 49
3.4.3 Hand augers 50
3.4.4 Boring 50
3.4.5 Backfilling of trial pits and boreholes 50
3.4.6 Soil sampling 50
3.4.7 Storage of samples 50
3.4.8 Frequency of sampling 50
3.4.9 Appointment of specialist soil investigator 51
3.5 Site examination of soils 52
3.6 Field (site) testing of soils 52
3.6.1 Standard Penetration Test (SPT) 52
3.6.2 Vane test 52
3.6.3 Plate bearing test 53
3.6.4 Pressuremeters 53
3.6.5 Groundwater (piezometers and standpipes) 53
3.6.6 Other field tests 55
3.7 Recording information – trial pit and borehole logs and soil profiles 55
3.8 Soil samples and soil profiles 56
3.9 Preliminary analysis of results 56
3.10 Site investigation report 61
3.10.1 Factors affecting quality of report 61
3.10.2 Sequence of report 62
3.10.3 Site description 62
3.10.4 The ground investigation 62
3.10.5 Results 62
3.10.6 Recommendations 62
3.11 Fills (made ground) 63
3.12 Legal issues 63
3.13 Time 64
3.14 Conclusions 64
3.15 Further information 65
3.16 References 65

PART 2: SPECIAL AND FURTHER CONSIDERATIONS 67

4 Topography and its Influence on Site Development 69
4.1 Introduction 69
4.2 Implications from surface observations 69
4.2.1 Changes in level, ground slopes and movements 69
4.2.2 Mounds, depressions and disturbed ground 70
4.2.3 Past or current activities 71
4.2.4 Vegetation 72
4.2.5 Surface ponding or watercourses 72
4.3 Effects on development arising from topographical features 73
4.3.1 Sloping sites 73
4.3.2 Slope stability 75
4.3.3 Groundwater 77
4.3.4 Settlement 78
4.4 Summary 79
4.5 References 79

5 Contaminated and Derelict Sites 80
5.1 Introduction 80
5.1.1 State of the art 80
5.1.2 Contamination implications 81
5.2 Redundant foundations and services 82
5.2.1 Identification 83
5.2.2 Sampling and testing 83
5.2.3 Site treatment 83
5.3 Chemical and toxic contamination 83
5.3.1 Part IIA risk-based approach 83
5.3.2 Soil Guideline Values 84
5.3.3 CLEA Model 84
5.3.4 Risk to humans and animals 85
5.3.5 Risks to plants and the wider ecosystem 89
5.3.6 Risk to the water environment 89
5.3.7 Risk to buildings and construction materials 89

5.3.8 Toxic contamination – site identification 91
5.3.9 Contaminant investigation 91
5.3.10 Sampling and testing 92
5.3.11 Site treatment 92
5.4 Foundation protection 93
5.5 Examples of site investigations on potentially contaminated sites 94
5.6 References 94

6 Mining and Other Subsidence 95
6.1 Introduction 95
6.2 Mechanics of mining subsidence 95
6.3 Methods of mining 97
6.3.1 Longwall workings 97
6.3.2 Pillar and stall workings (partial extraction methods) 97
6.3.3 'Bell-pits' 99
6.4 Associated and other workings 100
6.4.1 Abandoned mine shafts and adits 100
6.4.2 Fireclay and other clays 100
6.4.3 Iron ores 100
6.4.4 Other metals 100
6.4.5 Limestone 100
6.4.6 Salt 100
6.4.7 Chalk 100
6.5 Faulting 100
6.6 Natural and other cavities 100
6.6.1 *Dissolving* rock 100
6.6.2 *Dissolving* soils 100
6.7 Treatment of abandoned shallow workings 100
6.7.1 Introduction 100
6.7.2 Excavate and backfill 101
6.7.3 Partial and full grouting 101
6.8 Treatment of abandoned shafts 101
6.8.1 Capping 101
6.9 Effect of mining method and method of treatment 101
6.9.1 Introduction 101
6.9.2 Bell workings 101
6.9.3 Pillar and stall 102
6.9.4 Longwall workings 103
6.9.5 Rafts founded over longwall workings 103
6.10 Design principles and precautions in longwall mining subsidence areas 103
6.10.1 Introduction 103
6.10.2 Rafts and strips for low-rise, lightly loading buildings 104
6.10.3 Rafts for multi-storey structures or heavy industrial buildings 105
6.10.4 Jacking points 105
6.10.5 Service ducts 105
6.10.6 Piling 105
6.10.7 Articulated foundation 105
6.11 Superstructures 106
6.11.1 Introduction 106
6.11.2 Rigid superstructures 106
6.11.3 Flexible superstructures 106
6.12 Monitoring 107
6.13 References 107

7 Fill 108
7.1 Filled sites 108
7.1.1 Introduction 108
7.1.2 Movement and settlement 108
7.2 The container 108
7.2.1 The container surface 108
7.2.2 The container edges 108
7.2.3 The container base 110
7.2.4 The container sub-strata 110
7.3 Water 111
7.3.1 Effect of water on combustion 111
7.3.2 Effect of water on chemical solutions 111
7.3.3 Water lubrication 111
7.3.4 Water inundation 111
7.3.5 Organic decay 111
7.3.6 Information from water 111
7.4 The fill material 111
7.4.1 Introduction 111
7.5 Fill investigations 112
7.5.1 Special requirements 112
7.5.2 Suggested procedures 113
7.6 Settlement predictions 113
7.6.1 Settlement: fill only 113
7.6.2 Settlement: combined effects 115
7.7 The development and its services 116
7.7.1 Sensitivity 116
7.7.2 Treatment and solutions 117
7.7.3 New filling for development 118
7.8 Case examples 118
7.8.1 Introduction 118
7.8.2 Example 1: Movement of existing building on fill 118
7.8.3 Example 2: New development on existing colliery fill 119
7.8.4 Example 3: New development on new filling 120
7.8.5 Example 4: New developments on existing preloaded fill 120
7.8.6 Example 5: New development on existing backfilled quarry (purchase of coal rights) 121
7.8.7 Example 6: Development on new fill (prevention of flooding) 122
7.9 References 123
7.10 Further reading 123

8 Ground Improvement Methods 124
8.1 Introduction 124
8.2 Surface rolling 124
8.2.1 Introduction 124
8.2.2 Method 124
8.2.3 Soil suitability and variation 125
8.2.4 Site monitoring 125
8.3 Vibro-stabilization 126
8.3.1 Introduction 126
8.3.2 Working surfaces 127

8.3.3 Method 127
8.3.4 Vibro-compaction 128
8.3.5 Vibro-displacement 129
8.3.6 Vibro-replacement 129
8.3.7 Summary of vibro-stabilization 130
8.3.8 Design considerations – granular soils 130
8.3.9 Design considerations – cohesive soils 130
8.3.10 Testing 131
8.3.11 Vibro-concrete 131
8.4 Dynamic consolidation 133
8.4.1 Introduction 133
8.4.2 Method 133
8.4.3 Usage 133
8.4.4 Site checks 133
8.5 Preloading 133
8.5.1 Introduction 133
8.5.2 Method 134
8.5.3 Design of surcharge 134
8.5.4 Installation of drainage systems 134
8.6 Grout injections 135
8.6.1 Introduction 135
8.6.2 Loose soils 135
8.6.3 Swallow-holes 136
8.6.4 Shallow mining 136
8.6.5 Mine shafts, wells and bell-pits 136
8.7 Lime/cement stabilization 137
8.8 Reinforced soil 138
8.8.1 Introduction 138
8.8.2 Foundation applications 139
8.8.3 Patents 139
8.8.4 Research and development 139
8.9 Reference 139

PART 3: FOUNDATION TYPES: SELECTION AND DESIGN 141

9 Foundation Types 143
9.1 Introduction 143
9.2 Foundation types 143
9.3 Group one – strip and pad foundations 143
9.3.1 Strip footings 143
9.3.2 Masonry strips 143
9.3.3 Concrete strips – plain and reinforced 144
9.3.4 Concrete trench fill 145
9.3.5 Stone trench fill 145
9.3.6 Rectangular beam strips 145
9.3.7 Inverted T beam strips 145
9.3.8 Pad bases 147
9.3.9 Shallow mass concrete pads 147
9.3.10 Shallow reinforced concrete pads 147
9.3.11 Deep reinforced concrete pads 147
9.3.12 Deep mass concrete pads 147
9.3.13 Balanced pad foundations 148
9.3.14 Rectangular balanced pad foundations 148
9.3.15 Trapezoidal balanced pad foundations 148
9.3.16 Holed balanced pad foundations 148
9.3.17 Cantilever balanced pad foundations 149
9.4 Group two – surface spread foundations 149
9.4.1 Nominal crust raft 149
9.4.2 Crust raft 150
9.4.3 Blanket raft 150
9.4.4 Slip-plane raft 151
9.4.5 Cellular raft 151
9.4.6 Lidded cellular raft 151
9.4.7 Beam strip raft 151
9.4.8 Buoyancy (or ‘floating’) raft 151
9.4.9 Jacking raft 152
9.5 Group three – pile foundations 152
9.5.1 Introduction 152
9.5.2 Stone/gravel piles 153
9.5.3 Concrete piles 153
9.5.4 Timber piles 155
9.5.5 Steel piles 156
9.5.6 Anchor piles 156
9.5.7 Anchor blocks 156
9.5.8 Pile caps and ground beams 157
9.6 Group four – miscellaneous elements and forms 157
9.6.1 Suspended ground floor slabs 158
9.6.2 Floating ground floor slabs 159
9.6.3 Pier and beam foundations 159
9.6.4 Retaining walls 161
9.6.5 Grillage foundations 162

