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Fourth edition



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Fourth edition

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**E & FN SPON**

An Imprint of Chapman & Hall

London · Glasgow · Weinheim · New York · Tokyo · Melbourne · Madras

**Published by E & FN Spon, an imprint of Chapman & Hall,  
2–6 Boundary Row, London SE1 8HN, UK**

Chapman & Hall, 2–6 Boundary Row, London SE1 8HN, UK

Chapman & Hall GmbH, Pappelallee 3, 69469 Weinheim, Germany

Chapman & Hall USA, 115 Fifth Avenue, New York, NY10003, USA

Chapman & Hall Japan, ITP-Japan, Kyowa Building, 3F, 2–2–1  
Hirakawacho, Chiyoda-ku, Tokyo 102, Japan

Chapman & Hall Australia, Thomas Nelson Australia, 102 Dodds  
Street, South Melbourne, Victoria 3205, Australia

Chapman & Hall India, R.Seshadri, 32 Second Main Road, CIT East,  
Madras 600 035, India

First edition 1977

This edition published in the Taylor & Francis e-Library, 2004.

To purchase your own copy of this or any of Taylor & Francis or Routledge's collection of thousands of eBooks please go to [www.eBookstore.tandf.co.uk](http://www.eBookstore.tandf.co.uk).

Third edition 1987

Fourth edition 1994

© 1977, 1981, 1987 Palladian, 1991, 1994 E & FN Spon

ISBN 0-203-47457-0 Master e-book ISBN

ISBN 0-203-23885-0 (OEB Format)

ISBN 0 419 18450 3 (Print Edition)

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A Catalogue record for this book is available from the British Library

Library of Congress Cataloging-in-Publication Data available

# Contents

<a href="#"><u>Preface to fourth edition</u></a>	<a href="#"><u>xi</u></a>
<a href="#"><u>Preface to first edition</u></a>	<a href="#"><u>xiii</u></a>
<a href="#"><u>Chapter 1 General principles and practices</u></a>	<a href="#"><u>1</u></a>
<a href="#"><u>1.1 Function of piles</u></a>	<a href="#"><u>1</u></a>
<a href="#"><u>1.2 Historical</u></a>	<a href="#"><u>1</u></a>
<a href="#"><u>1.3 Calculations of load-carrying capacity</u></a>	<a href="#"><u>2</u></a>
<a href="#"><u>1.4 Dynamic piling formulae</u></a>	<a href="#"><u>3</u></a>
<a href="#"><u>1.5 Code of practice requirements</u></a>	<a href="#"><u>4</u></a>
<a href="#"><u>1.6 Responsibilities of engineer and contractor</u></a>	<a href="#"><u>5</u></a>
<a href="#"><u>1.7 References</u></a>	<a href="#"><u>6</u></a>
<a href="#"><u>Chapter 2 Types of pile</u></a>	<a href="#"><u>7</u></a>
<a href="#"><u>2.1 Classification of piles</u></a>	<a href="#"><u>7</u></a>
<a href="#"><u>2.2 Driven displacement piles</u></a>	<a href="#"><u>9</u></a>
<a href="#"><u>2.2.1 Timber piles</u></a>	<a href="#"><u>9</u></a>
<a href="#"><u>2.2.2 Precast concrete piles</u></a>	<a href="#"><u>13</u></a>
<a href="#"><u>2.2.3 Jointed precast concrete piles</u></a>	<a href="#"><u>23</u></a>
<a href="#"><u>2.2.4 Steel piles</u></a>	<a href="#"><u>24</u></a>
<a href="#"><u>2.2.5 Shoes for steel piles</u></a>	<a href="#"><u>32</u></a>
<a href="#"><u>2.2.6 Working stresses for steel piles</u></a>	<a href="#"><u>33</u></a>
<a href="#"><u>2.3 Driven-and-cast-in-place displacement piles</u></a>	<a href="#"><u>35</u></a>
<a href="#"><u>2.3.1 General</u></a>	<a href="#"><u>35</u></a>
<a href="#"><u>2.3.2 Withdrawable-tube types</u></a>	<a href="#"><u>37</u></a>
<a href="#"><u>2.3.3 Shell types</u></a>	<a href="#"><u>39</u></a>
<a href="#"><u>2.3.4 Working stresses on driven-and-cast-in-place piles</u></a>	<a href="#"><u>42</u></a>
<a href="#"><u>2.4 Replacement piles</u></a>	<a href="#"><u>42</u></a>
<a href="#"><u>2.4.1 General</u></a>	<a href="#"><u>42</u></a>
<a href="#"><u>2.4.2 Bored-and-cast-in-place piles</u></a>	<a href="#"><u>42</u></a>
<a href="#"><u>2.4.3 Drilled-in tubular piles</u></a>	<a href="#"><u>45</u></a>
<a href="#"><u>2.5 Composite piles</u></a>	<a href="#"><u>47</u></a>
<a href="#"><u>2.6 Minipiles and micropiles</u></a>	<a href="#"><u>48</u></a>
<a href="#"><u>2.7 Factors governing choice of type of pile</u></a>	<a href="#"><u>48</u></a>
<a href="#"><u>2.8 References</u></a>	<a href="#"><u>50</u></a>

<b><u>Chapter 3 Piling equipment and methods</u></b>	<b><u>51</u></b>
<u>3.1 Equipment for driven piles</u>	<u>51</u>
<u>3.1.1 Piling frames</u>	<u>51</u>
<u>3.1.2 Crane supported (hanging) leaders</u>	<u>52</u>
<u>3.1.3 Trestle guides</u>	<u>54</u>
<u>3.1.4 Piling hammers</u>	<u>57</u>
<u>3.1.5 Piling vibrators</u>	<u>63</u>
<u>3.1.6 Selection of type of piling hammer</u>	<u>65</u>
<u>3.1.7 Noise control in pile driving</u>	<u>67</u>

3.1.8	<a href="#">Pile helmets and driving caps</a>	72
3.1.9	<a href="#">Jetting piles</a>	74
3.2	<a href="#">Equipment for installing driven-and-cast-in-place piles</a>	76
3.3	<a href="#">Equipment for installing bored-and-cast-in-place piles</a>	79
3.3.1	<a href="#">Power augers</a>	79
3.3.2	<a href="#">Grabbing rigs with casing oscillators</a>	81
3.3.3	<a href="#">Continuous flight auger drilling rigs</a>	81
3.3.4	<a href="#">Reverse-circulation drilling rigs</a>	83
3.3.5	<a href="#">Tripod rigs</a>	83
3.3.6	<a href="#">Drilling for piles with bentonite slurry</a>	85
3.3.7	<a href="#">Base and skin grouting of bored and cast-in-place piles</a>	86
3.4	<a href="#">Procedure in pile installation</a>	87
3.4.1	<a href="#">Driving timber piles</a>	87
3.4.2	<a href="#">Driving precast (including prestressed) concrete piles</a>	88
3.4.3	<a href="#">Driving steel piles</a>	89
3.4.4	<a href="#">Driving and concreting steel shell piles</a>	90
3.4.5	<a href="#">The installation of withdrawable-tube types of driven-and-cast-in-place piles</a>	90
3.4.6	<a href="#">The installation of bored-and-cast-in-place piles by power auger equipment</a>	90
3.4.7	<a href="#">Concreting pile shafts under water</a>	93
3.4.8	<a href="#">The installation of bored-and-cast-in-place piles by grabbing, vibratory and reverse-circulation rigs</a>	95
3.4.9	<a href="#">The installation of bored-and-cast-in-place piles by tripod rigs</a>	95
3.4.10	<a href="#">The installation of raking piles</a>	95
3.4.11	<a href="#">Positional tolerances</a>	96
3.5	<a href="#">Constructing piles in groups</a>	97
3.6	<a href="#">References</a>	97
<b>Chapter 4</b>	<b><a href="#">Calculating the resistance of piles to compressive loads</a></b>	<b>99</b>
4.1	<a href="#">General considerations</a>	99
4.1.1	<a href="#">The basic approach to the calculation of pile resistance</a>	99
4.1.2	<a href="#">The behaviour of a pile under load</a>	100
4.1.3	<a href="#">Definition of failure load</a>	101
4.1.4	<a href="#">Allowable loads on piles</a>	102
4.2	<a href="#">Piles in cohesive soils</a>	103
4.2.1	<a href="#">Driven displacement piles</a>	103
4.2.2	<a href="#">Driven-and-cast-in-place displacement piles</a>	110
4.2.3	<a href="#">Bored-and-cast-in-place non-displacement piles</a>	111
4.2.4	<a href="#">The effects of time on pile resistance in clays</a>	113

<a href="#"><u>4.3 Piles in cohesionless soil</u></a>	<a href="#"><u>114</u></a>
<a href="#"><u>4.3.1 General</u></a>	<a href="#"><u>114</u></a>
<a href="#"><u>4.3.2 Driven piles in cohesionless soils</u></a>	<a href="#"><u>119</u></a>
<a href="#"><u>4.3.3 Piles with open-ends driven into cohesionless soils</u></a>	<a href="#"><u>121</u></a>
<a href="#"><u>4.3.4 Grouted driven piles</u></a>	<a href="#"><u>122</u></a>
<a href="#"><u>4.3.5 Driven-and-cast-in-place piles in cohesionless soils</u></a>	<a href="#"><u>123</u></a>
<a href="#"><u>4.3.6 Bored-and-cast-in-place piles in cohesionless soils</u></a>	<a href="#"><u>123</u></a>
<a href="#"><u>4.3.7 The use of in-situ tests to predict the ultimate resistance of piles in cohesionless soils</u></a>	<a href="#"><u>124</u></a>
<a href="#"><u>4.3.8 Time effects for piles in cohesionless soils</u></a>	<a href="#"><u>129</u></a>
<a href="#"><u>4.4 Piles in soils intermediate between sands and clays</u></a>	<a href="#"><u>129</u></a>
<a href="#"><u>4.5 Piles in layered cohesive and cohesionless soils</u></a>	<a href="#"><u>131</u></a>
<a href="#"><u>4.6 The settlement of the single pile at the working load for piles in soil</u></a>	<a href="#"><u>133</u></a>
<a href="#"><u>4.7 Piles bearing on rock</u></a>	<a href="#"><u>138</u></a>
<a href="#"><u>4.7.1 Driven piles</u></a>	<a href="#"><u>138</u></a>
<a href="#"><u>4.7.2 Driven-and-cast-in-place piles</u></a>	<a href="#"><u>142</u></a>
<a href="#"><u>4.7.3 Bored-and-cast-in-place piles</u></a>	<a href="#"><u>143</u></a>

4.7.4	<a href="#">The settlement of the single pile at the working load for piles in rocks</a>	147
4.8	<a href="#">Piles in fill—negative skin friction</a>	148
4.8.1	<a href="#">Estimating negative skin friction</a>	148
4.8.2	<a href="#">Safety factors for negative skin friction</a>	152
4.8.3	<a href="#">Minimizing negative skin friction</a>	152
4.9	<a href="#">References</a>	153
4.10	<a href="#">Worked examples</a>	154
<b>Chapter 5</b>	<b><a href="#">Pile groups under compressive loading</a></b>	<b>166</b>
5.1	<a href="#">Group action in piled foundations</a>	166
5.2	<a href="#">Pile groups in cohesive soils</a>	168
5.2.1	<a href="#">Ultimate bearing capacity</a>	168
5.2.2	<a href="#">Settlement</a>	170
5.3	<a href="#">Pile groups in cohesionless soils</a>	179
5.3.1	<a href="#">Estimating settlements from standard penetration tests</a>	179
5.3.2	<a href="#">Estimating settlements from static cone penetration tests</a>	182
5.4	<a href="#">Deep pile groups in cohesive and cohesionless soils</a>	185
5.5	<a href="#">Pile groups terminating in rock</a>	186
5.6	<a href="#">Pile groups in filled ground</a>	189
5.7	<a href="#">Effects on pile groups of installation methods</a>	190
5.8	<a href="#">Precautions against heave effects in pile groups</a>	193
5.9	<a href="#">Pile groups beneath basements</a>	193
5.10	<a href="#">The optimization of pile groups to reduce differential settlements in clay</a>	196
5.11	<a href="#">References</a>	198
5.12	<a href="#">Worked examples</a>	199
<b>Chapter 6</b>	<b><a href="#">The design of piled foundations to resist uplift and lateral loading</a></b>	<b>208</b>
6.1	<a href="#">The occurrence of uplift and lateral loading</a>	208
6.2	<a href="#">Uplift resistance of piles</a>	210
6.2.1	<a href="#">General</a>	210
6.2.2	<a href="#">The uplift resistance of friction piles</a>	210
6.2.3	<a href="#">Piles with base enlargements</a>	212
6.2.4	<a href="#">Anchoring piles to rock</a>	214
6.2.5	<a href="#">The uplift resistance of drilled-in rock anchors</a>	215
6.3	<a href="#">Single vertical piles subjected to lateral loads</a>	221
6.3.1	<a href="#">Calculating the ultimate resistance to lateral loads</a>	223
6.3.2	<a href="#">Bending and buckling of partly embedded single vertical piles</a>	232
6.3.3	<a href="#">The deflection of vertical piles carrying lateral loads</a>	233

6.3.4	<a href="#">Elastic analysis of laterally-loaded vertical piles</a>	<a href="#">236</a>
6.3.5	<a href="#">The use of <math>p</math>-<math>y</math> curves</a>	<a href="#">241</a>
6.3.6	<a href="#">Effect of method of pile installation on behaviour under lateral loads and moments applied to pile head</a>	<a href="#">247</a>
6.3.7	<a href="#">The use of pressuremeter test to establish <math>p</math>-<math>y</math> curves</a>	<a href="#">247</a>
6.3.8	<a href="#">Calculation of lateral deflections and bending moments by elastic continuum methods</a>	<a href="#">250</a>
6.4	<a href="#">Lateral loads on raking piles</a>	<a href="#">253</a>
6.5	<a href="#">Lateral loads on groups of piles</a>	<a href="#">253</a>
6.6	<a href="#">References</a>	<a href="#">257</a>
6.7	<a href="#">Worked examples</a>	<a href="#">258</a>
<b>Chapter 7</b>	<b><a href="#">The structural design of piles and pile groups</a></b>	<b><a href="#">272</a></b>
7.1	<a href="#">General design requirements</a>	<a href="#">272</a>
7.2	<a href="#">Designing reinforced concrete piles for lifting after fabrication</a>	<a href="#">272</a>
7.3	<a href="#">Designing piles to resist driving stresses</a>	<a href="#">275</a>
7.4	<a href="#">The effects of bending on piles below ground level</a>	<a href="#">277</a>
7.5	<a href="#">The design of axially-loaded piles as columns</a>	<a href="#">278</a>

7.6	<a href="#">Lengthening piles</a>	278
7.7	<a href="#">Bonding piles with caps and ground beams</a>	280
7.8	<a href="#">The design of pile caps</a>	281
7.9	<a href="#">The design of pile capping beams and connecting ground beams</a>	284
7.10	<a href="#">References</a>	289
7.11	<a href="#">Worked examples</a>	289
<b>Chapter 8</b>	<b><a href="#">Piling for marine structures</a></b>	<b>299</b>
8.1	<a href="#">Berthing structures and jetties</a>	299
8.1.1	<a href="#">Loading on piles from berthing impact forces</a>	301
8.1.2	<a href="#">Mooring forces on piles</a>	306
8.1.3	<a href="#">Wave forces on piles</a>	306
8.1.4	<a href="#">Current forces on piles</a>	309
8.1.5	<a href="#">Wind forces on piles</a>	311
8.1.6	<a href="#">Forces on piles from floating ice</a>	311
8.1.7	<a href="#">Materials for piles in jetties and dolphins</a>	312
8.2	<a href="#">Fixed offshore platforms</a>	313
8.3	<a href="#">Pile installations for marine structures</a>	315
8.4	<a href="#">References</a>	319
8.5	<a href="#">Worked examples</a>	319
<b>Chapter 9</b>	<b><a href="#">Miscellaneous piling problems</a></b>	<b>330</b>
9.1	<a href="#">Piling for machinery foundations</a>	330
9.1.1	<a href="#">General principles</a>	330
9.1.2	<a href="#">Pile design for static machinery loading</a>	331
9.1.3	<a href="#">Pile design for dynamic loading from machinery</a>	331
9.2	<a href="#">Piling for underpinning</a>	332
9.2.1	<a href="#">Requirements for underpinning</a>	332
9.2.2	<a href="#">Piling methods in underpinning work</a>	332
9.3	<a href="#">Piling in mining subsidence areas</a>	339
9.4	<a href="#">Piling in frozen ground</a>	342
9.4.1	<a href="#">General effects</a>	342
9.4.2	<a href="#">The effects of adfreezing on piled foundations</a>	342
9.4.3	<a href="#">Piling in permafrost regions</a>	343
9.5	<a href="#">Piled foundations for bridges on land</a>	344
9.5.1	<a href="#">Selection of pile type</a>	344
9.5.2	<a href="#">Imposed loads on bridge piling</a>	345
9.6	<a href="#">Piled foundations for over-water bridges</a>	349

9.6.1	<a href="#">Selection of pile type</a>	<a href="#">349</a>
9.6.2	<a href="#">Imposed loads on piers of over-water bridges</a>	<a href="#">350</a>
9.6.3	<a href="#">Pile caps for over-water bridges</a>	<a href="#">353</a>
9.7	<a href="#">References</a>	<a href="#">355</a>
9.8	<a href="#">Worked example</a>	<a href="#">355</a>
<b>Chapter 10</b>	<b><a href="#">The durability of piled foundations</a></b>	<b><a href="#">357</a></b>
10.1	<a href="#">General</a>	<a href="#">357</a>
10.2	<a href="#">Durability and protection of timber piles</a>	<a href="#">357</a>
10.2.1	<a href="#">Timber piles in land structures</a>	<a href="#">357</a>
10.2.2	<a href="#">Timber piles in river and marine structures</a>	<a href="#">361</a>
10.3	<a href="#">Durability and protection of concrete piles</a>	<a href="#">365</a>
10.3.1	<a href="#">Concrete piles in land structures</a>	<a href="#">365</a>
10.3.2	<a href="#">Concrete piles in marine structures</a>	<a href="#">368</a>
10.4	<a href="#">Durability and protection of steel piles</a>	<a href="#">369</a>
10.4.1	<a href="#">Steel piles for land structures</a>	<a href="#">369</a>
10.4.2	<a href="#">Steel piles for marine structures</a>	<a href="#">370</a>
10.5	<a href="#">References</a>	<a href="#">372</a>
<b>Chapter 11</b>	<b><a href="#">Site investigations, piling contracts, pile testing</a></b>	<b><a href="#">373</a></b>
11.1	<a href="#">Site investigations</a>	<a href="#">373</a>
11.1.1	<a href="#">Planning the investigation</a>	<a href="#">373</a>

11.1.2 <a href="#">Boring in soil</a>	<a href="#">374</a>
11.1.3 <a href="#">Drilling in rock</a>	<a href="#">375</a>
11.1.4 <a href="#">In-situ testing in soils and rocks</a>	<a href="#">376</a>
11.2 <a href="#">Piling contracts and specifications</a>	<a href="#">380</a>
11.2.1 <a href="#">Contract procedure</a>	<a href="#">380</a>
11.2.2 <a href="#">Piling specifications</a>	<a href="#">382</a>
11.3 <a href="#">Control of pile installation</a>	<a href="#">383</a>
11.3.1 <a href="#">Driven piles</a>	<a href="#">383</a>
11.3.2 <a href="#">Driven-and-cast-in-place piles</a>	<a href="#">385</a>
11.3.3 <a href="#">Bored-and-cast-in place piles</a>	<a href="#">386</a>
11.4 <a href="#">Load testing of piles</a>	<a href="#">386</a>
11.4.1 <a href="#">Compression tests</a>	<a href="#">386</a>
11.4.2 <a href="#">Interpretation of compression test records</a>	<a href="#">393</a>
11.4.3 <a href="#">Uplift tests</a>	<a href="#">396</a>
11.4.4 <a href="#">Lateral loading tests</a>	<a href="#">398</a>
11.5 <a href="#">Tests for the structural integrity of piles</a>	<a href="#">399</a>
11.6 <a href="#">References</a>	<a href="#">400</a>
<b><a href="#">Appendix Properties of materials</a></b>	<b><a href="#">402</a></b>
1. <a href="#">Cohesionless soils</a>	<a href="#">402</a>
2. <a href="#">Cohesive and organic soils</a>	<a href="#">402</a>
3. <a href="#">Rocks and other materials</a>	<a href="#">403</a>
<b><a href="#">Name index</a></b>	<b><a href="#">405</a></b>
<b><a href="#">Subject index</a></b>	<b><a href="#">408</a></b>

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## Preface to fourth edition

In this edition the chapters dealing with methods of calculating the bearing capacity and settlements of piles and pile groups have been extensively revised to take account of recent research and development on this subject. A draft of Eurocode No. 7, *Geotechnics*, had been completed at the time of preparing this edition. Reference is made to the draft requirements of the Eurocode in the chapters dealing with the design of single piles and pile groups.