10 Foundation Selection and Design Procedures 164

A: Foundation selection 164
10.1 Introduction 164
10.2 Foundation selection 164
10.3 Information collection/assessment 164
10.4 General approach to choice of foundations 165
10.5 Questioning the information and proposals 169
10.6 Exploitation of foundation stiffness and resulting ground pressure 172
10.7 Conclusions 173

B: Foundation design calculation procedure 173
10.8 Introduction 173
10.9 Definition of bearing pressures 173
10.10 Calculation of applied bearing pressures 174
10.11 Structural design of foundation members 178
10.12 General design method 180
10.13 References 185

11 Design of Pads, Strips and Continuous Foundations 186
11.1 Unreinforced concrete pads and strips 186
11.1.1 Introduction 186
11.1.2 Trench fill 186
11.1.3 Trench fill design decisions 187
11.1.4 Sizing of the design 189
11.1.5 Design Example 1: Trench fill strip footing 190

11.1.6 Design Example 2: Deep mass concrete pad base 192
11.1.7 Unreinforced concrete strips 193
11.2 Reinforced concrete pads and strips 194
11.2.1 Introduction 194
11.2.2 Design decisions 194
11.2.3 Sizing up of the design 194
11.2.4 Design Example 3: Reinforced strip foundation 195
11.2.5 Design Example 4: Reinforced pad base 198
11.3 Pad foundations with axial loads and bending moments 200
11.3.1 Design Example 5: Pad base – axial load plus bending moment (small eccentricity) 201
11.3.2 Design Example 6: Pad base – axial load plus bending moment (large eccentricity) 202
11.3.3 Design Example 7: Pad base – axial load plus bending moments about both axes 206
11.3.4 Design Example 8: Pad base – axial and horizontal loads 207
11.3.5 Design Example 9: Shear wall base – vertical loads and horizontal wind loads 209
11.4 Rectangular and Tee-beam continuous strips 212
11.4.1 Introduction 212
11.4.2 Design decisions 212
11.4.3 Sizing of the design 212
11.4.4 Design Example 10: Continuous Tee beam footing with uniform bearing pressure 213
11.4.5 Design Example 11: Continuous rectangular beam footing with trapezoidal bearing pressure 217
11.5 Grillage foundations 221
11.5.1 Introduction 221
11.5.2 Design decisions 221
11.5.3 Sizing of the design 221
11.5.4 Design Example 12: Grillage foundation 221
11.6 Floating slabs (ground slabs) 224
11.6.1 Introduction 224
11.6.2 Design decisions 224
11.6.3 Sizing of the slab 225
11.6.4 Design Example 13: Floating slab 225
11.7 References 226

12 Tied and Balanced Foundations 228
12.1 General introduction 228
12.2 Tied foundations 228
12.2.1 Introduction 228
12.2.2 Design decisions 228
12.2.3 Sizing the foundations 228
12.2.4 Design Example 1: Tied portal frame base 229
12.3 Balanced foundations (rectangular, cantilever, trapezoidal and holed) 230
12.3.1 Introduction 230
12.3.2 Design decisions 230
12.3.3 Sizing up the design 230
12.3.4 Design Example 2: Rectangular balanced foundation 232
12.3.5 Design Example 3: Cantilever balanced foundation 233
12.3.6 Design Example 4: Trapezoidal balanced foundation 235
12.3.7 Design Example 5: Holed balanced foundation 236

13 Raft Foundations 238
13.1 Design procedures for semi-flexible rafts 238
13.1.1 Design principles 238
13.1.2 Design of raft layouts 238
13.1.3 Bearing pressure design 239
13.1.4 Design span for local depressions 240
13.1.5 Slab design 240
13.1.6 Beam design 243
13.2 Nominal crust raft – semi-flexible 245
13.2.1 Design decisions 245
13.2.2 Sizing the design 245
13.2.3 Design Example 1: Nominal crust raft 249
13.3 Crust raft 251
13.3.1 Introduction 251
13.3.2 Design decisions 251
13.3.3 Design Example 2: Crust raft 252
13.4 Blanket raft 256
13.4.1 Introduction 256
13.4.2 Design decisions 257
13.4.3 Sizing the design 257
13.4.4 Design Example 3: Blanket raft 257
13.5 Slip sandwich raft 261
13.5.1 Introduction 261
13.5.2 Design decisions 262
13.5.3 Sizing the design 262
13.5.4 Design Example 4: Slip sandwich raft 263
13.6 Cellular raft 265
13.6.1 Introduction 265
13.6.2 Sizing the design 265
13.6.3 Design Example 5: Cellular raft 266
13.7 Lidded cellular raft 270
13.7.1 Introduction 270
13.7.2 Sizing the design 271
13.7.3 Design Example 6: Lidded cellular raft 271
13.8 Beam strip raft 271
13.8.1 Introduction 271
13.8.2 Sizing the design 271
13.8.3 Design Example 7: Beam strip raft 272
13.9 Buoyancy raft 272
13.9.1 Introduction 272
13.9.2 Sizing the design 274
13.9.3 Design Example 8: Buoyancy raft 274
13.10 Jacking raft 276
13.10.1 Introduction 276
13.10.2 Sizing the design 276
13.11 References 276

14 Piles **277**
14.1 Introduction 277
14.2 Applications 277
14.3 Types of piles 278
14.3.1 Load-bearing characteristics 278
14.3.2 Materials 278
14.4 Methods of piling 283
14.4.1 Driven piles 283
14.4.2 Driven cast-in-place piles 283
14.4.3 Bored cast-in-place piles 283
14.4.4 Screw piles 284
14.4.5 Jacked piles 284
14.4.6 Continuous flight auger piles 284
14.4.7 Mini or pin piles 284
14.5 Choice of pile 284
14.5.1 Ground conditions and structure 285
14.5.2 Durability 285
14.5.3 Cost 285
14.6 Design of piled foundations 285
14.6.1 Factor of safety 285
14.6.2 Determination of ultimate bearing capacity 286
14.6.3 Pile loading tests 288
14.6.4 Pile groups 288
14.6.5 Spacing of piles within a group 289
14.6.6 Ultimate bearing capacity of group 289
14.6.7 Negative friction 289
14.7 Pile caps 289
14.7.1 Introduction 289
14.7.2 The need for pile caps – capping beams 290
14.7.3 Size and depth 290
14.8 Design of foundations at pile head 291
14.9 Design examples 293
14.9.1 Design Example 1: Calculation of pile safe working loads 293
14.9.2 Design Example 2: Pile cap design 295
14.9.3 Design Example 3: Piled ground beams with floating slab 296
14.9.4 Design Example 4: Piled ground beams with suspended slab 299
14.9.5 Design Example 5: Piled foundation with suspended flat slab 300
14.10 References 303

15 Retaining Walls, Basement Walls, Slip Circles and Underpinning **304**
15.1 Introduction 304
15.2 Retaining walls and basements 304
15.3 Stability 305
15.4 Flotation 306
15.5 Buoyancy 306
15.6 Pressures 307
15.6.1 Liquid pressure 307
15.6.2 Earth pressure 307
15.6.3 Surcharge 307
15.7 Slip circle example 307
15.8 Continuous underpinning 308
15.9 Discontinuous underpinning 310
15.10 Spread underpinning 311
15.11 References 311

Appendices **313**
Introduction to appendices 313
Appendix A: Properties and Presumed Bearing Pressures of Some Well Known Engineering Soils and Rocks 314
Appendix B: Map Showing Areas of Shrinkable Clays In Britain 317
Appendix C: Map Showing Areas of Coal and Some Other Mineral Extractions 318
Appendix D: Foundation Selection Tables 319
Appendix E: Guide to Use of Ground Improvement 322
Appendix F: Tables Relating to Contaminated Sites/Soils 325
Appendix G: Factors of Safety 341
Appendix H: Design Charts for Pad and Strip Foundations 343
Appendix J: Table of Ground Beam Trial Sizes 348
Appendix K: Design Graphs and Charts for Raft Foundations Spanning Local Depressions 349
Appendix L: Table of Material Frictional Resistances 357
Appendix M: Cost Indices for Foundation Types 358
Appendix N: Allowable Bearing Pressure for Foundations on Non-Cohesive Soil 359

Index **361**

Preface

In this age of increasing specialism, it is important that the engineer responsible for the safe design of structures maintains an all-round knowledge of the art and science of foundation design. In keeping with the aims and aspirations of the original authors, this second edition of the *Structural Foundation Designers' Manual* provides an up-to-date reference book, for the use of structural and civil engineers involved in the foundation design process.