Generally the descriptions of types of pile, piling equipment and methods of installation have been brought up-to-date with current practice and a new section has been added on piled foundations for bridges.

The author is grateful to Mr Malcolm J. Brittain, MICE, of Grove Structural Consultants, for assistance in bringing [Chapter 7](#) into line with British Standard Code of Practice BS 8110 for structural concrete and for revising the worked examples in this chapter. The help of Mr Keith Brook, FICE in compiling the revised [Table 10.1](#) is also gratefully acknowledged.

Many specialist piling contractors and manufacturers of piling equipment have kindly supplied technical information and illustrations of their processes and products. Where appropriate the source of this information is given in the text.

In addition, the author wishes to thank the following for the supply of photographs and illustrations from technical publications and brochures:

Akermanns Industries (UK) Limited

Figures [3.4](#) and [3.12](#)

American Society of Civil Engineers

Figures [4.9](#), [4.15](#), [4.16](#), [4.44](#), [5.24](#), [6.25](#), [6.26](#), [6.30](#), [6.32](#), [6.33](#), [6.35](#) and [6.40](#)

Ballast Nedam Groep N.V.

Figures [9.23](#) and [9.24](#)

Brendan Butler Limited

[Figure 3.26](#)

The British Petroleum Company Limited

[Figure 8.15](#)

BSP International Foundations Limited

Figures [3.6](#), [3.13](#), [3.14](#), [3.15](#), [3.25](#), [3.27](#), [3.28](#) and [3.30](#)

Building Research Establishment Princes Risborough Laboratory

Figures [10.2a](#) and [10.2b](#)

Canadian Geotechnical Journal

Figures [4.34](#), [4.41](#), [4.42](#), [5.11](#), [5.33](#) and [6.9](#)

Cement and Concrete Association

[Figure 7.12](#)

Cementation Piling and Foundations Limited

Figures [3.24](#), [3.30](#), [3.34](#), [9.6](#) and [11.6](#)

Central Electricity Generating Board

[Figure 2.17](#)

C.E.T. Plant Limited

Figures [3.2](#) and [3.3](#)

CIRIA/Butterworth

Figures [4.14](#) and [5.22](#)

Construction Industry Research and Information Association (CIRIA)

[Figure 4.11](#)

Danish Geotechnical Institute

[Figure 6.21](#)

Dar-al-Handasah Consultants

[Figure 9.15](#)

Department of the Environment

[Figure 10.1](#)

C. Evans and Sons Limited

[Figure 3.17](#)

Hans Feibusch, Consulting Engineer

[Figure 3.5](#)

Fondedile Foundations Limited

[Figure 9.5](#)

The Geological Society

[Figure 8.9](#)

International Society for Soil Mechanics and Foundation Engineering

Figures [3.35](#), [5.18](#), [5.19](#), [6.18](#), [6.41](#), [9.20](#) and [9.21](#)

Institution of Civil Engineers

Figures [4.32](#), [5.20](#), [5.21](#), [5.28](#), [5.29](#), [5.30](#), [5.36](#), [5.37](#), [6.59](#), [9.22](#), [9.26](#) and [9.27](#)

Keilawarra Limited

[Figure 3.32](#)

McEvoy Oilfield Equipment Limited

[Figure 2.16](#)

National Coal Board

Figures [2.17](#), [4.30](#) and [8.2](#)

Pentech Press	Figures <a href="#">4.40</a> and <a href="#">5.14</a>
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Sheet Piling Contractors Limited	<a href="#">Figure 3.20</a>
Soil Mechanics Limited	Figures <a href="#">2.10</a> and <a href="#">2.11</a>
Swedish Geotechnical Society	<a href="#">Figure 5.15</a>
Trans-Tech Publications	Figures <a href="#">6.49</a> and <a href="#">6.50</a>
University of Austin in Texas	Figures <a href="#">6.36</a> , <a href="#">6.37</a> , <a href="#">6.38</a> and <a href="#">6.39</a>
United States Department of Transportation	<a href="#">Figure 4.33</a>
Vales Plant Register Limited	Figures <a href="#">3.1</a> and <a href="#">3.13</a>
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M.J.T.  
Deal, 1993

## Preface to first edition

Piling is both an art and a science. The art lies in selecting the most suitable type of pile and method of installation for the ground conditions and the form of the loading. Science enables the engineer to predict the behaviour of the piles once they are in the ground and subject to loading. This behaviour is influenced profoundly by the method used to install the piles and it cannot be predicted solely from the physical properties of the pile and of the undisturbed soil. A knowledge of the available types of piling and methods of constructing piled foundations is essential for a thorough understanding of the science of their behaviour. For this reason the author has preceded the chapters dealing with the calculation of allowable loads on piles and deformation behaviour by descriptions of the many types of proprietary and non-proprietary piles and the equipment used to install them.

In recent years substantial progress has been made in developing methods of predicting the behaviour of piles under lateral loading. This is important in the design of foundations for deep-water terminals for oil tankers and oil carriers and for offshore platforms for gas and petroleum production. The problems concerning the lateral loading of piles have therefore been given detailed treatment in this book.

The author has been fortunate in being able to draw on the world-wide experience of George Wimpey and Company Limited, his employers for nearly 30 years, in the design and construction of piled foundations. He is grateful to the management of Wimpey Laboratories Ltd. and their parent company for permission to include many examples of their work. In particular, thanks are due to P.F. Winfield, F.I.Struct.E., for his assistance with the calculations and his help in checking the text and worked examples.

Burton-on-Stather, 1977

M.J.T.

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# CHAPTER 1

## General principles and practices

### 1.1 Function of piles

Piles are columnar elements in a foundation which have the function of transferring load from the superstructure through weak compressible strata or through water, onto stiffer or more compact and less compressible soils or onto rock. They may be required to carry uplift loads when used to support tall structures subjected to overturning forces from winds or waves. Piles used in marine structures are subjected to lateral loads from the impact of berthing ships and from waves. Combinations of vertical and horizontal loads are carried where piles are used to support retaining walls, bridge piers and abutments, and machinery foundations.

### 1.2 Historical

The driving of bearing piles to support structures is one of the earliest examples of the art and science of the civil engineer. In Britain there are numerous examples of timber piling in bridge works and riverside settlements constructed by the Romans. In mediaeval times, piles of oak and alder were used in the foundations of the great monasteries constructed in the fenlands of East Anglia. In China, timber piling was used by the bridge builders of the Han Dynasty (200 BC to AD 200). The carrying capacity of timber piles is limited by the girth of the natural timbers and the ability of the material to withstand driving by hammer without suffering damage due to splitting or splintering. Thus primitive rules must have been established in the earliest days of piling by which the allowable load on a pile was determined from its resistance to driving by a hammer of known weight and with a known height of drop. Knowledge was also accumulated regarding the durability of piles of different species of wood, and measures taken to prevent decay by charring the timber or by building masonry rafts on pile heads cut off below water level.

Timber, because of its strength combined with lightness, durability and ease of cutting and handling, remained the only material used for piling until comparatively recent times. It was replaced by concrete and steel only because these newer materials could be fabricated into units that were capable of sustaining compressive, bending and tensile forces far beyond the capacity of a timber pile of like dimensions. Concrete, in particular, was adaptable to in-situ forms of construction which facilitated the installation of piled foundations in drilled holes in situations where noise, vibration and ground heave had to be avoided.

Reinforced concrete, which was developed as a structural medium in the late nineteenth and early twentieth centuries, largely replaced timber for high-capacity piling for works on land. It could be precast in various structural forms to suit the imposed loading and ground conditions, and its durability was satisfactory for most soil and immersion conditions. The partial replacement of driven precast concrete piles by numerous forms of cast-in-situ piles has been due more to the development of highly efficient machines for drilling pile boreholes of large diameter and great depth in a wide range of soil and rock conditions, than to any deficiency in the performance of the precast concrete element.

Steel has been used to an increasing extent for piling due to its ease of fabrication and handling and its ability to withstand hard driving. Problems of corrosion in marine structures have been overcome by the introduction of durable coatings and cathodic protection.

### 1.3 Calculations of load-carrying capacity

While materials for piles can be precisely specified, and their fabrication and installation can be controlled to conform to strict specification and code of practice requirements, the calculation of their load-carrying capacity is a complex matter which at the present time is based partly on theoretical concepts derived from the sciences of soil and rock mechanics, but mainly on empirical methods based on experience. Practice in calculating the ultimate carrying capacity of piles based on the principles of soil mechanics differs greatly from the application of these principles to shallow spread foundations. In the latter case the entire area of soil supporting the foundation is exposed and can be inspected and sampled to ensure that its bearing characteristics conform to those deduced from the results of exploratory boreholes and soil tests. Provided that the correct constructional techniques are used the disturbance to the soil is limited to a depth of only a few centimetres below the excavation level for a spread foundation. Virtually the whole mass of soil influenced by the bearing pressure remains undisturbed and unaffected by the constructional operations (Figure 1.1 a). Thus the safety factor against general shear failure of the spread foundation and its settlement under the design working load can be predicted from a knowledge of the physical characteristics of the *undisturbed* soil with a degree of certainty which depends only on the complexity of the soil stratification.

The conditions which govern the supporting capacity of the piled foundation are quite different. No matter whether the pile is installed by driving with a hammer, by jetting, by vibration, by jacking, screwing or drilling, the soil in contact with the pile face, from which the pile derives its support by skin friction, and its resistance to lateral loads, is completely disturbed by the method of installation. Similarly the soil or rock beneath the toe of a pile is compressed (or sometimes loosened) to an extent which may affect significantly its end-bearing resistance (Figure 1.1b). Changes take place in the conditions at the pile-soil interface over periods of days, months or years which materially affect the skin-friction resistance of a pile. These changes may be due to the dissipation of excess pore pressure set up by installing the pile, to the relative effects of friction and cohesion which in turn depend on the relative pile-to-soil movement, and to chemical or electro-chemical effects caused by the hardening of the concrete or the corrosion of the steel in contact with the soil. Where piles are installed in groups to carry heavy foundation loads, the operation of driving or drilling for adjacent piles can cause changes in the carrying capacity and load-settlement characteristics of the piles in the group that have already been driven.

In the present state of knowledge, the effects of the various methods of pile installation on the carrying capacity and deformation characteristics cannot be calculated by the strict application of soil or rock mechanics theory. The general procedure is to apply simple empirical factors to the strength density, and compressibility properties of the undisturbed soil or rock. The various factors which can be used depend on the particular method of installation and are based on experience and on the results of field loading tests.

The basis of the 'soil mechanics approach' to calculating the carrying capacity of piles is that the

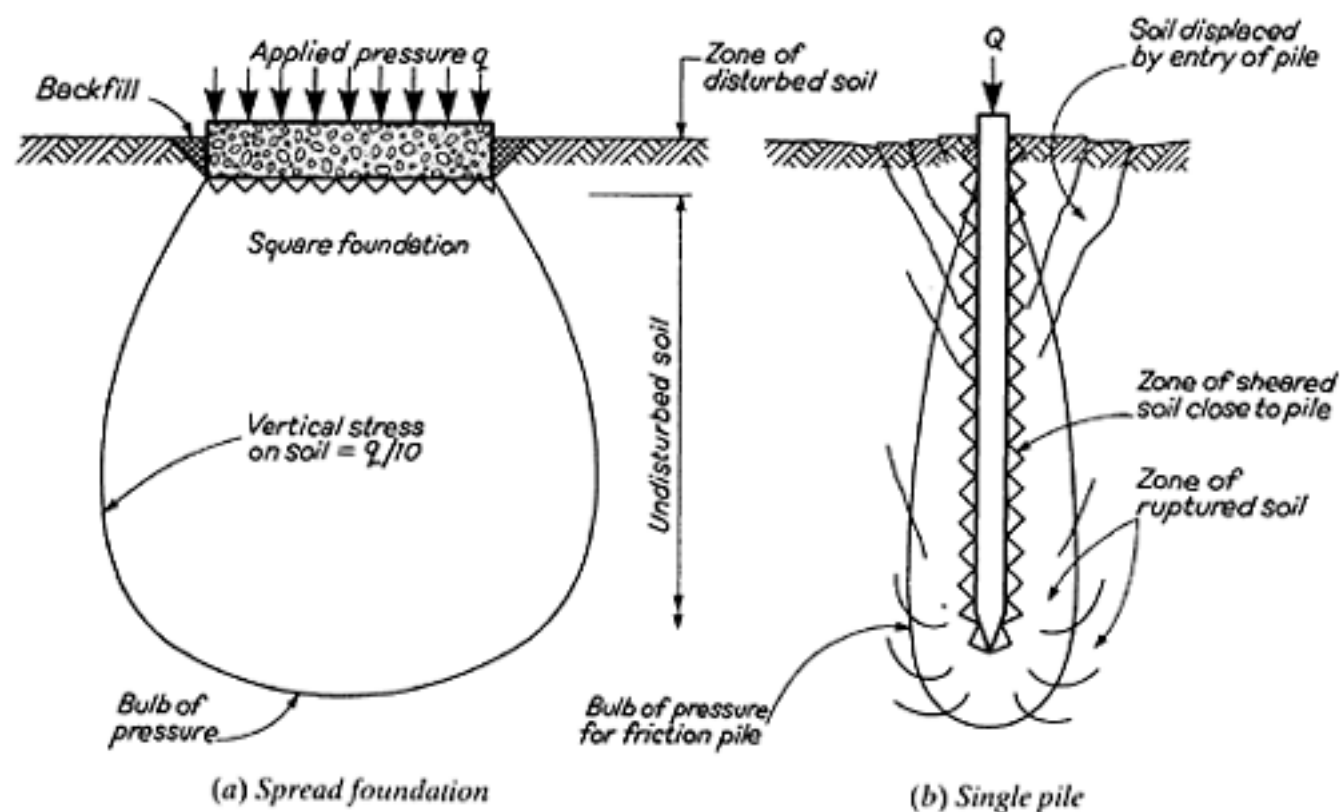


Fig. 1.1 Comparison of pressure distribution and soil disturbance beneath spread and piled foundations

total resistance of the pile to compression loads is the sum of two components, namely skin friction and end resistance. A pile in which the skin-frictional component predominates is known as a friction pile ([Figure 1.2a](#)), while a pile bearing on rock or some other hard incompressible material is known as an end-bearing pile ([Figure 1.2b](#)). However, even if it is possible to make a reliable estimate of total pile resistance a further difficulty arises in predicting the problems involved in installing the piles to the depths indicated by the empirical or semi-empirical calculations. It is one problem to calculate that a precast concrete pile must be driven to a depth of, say, 20 metres to carry safely a certain working load, but quite another problem to decide on the energy of the hammer required to drive the pile to this depth, and yet another problem to decide whether or not the pile will be irredeemably shattered while driving it to the required depth. In the case of driven and cast-in-place piles the ability to drive the piling tube to the required depth and then to extract it within the pulling capacity of the piling

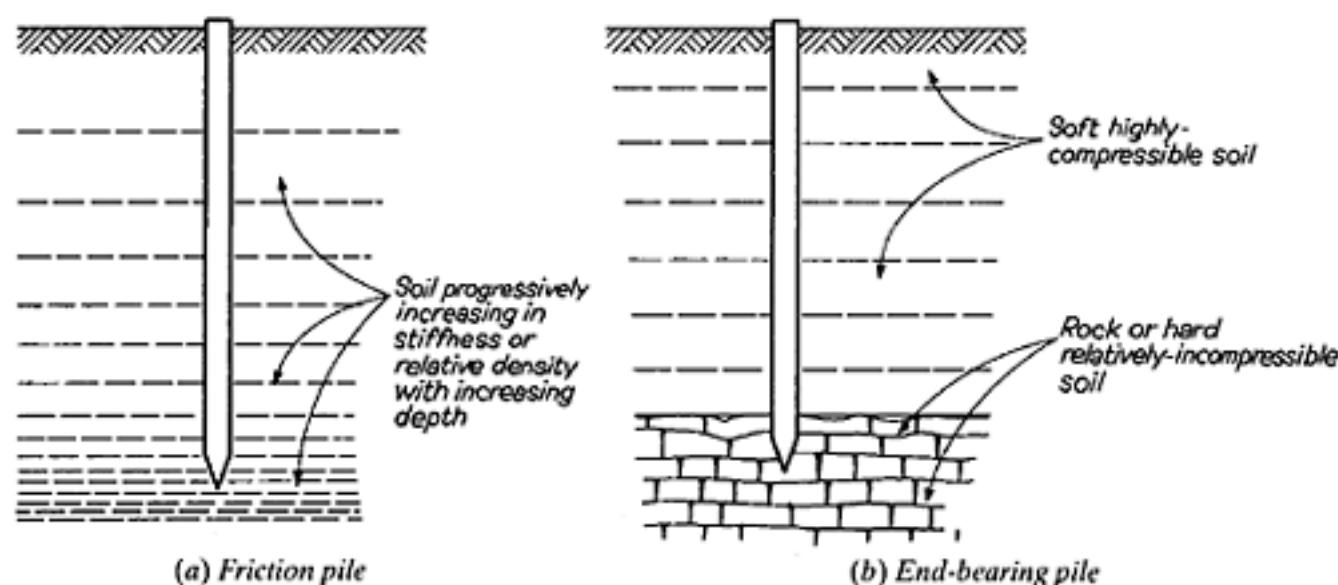


Fig. 1.2 Types of bearing pile

rig must be correctly predicted.

Bjerrum([1.1](#)) has drawn attention to the importance of time effects in calculating the resistance of a pile in clay. The time effects include the rate of applying load to a pile, and the time interval between installing and testing a pile. The skin-frictional resistance of a pile in clay loaded very slowly may only be one-half of that which is measured under the rate at which load is normally applied during a pile loading test. The slow rate of loading may correspond to that of a building under construction, yet the ability of a pile to carry its load is judged on its behaviour under a comparatively rapid loading test made only a few days after installation. The carrying capacity of a pile in sands may also diminish with time, but in spite of the importance of such time effects both in cohesive and cohesionless soils the only practicable way of determining the load-carrying capacity of a piled foundation is to confirm the design calculations by short-term tests on isolated single piles, and then to allow in the safety factor for any reduction in the carrying capacity with time. The effects of grouping piles can be taken into account by considering the pile group to act as a block foundation, as described in [Chapter 5](#).

## 1.4 Dynamic piling formulae

The soil mechanics approach to calculating allowable working loads on piles is that of determining the resistance of static loads applied at the test-loading stage or during the working life of the structure. Methods of calculation based on the measurement of the resistance encountered when driving a pile were briefly mentioned in the context of history. Until comparatively recently all piles were installed by driving them with a simple falling ram or drop hammer. Since there is a relationship between the downward movement of a pile under a blow of given energy and its ultimate resistance to static loading, when all piles were driven by a falling ram a considerable body of experience was built up and simple empirical formulae established from which the ultimate resistance of the pile could be calculated from the 'set' of the pile due to each hammer blow at the final stages of driving. However, there are many drawbacks to the use of these formulae with modern pile-driving equipment particularly when used in conjunction with diesel hammers. The energy of blow delivered to the pile by these types increases as the resistance of the ground increases. The energy can also vary with the mechanical condition of the hammer and its operating temperature. They now are largely discredited as a means of predicting the

resistance of piles to static loading unless the driving tests are performed on piles instrumented to measure the energy transferred to the pile head. If this is done the dynamic analyser (see [Section 7.3](#)) provides the actual rather than the assumed energy of blow enabling the dynamic formula to be used as a means of site control when driving the working piles. Dynamic pile formulae are allowed to be used by Eurocode EC7 provided that their validity has been demonstrated by experience in similar ground conditions or verified by static loading tests.

Steady progress has been made in the development of ‘static’ formulae and, with increasing experience of their use backed by research, the soil mechanics approach can be applied to all forms of piling in all ground conditions, whereas even if a reliable dynamic formula could be established its use would be limited to driven piles only. Furthermore, by persevering with static formulae the desirable goal of predicting accurately the load-deformation characteristics will eventually be attained. However, dynamic formulae still have their uses in predicting the stresses within the material forming the pile during driving and hence in assessing the risk of pile breakage, and their relevance to this problem is discussed in [Chapter 7](#).

## 1.5 Code of practice requirements

The uncertainties in the methods of predicting allowable or ultimate loads on piles are reflected in the information available to designers in the various codes of practice which cover piling. The British Standard Code of Practice BS 8004 (Foundations) defines the ultimate bearing capacity of a pile as ‘The load at which the resistance of the soil becomes fully mobilized’ and goes on to state that this is generally taken as the load causing the head of the pile to settle a depth of 10% of the pile width or diameter. BS 8004 does not define ultimate loads for uplift or lateral loading. Specific design information is limited to stating the working stresses on the pile material and the cover required to the reinforcement, the requirements for positional tolerance and verticality also being stated. No quantitative information is given on skin friction or end-bearing values in soils or rocks, whereas it will be seen from [Chapter 2](#) that many countries place limits on these values or on maximum pile loads in order to ensure that piles are not driven very heavily so as to achieve the maximum working load that can be permitted by the allowable stress on the cross-sectional area of the pile shaft.

A conflict can arise in British practice where structures, including foundation substructures, are designed to the requirements of BS 8110 and their foundations to those of BS 8004. In the former document partial safety factors are employed to increase the characteristic dead and imposed loads to amounts which are defined as the ultimate load. The ultimate resistance of the structure is calculated on the basis of the characteristic strength of the material used for its construction which again is multiplied by a partial safety factor to take into account the possibility of the strength of the material used being less than the designed characteristic strength. Then, if the ultimate load on the structure does not exceed its ultimate resistance to load, the *ultimate or collapse limit-state* is not reached and the structure is safe. Deflections of the structure are also calculated to ensure that these do not exceed the maximum values that can be tolerated by the structure or user, and thus to ensure that the *serviceability limit-state* is not reached.

When foundations are designed in accordance with BS 8004, the maximum working load is calculated. This is comparable to the characteristic loading specified in BS 8110, *i.e.* the sum of the maximum dead and imposed loading. The resistance offered by the ground to this loading is calculated. This is based on *representative* shearing strength parameters of the soils or rocks concerned. These are not necessarily minimum or average values but are parameters selected by the engineer using his experience and judgement and taking into account the variability in the geological conditions, the number of test results available, the care used in taking samples and selecting them for test, and experience of other site investigations and of the behaviour of existing structures in the locality. The maximum load imposed by the sub-structure on the ground must not exceed the calculated resistance of the ground multiplied by the appropriate safety factor. The latter takes into account the risks of excessive total and differential settlements of the structure as well as allowing for uncertainties in the design method and in the values selected for the shearing strength parameters.

The settlements of the foundations are then calculated, the loading adopted for these calculations being not necessarily the same as that used to obtain the maximum working load. It is the usual practice to take the actual dead load and the whole or some proportion of the imposed load, depending on the type of loading; *i.e.* the full imposed load is taken for structures such as grain silos, but the imposed wind loading may not be taken into account when calculating long-term settlements.

There is no reason why this dual approach should not be adopted when designing structures and their foundations, but it is important that the designer of the structure should make an unambiguous

statement of the loading conditions which are to be supported by the ground. If he provides the foundation engineer with a factored ultimate load, and the foundation engineer then uses this load with a safety factor of, say, 2.5 or 3 on the calculated shearing resistance of the ground, the resulting design may be over-conservative. Similarly, if the ultimate load is used to calculate settlements the values obtained will be unrealistically large. The foundation engineer must know the actual dead load of the superstructure and sub-structure and he must have full details of the imposed loading, *i.e.* its type and duration. The conflict between the design of structures and sub-structures to BS 8110 or similar structural codes, and the design of piled foundations to BS 8004 should be ended if, and when, Eurocode No. 7<sup>(1.2)</sup> is adopted as general practice for foundation design. [Chapter 7](#) of the Eurocode deals with piled foundations from the aspects of actions (forces) on piles from superimposed loading or ground movements, design methods for piles subjected to compression, tension, and lateral loading, pile-loading tests, structural design and supervision of construction. In using [Chapter 7](#) of the Eurocode the designer is required to demonstrate that the sum of the ultimate limit-state components of bearing capacity of the pile or pile group exceeds the ultimate limit-state design loading and that the serviceability limitstate is not reached.

At the time of preparing this edition Eurocode No. 7 was published only in the form of a draft for comment. It is likely that some revisions to the draft will be made before final publication. Brief references are made to the draft code in the chapters of this book dealing with pile design. These references are necessarily brief because the EC7 Code does not make recommendations on methods of pile design. Essentially it prescribes the succession of stages in the design process. If the reader wishes to apply the Eurocode rules it will be essential to study the draft or final publication so that the step-by-step design process can be followed and account taken of the various qualifications to the application of the code rules. Whether or not the Eurocode is used for design in preference to present conventional methods it does provide a very useful design check itemising all the factors which can influence foundation design.

## 1.6 Responsibilities of engineer and contractor

In Britain and in many other countries piling is regarded as a specialist operation and the procedure for calling for tendered prices for this work may result in a division of responsibility which can lead to undesirable practices. When the engineer is wholly responsible for design or supervision of construction he will specify the type, width and overall length of the piles based on the ground information. He will then prepare detailed designs for concrete piles showing the reinforcement, concrete mix proportions, cover, and cube crushing strengths. In the case of steel piles he will specify the standard sections, grade of steel, and welding requirements. The engineer will decide on the depth of penetration of each pile from the results of preliminary calculations checked by field observations during driving. He will accept responsibility for paying the contractor for any costs involved in shortening or lengthening piles, or of providing additional piles should the ground conditions differ from those envisaged or should the piles fail a loading test or fail to achieve the 'set' criterion given by a dynamic formula when at the design length.

Quite a different procedure is adopted when the contractor is responsible for design. The engineer will provide the piling contractor with whatever ground information is available, and he will state either the required working load on a single pile, or he may simply provide a building layout plan showing the column loads or the load per metre run from the load-bearing walls. In the latter case the contractor will be responsible for deciding the required piling layout. In all cases the contractor will determine the type and required diameter and length of the piles, but he will be careful to quote his price for lengthening the piles should the actual ground conditions differ from the information supplied at the time of tendering. The contractor's tender is usually accompanied by financial provisions to guarantee the performance and safety of his design.

The engineer may not always specify allowable working stresses on the pile shaft, minimum cube crushing strengths, or minimum cement contents in concrete mixes. He may consider it the proper duty of the piling contractor to decide on these values since they may be governed by the particular piling process employed.\* In all cases the engineer *must* specify the maximum permissible settlement at the working load and at some simple multiple, say 1.5 times or twice the working load, either on test piles or on working piles or both. This is essential as it is the only means that the engineer possesses of checking that the contractor's design assumptions and the piles as installed will fulfil their function in supporting the structure. Only the engineer can state the requirement for settlement at the working

\* The need to specify allowable working stresses and the crushing strength and minimum cement content of concrete piles is dealt with in [Chapters 2](#) and [10](#).

load since only he knows what the structure can tolerate in the way of total and differential settlement. It frequently happens that the maximum settlements specified are so unrealistically small that they will be exceeded by the inevitable elastic compression of the pile shaft, irrespective of any elastic compression or yielding of the soil or rock supporting the pile. However, the specified permissible settlement should not be so large that the safety factor is compromised (see 4.1.4) and it should be remembered that the settlement of a pile group is related to the settlement of a single pile within the group (Chapter 5). It is unrealistic to specify the maximum movement of a pile under lateral loading, since this can be determined only by field trials.

The above procedure for contractor-designed piling has been advantageous in that it has promoted the development of highly efficient piling systems. However, they have the drawback that they place the engineer in a difficult position when checking the contractor's designs and in deciding whether or not to approve a request for pile lengths that are greater than those on which the tendered price was based. If the engineer declines to authorise extra pile lengths the contractor will withdraw his guarantee of performance. Nevertheless the engineer has a duty to his employer or client to check the specialist contractor's designs as far as he is able (guidance regarding this is given in Chapter 4), to enquire as to whether or not the contractor has made proper provision for difficult ground conditions such as obstructions or groundwater flow, to check on site that the piles are being installed in a sound manner, and that they comply with the requirements for test loading. In the interests of his client he should not allow extra pile lengths if he considers the contractor is being over-cautious in his assessment of the conditions. However, he should not make this decision without test-pile observations or previous knowledge of the performance of piles in similar soil conditions.

The contractor's guarantee is usually limited to that of the load-settlement characteristics of a single pile and for soundness of workmanship, but his responsibilities regarding effects due to installation extend to the complete structure and to any nearby existing buildings or services. For example, if a building were to suffer damage due to the settlement of a group of piles and the settlement were due to the consolidation of a layer of weak compressible soil beneath the zone of disturbance caused by pile driving (Figure 1.3), the contractor could reasonably argue that this was not his responsibility. The engineer should have considered this in his overall design and specified a minimum pile length to take account of this compressible layer. On the other hand, a contractor is regarded as responsible for any damage to surrounding structures caused by vibrations or ground heave when driving a group of piles, or by any loss of ground when drilling for groups of bored and cast-in-place piles.

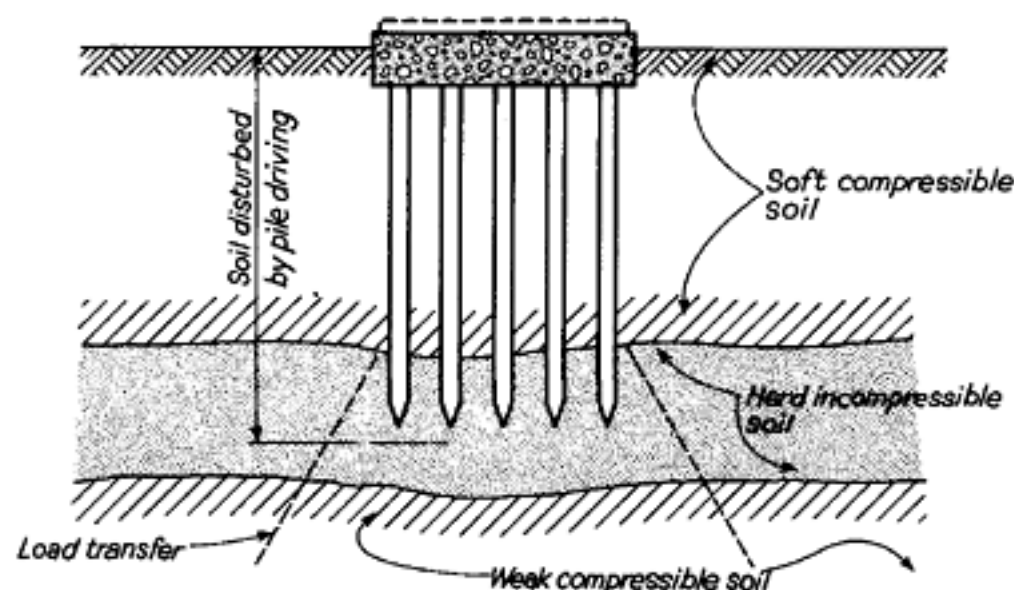


Fig. 1.3 Pile group terminating in hard incompressible soil layer underlain by weak compressible soil

Because of the great importance of installation effects on pile behaviour, the various types of pile available and their methods of installation are first described in Chapters 2 and 3, before going on to discuss the various methods of calculating allowable loads on single piles and groups of piles in Chapters 4 to 6.

## 1.7 References

- 1.1 BJERRUM, L. Problems of soil mechanics and construction on soft clays, *Proceedings 8th International Conference, ISSMFE, Moscow*, Vol. 3, 1973, pp. 111–59.
- 1.2 EUROCODE NO. 7, *Geotechnics*, European Committee for Standardization, Commission of the European Communities, draft code 1991.

## CHAPTER 2

# Types of pile

### 2.1 Classification of piles

The British Standard Code of Practice for Foundations (BS 8004) places piles in three categories. These are as follows.

*Large displacement piles* comprise solid-section piles or hollow-section piles with a closed end, which are driven or jacked into the ground and thus displace the soil. All types of driven and cast-in-place piles come into this category.

*Small-displacement piles* are also driven or jacked into the ground but have a relatively small cross-sectional area. They include rolled steel H- or I-sections, and pipe or box sections driven with an open end such that the soil enters the hollow section. Where these pile types plug with soil during driving they become large displacement types.

*Replacement piles* are formed by first removing the soil by boring using a wide range of drilling techniques. Concrete may be placed into an unlined or lined hole, or the lining may be withdrawn as the concrete is placed. Preformed elements of timber, concrete, or steel may be placed in drilled holes.

Types of piles in each of these categories can be listed as follows.

#### **Large displacement piles (driven types)**

1. Timber (round or square section, jointed or continuous).
2. Precast concrete (solid or tubular section in continuous or jointed units).
3. Prestressed concrete (solid or tubular section).
4. Steel tube (driven with closed end).
5. Steel box (driven with closed end).
6. Fluted and tapered steel tube.
7. Jacked-down steel tube with closed end.
8. Jacked-down solid concrete cylinder.

#### **Large displacement piles (driven and cast-in-place types)**

1. Steel tube driven and withdrawn after placing concrete.
2. Precast concrete shell filled with concrete.
3. Thin-walled steel shell driven by withdrawable mandrel and then filled with concrete.

#### **Small-displacement piles**

1. Precast concrete (tubular section driven with open end).
2. Prestressed concrete (tubular section driven with open end).
3. Steel H-section.
4. Steel tube section (driven with open end and soil removed as required).
5. Steel box section (driven with open end and soil removed as required).

## Replacement piles

1. Concrete placed in hole drilled by rotary auger, baling, grabbing, airlift or reverse-circulation methods (bored and cast-in-place).
2. Tubes placed in hole drilled as above and filled with concrete as necessary.
3. Precast concrete units placed in drilled hole.
4. Cement mortar or concrete injected into drilled hole.
5. Steel sections placed in drilled hole.
6. Steel tube drilled down.

## Composite piles

Numerous types of piles of composite construction may be formed by combining units in each of the above categories, or by adopting combinations of piles in more than one category. Thus composite piles of a displacement type can be formed by jointing a timber section to a precast concrete section, or a precast concrete pile can have an H-section jointed to its lower extremity. Composite piles consisting of more than one type can be formed by driving a steel or precast concrete unit at the base of a drilled hole, or by driving a tube and then drilling out the soil and extending the drill hole to form a bored and cast-in-place pile.

## Selection of pile type

The selection of the appropriate type of pile from any of the above categories depends on the following three principal factors.

The location and type of structure.

The ground conditions.

Durability.

Considering the first factor, some form of displacement pile is the first choice for a *marine structure*. A solid precast or prestressed concrete pile can be used in fairly shallow water, but in deep water a solid pile becomes too heavy to handle and either a steel tubular pile or a tubular precast concrete pile is used. Steel tubular piles are preferred to H-sections for exposed marine conditions because of the smaller drag forces from waves and currents. Large-diameter steel tubes are also an economical solution to the problem of dealing with impact forces from waves and berthing ships. Timber piles are used for temporary works in fairly shallow water. Bored and cast-in-place piles would not be considered for any marine or river structure unless used in a composite form of construction, say as a means of extending the penetration depth of a tubular pile driven through water and soft soil to a firm stratum.

Piling for a structure on *land* is open to a wide choice in any of the three categories. Bored and cast-in-place piles are the cheapest type where unlined or only partly-lined holes can be drilled by rotary auger. These piles can be drilled in very large diameters and provided with enlarged or grout-injected bases, and thus are suitable to withstand high working loads. Augered piles are also suitable where it is desired to avoid ground heave, noise and vibration, *i.e.* for piling in urban areas, particularly where stringent noise regulations are enforced. Driven and cast-in-place piles are economical for land structures where light or moderate loads are to be carried, but the ground heave, noise and vibration associated with these types may make them unsuitable for some environments.

Timber piles are suitable for light to moderate loadings in countries where timber is easily obtainable. Steel or precast concrete driven piles are not as economical as driven or bored and cast-in-place piles for land structures. Jacked-down steel tubes or concrete units are used for underpinning work.

The second factor, *ground conditions*, influences both the material forming the pile and the method of installation. Firm to stiff cohesive soils favour the augered bored pile, but augering without support of the borehole by a bentonite slurry, cannot be performed in very soft clays, or in loose or water-bearing granular soils, for which driven or driven-and-cast-in-place piles would be suitable. Piles with enlarged bases formed by auger drilling can be installed only in firm to stiff or hard cohesive soils or in weak rocks. Driven and driven-and-cast-in-place piles cannot be used in ground containing boulders or other massive obstructions, nor can they be used in soils subject to ground heave, in situations where this phenomenon must be prevented.

Driven-and-cast-in-place piles which employ a withdrawable tube cannot be used for very deep penetrations because of the limitations of jointing and pulling out the driving tube. For such conditions either a driven pile or a mandrel-driven thin-walled shell pile would be suitable. For hard driving conditions, *e.g.*, boulder clays or gravelly soils, a thick-walled steel tubular pile or a steel H-section can withstand

heavier driving than a precast concrete pile of solid or tubular section. Thin steel shell piles are liable to tearing when being driven through soils containing boulders or similar obstructions.

Some form of drilled pile, such as a drilled-in steel tube, would be used for piles taken down into a rock for the purpose of mobilizing resistance to uplift or lateral loads.

The factor of *durability* affects the choice of material for a pile. Although timber piles are cheap in some countries they are liable to decay above ground-water level, and in marine structures they suffer damage by destructive mollusc-type organisms. Precast concrete piles do not suffer corrosion in saline water below the 'splash zone', and rich well-compacted concrete can withstand attack from quite high concentrations of sulphates in soils and ground waters. Cast-in-place concrete piles are not so resistant to aggressive substances because of difficulties in ensuring complete compaction of the concrete, but protection can be provided against attack by placing the concrete in permanent linings of coated light-gauge metal or plastics. Steel piles can have a long life in ordinary soil conditions if they are completely embedded in undisturbed soil but the portions of a pile exposed to sea water or to disturbed soil must be protected against corrosion by cathodic means if a long life is required.

Other factors influence the choice of one or another type of pile in each main classification, and these are discussed in the following pages, in which the various types of pile are described in detail. In UK practice specifications for pile materials, manufacturing requirements (including dimensional tolerances) and workmanship are given in a publication of the Institution of Civil Engineers(2.1).

Having selected a certain type or types of pile as being suitable for the location and type of structure, for the ground conditions at the site, and for the requirements of durability, the final choice is then made on the basis of cost. However, the total cost of a piled foundation is not simply the quoted price per metre run of piling or even the more accurate comparison of cost per pile per kN of working load carried. The most important consideration is the overall cost of the foundation work including the main contractor's costs and overheads.

It has been noted in [Chapter 1](#) that a piling contractor is unlikely to quote a fixed price based on a predetermined length of pile. Extra payment will be sought if the piles are required to depths greater than those predicted at the tendering stage. Thus a contractor's previous experience of the ground conditions in a particular locality is important in assessing the likely pile length on which to base his tender. Experience is also an important factor in determining the extent and cost of a preliminary test piling programme. This preliminary work can be omitted if a piling contractor can give an assurance from his knowledge of the site conditions that he can comply with the engineer's requirements for load-settlement criteria. The cost of test piling can then be limited to that of proof-loading selected working piles.

If this experience is not available, preliminary test piling may be necessary to prove the feasibility of the contractor's installation method and to determine the load-settlement relationship for a given pile diameter and penetration depth. If a particular piling system is shown to be impracticable, or if the settlements are shown by the test loading to be excessive, then considerable time and money can be expended in changing to another piling system or adopting larger-diameter or longer piles. During the period of this preliminary work the main contractor continues to incur the overhead costs of his site organization and he may well claim reimbursement of these costs if the test-piling work extends beyond the time allowed in his constructional programme. To avoid such claims it is often advantageous to conduct the preliminary test piling before the main contractor commences work on the site.

Finally, a piling contractor's resources for supplying additional rigs and skilled operatives to make up time lost due to unforeseen difficulties, and his technical ability in overcoming these difficulties, are factors which may influence the choice of a particular piling system.