The inspiration provided by Bill Curtin who was the driving force behind the practical approach and no-nonsense style of the original book, has not been sacrificed and the book continues to provide assistance for the new graduate and the experienced design engineer in the face of the myriad choices available when selecting a suitable foundation for a tricky structure on difficult ground.

Since the first edition was written, there have been changes to the many technical publications and British Standards relevant to the subject area and the opportunity has been taken to revise and update the original material in line with these new references. In particular, the chapter on contaminated and derelict sites has been rewritten incorporating current UK guidelines contained within the Part IIA Environmental Protection Act 1990 and guidance provided by DEFRA, the Environment Agency and BS 10175.

The work continues to draw on the practical experience gained by the directors and staff of Curtins Consulting over 45 years of civil and structural engineering consultancy, who I thank for their comments and feedback. Thanks also go to the Department of Engineering at the University of Wales, Newport for providing secretarial support and editing facilities.

N.J. Seward

Preface to First Edition

'Why yet another book on foundations when so many good ones are already available?' – a good question which deserves an answer.

This book has grown out of our consultancy's extensive experience in often difficult and always cost-competitive conditions of designing structural foundations. Many of the existing good books are written with a civil engineering bias and devote long sections to the design of aspects such as bridge caissons and marine structures. Furthermore, a lot of books give good explanations of soil mechanics and research – but mainly for *green field* sites. We expect designers to know soil mechanics and where to turn for reference when necessary. However there are few books which cover the new advances in geotechnical processes necessary now that we have to build on derelict, abandoned inner-city sites, polluted or toxic sites and similar problem sites. And no book, yet, deals with the developments we and other engineers have made, for example, in raft foundations. Some books are highly specialized, dealing only (and thoroughly) with topics such as piling or underpinning.

Foundation engineering is a wide subject and designers need, primarily, one reference for guidance. Much has been written on foundation construction work and methods – and that deserves a treatise in its own right. Design and construction should be interactive, but in order to limit the size of the book, we decided, with regret to restrict discussion to design and omit discussion of techniques such as dewatering, bentonite diaphragm wall construction, timbering, etc.

Foundation construction can be the biggest bottleneck in a building programme so attention to speed of construction is vital in the design and detailing process. Repairs to failed or deteriorating foundations are frequently the most costly of all building remedial measures so care in safe design is crucial, but extravagant design is wasteful. Too much foundation design is unnecessarily costly and the advances in civil engineering construction have not always resulted in a spin-off for building foundations. Traditional building foundations, while they may have sometimes been over-costly were quick to construct and safe – on good ground. But most of the good ground is now used up and we have to build on sites which would have been rejected on the basis of cost and difficulty as recently as a decade ago. Advances in techniques and developments can now make such sites a cost-and-construction viable option. All these aspects have been addressed in this book.

Though the book is the work of four senior members of the consultancy, it represents the collective experience of all directors, associates and senior staff, and we are grateful for their support and encouragement. As in all engineering design there is no unique 'right' answer to a problem – designers differ on approach, priorities, evaluation of criteria, etc. We discussed, debated and disagreed – the result is a reasonable consensus of opinion but not a compromise. Engineering is an art as well as a science, but the art content is even greater in foundation design. No two painters would paint a daffodil in the same way (unless they were painting by numbers!). So no two designers would design a foundation in exactly the same manner (unless they chose the same computer program and fed it with identical data).

So we do not expect experienced senior designers to agree totally with us and long may individual preference be important. All engineering design, while based on the same studies and knowledge, is an exercise in judgement backed by experience and expertise. Some designers can be daring and others over-cautious; some are innovative and others prefer to use stock solutions. But all foundation design must be safe, cost-effective, durable and buildable, and these have been our main priorities. We hope that all designers find this book useful.

The Book's Structure and What It Is About

The book is arranged so that it is possible for individual designers to use the manual in different ways, depending upon their experience and the particular aspects of foundation design under consideration.

The book, which is divided into three parts, deals with the whole of foundation design from a practical engineering viewpoint. Chapters 1–3, i.e. Part 1, deal with soil mechanics and the behaviour of soils, and the commission and interpretation of site investigations are covered in detail.

In Part 2 (Chapters 4–8), the authors continue to share their experience – going back over 45 years – of dealing with filled and contaminated sites and sites in mining areas; these 'problem' sites are increasingly becoming 'normal' sites for today's engineers.

In Part 3 (Chapters 9–15), discussion and practical selection of foundation types are covered extensively, followed by detailed design guidance and examples for the various foundation types. The design approach ties together the safe working load design of soils with the limit-state design of structural foundation members.

The emphasis on practical design is a constant theme running through this book, together with the application of engineering judgement and experience to achieve appropriate and economic foundation solutions for difficult sites. This is especially true of raft design, where a range of raft types, often used in conjunction with filled sites, provides an economic alternative to piled foundations.

It is intended that the experienced engineer would find Part 1 useful to recapitulate the basics of design, and refresh his/her memory on the soils, geology and site investigation aspects. The younger engineer should find Part 1 of more use in gaining an overall appreciation of the starting point of the design process and the interrelationship of design, soils, geology, testing and ground investigation.

Part 2 covers further and special considerations which may affect a site. Experienced and young engineers should find useful information within this section when dealing with sites affected by contamination, mining, fills or when considering the treatment of sub-soils to improve bearing or settlement performance. The chapters in Part 2 give information which will help when planning site investigations and assist in the foundation selection and design process.

Part 3 covers the different foundation types, the selection of an appropriate foundation solution and the factors affecting the choice between one foundation type and another. Also covered is the actual design approach, calculation method and presentation for the various foundation types. Experienced and young engineers should find this section useful for the selection and design of pads, strips, rafts and piled foundations.

The experienced designer can refer to Parts 1, 2 and 3 in any sequence. Following an initial perusal of the manual, the young engineer could also refer to the various parts out of sequence to assist with the different stages and aspects of foundation design.

For those practising engineers who become familiar with the book and its information, the tables, graphs and charts grouped together in the Appendices should become a quick and easy form of reference for useful, practical and economic foundations in the majority of natural and man-made ground conditions.

Occasional re-reading of the text, by the more experienced designer, may refresh his/her appreciation of the basic important aspects of economical foundation design, which can often be forgotten when judging the merits of often over-emphasized and over-reactive responses to relatively rare foundation problems. Such problems should not be allowed to dictate the 'norm' when, for the majority of similar cases, a much simpler and more practical solution (many of which are described within these pages) is likely still to be quite appropriate.

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We are grateful for the trust and confidence of many clients in the public and private sectors who readily gave us freedom to develop innovative design. We appreciate the help given by many friends in the construction industry, design professions and organizations and we learnt much from discussions on site and debate in design team meetings. We are happy to acknowledge (in alphabetical order) permission to quote from:

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- Building Research Establishment
- Cement and Concrete Association
- Corus
- CIRIA
- DEFRA
- Institution of Civil Engineers
- John Wiley & Sons.

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Bill Curtin's interest and involvement in foundation engineering dated back to his lecturing days at Brixton and Liverpool in the 1950–60s. In 1960 he founded the Curtins practice in Liverpool and quickly gained a reputation for economic foundation solutions on difficult sites in the north-west of England and Wales. He was an active member of both the Civil and Structural Engineering Institutions serving on and chairing numerous committees and working with BSI and CIRIA. He produced numerous technical design guides and text books including *Structural Masonry Designers' Manual*.

G. SHAW (1940–1997) CEng, FICE, FIStructE, MConsE

Gerry Shaw was a director of Curtins Consulting Engineers plc with around 40 years' experience in the building industry, including more than 30 years as a consulting engineer. He was responsible for numerous important foundation structures on both virgin and man-made soil conditions and was continuously involved in foundation engineering, innovative developments and monitoring advances in foundation solutions. He co-authored a number of technical books and design notes and was external examiner for Kingston University. He acted as expert witness in legal cases involving building failures, and was a member of the BRE/CIRIA Committee which investigated and analysed building failures in 1980. He co-authored both *Structural Masonry Designers' Manual* and *Structural Masonry Detailing Manual*. He was a Royal Academy of Engineering Visiting Professor of Civil Engineering Design to the University of Plymouth.

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Gary Parkinson was a director of Curtins Consulting Engineers plc responsible for the Liverpool office. He has over 40 years' experience in the building industry, including 35 years as a consulting engineer. He has considerable foundation engineering experience, and has been involved in numerous land reclamation and development projects dealing with derelict and contaminated industrial land and dockyards. He is co-author of *Structural Masonry Detailing Manual*.