## 2.2 Driven displacement piles

### 2.2.1 Timber piles

In many ways, timber is an ideal material for piling. It has a high strength to weight ratio, it is easy to handle, it is readily cut to length and trimmed after driving, and in favourable conditions of exposure durable species have an almost indefinite life. Timber piles used in their most economical form consist of round untrimmed logs which are driven butt uppermost. The traditional British practice of using squared timber may have become established because of the purchase for piling work of imported timber which had been squared for general structural purposes in the sawmills of the country of origin. The practice of squaring the timber can be detrimental to its durability since it removes the outer sapwood which is absorptive to creosote or some other liquid preservative. The less absorptive heartwood is thus exposed and instead of a pile being encased by a thick layer of well-impregnated sapwood,

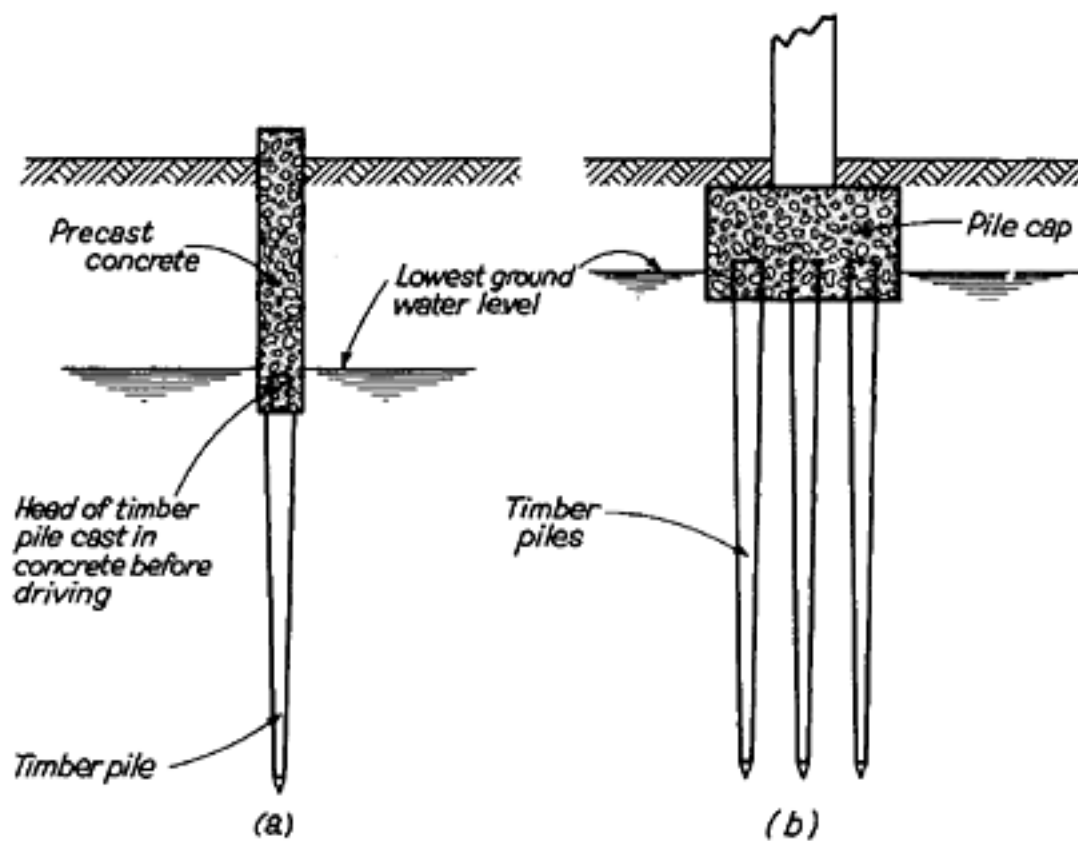


Fig. 2.1 Protecting timber piles from decay

(a) by precast concrete upper section above water level; (b) by extending pile cap below water level

there is only a thin layer of treated timber which can be penetrated by the hooks or slings used in handling the piles, or stripped off by obstructions in the ground.

Timber piles, when situated wholly below ground-water level, are resistant to fungal decay and have an almost indefinite life. However, the portion above ground-water level in a structure on land is liable to decay. Although creosote or other preservatives extend the life of timber in damp or dry conditions they will not prolong its useful life indefinitely. Therefore it is the usual practice to cut off timber piles just below the lowest predicted ground-water level and to extend them above this level in concrete ([Figure 2.1a](#)). If the ground-water level is shallow the pile cap can be taken down below the water level ([Figure 2.1b](#)).

Timber piles in marine structures are liable to be severely damaged by the mollusc-type borers which infest the sea-water in many parts of the world, particularly in tropical seas. The severity of this form of attack can be reduced to some extent by using softwood impregnated with creosote, or greatly minimized by the use of a hardwood of a species known to be resistant to borer attack. The various forms of these organisms, the form of their attack, and the means of overcoming it are discussed in greater detail in [Chapter 10](#).

Bark should be removed from round timbers where these are to be treated with preservative. If this is not done the bark reduces the depth of impregnation. Also the bark should be removed from piles carrying uplift loads by skin friction in case it should become detached from the trunk, thus causing the latter to slip. Bark need not be removed from piles carrying compression loads or from fender piles of uncreosoted timber (hardwoods are not treated because they will not absorb creosote or other liquid preservatives).

Commercially-available timbers which are suitable for piling include Douglas fir, pitch pine, larch, and Western red cedar, in the softwood class, and greenheart, jarrah, opepe, teak and European oak in the hardwood class. The timber should be straight-grained and free from defects which could impair its strength and durability. BS 8004 states that a deviation in straightness from the centre-line of up to 25mm on a 6m chord is permitted for round logs but the centre-line of a sawn timber pile must not deviate by more than 25mm from a straight line throughout its length. The Swedish Code SBS-S23:6 (1968) permits a maximum deviation of 1% of length between two arbitrarily selected measuring points which must be at least 3m apart.

The requirements of BS 8004 of the working stresses in timber piles merely state that these should not exceed the green permissible stresses given in CP 112 for compression parallel to the grain for the species and grade of timber being used. The Code suggests that suitable material will be obtained from stress grades ss and better. Grade stresses in accordance with BS 5268 (which replaced CP 112) are shown in [Table 2.1](#), for various classes of softwood and hardwood suitable for piling work. The working stresses shown in [Table 2.1](#) for the hardwoods are considerably higher than those of the comparable grades of softwood. It should be noted that the stresses in [Table 2.1](#) are for dry timber. Timber piles are usually in a wet environment when the multiplying factors shown in [Table 2.2](#) should be used to convert

Table 2.1 Grade stresses and moduli of elasticity of some softwoods and tropical hardwoods suitable for bearing piles

Standard name	Grade	BS 5268: Part 2:1984 (values in N/mm <sup>2</sup> )						
		Bending parallel to grain*	Tension parallel to grain	Compression parallel to grain	Compression perpendicular to grain	Shear parallel to grain*	Modulus of elasticity	
							Mean	Minimum
Douglas fir	SS/MSS	6.2	3.7	6.6	2.4	0.88	11000	7000
Pitch pine	SS	10.5	6.3	11.0	3.2	1.16	13500	9000
Larch	SS	7.5	4.5	7.9	2.1	0.82	10500	7000
Douglas fir-larch	SS	7.5	4.5	7.9	2.4	0.85	11000	7500
Western red cedar (imported)	GS	5.3	3.2	6.8	2.2	0.85	10000	6500
	SS	5.7	3.4	6.1	1.7	0.63	8500	5500
	GS	4.1	2.5	5.2	1.6	0.63	7000	4500
Greenheart	HS	26.1	15.6	23.7	5.9	2.6	21600	18000
Jarrah	HS	13.8	8.2	14.2	3.1	2.0	12400	8700
Opepe	HS	17.0	10.2	17.6	3.8	2.1	14500	11300
Teak	HS	13.7	8.2	13.4	3.1	1.7	10700	7400

Notes: \* Stresses applicable to timber 300mm deep (or wide).

† When the specifications specifically prohibit wane at bearing areas, the SS and HS grade perpendicular to the grain, stress may be multiplied by 1.33 and used for all grades.

SS denotes special structural grade (visually stressed graded).

HS denotes special structural grade (machine stress graded).

All stresses apply to long-term loading.

Table 2.2 Modification factor  $K_2$  by which dry stresses and moduli should be multiplied to obtain wet stresses and moduli applicable to wet exposure conditions

Property	Value of $K_2$
Bending parallel to grain	0.8
Tension parallel to grain	0.8
Compression parallel to grain	0.6
Compression perpendicular to grain	0.6
Shear parallel to grain	0.9
Mean and minimum modulus of elasticity	0.8

the dry stress properties to the wet conditions. When calculating the working stress on a pile, allowance must be made for bending stresses due to eccentric and lateral loading and to eccentricity caused by deviations in the straightness and inclination of a pile. Allowance must also be made for reductions in the cross-sectional area due to drilling or notching and to the taper on a round log.

The requirements of codes of practice in various countries are shown in [Table 2.3](#). It may be seen from this table that, in addition to specifying a maximum working stress, some codes limit the maximum load which can be carried by a pile of any diameter. This limitation is applied in order to avoid the risk of damage to a pile by driving it to some arbitrary 'set' as required by a dynamic pile-driving formula and also to avoid a high concentration of stress at the toe of a pile end bearing on a hard stratum. Damage to a pile during driving is most likely to occur at its head and toe.

The problems of splitting of the heads and unseen 'brooming' and splitting of the toes of timber piles occur when it is necessary to penetrate layers of compact or cemented soils to reach the desired founding level. This damage can also occur when attempts are made to drive deeply into dense sands and gravels or into soils containing boulders, in order to mobilize the required skin-frictional resistance for a given uplift or compressive load. Judgement is required to assess the soil conditions at a site so as to decide whether or not it is feasible to drive a timber pile to the depth required for a given load without damage, or whether it is preferable to reduce the working load to a value which permits a shorter pile to be used. As

an alternative, jetting or pre-boring may be adopted to reduce the amount of driving required. The temptation to continue hard driving in an attempt to achieve an arbitrary set for compliance with some dynamic formula must be resisted. Cases have occurred where the measured

Table 2.3 Code of practice requirements for working stresses in timber piles

Country	Code	Working stress	Other requirements
United Kingdom	BS 8004	Not to exceed permissible green stress in CP 112 for compression parallel to grain (See Tables <a href="#">2.1</a> and <a href="#">2.2</a> )	Allowance to be made for drilling or notching. Higher stresses permitted during driving
USA	New York City Building Code (1985)	8.3N/mm <sup>2</sup> for southern pine, Douglas fir, oak or other wood of comparable strength. 5.9N/mm <sup>2</sup> for cedar, Norway pine, spruce or other wood of comparable strength	Piles 12m or more in length and of 300kN capacity or less shall be deemed to be adequate if they conform as follows. Piles of 250 to 300kN capacity shall be in Class A timber or minimum 200mm tip with uniform taper. Piles of less than 250kN capacity, shall be in Class A or B timber or minimum 150mm tip with uniform taper. All piles driven to end bearing on to rock or hardpan shall be in Class A timber with minimum 200mm tip and with uniform taper. (Classes of timber defined in Reference Standard RS 11–7)
Germany	DIN 4026	Class II DIN 4074 Sheets 1 and 2	If type of timber is not specified, contractor must use only coniferous wood. Taper not to be more than 10mm in 1m. Mean diameter can be up to $\pm 30$ mm on specified diameter. Sawn timber not to be less than 160mm wide. For pile length less than 6m, mean diameter to be 250mm ( $\pm 20$ mm). For pile length equal to or greater than 6m, mean diameter to be 200mm ( $\pm 20$ mm) plus 10Lmm, where $L$ is embedded length in metres
Sweden	SBS—S23:6	4.4N/mm <sup>2</sup>	Working stress should be 20% lower for a jointed pile

set achieved per blow has been due to the crushing and brooming of the pile toe and not to the deeper penetration required to reach the bearing stratum.

Damage to a pile can be minimized by reducing as far as possible the *number* of hammer blows necessary to achieve the desired penetration, and also by limiting the height of drop of the hammer. This necessitates the use of a heavy hammer which should at least be equal in weight to the weight of the pile for hard driving conditions, and to one-half of the pile weight for easy driving. The German Code (DIN 18304) limits the hammer drop to 2.0m normally and to 2.5m exceptionally. The lightness of a timber pile can be an embarrassment when driving groups of piles through soft clays or silts to a point bearing on rock. Frictional resistance in the soft materials can be very low for a few days after driving, and the effect of pore pressures caused by driving adjacent piles in the group may cause the piles already driven to rise out of the ground due to their own buoyancy relative to that of the soil. The only remedy is to apply loads to the pile heads until all the piles in the area have been driven.

Heads of timber piles should be protected against splitting during driving by means of a mild steel hoop slipped over the pile head or screwed to it ([Figures 2.2a](#) and [2.2b](#)). A squared pile toe can be provided where piles are terminated in soft to moderately stiff clays ([Figure 2.2a](#)). Where it is necessary to drive them into dense or hard materials a cast steel point should be provided ([Figure 2.2b](#)). As an alternative to a hoop, a cast steel helmet can be fitted to the pile head during driving. The helmet must be deeply recessed and tapered to permit it to fit well down over the pile head, allowing space for the insertion of hardwood packing.

Commercially available timbers are imported in lengths of up to 18m. If longer piles are required they may be spliced as shown in [Figure 2.3](#). A splice near the centre of the length of a pile should be avoided since this is the point of maximum bending moment when the pile is lifted from a horizontal position by attachments to one end, or at the centre. Timber piles can be driven in very long lengths in soft to firm clays by splicing them in the leaders of the piling frame as shown in [Figure 2.4](#). The abutting surfaces of the timber should be cut truly square at the splice positions in order to distribute the stresses caused by driving and loading evenly over the full cross-section. The Swedish piling code SBS-S23:6 (1968) requires joints between two timber elements or between a timber and a concrete element to be capable of carrying a tensile force of 150kN without exceeding the yield load of the joint structure.

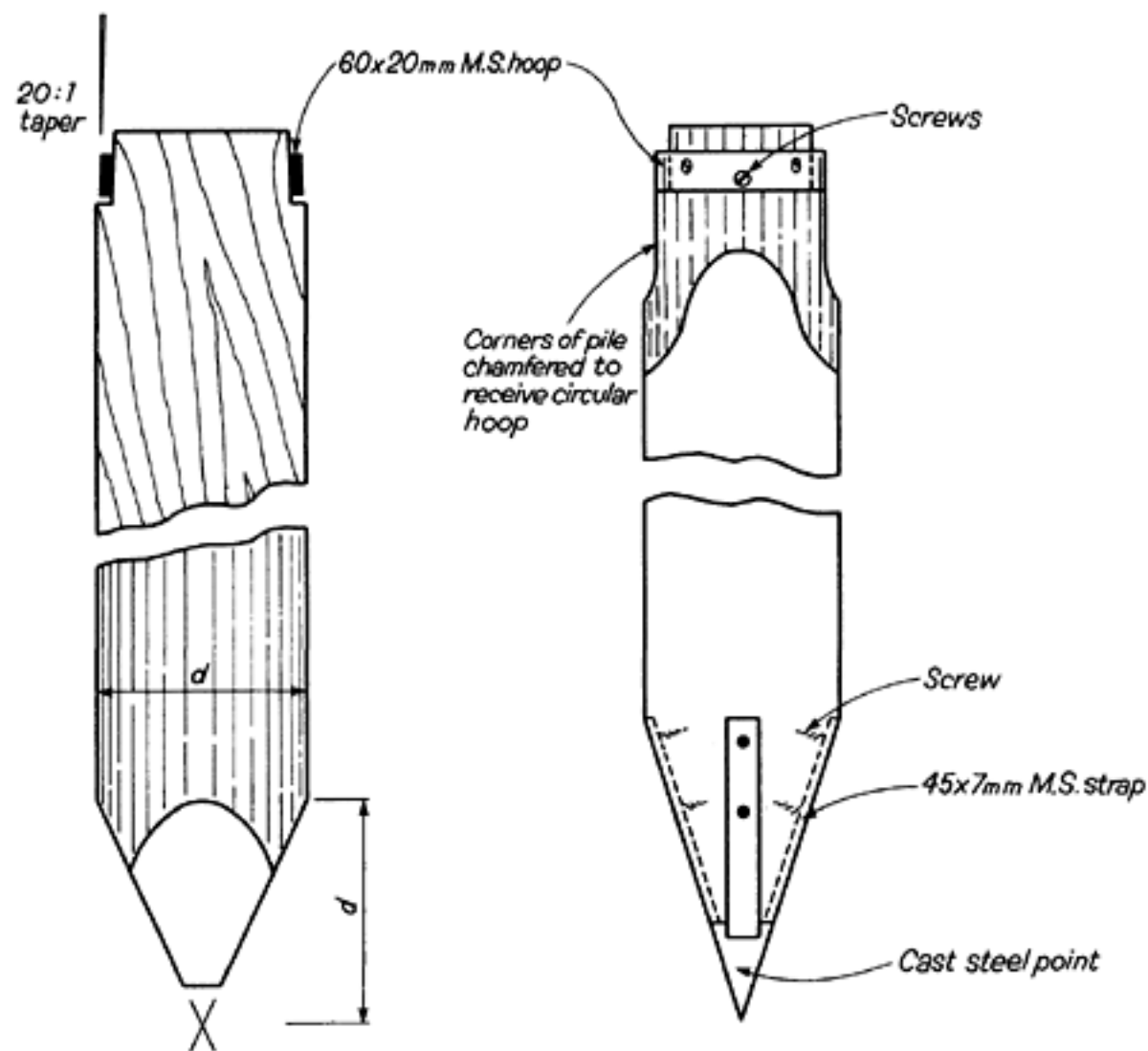


Fig. 2.2 Protecting timber piles from splitting during driving

(a) Protecting head by mild steel hoop (b) Protecting toe by cast steel point

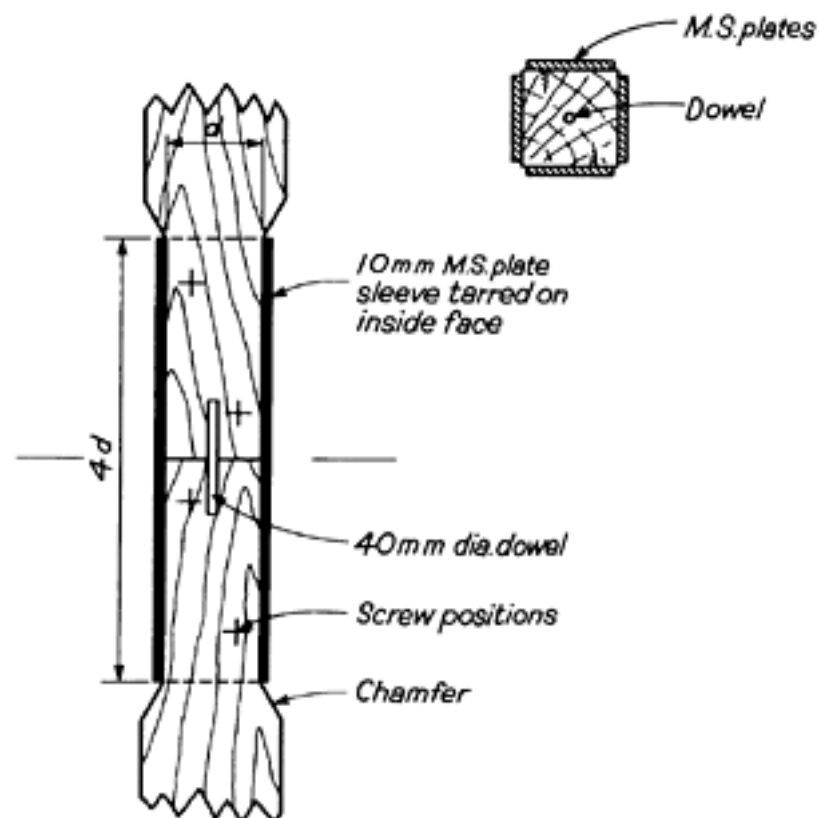
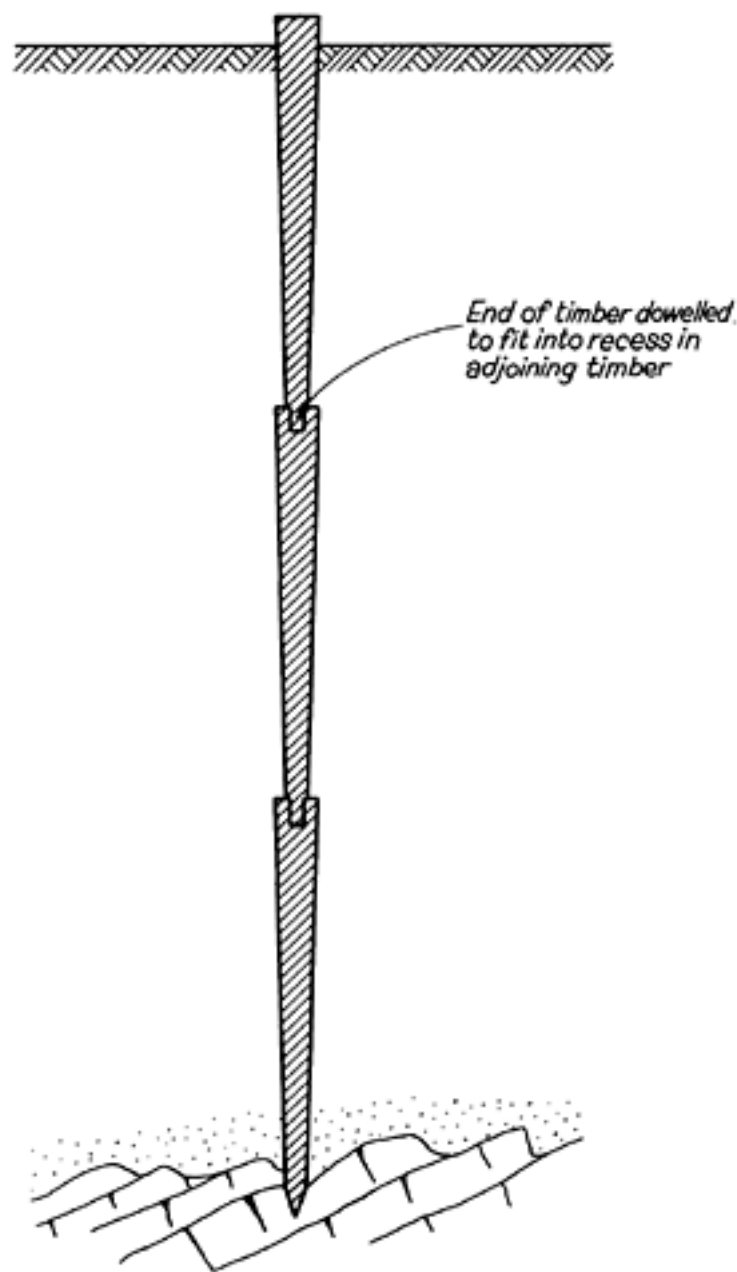


Fig. 2.3 Splice in squared timber pile

### 2.2.2 Precast concrete piles

Precast concrete piles have their principal use in marine and river structures, *i.e.* in situations where the use of driven-and-cast-in-situ piles is impracticable or uneconomical. For land structures unjointed precast concrete piles are frequently more costly than driven-and-cast-in-situ types for two main reasons.

1 Reinforcement must be provided in the precast concrete pile to withstand the bending and tensile stresses which occur during handling and driving. Once the pile is in the ground, and if mainly compressive loads are carried, the majority of this steel is redundant.



*Fig. 2.4 Splicing timber piles in multiple lengths*

2 The precast concrete pile is not readily cut down or extended to suit variations in the level of the bearing stratum to which the piles are driven.

However, there are many situations for land structures where the precast concrete pile can be the more economical. Where large numbers of piles are to be installed in easy driving conditions the savings in cost due to the rapidity of driving achieved may outweigh the cost of the heavier reinforcing steel necessary. Reinforcement may be needed in any case to resist bending stresses due to lateral loads or tensile stresses from uplift loads. Where high-capacity piles are to be driven to a hard stratum savings in the overall quantity of concrete compared with cast-in-situ piles can be achieved since higher working stresses can be used. Where piles are to be driven in sulphate-bearing ground or into aggressive industrial waste materials, the provision of sound high-quality dense concrete is ensured. The problem of varying the length of the pile can be overcome by adopting a jointed type.

From the above remarks it can be seen that there is still quite a wide range of employment for the precast concrete pile, particularly for projects where the costs of establishing a precasting yard can be spread over a large number of piles. The piles can be designed and manufactured in ordinary reinforced concrete, or in the form of pre-tensioned or post-tensioned prestressed concrete members. The ordinary reinforced concrete pile is likely to be preferred for a project requiring a fairly small number of piles, where the cost of establishing a production line for prestressing work on site is not justifiable and where the site is too far from an established factory to allow the economical transportation of prestressed units from the factory to the site. In countries where the precast concrete pile is used widely, *e.g.*, in Holland and Sweden, the ordinary reinforced concrete pile is preferred to the prestressed design in almost all circumstances.

Precast concrete piles in ordinary reinforced concrete are usually square or hexagonal and of solid cross-section for units of short or moderate length, but for saving weight long piles are usually manufactured with a hollow interior in hexagonal, octagonal or circular sections. The interiors of the piles can be filled with concrete after driving. This is necessary to avoid bursting where piles are exposed to severe frost action. Alternatively drainage holes can be provided to prevent water accumulating in the hollow interior. To avoid excessive flexibility while handling and driving the usual maximum lengths of square section piles and the range of working loads applicable to each size are shown in [Table 2.4](#).

Where piles are designed to carry the applied loads mainly in end bearing, *e.g.*, piles driven through soft clays into medium-dense or dense sands, economies in concrete and reductions in weight for handling can be achieved by providing the piles with an enlarged toe. This is practised widely in Holland where the standard enlargements are 1.5 to 2.5 times the shaft width with a length equal to or greater than the width of the enlargement.

Table 2.4 Working loads and maximum lengths for ordinary precast concrete piles of square section

Pile size (mm square)	Range of working loads (kN)	Maximum length (m)
250	200–300	12
300	300–450	15
350	350–600	18
400	450–750	21
450	500–900	25

BS 8004 requires that piles should be designed to withstand the loads or stresses and to meet other serviceability requirements during handling, pitching, driving and in service in accordance with the current standard Code of Practice for the structural use of concrete. If nominal mixes are adopted a 40-grade concrete with a minimum 28-day cube strength of 40N/mm<sup>2</sup> is suitable for hard to very hard driving and for all marine construction. For normal or easy driving, a 25-grade concrete is suitable. This concrete has a minimum 28-day cube strength of 25N/mm<sup>2</sup>. High stresses, which may exceed the handling stresses, can occur during driving and it is necessary to consider the serviceability limit of cracking. BS 8110 states that National Standards and Codes of Practice require cracks to be controlled to maximum widths close to the main reinforcement ranging from 0.3mm down to 0.1mm in an aggressive environment, or they require that crack widths shall at no point on the surface of the structure exceed a specified width, usually 0.3mm. The German Code (DIN 4026) does not regard cracks\* due to driving that are narrower in width than 0.15mm as detrimental. In Germany the concrete quality must be in accordance with DIN 1045 with a crushing strength at the time of lifting of 22.7N/mm<sup>2</sup>. The Swedish Code also permits cracks of up to 0.2mm in width with a length not exceeding one-half of the pile circumference for transverse cracks, or 100mm for longitudinal cracks.

To comply with the requirements of BS 8110 precast piles of either ordinary or prestressed concrete should have nominal cover to the reinforcement as follows.

Exposure conditions	Nominal cover for concrete grade of			
	25	30	40	50 and over
Buried concrete and concrete continuously under water	40mm	30mm	25mm	20mm
Alternate wetting and drying and freezing	50mm	40mm	30mm	25mm
Exposed to sea water and moorland water with abrasion	—	—	60mm	50mm

The requirements of BS 8004 and other foundation codes are shown in [Table 2.5](#).

The proportion of main reinforcing steel in the form of longitudinal bars is determined by the bending moments induced when the pile is lifted from its casting bed to the stacking area. The magnitude of the bending moments depends on the number and positioning of the lifting points. Design data for various lifting conditions are dealt with in 7.2. In some cases the size of the externally applied lateral or uplift loads may necessitate more main steel than is required by lifting considerations. Lateral steel in the form of hoops and links is provided to prevent shattering or splitting of the pile during driving. Code of practice requirements for the proportion of longitudinal steel, hoops and links are shown in [Table 2.5](#). In hard driving conditions it is advantageous to place additional lateral steel in the form of a helix at the head of the pile. The helix should be about two pile widths in length with a pitch equal to the spacing of the link steel at the head. It can have zero cover where the pile head is to be cut down for bonding to the cap.

A design for a precast concrete pile to comply with BS 8004 for easy driving conditions is shown in [Figure 2.5a](#). A design for a longer octagonal pile suitable for driving to end bearing on rock is shown

\* These are permissible provided that the piles are not damaged to any degree judged to be detrimental.

Table 2.5 Code of practice requirements for reinforcement in precast concrete piles

Country	Code	Longitudinal steel	Type and diameter of lateral steel
United Kingdom	BS 8004	To provide for lifting, handling and superstructure loads and for tensile forces caused by ground heave	
Germany	DIN 4026	Ditto. Not less than 0.8% cross-sectional area for piles more than 10m long. Solid rectangular piles: 4 bars not less than 14mm diameter placed in corners. Round piles: 5 bars 14mm diameter equally spaced	Hoops or links not less than 5mm diameter
USA	New York City Building Code (1985)  American Concrete Institute Recommendations 2.8 (1974)	Min 2% of cross section in symmetrical pattern of at least 4 bars  Min 1.5%, max 8% of cross-section. At least 6 bars for round and octagonal piles. At least 4 bars for square piles	Hoops or links not less than 5.7mm diameter  Spiral or not less than 6mm diameter
Sweden	SBS-S23:6 (1968)	Unstressed: Min diameter 12mm, max diameter 25mm. Effective cross-sectional area at least 1.2% of cross-section for Class B piles and 0.6% of cross-section for Class c piles	

in [Figure 2.5b](#). The design of a prestressed concrete pile in accordance with the recommendations of BS 8110 and the Concrete Society's data sheet<sup>(2.2)</sup> is shown in [Figure 2.6](#).

Prestressed concrete piles have certain advantages over those of ordinary reinforced concrete. Their principal advantage is in their higher strength to weight ratio, enabling long slender units to be lifted and driven. However, slenderness is not always advantageous since a large cross-sectional area may be needed to mobilize sufficient resistance in skin friction and end bearing. The second main advantage is the effect of the prestressing in closing up cracks caused during handling and driving. This effect, combined with the high-quality concrete necessary for economic employment of prestressing, gives the prestressed pile increased durability which is advantageous in marine structures and corrosive soils.

The nominal mixes for precast reinforced concrete piles are related to the severity of driving, and the working stresses appropriate to these mixes are shown in [Table 2.6](#).

For economy in materials, prestressed concrete piles should be made with designed concrete mixes with a minimum 28-day works cube strength of 40N/mm<sup>2</sup>. It may be noted from [Table 2.6](#) that some codes specify a maximum load which can be applied to a precast concrete pile of any dimensions. As in the case of timber piles this limitation is to prevent unseen damage to piles which may be over-driven to achieve an arbitrary set given by a dynamic pile-driving formula.

Concrete made with ordinary Portland cement is suitable for all normal exposure conditions but sulphate-resisting cement may be needed for aggressive ground conditions as discussed in [Chapter 10](#).

Metal shoes are not required at the toes of precast concrete piles where they are driven through

Volume of steel at head and toe of pile	Volume of steel in body of pile	Cover	Other requirements
0.6% gross volume over distance of $3 \times$ pile width from each end	0.2% of gross volume spaced at not more than $\frac{1}{2} \times$ pile width	As BS 8110	Lapping of short bars with main reinforcement to be arranged to avoid sudden discontinuity
Spaced at 50mm centres over 1m length at each end	Spaced at 120mm centres	Not less than 30mm for main steel. Increase to 40mm for corrosive conditions	
Spaced at 75mm centres over distance of $3 \times$ pile width	Spaced at 305mm centres	Not less than 40mm	For hollow piles, min thickness of wall not less than 100mm. Hoops or links for hollow piles to extend over distance of 3.66m from each end or $\frac{1}{3} \times$ pile length whichever is smaller
	Spaced at not more than 150mm centres	Normal exposure 50mm. Marine exposure 75mm	
Within distance of 1m from ends of pile or element. Links (calculated on $2 \times$ area of each bar) must be able to carry total force of 98kN when stress in links may be max. of $f_y/1.5$ or 255N/mm <sup>2</sup>	Spaced at not more than 20mm for unstressed piles and not more than 150mm for prestressed piles	Normal exposure: 30mm. Aggressive conditions 45mm	Must be deformed bars for longitudinal steel. Longitudinal reinforcement for unjointed pile longer than 13m and all jointed piles to carry tensile force equal to tensile stress of at least 4.9N/mm <sup>2</sup> on pile cross-sectional area for which tensile stress may be max of $0.8f_y$ for Class B piles and $f_y$ for Class c piles

soft or loose soils into dense sands and gravels or firm to stiff clays. A blunt pointed end ([Figure 2.7a](#)) appears to be just as effective in achieving the desired penetration in these soils as a more sharply pointed end ([Figure 2.7b](#)) and the blunt point is better for maintaining alignment during driving. A cast-iron or cast-steel shoe fitted to a pointed toe may be used for penetrating rocks or for splitting cemented soil layers. The shoe ([Figure 2.7c](#)) serves to protect the pointed end of the pile. Where piles are to be driven to refusal on a sloping hard rock surface, the 'Oslo point' ([Figure 2.7d](#)) is desirable. This is a hollow-ground hardened steel point. When the pile is judged to be nearing the rock surface the hammer drop is reduced and the pile point is seated on to the rock by a number of blows with a small drop. As soon as there is an indication that a seating has been obtained the drop can be increased and the pile driven to refusal or some other predetermined set. The Oslo point was used by George Wimpey and Co. on the piles illustrated in [Figure 2.5b](#), which were driven on to hard rock at the site of the Whitegate Refinery, Cork. A hardened steel to BS 970:EN2 with a Brinell hardness of 400 to 600 was employed. The 89mm point was machined concave to 12.7mm depth and embedded in a chilled cast-iron shoe. Flame treatment of the point was needed after casting into the shoe to restore the hardness lost during this operation.

Piles may be cast on mass concrete beds using removable side forms of timber or steel ([Figure 2.8](#)). The reinforcing cage is suspended from bearers with spacing forks to maintain alignment. Spacer blocks to maintain cover are undesirable. The stop ends must be set truly square with the pile axis to ensure an even distribution of the hammer blow during driving. Vibrators are used to obtain thorough compaction of the concrete and the concrete between the steel and the forms should be worked with a slicing tool

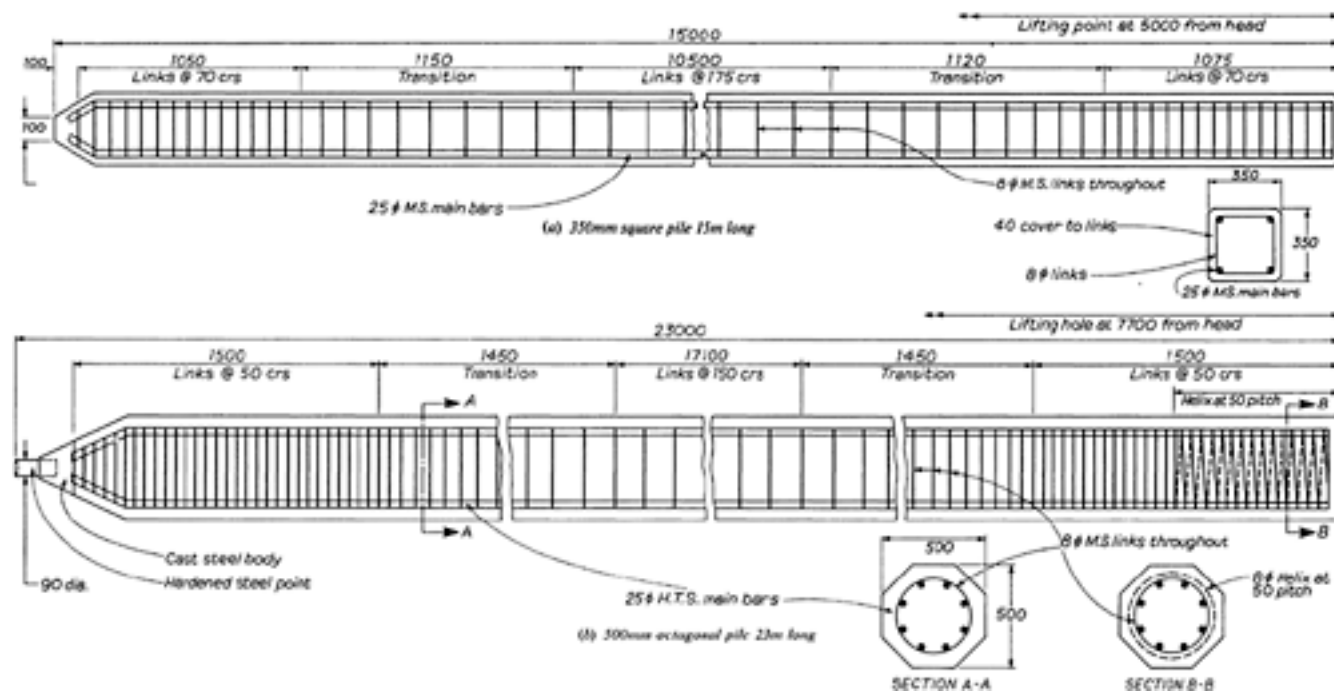


Fig. 2.5 Designs for precast concrete piles

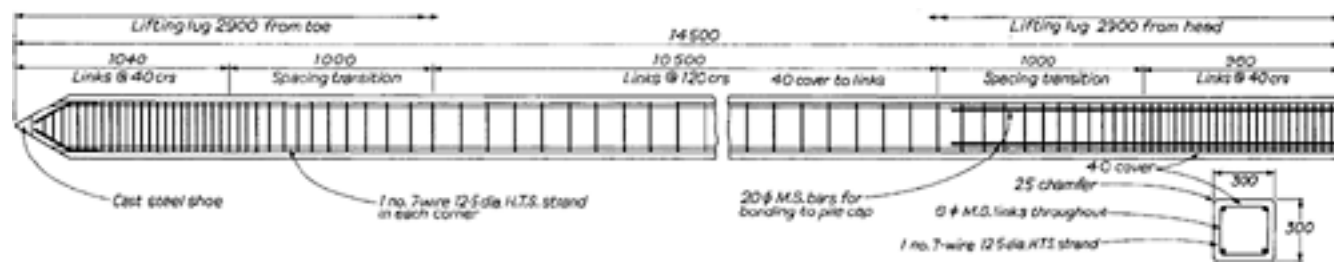


Fig. 2.6 Design for prestressed concrete pile

Table 2.6 Code of practice requirements for working stresses in precast concrete piles

Country	Code	Compression in concrete Related to compressive strength	Tension in reinforcing steel	Remarks
United Kingdom	BS 8004	—		Required to conform to BS 8110
Germany	DIN 4026			Compressive strength to be at least 22N/mm <sup>2</sup> at time of lifting and 34N/mm <sup>2</sup> at time of driving
USA	American Concrete Institute	$0.33u_w$	$0.5f_y$ or 165N/mm <sup>2</sup>	Use 10% less for trestle piles and piles supporting piers, docks and other marine structures
	Recommendations 2.8 (1974)	$0.225u_w$	—	
	National Building Code (1967) New York City Building Code (1985)	$0.25u_w$	$0.40f_y$	Tensile stress in steel $\nless 207\text{N/mm}^2$ Max load on pile driven to hardpan over rock not to exceed 996kN unless higher load substantiated by loading tests
Sweden	SBS-S23:6 (1968)	—	—	Class C: Square piles with concrete strength of 39N/mm <sup>2</sup> and 55 000mm <sup>2</sup> and 75 000mm <sup>2</sup> in area. Working loads not to exceed 330kN and 450kN respectively Class B: Square piles with concrete strength of 49N/mm <sup>2</sup> and 55 000mm <sup>2</sup> and 75 000mm <sup>2</sup> in area. Working loads not to exceed 450kN and 660kN respectively Class A: Quality of concrete not inferior to Class B. Allowable loads in excess of 600kN checked by driving and loading tests
France	DTU 13.2 (1978)	$0.25u_w$		Maximum permissible compression stress on concrete 8N/mm <sup>2</sup>

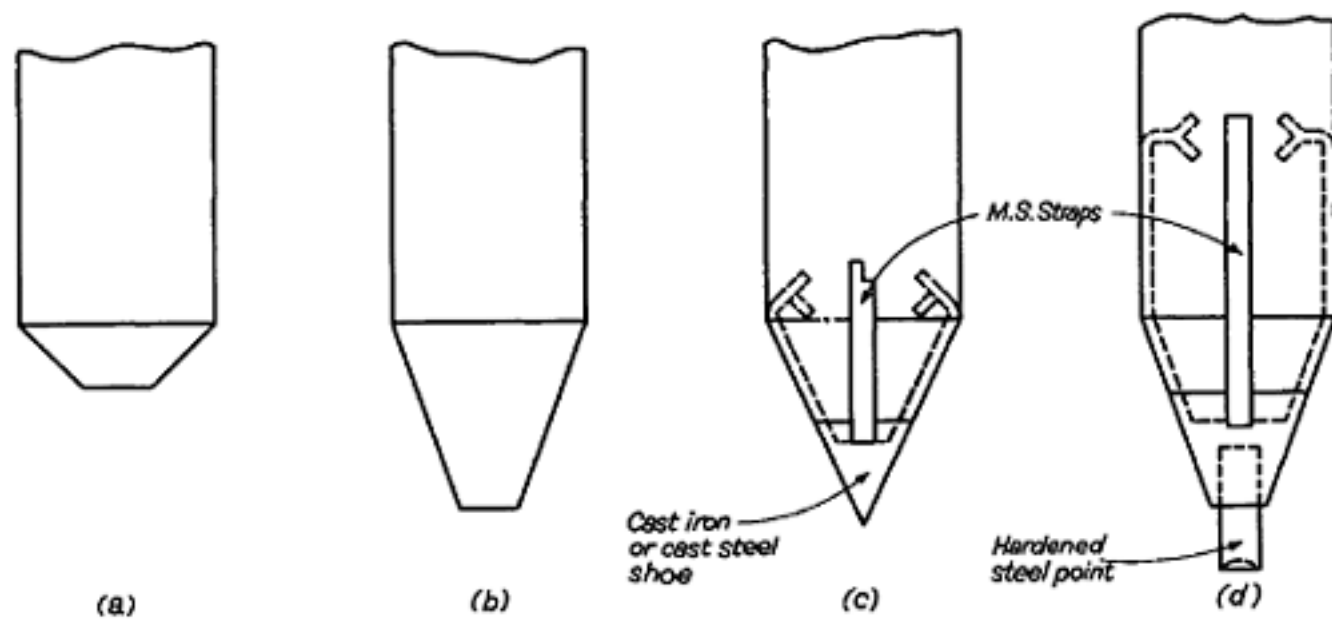


Fig. 2.7 Shoes for precast (including prestressed) concrete piles

- (a) For driving through soft or loose soils to shallow penetration into dense granular soils or firm to stiff clays;  
 (b) Pointed end, suitable for moderately deep penetration into medium-dense to dense sands and firm to stiff clays; (c) Cast-iron or cast-steel shoe for seating pile into weak rocks or breaking through cemented soil layer;  
 (d) 'Oslo' point for seating pile into hard rock

to eliminate honeycombed patches. The casting beds must be sited on firm ground in order to prevent bending of the piles during and soon after casting. After removing the side forms the piles already cast may be used as side forms for casting another set of piles in between them. If this is done the side forms should be set to give a trapezoidal cross-section in order to facilitate release. Piles may also be cast in tiers on top of each other, but a space between them should be maintained to allow air to circulate (Figure 2.9). Casting in tiers involves a risk of distortion of the piles due to settlement of the stacks. In addition, the piles which are first to be cast are the last to be lifted which is in the wrong order, since the most-mature piles should be the first to be lifted and driven.