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John Golding spent seven years working with Curtins Consulting Engineers and is now an associate with WSP Cantor Seinuk. He has recently completed the substructure design for the award-winning Wellcome Trust Headquarters, and is currently responsible for the design of the UK Supreme Court and the National Aquarium. He has over 25 years' experience in the design of commercial, residential and industrial structures, together with civil engineering water treatment works, road tunnels and subway stations. Many of the associated foundations have been in difficult inner-city sites, requiring a range of ground improvement and other foundation solutions. He has been involved in research and development of innovative approaches to concrete, masonry and foundation design, and is the author of published papers on all of these topics.

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Norman Seward is a senior lecturer at the University of Wales, Newport. Prior to this he spent 28 years in the building industry, working on the design of major structures both in the UK and abroad with consulting engineers Turner Wright, Mouchel, the UK Atomic Energy Authority and most recently as associate director in Curtins Cardiff office. He was Wales Branch chairman of the IStructE in 1998 and chief examiner for the Part III examination from 2000 to 2004. He has experience as an expert witness in cases of structural failure, has been technical editor for a number of publications including the IStructE *Masonry Handbook* and is a member of the IStructE EC6 Handbook Editorial Panel. He currently teaches on the honours degree programme in civil engineering, in addition to developing his research interests in the field of foundations for lightweight structures.

Notation

APPLIED LOADS AND CORRESPONDING PRESSURES AND STRESSES

Loads		**Pressures and stresses**	
$F = F_B + F_S$	foundation loads	$f = F/A$	pressure component resulting from F
F_B	buried foundation/backfill load	$f_B = F_B/A$	pressure component resulting from F_B
F_S	new surcharge load	$f_S = F_S/A$	pressure component resulting from F_S
G	superstructure dead load	g	pressure component resulting from G
H	horizontal load		
H_f	horizontal load capacity at failure		
M	bending moment		
$N = T - S$	net load	$n = t - s$	pressure component resulting from N
		$n' = n - \gamma_w z_w$	net effective stress
		n_f	net ultimate bearing capacity at failure
P	superstructure vertical load	$p = t - f$	pressure component resulting from P
		$p_u = t_u - f_u$	resultant ultimate design pressure
		p_z	pressure component at depth z resulting from P
Q	superstructure imposed load	q	pressure component resulting from Q
$S = S_B + S_S$	existing load	$s = S/A$	pressure component resulting from S
S_B	'buried' surcharge load (i.e. $\approx F_B$)	$s_B = S_B/A$	pressure component resulting from S_B
S_S	existing surcharge load	$s_S = S_S/A$	pressure component resulting from S_S
		$s' = s - \gamma_w z_w$	existing effective stress
$T = P + F$	total vertical load	t	pressure resulting from T
		$t' = t - \gamma_w z_w$	total effective stress
		t_f	total ultimate bearing capacity at failure
V	shear force	v	shear stress due to V
W	superstructure wind load	w	pressure component resulting from W

General subscripts for loads and pressures

a	allowable (load or bearing pressure)
f	failure (load or bearing pressure)
u	ultimate (limit-state)
G	dead
Q	imposed
W	wind
F	foundation
P	superstructure
T	total

Partial safety factors for loads and pressures

γ_G	partial safety factor for dead loads
γ_Q	partial safety factor for imposed loads
γ_W	partial safety factor for wind loads
γ_F	combined partial safety factor for foundation loads
γ_P	combined partial safety factor for superstructure loads
γ_T	combined partial safety factor for total loads

Notation principles for loads and pressures

(1) *Loads* are in capitals, e.g.
P = load from superstructure (kN)
F = load from foundation (kN)

(2) *Loads per unit length* are also in capitals, e.g.
P = load from superstructure (kN/m)
F = load from foundation (kN/m)

(3) Differentiating between *loads* and *loads per unit length*.
This is usually made clear by the context, i.e. pad foundation calculations will normally be in terms of *loads* (in kN), and strip foundations will normally be in terms of *loads per unit length* (kN/m). Where there is a need to differentiate, this is done, as follows:
$\sum P$ = load from superstructure (kN)
P = load from superstructure per unit length (kN/m)

(4) *Distributed loads* (loads per unit area) are lower case, e.g.
f = uniformly distributed foundation load (kN/m^2)

(5) *Ground pressures* are also in lower case, e.g.
p = pressure distribution due to superstructure loads (kN/m^2)
f = pressure distribution due to foundation loads (kN/m^2)

(6) *Characteristic* versus *ultimate* (u subscript).
Loads and pressures are either *characteristic* values or *ultimate* values. This distinction is important, since *characteristic* values (working loads/pressures) are used for bearing pressure checks, while *ultimate* values (factored loads/pressures) are used for structural member design. All ultimate values have u subscripts. Thus
p = characteristic pressure due to superstructure loads
p_u = ultimate pressure due to superstructure loads

GENERAL NOTATION

Dimensions

a	distance of edge of footing from face of wall/beam
A	area of base
A_b	effective area of base (over which compressive bearing pressures act)
A_s	area of reinforcement OR surface area of pile shaft
b	width of the section for reinforcement design
B	width of base
B_b	width of beam thickening in raft
B_{conc}	assumed width of concrete base
B_{fill}	assumed spread of load at underside of compacted fill material
d	effective depth of reinforcement
D	depth of underside of foundation below ground level OR diameter of pile
D_w	depth of water-table below ground level
e	eccentricity
h	thickness of base
h_b	thickness of beam thickening in raft
h_{fill}	thickness of compacted fill material
h_{conc}	thickness of concrete
H	length of pile OR height of retaining wall
H_1, H_2	thickness of soil strata ‘1’, ‘2’, etc.
L	length of base OR length of depression
L_b	effective length of base (over which compressive bearing pressures act)
t_w	thickness of wall
u	length of punching shear perimeter
x	projection of external footing beyond line of action of load

z	depth below ground level
z_w	depth below water-table
ρ_1, ρ_2	settlement of strata '1', '2', etc.

Miscellaneous

c	cohesion
c_b	undisturbed shear strength at base of pile
c_s	average undrained shear strength for pile shaft
e	void ratio
f_{bs}	characteristic local bond stress
f_c	ultimate concrete stress (in pile)
f_{cu}	characteristic concrete cube strength
I	moment of inertia
k	permeability
K	earth pressure coefficient
K_a	active earth pressure coefficient
K_m	bending moment factor (raft design)
m_v	coefficient of volume compressibility
N	SPT value
N_c	Terzaghi bearing capacity factor
N_q	Terzaghi bearing capacity factor
N_γ	Terzaghi bearing capacity factor
v_c	ultimate concrete shear strength
V	total volume
V_s	volume of solids
V_v	volume of voids
Z	section modulus
α	creep compression rate parameter OR adhesion factor
γ	unit weight of soil
γ_{dry}	dry unit weight of soil
γ_{sat}	saturated unit weight of soil
γ_w	unit weight of water
δ	angle of wall friction
ε	strain
μ	coefficient of friction
σ	(soil) stress normal to the shear plane
σ'	(soil) effective normal stress
τ	(soil) shear stress
ϕ	angle of internal friction

Occasionally it has been necessary to vary the notation system from that indicated here. Where this does happen, the changes to the notation are specifically defined in the accompanying text or illustrations.

Part 1

Approach and First Considerations

1 Principles of Foundation Design

1.1 Introduction

Foundation design could be thought of as analogous to a beam design. The designer of the beam will need to know the load to be carried, the load-carrying capacity of the beam, how much it will deflect and whether there are any long-term effects such as creep, moisture movement, etc. If the calculated beam section is, for some reason, not strong enough to support the load or is likely to deflect unduly, then the beam section is changed. Alternatively, the beam can either be substituted for another type of structural element, or a stronger material be chosen for the beam.

Similarly the soil supporting the structure must have adequate load-carrying capacity (bearing capacity) and not deflect (settle) unduly. The long-term effect of the soil's bearing capacity and settlement must be considered. If the ground is not strong enough to bear the proposed initial design load then the structural contact load (bearing pressure) can be reduced by spreading the load over a greater area – by increasing the foundation size or other means – or by transferring the load to a lower stratum. For example, rafts could replace isolated pad bases – or the load can be transferred to stronger soil at a lower depth beneath the surface by means of piles. Alternatively, the ground can be strengthened by compaction, stabilization, preconsolidation or other means. The structural materials in the superstructure are subject to stress, strain, movement, etc., and it can be helpful to consider the soil supporting the superstructure as a structural material, also subject to stress, strain and movement.

Structural design has been described as using materials not fully understood, to make frames which cannot be accurately analysed, to resist forces which can only be estimated. Foundation design is, at best, no better. 'Accuracy' is a chimera and the designer must exercise judgement.

Sections 1.2–1.6 outline the general principles before dealing with individual topics in the following sections and chapters.