Where piles are made in a factory, permanent casting beds can be formed in reinforced concrete with heating elements embedded in them to allow a 24-hour cycle of casting and lifting from the moulds. This method of construction was used by Soil Mechanics Ltd. to cast prestressed concrete piles at Drax Power Station in Yorkshire<sup>(2.3)</sup>, where the large number of piles cast (18500) justified the establishment on site of an elaborate casting yard such as would be used in a precast concrete factory. The reinforced concrete formwork is shown in Figure 2.10. This type, which does not have removable side forms, necessitates the embedment of lifting plugs or loops into the tops of the piles.

The layout of the casting yard at Drax is shown in Figure 2.11. The strand reels were set on carriers at one end of the four rows of casting beds, with the winches for tensioning the strand at the opposite end. Each casting bed had five lines of forms. The provision of electric heating elements enabled the concrete to achieve its release strength of  $27.6\text{N/mm}^2$  in 40 to 48 hours. An average of 300 piles per week, with a peak of 400 in a week, were manufactured. Two coats of whitewash were used as a release agent, as it was found that mould oil did not give a sufficiently thick coating to prevent the piles occasionally 'locking-in' to the moulds, in spite of a 1 in 10 taper on the sides. The oil also contaminated the prestressing strand.

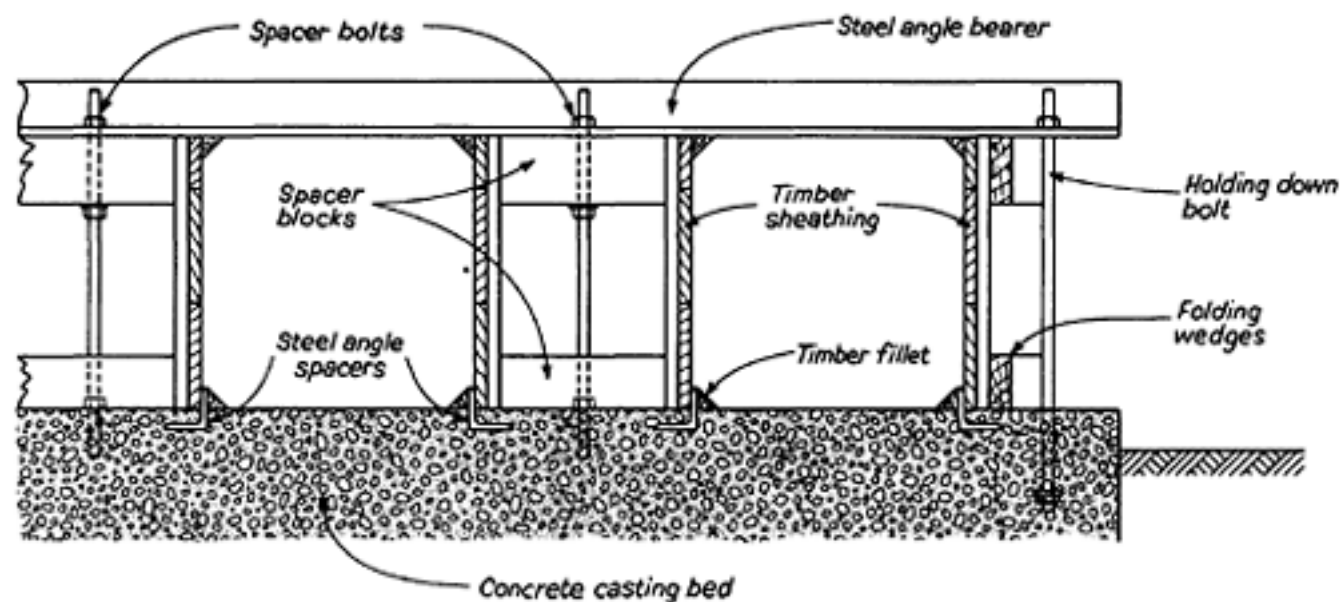


Fig. 2.8 Timber form work for precast concrete piles

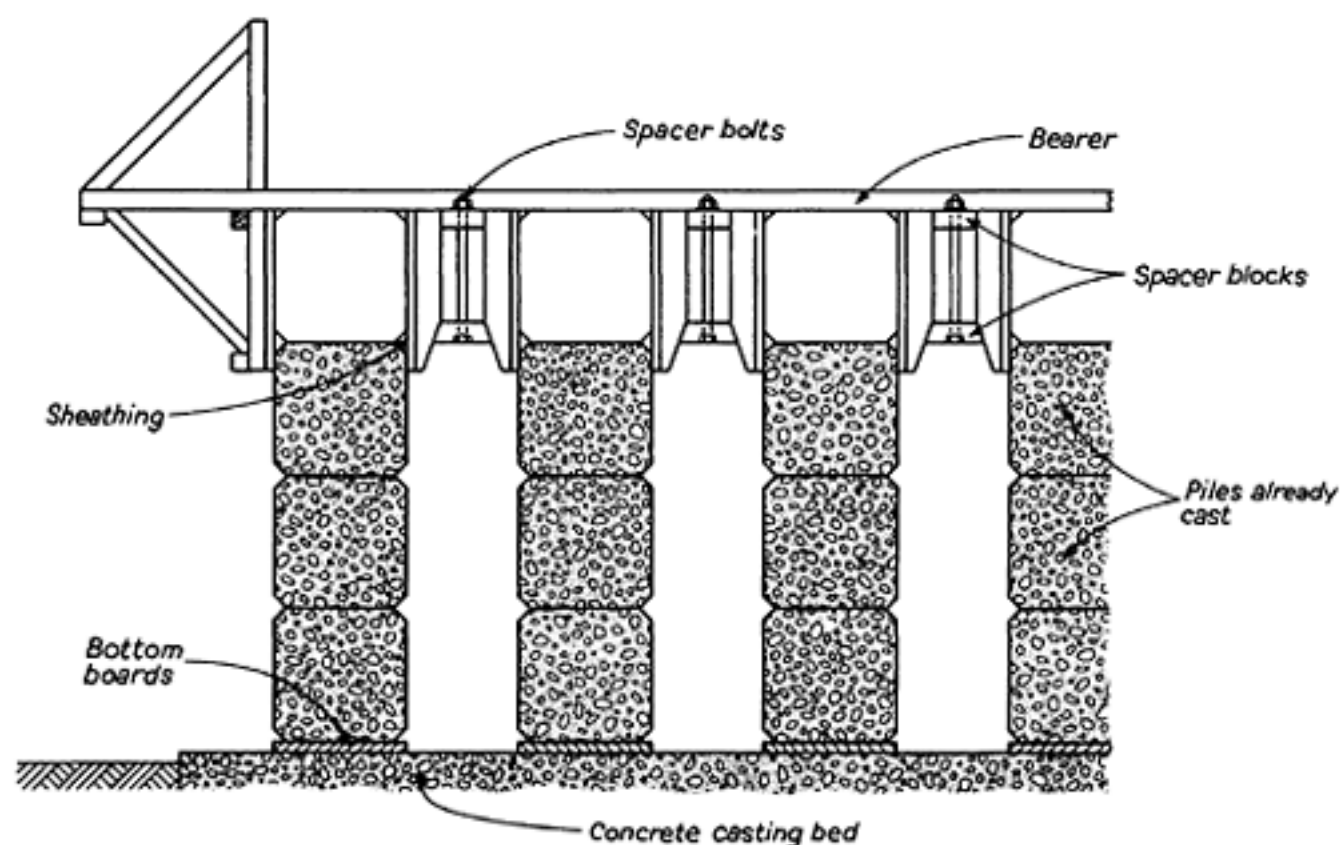


Fig. 2.9 Timber form work for precast concrete piles

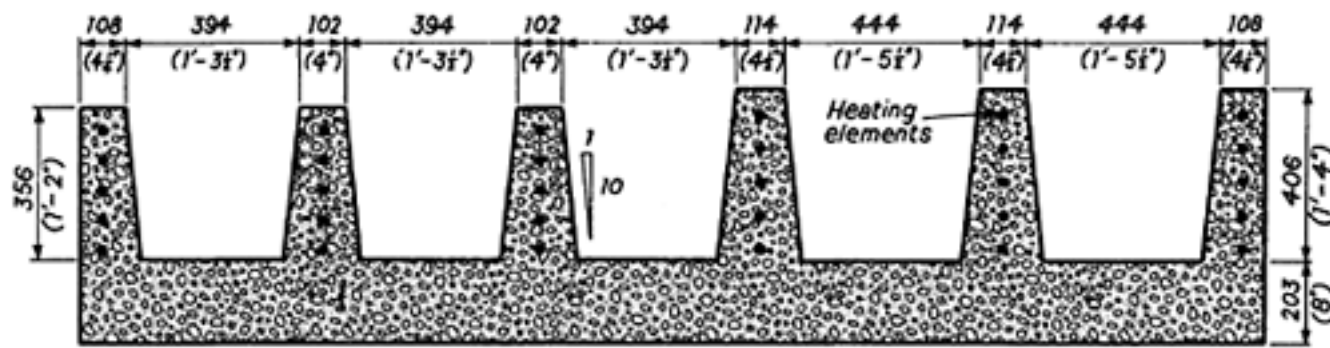


Fig. 2.10 Heated concrete moulds for prestressed concrete piles

When piles are cast within wooden side forms the latter should be removed as soon as possible, and wet curing by water spray and hessian maintained for a seven-day period. As soon as crushing tests on cubes indicate that the piles are strong enough to be lifted they should be slightly canted by careful levering with a bar and packing with wedges to release the suction between the pile and the bed. The lifting slings or bolt inserts may then be fixed and the pile lifted for transporting to the stacking area. This operation of first canting and lifting must be undertaken with great care since the piles have still only a comparatively immature strength and any cracks or incipient cracks formed at this stage will open under driving stresses. The piles should be clearly marked with a reference number, length, and date of casting at or before the time of lifting, to ensure that they are driven in the correct sequence. Timber bearers should be placed between the piles in the stacks to allow air to circulate around them. They should be protected against too-rapid drying in hot weather by covering the stack with a tarpaulin or polyethylene sheeting. Care must be taken to place the bearers only at the lifting positions. If they are misplaced there could be a risk of excessive bending stresses developing and cracking occurring, as shown in [Figure 2.12](#).

Prestressed concrete piles of hollow cylindrical section are manufactured by centrifugal spinning in diameters ranging from 400 to 1626mm.

John Mowlem and Co. cast 915 hollow cylindrical piles near the site of the Esso Oil Jetty at Milford Haven<sup>(2.4)</sup>. Five piles were cast simultaneously on a 150m long casting bed. The 698mm and 559mm outside-diameter sections were formed to the required 76mm wall thickness by using internal moulds consisting of tarred building paper wrapped around lightly tensioned wires. The wires were kept in position by internal spacers of weak concrete. After completion of casting the concrete spacer discs were pulled out by a 'go-devil' after which the wires were withdrawn by pulling them with a tractor.

Bridges totalling 12.5km in length form part of the 25km causeway linking Bahrain Island with the mainland of Saudi Arabia<sup>(2.5)</sup>. Each of the two two-lane box girder spans are supported by single 3.5m OD prestressed concrete piles. The tubular sections have a wall thickness of 350mm and were cast vertically in short sections, then assembled horizontally linked by prestressed wires into units, and lifted by a 1000-tonne crane barge for pitching and installing by a drilling process.

The precautions for driving precast concrete piles are described in [3.4.2](#), and the procedures for bonding piles to caps and ground beams and lengthening piles are described in [7.6](#) and [7.7](#).

One of the principal problems associated with precast concrete piles is unseen breakage due to hard driving conditions. These conditions are experienced in Sweden where the widely used jointed or unjointed precast concrete piles are driven through soft or loose soils onto hard rock. On some sites the rock surface may slope steeply, causing the piles to deviate from a true line and break into short sections near the toe. Accumulations of boulders over bedrock can also cause the piles to be deflected with consequent breakage. Because of these experiences the Swedish piling code recommends quite elaborate precautions in the driving process and of means to detect breakage. Considerable importance is attached to the provision of a central inspection hole in the pile. This is provided in test piles and sometimes in a proportion of the working piles. Also, whenever a pile is known or suspected to be broken the adjacent replacement pile must have such a central hole, but for reasons of economy the hole is not specified for all working piles on a typical site; however, all high-capacity piles (Class A in [Table 2.6](#)) are required to have the hole.

The standard central inspection hole has an internal diameter of 42mm; it is made by embedding a metal tube with a wall thickness of 1.2 to 1.5mm along the axis of the pile. Before driving the pile the axis of the hole should not deviate from the true alignment by more than 5mm in an unjointed pile (or in the body of a 5m long jointed pile) or by more than 1mm through a pile joint. A check for deviation of the pile from line is made by lowering a steel tube of 36mm outside diameter with a wall thickness of 8mm and 1.8m in length down the hole. If such a tube can be lowered to the bottom of the hole under its own weight the pile should not be bent to a radius which would impair its structural integrity. If the rod jams in the hole it is the usual practice to bring an inclinometer to the site to



*Fig. 2.11 Casting yard for prestressed concrete piles at Drax Power Station*

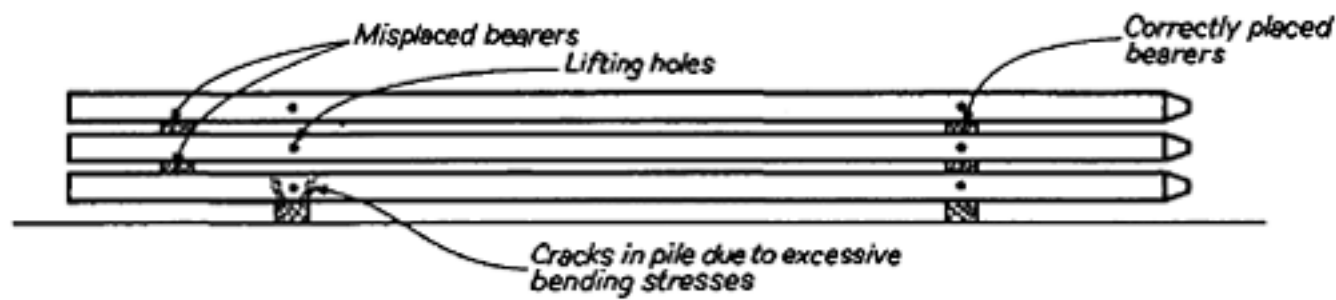


Fig. 2.12 Misplaced packing in stacks of precast concrete piles

record the actual deviation, and hence to decide whether or not the pile should be rejected and replaced. The testing tube also detects deviations in the position or alignment of a jointed pile.

Breakages are due either to tensile forces caused by driving with too light a hammer in soft or loose soils, or to compressive forces caused by driving with too great a hammer drop on to a pile seated on a hard stratum: in both cases the damage occurs in the buried portion of the pile. In the case of compression failure it occurs by crushing or splitting near the pile toe. Such damage is not indicated by any form of cracking in the undriven portion of the pile above ground level. The provision of a central test hole will again enable crushing of the pile due to failure in compression to be detected.

### 2.2.3 Jointed precast concrete piles

The disadvantages of having to adjust the lengths of precast concrete piles either by cutting off the surplus or casting on additional lengths to accommodate variations in the depth to a hard bearing stratum will be evident. These drawbacks can be overcome by employing jointed piles in which the adjustments in length can be made by adding or taking away short lengths of pile which are jointed to each other by devices capable of developing the same bending and tensile resistance as the main body of the pile.

The ‘Hercules’ pile, developed in Sweden by AB Scanpile and driven under licence in the UK by Hercules Piling Ltd. has a hexagonal cross-section, and is shown in [Figure 2.13](#) with a rock shoe incorporating an Oslo point. The precast concrete units are locked together by a steel bayonet-type joint to obtain the required bending and tensile resistance.

The Hercules piles are factory-made and in the UK two square and two hexagonal sizes are manufactured in standard lengths of 6.1m, 9.2m and 12.2m. The properties of the available sizes are shown in [Table 2.7](#). A length is chosen for the initial driving which is judged to be suitable for the shallowest predicted penetration in a given area. Additional lengths are locked on if deeper penetrations are necessary, or if very deep penetrations requiring multiples of the standard lengths are necessary. It is claimed that penetrations of up to 90m are possible. The Swedish code requires all piles less than 12m long to be unjointed.

Other types of jointed precast concrete piles are the West’s ‘Hardrive’ made in lengths of 2.5, 5 and 12m in 285mm square sections suitable for working loads of up to 800kN; the Europile 500 which is a 272×290mm triangular section in unjointed lengths up to 14m for working loads up to 707kN; and the Europile 750 which is a 275mm square section in unjointed lengths up to 15m for working loads up to 1080kN. It has been pointed out that the skin friction mobilized by clay on the shaft of a triangular pile may be less than that of a circular or hexagonal pile of the same volume per unit length([2.6a](#)). The West

Table 2.7 Dimensions and properties of ‘Hercules’ piles as manufactured in the UK

Type of pile	Hexagonal		Square	
	H 800	H 1300	S 550	S 730/750
Maximum safe working load* (kN)	1300	2000	700	1200
Cross-sectional area† (mm <sup>2</sup> )	80000	130000	55225	72900/75625
Dimension <i>b</i> ( <a href="#">Figure 2.13</a> )	505	388	235	270/275
Dimension <i>d</i> ( <a href="#">Figure 2.13</a> )	176	224	235	270/275
Perimeter (mm)	1056	1344	940	1080/1100
Volume (m <sup>3</sup> /m)	0.080	0.130	0.055	0.073/0.076
Mass (kg/m)	200	325	137	182/190
Surface area (m <sup>2</sup> /m)	1.06	1.34	0.94	1.08/1.10

\* Safe working load is dependent on length of pile and soil properties.

† Units of up to 206000mm<sup>2</sup> are manufactured in Sweden by AB Scanpile.

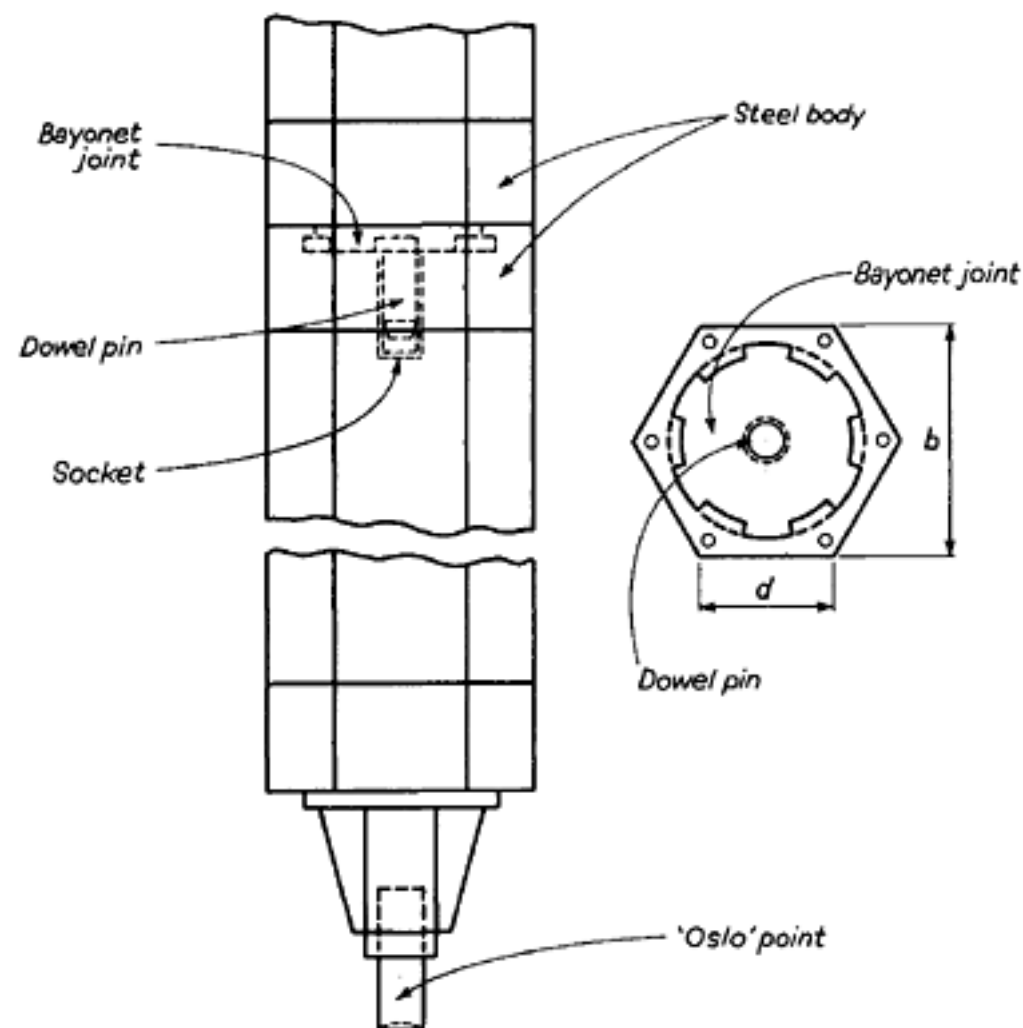


Fig. 2.13 'Hercules' jointed precast concrete pile with 'Oslo' rock point

segmental pile which consists of 280mm cylindrical sections 1m long with a 70mm central hole is suitable for working loads of up to 300kN. The West's Hardrive and the Europile sections can be provided with rock shoes similar to that shown in [Figure 2.13](#) or flat butt ends.

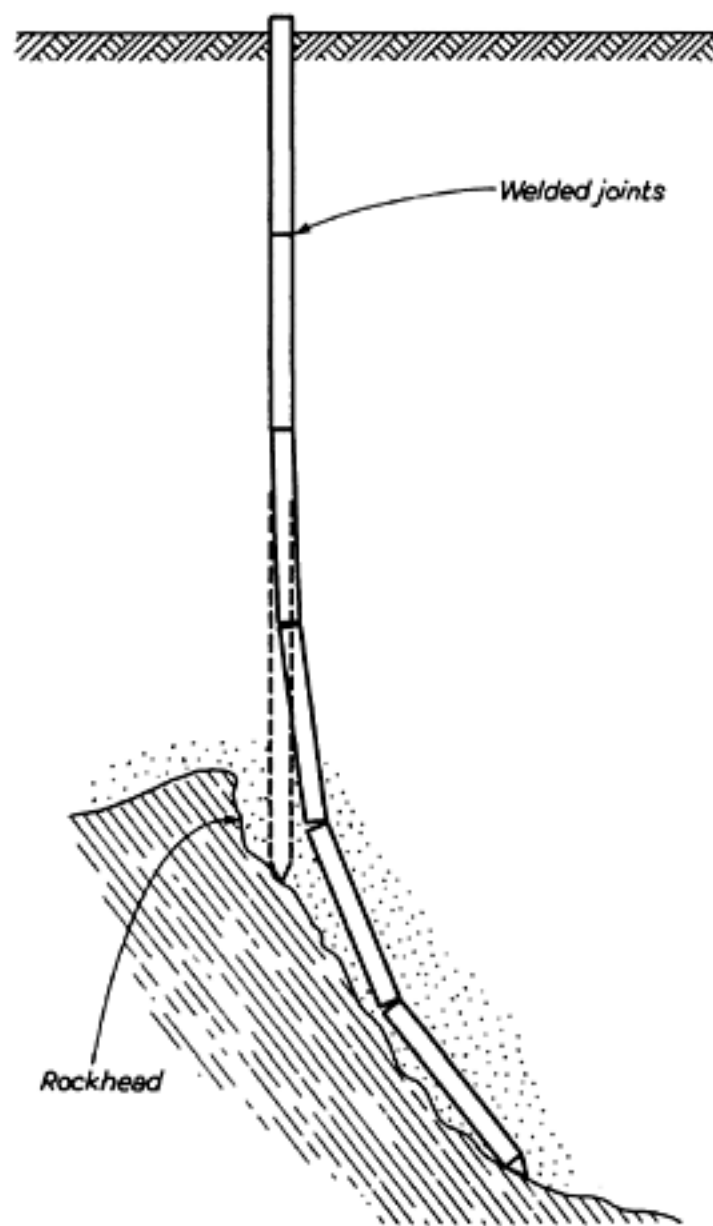
Precast concrete piles which consist of units joined together by simple steel end plates with welded butt joints are not always suitable for hard driving conditions, or for driving on to a sloping hard rock surface. Welds made in exposed site conditions with the units held in the leaders of a piling frame may not always be sound. If the welds break due to tension waves set up during driving or to bending caused by any deviation from alignment the pile may break up into separate units with a complete loss of bearing capacity ([Figure 2.14](#)). This type of damage can occur with keyed or locked joints when the piles are driven heavily, for example to break through thin layers of dense gravel. The design of the joint is, in fact, a critical factor in the successful employment of these piles. The Swedish Code SBS-S 23:6 requires that the splice must be equal in ultimate resistance to the unspliced pile section in bending, tension, and compression. Tests to determine these properties of a jointed section are made after subjecting the section to test driving when jointed to a pile section already seated on rock. Where hard driving conditions are anticipated the jointed pile should be provided with a central hole for all preliminary test piles and for a proportion of the working piles.

The PMI pile was developed in Malaysia mainly for the foundations of light structures. It consists of 82×82mm solid square sections 3m long. The sections are precast and pre-tensioned with a single 4mm wire using 50N/mm<sup>2</sup> concrete. Each section is pushed into the ground by a 20-tonne jack with a stroke of 3.65m and successively jointed by a mild steel box-section sleeve and epoxy resin. A jacking force of 157kN is used for the nominal maximum working load of 78kN.

The Franki 'Miga' pile is a jointed precast concrete pile consisting of short cylindrical units which are jacked into the soil. It is used for underpinning work as described in [Chapter 9](#).

## 2.2.4 Steel piles

Steel piles have the advantages of being robust, light to handle, capable of carrying high compressive loads when driven on to a hard stratum, and capable of being driven hard to a deep penetration to reach a bearing stratum or to develop a high skin-frictional resistance, although their cost per metre run is high compared with precast concrete piles. They can be designed as small displacement piles, which is advantageous in situations where ground heave and lateral displacement must be avoided. They can be readily cut down and extended where the level of the bearing stratum varies; also the head of a pile which buckles during driving can be cut down and re-trimmed for further driving. They have a good resilience and high resistance to buckling and bending forces.



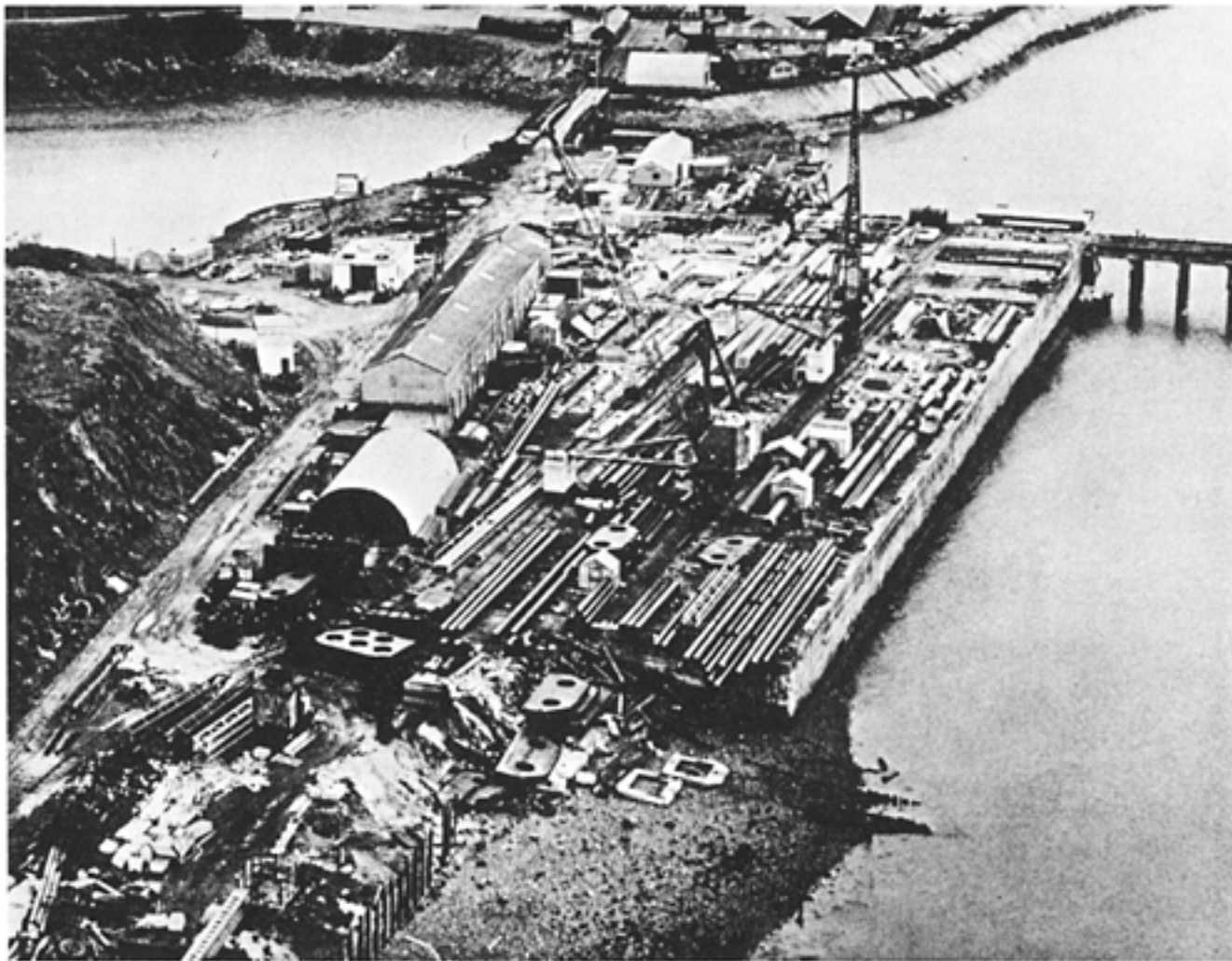
*Fig. 2.14 Unseen breakage of precast concrete piles with welded butt joints*

Types of steel piles include plain tubes, box-sections, H-sections, and tapered and fluted tubes (Monotubes). Hollow-section piles can be driven with open ends. If the base resistance must be eliminated when driving hollow-section piles to a deep penetration, the soil within the pile can be cleaned out by grabbing, by augers, by reverse water-circulation drilling, or by airlift (see [3.4.3](#)). It is not always necessary to fill hollow-section piles with concrete. In normal undisturbed soil conditions they should have an adequate resistance to corrosion during the working life of a structure, and the portion of the pile above the sea bed in marine structures or in disturbed ground can be protected by cathodic means, supplemented by bituminous or resin coatings (see [Section 10.4](#)). Concrete filling may be undesirable in marine structures where resilience, rather than rigidity, is required to deal with bending and impact forces.

Where hollow-section piles are required to carry high compressive loads they may be driven with a closed end to develop the necessary end-bearing resistance over the pile base area. Where deep penetrations are required they may be driven with open ends and with the interior of the pile closed by a stiffened steel plate bulkhead located at a predetermined height above the toe. An aperture should be provided in the bulkhead for the release of water, silt or soft clay trapped in the interior during driving. In some circumstances the soil plug within the pile may itself develop the required base resistance (Figure [4.10](#) and [4.3.2](#)).

Concrete filling of light-gauge steel tubes is required after driving is completed because the steel may be torn or buckled or may suffer corrosion losses. Piles formed from thin steel shells driven by means of an internal mandrel, which is withdrawn before filling the shells with concrete, are described in 2.3.3.

The facility of extending steel piles for driving to depths greater than predicted from soil investigation data has already been mentioned. The practice of welding-on additional lengths of pile in the leaders of the piling frame is satisfactory for land structures where the quality of welding may not be critical. A steel pile supported by the soil can continue to carry high compressive loads even though the weld is partly fractured by driving stresses. However, this practice is not desirable for marine structures where the weld joining the extended pile may be above sea-bed level in a zone subjected to high lateral forces and corrosive influences. Conditions are not conducive to first-class welding when the extension pile is held in leaders or guides on a floating vessel, or on staging supported by piles swaying under the



*Fig. 2.15 Fabrication yard for steel tubular piles at Milford Haven*

influence of waves and currents. It is preferable to do all welding on a prepared fabrication bed with the pile in a horizontal position where it can be rotated in a covered welding station ([Figure 2.15](#)). The piles should be fabricated to cover the maximum predicted length and any surplus length cut off, rather than be initially of only medium length and then be extended. Cut-off portions of steel piles usually have some value as scrap, or they can be used in other fabrications. However, there are many situations where in-situ welding of extensions cannot be avoided. The use of a stable jack-up platform ([Figure 8.14](#)) from which to install the piles is then advantageous.

Where very long lengths of steel tubular piles are required to be driven, as in the case of off-shore petroleum production platforms ([Section 8.2](#)), they cannot be handled in a single length by cranes. They can be driven by underwater hammers, but for top-driven piles the Rockwell pile connector is a useful device for joining such lengths of pile without the delays which occur when making welded joints. This form of connector ([Figure 2.16](#)) was developed for joining lengths of oil well conductor pipe and has been adapted for making connections in piles. It consists of a collar on which are formed specially-shaped segmental projections that lock with projections on a similar collar welded to the mating pile and are capable of resisting tensile forces. The collar can either be a projecting type giving a flush internal surface or can be made flush with the outer pile surface. Internal flush types are used where insert piles are required to be driven, but external flush types are needed where the clearance with the sleeve in jacket-type structures is small and where skin friction must be mobilized if the connectors are driven below the soil surface. Long steel tubular piles driven within the tubular members of a jacket-type structure are redundant above their point of connection by annular grouting to the lower part of the tubular sleeve. This redundant part of the pile, which acts as a 'dolly' or follower for the final stages of driving, can be withdrawn for use when driving other piles, by means of a joint made with the Rockwell connector. The latter can be recovered by a special latching device as the topmost length of pile is lifted out of the jacket.

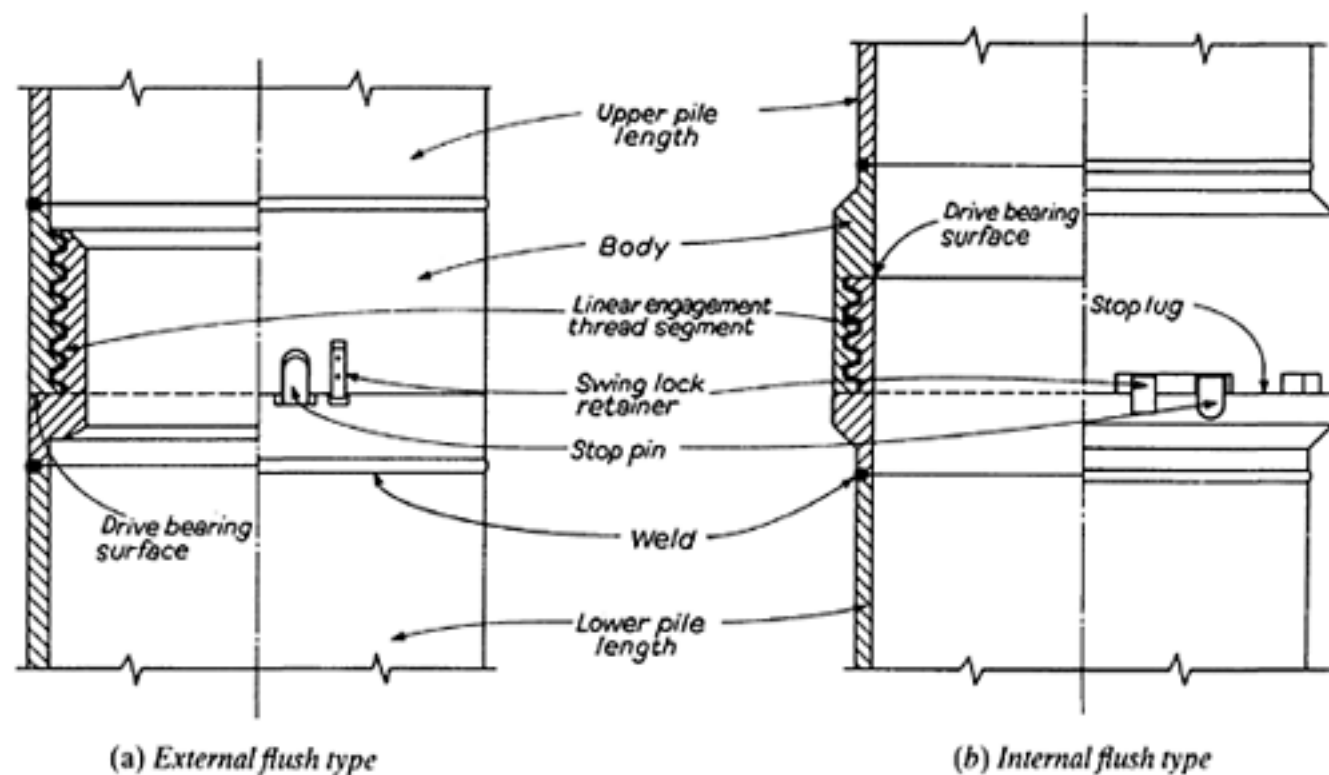


Fig. 2.16 The Rock well connector for joining steel tubular piles

Where steel tubular piles are required to be spliced below the ground surface and compressive loads only are carried, the 'Advance' purpose-made splicing devices manufactured by the Associated Pile and Fitting Corporation of USA can be used. The splicer consists of an external collar which is slipped on to the upper end of the pile section already driven and is held in position by an internal lug. The next length of pile is then entered into the collar and driven down. The APF 'Champion' splicer is used for H-piles and consists of a pair of channel sections set on the head of the pile length already driven to act as a guide for placing and then welding-on the next length.

Steel tubular piles are the preferred shape when soil has to be cleaned out for subsequent placement of concrete, since there are no corners from which the soil may be difficult to dislodge by the cleaning-out tools. They are also preferred for marine structures where they can be fabricated and driven in large diameters to resist the lateral forces in deep-water structures. The circular shape is also advantageous in minimizing drag and oscillation from waves and currents (see [8.1.3](#) and [8.1.4](#)). The hollow section of a tubular pile is also an advantage when inspecting a closed-end pile for buckling. A light can be lowered down the pile and if it remains visible when lowered to the bottom, no deviation has occurred. If a large deviation is shown by complete or partial disappearance of the light, then measures can be taken to strengthen the buckled section by inserting a reinforcing cage and placing concrete.

Steel tubes are manufactured in Britain in standard outside diameters ranging from 273 to 2134mm. The Japanese steel industry produces tubes in the standard range of 318.5 to 3048mm OD. Tubes for piles are manufactured as seamless, spirally-welded and longitudinally-welded units. There is nothing to choose between the latter two types from the aspect of strength to resist driving stresses. In the spiral welding process the coiled steel strip is continuously unwound and spirally bent cold into the tubular form. The joints are then welded from both sides. A 2m diameter spirally-welded pile is shown in [Figure 3.8](#). In the longitudinally-welding process a steel plate is cut and bevelled to the required dimensions then pressed or rolled into tubular form and welded along the linear joints. The spiral method has the advantage that a number of different sizes can be formed on the same machine, but there is a limitation that plate thicknesses more than about 25mm cannot be handled. There is also some risk of weld 'unzipping' from the pile toe under hard driving conditions. This can be prevented by a circumferential shoe of a type described below.

Piles driven in exposed deep water locations are fabricated from steel plate in thicknesses up to 62mm by the longitudinal welding process. Special large-diameter piles can be manufactured by the process. Piles for marine terminals at Cook Inlet, Alaska, are subjected to lateral forces from floating ice. Gerwick<sup>(2.6)</sup> described the installation of 4m diameter by 50mm wall thickness piles by a combination of driving and peripheral jetting (see [3.1.9](#)).