1.2 Foundation safety criteria

It is a statement of the obvious that the function of a foundation is to transfer the load from the structure to the ground (i.e. soil) supporting it – and it must do this safely, for if it does not then the foundation will fail in bearing and/or settlement, and seriously affect the structure which may also fail. The history of foundation failure is as old as the history of building itself, and our language abounds in such idioms as 'the god with feet of clay', 'build not thy house on sand', 'build on a firm foundation', 'the bedrock of our policy'. The foundation must also be economical in construction costs, materials and time.

There are a number of reasons for foundation failure, the two major causes being:

(1) *Bearing capacity*. When the shear stress within the soil, due to the structure's loading, exceeds the shear strength of the soil, catastrophic collapse of the supporting soil can occur. Before ultimate collapse of the soil occurs there can be large deformations within it which may lead to unacceptable differential movement or settlement of, and damage to, the structure. (In some situations however, collapse can occur with little or no advance warning!)

(2) *Settlement*. Practically all materials contract under compressive loading and distort under shear loading – soils are no exception. Provided that the settlement is either acceptable (i.e. will not cause structural damage or undue cracking, will not damage services, and will be visually acceptable and free from practical problems of door sticking, etc.) or can be catered for in the structural design (e.g. by using three-pinned arches which can accommodate settlement, in lieu of fixed portal frames), there is not necessarily a foundation design problem. Problems will occur when the settlement is significantly excessive or differential.

Settlement is the combination of two phenomena:

(i) *Contraction of the soil* due to compressive and shear stresses resulting from the structure's loading. This contraction, partly elastic and partly plastic, is relatively rapid. Since soils exhibit non-linear stress/strain behaviour and the soil under stress is of complex geometry, it is not possible to predict accurately the magnitude of settlement.

(ii) *Consolidation of the soil* due to volume changes. Under applied load the moisture is 'squeezed' from the soil and the soil compacts to partly fill the voids left by the retreating moisture. In soils of low permeability, such as clays, the consolidation process is slow and can even continue throughout the life of the structure (for example, the leaning tower of Pisa). Clays of relatively high moisture content will consolidate by greater amounts than clays with lower moisture contents. (Clays are susceptible to volume change with change in moisture content – they can shrink on drying out and *heave*, i.e. expand, with increase in moisture content.) Sands tend to have higher permeability and lower moisture content than clays. Therefore the consolidation of sand is faster but less than that of clay.

1.3 Bearing capacity

1.3.1 Introduction

Some designers, when in a hurry, tend to want simple 'rules of thumb' (based on local experience) for values of bearing capacity. But like most rules of thumb, while safe for typical structures on normal soils, their use can produce uneconomic solutions, restrict the development of improved methods of foundation design, and lead to expensive mistakes when the structure is not *typical*.

For *typical* buildings:

(1) The dead and imposed loads are built up gradually and relatively slowly.
(2) *Actual* imposed loads (as distinct from those assumed for design purposes) are often only a third of the dead load.
(3) The building has a height/width ratio of between 1/3 and 3.
(4) The building has regularly distributed columns or load-bearing walls, most of them fairly evenly loaded.

Typical buildings have changed dramatically since the Second World War. The use of higher design stresses, lower factors of safety, the removal of robust non-load-bearing partitioning, etc., has resulted in buildings of half their previous weight, more susceptible to the effects of settlement, and built for use by clients who are less tolerant in accepting relatively minor cracking of finishes, etc. Because of these changes, *practical* experience gained in the past is not always applicable to present construction.

For *non-typical* structures:

(1) The imposed load may be applied rapidly, as in tanks and silos, resulting in possible settlement problems.
(2) There may be a high ratio of imposed to dead load. Unbalanced imposed-loading cases – imposed load over part of the structure – can be critical, resulting in differential settlement or bearing capacity failures, if not allowed for in design.
(3) The requirement may be for a tall, slender building which may be susceptible to tilting or overturning and have more critical wind loads.
(4) The requirement may be for a non-regular column/wall layout, subjected to widely varying loadings, which may require special consideration to prevent excessive differential settlement and bearing capacity failure.

There is also the danger of going to the other extreme by doing complicated calculations based on numbers from unrepresentative soil tests alone, and ignoring the important evidence of the soil profile and local experience. Structural design and materials are not, as previously stated, mathematically precise; foundation design and materials are even less precise. Determining the bearing capacity solely from a 100 mm thick small-diameter sample and applying it to predict the behaviour of a 10 m deep stratum, is obviously not sensible – particularly when many structures could fail, in serviceability, by settlement at bearing pressures well below the soil's ultimate bearing capacity.

1.3.2 Bearing capacity

Probably the happy medium is to follow the sound advice given by experienced engineers in the British Standard Institution's *Code of practice for foundations*, BS 8004. There they define *ultimate bearing capacity* as 'the value of the gross loading intensity for a particular foundation at which the resistance of the soil to displacement of the foundation is fully mobilized.' (*Ultimate* in this instance does *not* refer to ultimate limit state.)

The *net loading intensity* (net bearing pressure) is the additional intensity of vertical loading at the base of a foundation due to the weight of the new structure and its loading, including any earthworks.

The ultimate bearing capacity divided by a suitable factor of safety – typically 3 – is referred to as the *safe bearing capacity*.

It has not been found possible, yet, to apply limit state design fully to foundations, since bearing capacity and settlement are so intertwined and influence both foundation and superstructure design (this is discussed further in section 1.5). Furthermore, the superstructure itself can be altered in design to accommodate, or reduce, the effects of settlement. A reasonable compromise has been devised by engineers in the past and is given below.

1.3.3 Presumed bearing value

The pressure within the soil will depend on the net loading intensity, which in turn depends on the structural loads and the foundation type. This pressure is then compared with the ultimate bearing capacity to determine a factor of safety. This appears reasonable and straightforward – but there is a catch-22 snag. It is not possible to determine the net loading intensity without first knowing the foundation type and size, but the foundation type and size cannot be designed without knowing the acceptable bearing pressure.

The deadlock has been broken by BS 8004, which gives *presumed allowable bearing values* (estimated bearing pressures) for different types of ground. This enables a preliminary foundation design to be carried out which can be adjusted, up or down, on further analysis. The presumed bearing value is defined as: 'the net loading intensity considered appropriate to the particular type of ground for preliminary design purposes'. The value is based on either local experience or on calculation from laboratory strength tests or field loading tests using a factor of safety against bearing capacity failure.

Foundation design, like superstructure design, is a trial-and-error method – a preliminary design is made, then checked and, if necessary, amended. Amendments would be necessary, for example, to restrict settlement or overloading; in consideration of economic and construction implications, or designing the superstructure to resist or accommodate settlements. The Code's presumed bearing values are given in Table 1.1 and experience shows that these are valuable and reasonable in preliminary design.

Table 1.1 Presumed bearing values (BS 8004, Table 1)$^{(1)}$

NOTE. These values are for preliminary design purposes only, and may need alteration upwards or downwards. No addition has been made for the depth of embedment of the foundation (see 2.1.2.3.2 and 2.1.2.3.3).

Category	Types of rocks and soils	Presumed allowable bearing value		Remarks
		kN/m^2*	kgf/cm^2* tonf/ft^2	
Rocks	Strong igneous and gneissic rocks in sound condition	10 000	100	These values are based on the assumption that the foundations are taken down to unweathered rock. For weak, weathered and broken rock, see 2.2.2.3.1.12
	Strong limestones and strong sandstones	4000	40	
	Schists and slates	3000	30	
	Strong shales, strong mudstones and strong siltstones	2000	20	
Non-cohesive soils	Dense gravel, or dense sand and gravel	>600	>6	Width of foundation not less than 1 m. Groundwater level assumed to be a depth not less than below the base of the foundation. For effect of relative density and groundwater level, see 2.2.2.3.2
	Medium dense gravel, or medium dense sand and gravel	<200 to 600	<2 to 6	
	Loose gravel, or loose sand and gravel	<200	<2	
	Compact sand	>300	>3	
	Medium dense sand	100 to 300	1 to 3	
	Loose sand	<100	<1	
		Value depending on degree of looseness		
Cohesive soils	Very stiff boulder clays and hard clays	300 to 600	3 to 6	Group 3 is susceptible to long-term consolidation settlement (see 2.1.2.3.3). For consistencies of clays, see table 5
	Stiff clays	150 to 300	1.5 to 3	
	Firm clays	75 to 150	0.75 to 1.5	
	Soft clays and silts	<75	<0.75	
	Very soft clays and silts	Not applicable		
Peat and organic soils		Not applicable		See 2.2.2.3.4
Made ground or fill		Not applicable		See 2.2.2.3.5

* 107.25 kN/m^2 = 1.094 kgf/cm^2 = 1 tonf/ft^2
All references within this table refer to the original document

1.3.4 *Allowable bearing pressure*

Knowing the structural loads, the preliminary foundation design and the ultimate bearing capacity, a check can be made on the *allowable bearing pressure*. The allowable net bearing pressure is defined in the Code as 'the maximum allowable net loading intensity at the base of the foundation' taking into account:

(1) The ultimate bearing capacity.
(2) The amount and kind of settlement expected.
(3) The ability of the given structure to accommodate this settlement.

This practical definition shows that the allowable bearing pressure is a combination of three functions; the strength and settlement characteristics of the ground, the foundation type, and the settlement characteristics of the structure.

1.3.5 *Non-vertical loading*

When horizontal foundations are subject to inclined forces (portal frames, cantilever structures, etc.) the passive resistance of the ground must be checked for its capacity to resist the horizontal component of the inclined load. This could result in reducing the value of the allowable bearing pressure to carry the vertical component of the inclined load. BS 8004 (*Code of practice for foundations*) suggests a simple *rule* for design of foundations subject to non-vertical loads as follows:

$$\frac{V}{P_{\mathrm{v}}}+\frac{H}{P_{\mathrm{h}}}<1$$

where V = vertical component of the inclined load,
H = horizontal component of the inclined load,
P_{v} = allowable vertical load – dependent on allowable bearing pressure,
P_{h} = allowable horizontal load – dependent on allowable friction and/or adhesion on the horizontal base, plus passive resistance where this can be relied upon.

However, like all simple *rules* which are on the safe side, there are exceptions. A more conservative value can be necessary when the horizontal component is relatively high and is acting on shallow foundations (where their depth/breadth ratio is less than 1/4) founded on non-cohesive soils.

In the same way that allowable bearing pressure is reduced to prevent excessive settlement, so too may allowable passive resistance, to prevent unacceptable horizontal movement.

If the requirements of this rule cannot be met, provision should be made for the horizontal component to be taken by some other part of the structure or by raking piles, by tying back to a line of sheet piling or by some other means.

1.4 Settlement

If the building settles excessively, particularly differentially – e.g. adjacent columns settling by different amounts – the settlement may be serious enough to endanger the stability of the structure, and would be likely to cause serious serviceability problems.

Less serious settlement may still be sufficient to cause cracking which could affect the building's weathertightness, thermal and sound insulation, fire resistance, damage finishes and services, affect the operation of plant such as overhead cranes, and other *serviceability* factors. Furthermore, settlement, even relatively minor, which causes the building to tilt, can render it visually unacceptable. (Old Tudor buildings, for example, may look charming and quaint with their tilts and leaning, but clients and owners of modern buildings are unlikely to accept similar tilts.)

Differential settlement, sagging, hogging and relative rotation are shown in Fig. 1.1.

In general terms it should be remembered that foundations are no different from other structural members and deflection criteria similar to those for superstructure members would also apply to foundation members.

From experience it has been found that the magnitude of *relative* rotation – sometimes referred to as angular distortion – is critical in framed structures, and the magnitude of the *deflection ratio*, Δ/L, is critical for load-bearing walls. Empirical criteria have been established to minimize cracking, or other damage, by limiting the movement, as shown in Table 1.2.

The length-to-height ratio is important since according to some researchers the greater the length-to-height ratio the greater the limiting value of Δ/L. It should be noted that cracking due to hogging occurs at half the deflection ratio of that for sagging. Sagging problems appear to occur more frequently than hogging in practice.

Since separate serviceability and ultimate limit state analyses are not at present carried out for the soil – see section 1.5 – it is current practice to adjust the factor of safety which is applied to the soil's ultimate bearing capacity, in order to obtain the allowable bearing pressure.

Similarly, the partial safety factor applied to the characteristic structural loads will be affected by the usual superstructure design factors and then adjusted depending on the structure (its sensitivity to movement, design life, damaging effects of movement), and the type of imposed loading. For example, full imposed load occurs infrequently in theatres and almost permanently in grain stores. Overlooking this permanence of loading in design has caused foundation failure in some grain stores. A number of failures due to such loading conditions have been investigated by the authors' practice. A typical example is an existing grain store whose foundations performed satisfactorily until a new grain store was built alongside. The

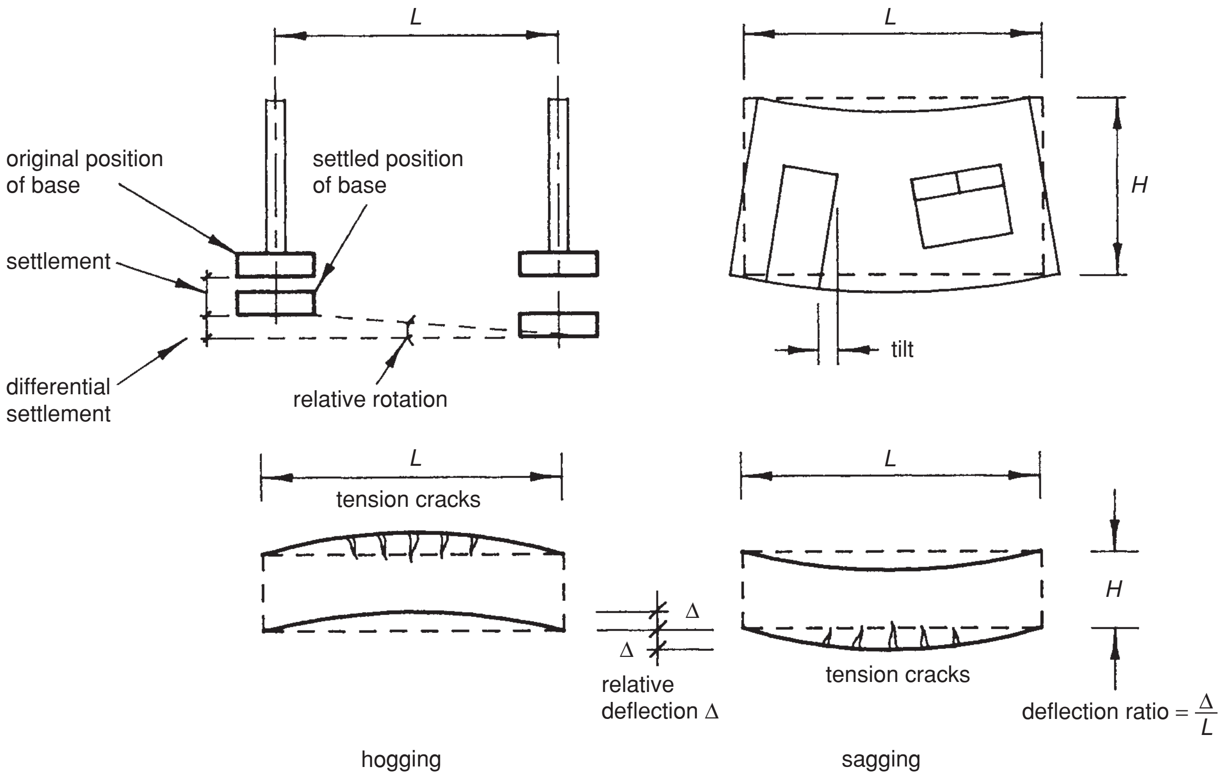

Fig. 1.1 Settlement definitions.

Table 1.2 Typical values of angular distortion to limit cracking (*Ground Subsidence*, Table 1, Institution of Civil Engineers, 1977)[(2)]

Class of structure	Type of structure	Limiting angular distortion
1	Rigid	Not applicable: tilt is criterion
2	Statically determinate steel and timber structures	1/100 to 1/200
3	Statically indeterminate steel and reinforced concrete framed structures, load-bearing reinforced brickwork buildings, all founded on reinforced concrete continuous and slab foundations	1/200 to 1/300
4	As class 3, but not satisfying one of the stated conditions	1/300 to 1/500
5	Precast concrete large-panel structures	1/500 to 1/700

ground pressure from the new store increased the pressure in the soil below the existing store – which settled and tilted. Similarly, any bending moments transferred to the ground (by, for example, fixing moments at the base of fixed portal frames) must be considered in the design, since they will affect the structure's contact pressure on the soil.

There is a rough correlation between bearing capacity and settlement. Soils of high bearing capacity tend to settle less than soils of low bearing capacity. It is therefore even more advisable to check the likely settlement of structures founded on weak soils. As a guide, care is required when the safe bearing capacity (i.e. ultimate bearing capacity divided by a factor of safety) falls below 125 kN/m^2; each site, and each structure, must however be judged on its own merits.

1.5 Limit state philosophy

1.5.1 Working stress design

A common design method (based on *working stress*) used in the past was to determine the ultimate bearing capacity of the soil, then divide it by a factor of safety, commonly 3, to determine the *safe bearing capacity*. The safe bearing capacity is the maximum allowable design loading intensity on the soil. The *ultimate bearing capacity* is exceeded when the loading intensity causes the soil to fail in shear. Typical ultimate bearing capacities are 150 kN/m^2 for soft clays, 300–600 kN/m^2 for firm clays and loose sands/gravels, and 1000–1500 kN/m^2 for hard boulder clays and dense gravels.

Consider the following example for a column foundation. The ultimate bearing capacity for a stiff clay is 750 kN/m^2. If the factor of safety equals 3, determine the area of a pad base to support a column load of 1000 kN (ignoring the weight of the base and any overburden).

$$\text{Safe bearing capacity} = \frac{\text{ultimate bearing capacity}}{\text{factor of safety}}$$

$$= \frac{750}{3} = 250\ \text{kN/m}^2$$

$$\text{actual bearing pressure} = \frac{\text{column load}}{\text{base area}}$$

therefore,

$$\text{required base area} = \frac{\text{column load}}{\text{safe bearing capacity}}$$

$$= \frac{1000}{250} = 4\ \text{m}^2$$

The method has the attraction of simplicity and was generally adequate for traditional buildings in the past. However, it can be uneconomic and ignores other factors. A nuclear power station, complex chemical works housing expensive plant susceptible to foundation movement or similar buildings, can warrant a higher factor of safety than a supermarket warehouse storing tinned pet food. A crowded theatre may deserve a higher safety factor than an occasionally used cow-shed. The designer should exercise judgement in the choice of factor of safety.

In addition, while there must be precautions taken against foundation *collapse limit state* (i.e. total failure) there must be a check that the *serviceability limit state* (i.e. movement under load which causes structural or building use distress) is not exceeded. Where settlement criteria dominate, the bearing pressure is restricted to a suitable value below that of the safe bearing capacity, known as the *allowable bearing pressure*.

1.5.2 Limit state design

Attempts to apply limit state philosophy to foundation design have, so far, not been considered totally successful. So a compromise between *working stress* and *limit state* has developed, where the designer determines an estimated *allowable bearing pressure* and checks for settlements and building serviceability. The actual bearing pressure is then factored up into an *ultimate design pressure*, for structural design of the foundation members.

The partial safety factors applied for ultimate design loads (i.e. typically 1.4 × dead, 1.6 × imposed, 1.4 × wind and 1.2 for dead + imposed + wind) are for superstructure design and should *not* be applied to foundation design for allowable bearing calculations.

For dead and imposed loads the actual working load, i.e. the unfactored characteristic load, should be used in most foundation designs. Where there are important isolated foundations and particularly when subject to significant eccentric loading (as in heavily loaded gantry columns, water towers, and the like), the engineer should exercise discretion in applying a partial safety factor to the imposed load. Similarly when the imposed load is very high in relation to the dead load (as in large cylindrical steel oil tanks), the engineer should apply a partial safety factor to the imposed load.

In fact when the foundation load due to wind load on the superstructure is relatively small – i.e. less than 25% of (dead + imposed) – it may be ignored. Where the occasional foundation load due to wind exceeds 25% of (dead + imposed), then the foundation area should be proportioned so that the pressure due to wind + dead + imposed loads does not exceed 1.25 × (allowable bearing pressure). When wind uplift on a foundation exceeds dead load, then this becomes a critical load case.

1.6 Interaction of superstructure and soil

The superstructure, its foundation, and the supporting soil should be considered as a structural entity, with the three elements interacting.

Adjustments to the superstructure design to resist the effects of bearing failure and settlements, at minor extra costs, are often more economic than the expensive area increase or stiffening of the foundations. Some examples from the authors' practice are given here to illustrate these adjustments. Adjustments to the soil to improve its properties are briefly discussed in section 1.8. The choice of foundation type is outlined in section 1.7. Adjustments and choices are made to produce the most economical solution.

1.6.1 Example 1: Three pinned arch

The superstructure costs for a rigid-steel portal-frame shed are generally cheaper than the three pinned arch solution (see Fig. 1.2).

Differential settlement of the column pad bases will however seriously affect the bending moments (and thus the stresses) in the rigid portal, but have insignificant effect on the three pinned arch. Therefore the pad foundations for the rigid portal will have to be bigger and more expensive than those for the arch, and may far exceed the saving in superstructure steelwork costs for the portal. (In some cases it can be worthwhile to place the column eccentric to the foundation base to counteract the moment at the base of the foundation due to column fixity and/or horizontal thrust.)

1.6.2 Example 2: Vierendeel superstructure

The single-storey reinforced concrete (r.c.) frame structure shown in Fig. 1.3 was founded in soft ground liable to excessive sagging/differential settlement. Two main solutions were investigated:

(1) Normal r.c. superstructure founded on deep, stiff, heavily reinforced strip footings.

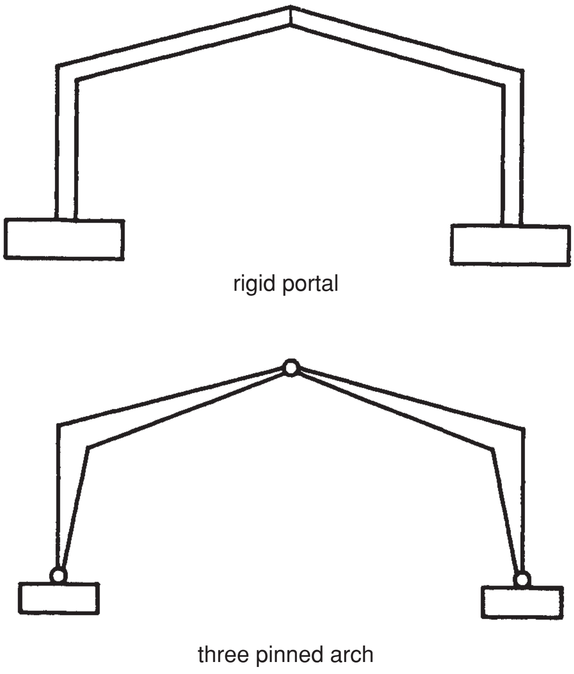

Fig. 1.2 Rigid portal versus three pinned arch.

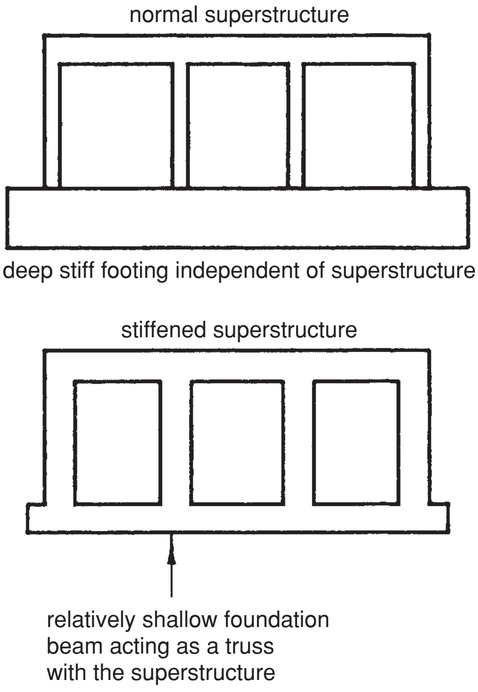

Fig. 1.3 Stiff footing versus Vierendeel truss.

(2) Stiffer superstructure, to act as a Vierendeel truss and thus in effect becoming a stiff *beam*, with the foundation beam acting as the bottom boom of the truss.

The *truss* solution (2) showed significant savings in construction costs and time.

1.6.3 Example 3: Prestressed brick diaphragm wall

A sports hall was to be built on a site with severe mining subsidence. At first sight the economic superstructure

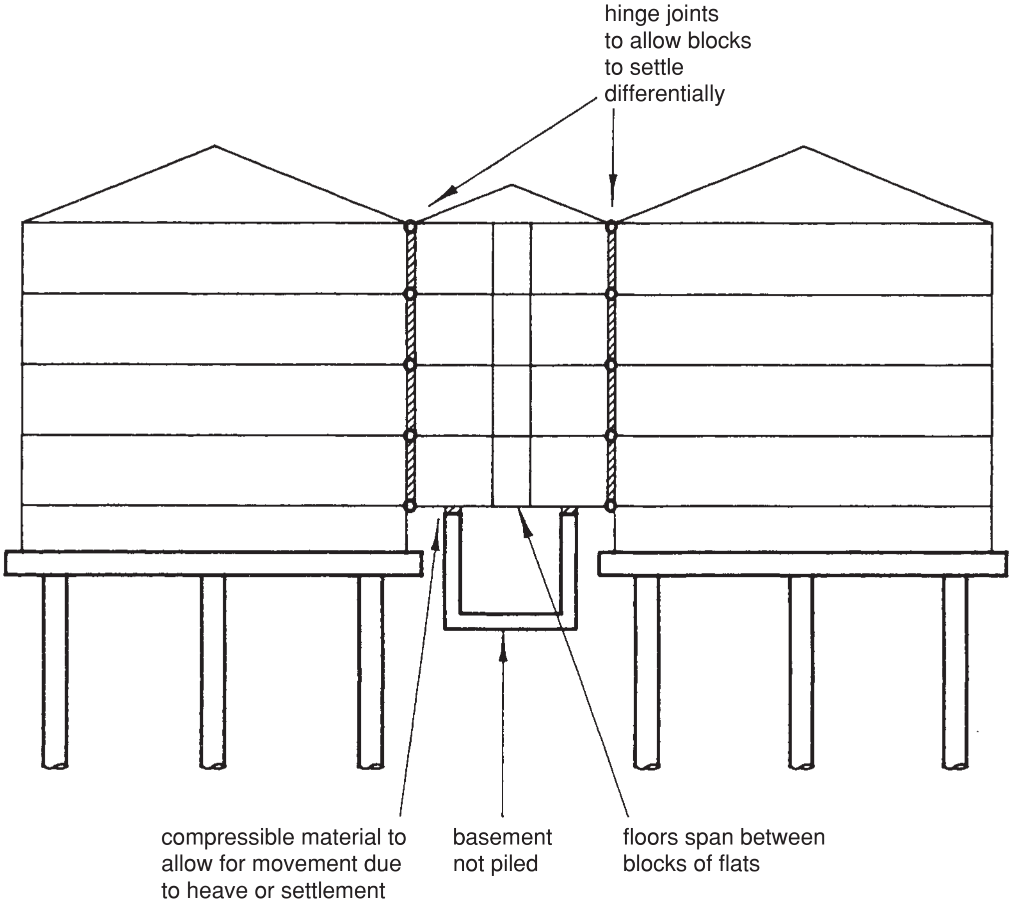

Fig. 1.4 Buoyancy raft.

solution of a brickwork diaphragm wall was ruled out, since the settlement due to mining would result in unacceptable tensile stresses in the brickwork. The obvious solutions were to cast massive, expensive foundation beams to resist the settlement and support the walls, or to abandon the brickwork diaphragm wall solution in favour of a probably more expensive structural steelwork superstructure. The problem was economically solved by prestressing the wall to eliminate the tensile stresses resulting from differential settlement.

1.6.4 Example 4: Composite deep beams

Load-bearing masonry walls built on a soil of low bearing capacity containing *soft spots* are often founded on strip footings reinforced to act as beams, to enable the footings to span over local depressions. The possibility of composite action between the wall and strip footing, acting together as a deeper beam, is not usually considered. Composite action significantly reduces foundation costs with only minor increases in wall construction costs (i.e. engineering bricks are used as a d.p.c. in lieu of normal d.p.c.s, which would otherwise act as a slip plane of low shear resistance). Bed joint reinforcement may also be used to increase the strength of the wall/foundation composite.

1.6.5 Example 5: Buoyancy raft

A four-storey block of flats was to be built on a site where part of the site was liable to ground heave due to removal of trees. The sub-soil was of low bearing capacity overlying dense gravel. The building plan was amended to incorporate two sections of flats interconnected by staircase and lift shafts, see Fig. 1.4. A basement was required beneath the staircase section and the removal of overburden enabled the soil to sustain structural loading. To have piled this area would have added unnecessary expense. The final design was piling for the two, four-storey sections of the flats, and a buoyancy raft (see section 13.9) for the basement.

It is hoped that these five simple examples illustrate the importance of considering the soil/structure interaction and encourage young designers not to consider the foundation design in isolation.

Bearing capacity, pressure, settlement, etc., are dealt with more fully in Chapter 2 and in section B of Chapter 10.

1.7 Foundation types

Foundation types are discussed in detail in Chapter 9; a brief outline only is given here to facilitate appreciation of the philosophy.

Basically there are four major foundation types: pads, strips, rafts, and piles. There are a number of variations within each type and there are combinations of types. Full details of the choice, application and design is dealt with in detail in later chapters. The choice is determined by the structural loads, ground conditions, economics of design,

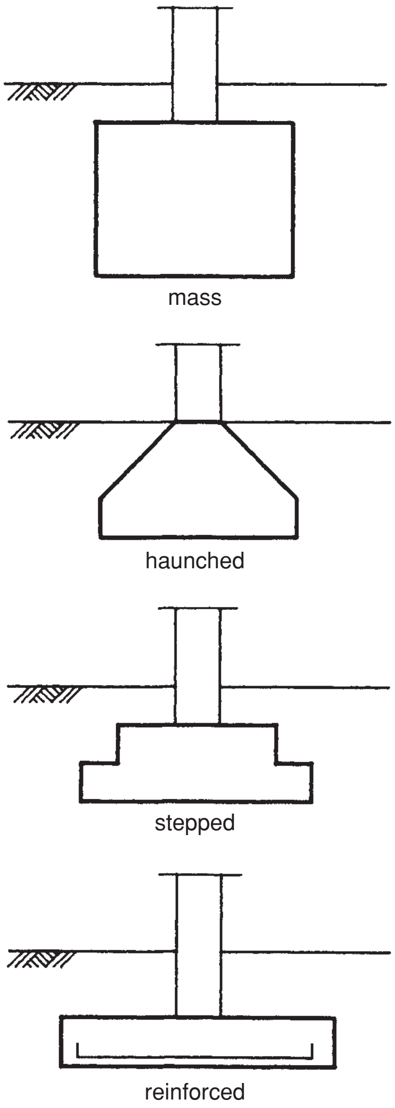

Fig. 1.5 Pad foundations.

economics of scale of the contract and construction costs, buildability, durability – as is all structural design choice. Only a brief description is given in this section to help understand the soil behaviour.

1.7.1 Pad foundations

Pad foundations tend to be the simplest and cheapest foundation type and are used when the soil is relatively strong or when the column loads are relatively light. They are usually square or rectangular on plan, of uniform thickness and generally of reinforced concrete. They can be stepped or haunched, if material costs outweigh labour costs. The reinforcement can vary from nothing at one extreme through to a heavy steel grillage at the other, with lightly reinforced sections being the most common. Typical types are shown in Fig. 1.5.

1.7.2 Strip footings

Strip footings are commonly used for the foundations to load-bearing walls. They are also used when the pad foundations for a number of columns in line are so closely spaced that the distance between the pads is approximately equal to the length of the side of the pads. (It is usually more economic and faster to excavate and cast concrete in one long strip, than as a series of closely spaced isolated pads.) They are also used on weak ground to increase the foundation bearing area, and thus reduce the bearing pressure – the weaker the ground then the wider the strip. When it is necessary to stiffen the strip to resist differential settlement, then *tee* or *inverted tee* strip footings can be adopted. Typical examples are shown in Fig. 1.6.

1.7.3 Raft foundations

When strips become so wide (because of heavy column loads or weak ground) that the clear distance between them is about the same as the width of the strips (or when the depth to suitable bearing capacity strata for strip footing loading becomes too deep), it is worth considering raft foundations. They are useful in restricting the differential settlement on variable ground, and to distribute variations of superstructure loading from area to area. Rafts can be stiffened (as strips can) by the inclusion of tee beams.

Rafts can also be made *buoyant* by the excavation (displacement) of a depth of soil, similar to the way that seagoing rafts are made to float by displacing an equal weight of

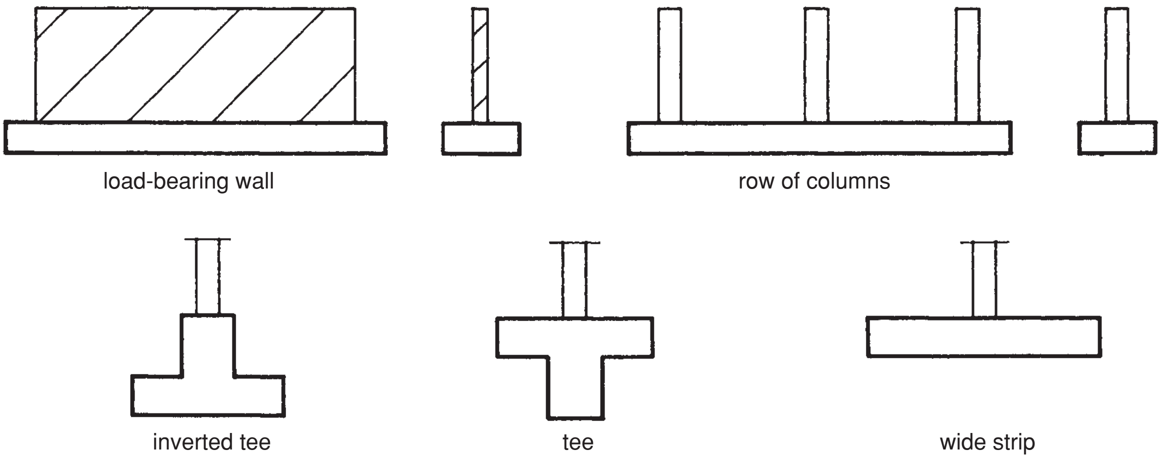

Fig. 1.6 Strip footings.