Economies in steel can be achieved by varying the wall thickness and quality of the steel. Thus in marine structures the upper part of the pile can be in mild steel which is desirable for welding on bracing and other attachments, the middle section can be in high-tensile steel with a thicker wall where bending moments are greatest, and the lower part, below sea bed, can be in a thinner mild steel or high-tensile steel depending on the severity of the driving conditions. The 1.3m OD steel tubular piles used for breasting

dolphins for the Abu Dhabi Marine Areas Ltd. tanker berth at Das Island ([Figure 8.8](#)) were designed by The British Petroleum Company to have an upper section 24mm in thickness, a middle section 30mm in thickness and a lower section of 20mm in thickness. The overall length was 36.6m.

BSP International Foundations Ltd. also fabricate light spirally-welded mild steel tubular piles in the range of sizes and nominal working loads listed in [Table 2.8](#). They are designed to be driven by a hammer working on top of the pile or by a drop hammer acting on a plug of concrete in the bottom of the pile (see [3.2](#)). These piles, known as ‘cased piles’ are essentially of light section and are designed to be filled with concrete after driving. In countries where heavy timbers are scarce they have to some extent replaced timber piling for temporary stagings in marine or river work. The end of each pile is closed by a flat mild steel plate welded circumferentially to the pile wall.

Concrete-filled steel tubular piles need not be reinforced unless required to carry uplift or bending stresses which would overstress a plain concrete section cast in the lighter gauges of steel.

*Steel box piles* are fabricated by welding together trough-section sheet piles (Larssen, Krupp, Hoesch, Unimetal and Esch-Belval types), or specially-rolled trough plating (Frodingham, Arbed, Peine, and Union types). The types fabricated from sheet piles are useful for connection with sheet piling forming retaining walls, for example to form a wharf wall capable of carrying heavy compressive loads in addition to the normal earth pressure. However, if the piles rotate during driving there can be difficulty in making welded connections to the flats. Plain flat steel plates can also be welded together to form box piles of square or rectangular section. Simple piles of this type were designed by George Wimpey and Co. for the approach section of the Regent Oil Refining Company’s jetty at Milford Haven. They were 692×457mm in section, made up from 16mm steel plate.

MV piles are small square-section box piles ranging in size from 70mm square to 100mm square. They are driven with a shoe of larger overall dimensions which forms an enlarged hole. This eliminates skin friction and enables the piles to be driven to the deep penetration required for their principal use as anchors to retaining walls. On reaching the design anchorage depth a cement grout is injected to fill the annular space around the shaft. The grouted zone provides the necessary skin frictional resistance to enable them to perform as anchors. They are fabricated by Hoesch AG Huttenwerke.

*H-section piles* have a small volume displacement and are suitable for driving in groups at close centres in situations where it is desired to avoid substantial ground heave or lateral displacement. They can withstand hard driving and are useful for penetrating soils containing cemented layers and for punching into rock. Their small displacement makes them suitable for driving deeply into loose or medium dense sands without the ‘tightening’ of the ground that occurs with large displacement piles. They were used for this purpose for the Tay Road Bridge pier foundations, where it was desired to take the piles below a zone of deep scour on the bed of the Firth of Tay. Test piles 305×305mm in section were driven to depths of up to 49m entirely in loose becoming medium-dense to dense sands, gravels, cobbles and boulders, which is indicative of the penetrating ability of the H-pile.

The ability of these piles to be driven deeply into stiff to very stiff clays and dense sands and gravels on the site of the Hartlepool Nuclear Power Station is illustrated in [Figure 2.17](#). On this site driving resistances of 355×368mm H-piles were compared with those of precast concrete piles of similar overall dimensions. Both types of pile were driven by a Delmag D-22 diesel hammer (see [3.1.4](#)). Although the driving resistances of both types were roughly the same to a depth of about 14m, (indicating that the ends of the H-piles were plugged solidly with clay) at this level the heads of the concrete piles commenced to spall and they could not be driven below 14.9m, whereas the H-piles were driven on to 29m without serious damage, even though a driving resistance of 0.5mm/blow was encountered.

Table 2.8 Dimensions and nominal working loads for concrete-filled cased piles

Internal diameter mm	Area of concrete mm <sup>2</sup>	Working load (kN) for ordinary soil*	Working load (kN) for rock, etc.†
254	50670	150	200
305	72960	300	350–450
356	99300	400	500–650
406	129700	500	600–850
457	164 100	650	800–1000
508	202700	800	1000–1300
559	245200	1000	1250
610	291 800	1200	1500

\* Ordinary soil—Sand, gravel or very stiff clay.

† Rock, etc.—Rock, very dense sand or gravel, very hard marl or hard shale. (Information from BSP International Foundations Ltd., Note CP43)

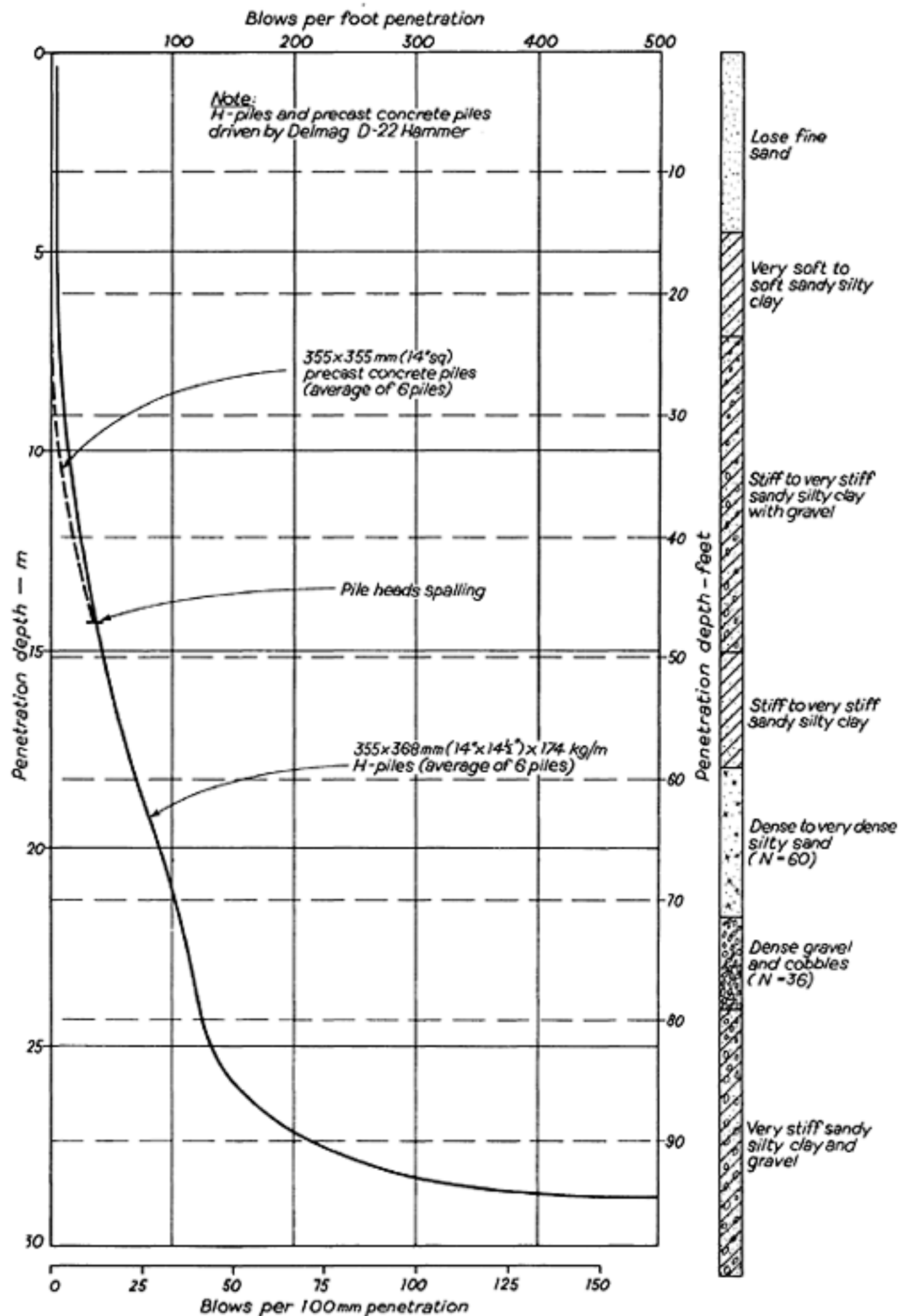


Fig. 2.17 Comparison of driving resistances of 355x355mm precast concrete piles and 355x368mm H-section piles driven into glacial clays, sands and gravels in Hartlepool Power Station

Three of the H-piles were loaded to 3000kN without failure but three of the precast concrete piles failed at test loads of between 1100 and 1500kN.

Because of their relatively small cross-sectional area, H-piles cannot develop a high end-bearing resistance when terminated in soils or in weak or broken rocks. In Germany and Russia it is frequently the practice to weld short H-sections on to the flanges of the piles near their toes to form 'winged piles' (Figure 2.18a). These provide an increased cross-sectional area in end bearing without appreciably reducing their penetrating ability. The bearing capacity of tubular piles can be increased by welding T-sections on to their outer periphery when the increased capacity is provided by a combination of skin friction and end bearing on the T-sections. This method was used to reduce the penetration depth of

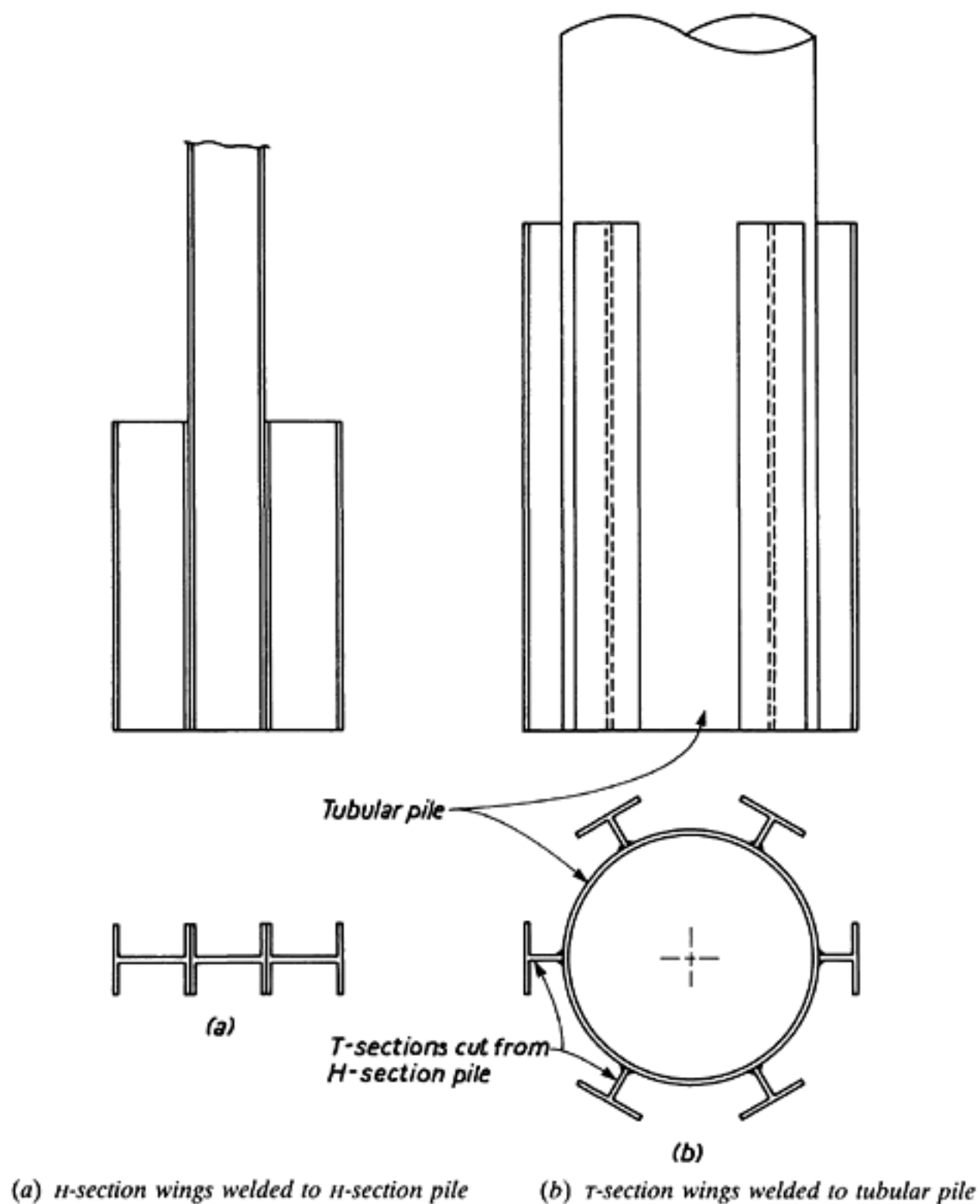


Fig. 2.18 Increasing the bearing capacity of steel piles with welded-on wings

1067mm OD tubular steel piles used in the breasting dolphins of the Britoil Marine Terminal in Cromarty Firth. A trial pile was driven with an open end through 6.5m of loose silty sand for a further 16m into a dense silty sand with gravel and cobbles. The pile was driven by a Menck MRB 1000 single-acting hammer with a 1.25m drop of the 10 tonne ram. It will be seen from [Figure 2.19](#) that there was only a gradual increase in driving resistance finishing with the low value of 39 blows/200mm at 22.6m penetration. The pile was then cleaned out and plugged with concrete but failed under a test load of 6300kN.

It was evident from the driving records that the plain piles showed little evidence of developing base resistance by plugging and would have had to be driven much deeper to obtain the required bearing capacity. In order to save the cost and time of welding-on additional lengths of pile it was decided to provide end enlargements in the form of six  $0.451 \times 0.303 \times 7.0$ m long T-sections welded to the outer periphery in the pattern shown in [Figure 2.18b](#). The marked increase in driving resistance of the trial pile is shown in [Figure 2.19](#). The final resistance was approaching refusal at 194 blows/200mm at 19m below sea bed. The winged pile did not fail under the test load of 6300kN.

A disadvantage of the H-pile is a tendency to bend about its weak axis during driving. The curvature may be sharp enough to cause failure of the pile in bending. Bjerrum<sup>(2.7)</sup> recommends that any H-pile having a radius of curvature of less than 366m after driving should be regarded as incapable of carrying load. A further complication arises when H-piles are driven in groups to an end bearing on a dense cohesionless soil or weak rock. If the piles bend during driving so that they converge there may be an excessive concentration of load at the toe and a failure in end bearing when the group is loaded ([Figure 5.7](#)). The author observed a deviation of the toes of H-piles of about 500mm after they had been driven only 13m through sands and gravels to an end bearing on sandstone at Nigg Bay in Scotland.

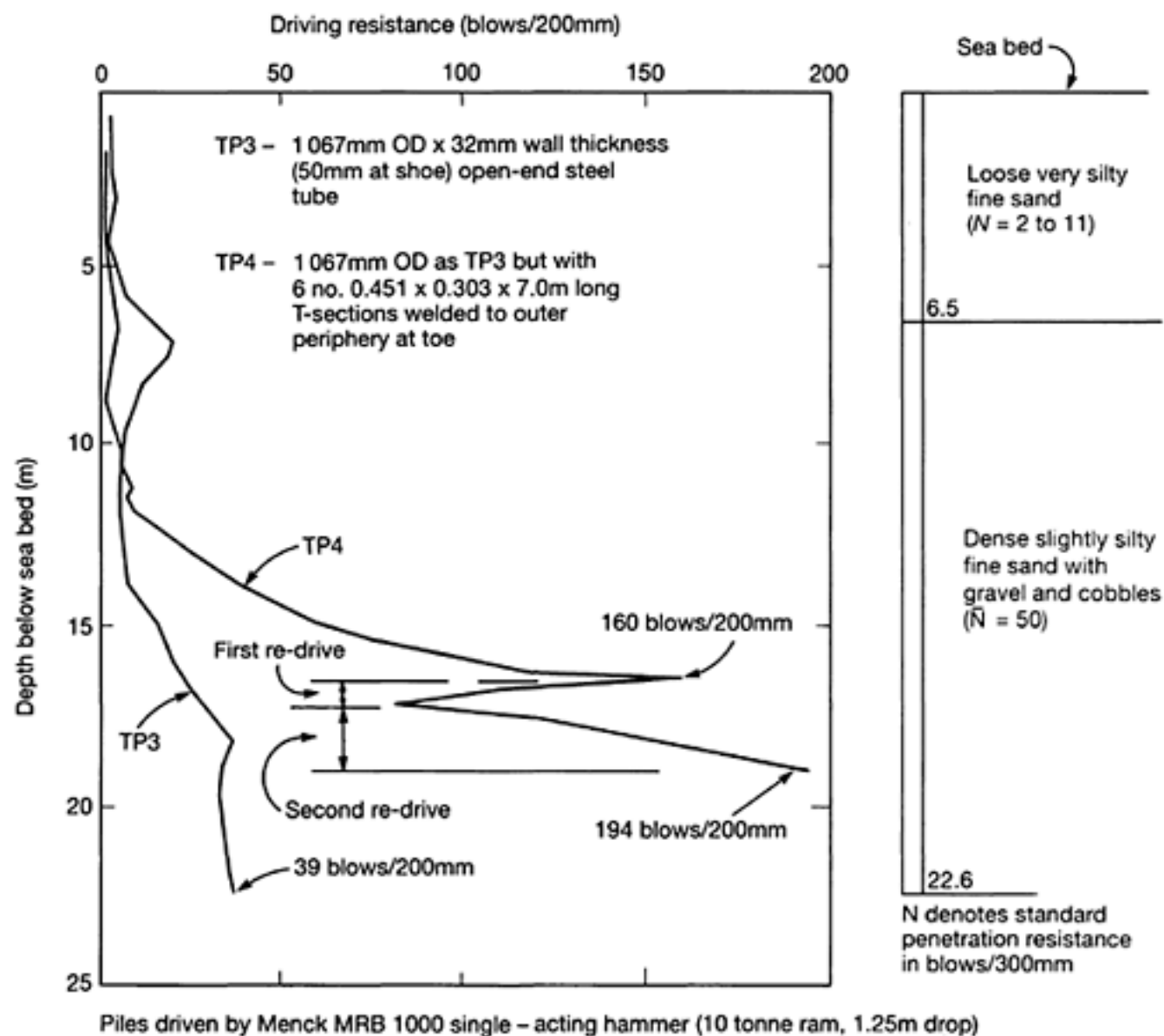


Fig. 2.19 Comparison of driving resistance of open-end plain and winged tubular steel piles at Britoil Tanker Terminal, Cromarty Firth

The curvature of H-piles can be measured by welding a steel angle or channel to the web of the pile (Figure 2.20). After driving, an inclinometer is lowered down the square-shaped duct to measure the deviation from the axis of the pile. This method was used by Hanna<sup>(2.8)</sup> at Lambton Power Station, Ontario, where 305mm and 355mm H-piles that were driven through 46m of clay into shale had deviated 1.8 to 2.1m from the vertical with a minimum radius of curvature of 52m. The piles failed under a test load, and the failure was attributed to plastic deformation of the pile shaft in the region of maximum curvature.

In the UK H-piles are rolled to BS 2566 as universal bearing piles (Figure 2.21a). *Peine piles* are broad-flanged H-sections rolled by Stahlwerke Peine-Salzgitter AG of Germany. They are rolled with bulbs at the tips of the flanges (Figure 2.21b). Loose clutches are used to interlock the piles into groups suitable for dolphins or fenders in marine structures. They can also be interlocked with Larssen sections to strengthen sheet-pile walls. The Arbed-HZ piles rolled by Columeta of Luxembourg are of similar design.

The *Monotube pile* fabricated by the Union Metal Manufacturing Company of USA is a uniformly tapering hollow steel tube. It is formed from steel which is cold-worked to a fluted section having a tensile yield strength of 345N/mm<sup>2</sup> or more. The strength of the fluted section is adequate for the piles to be driven from the top by hammer without an internal mandrel or concrete filling. The tubes have a standard tip diameter of 203mm and the shaft diameter increases to 305mm, 356mm, 406mm, or 457mm at rates of taper which can be varied to suit the required pile length. An upper section of uniform diameter can be fitted (Figure 2.22), which is advantageous for marine work where the fluted section has satisfactory strength and resilience for resisting wave forces and impact forces from small to medium-size ships. The tubes are fabricated in 3, 5, 7, 9 and 11 gauge steel. The heavier gauges enable piles

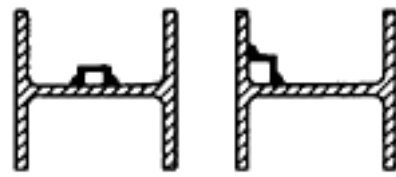


Fig. 2.20 Ducts welded to webs of H-section piles for measurements of curvature by inclinometer

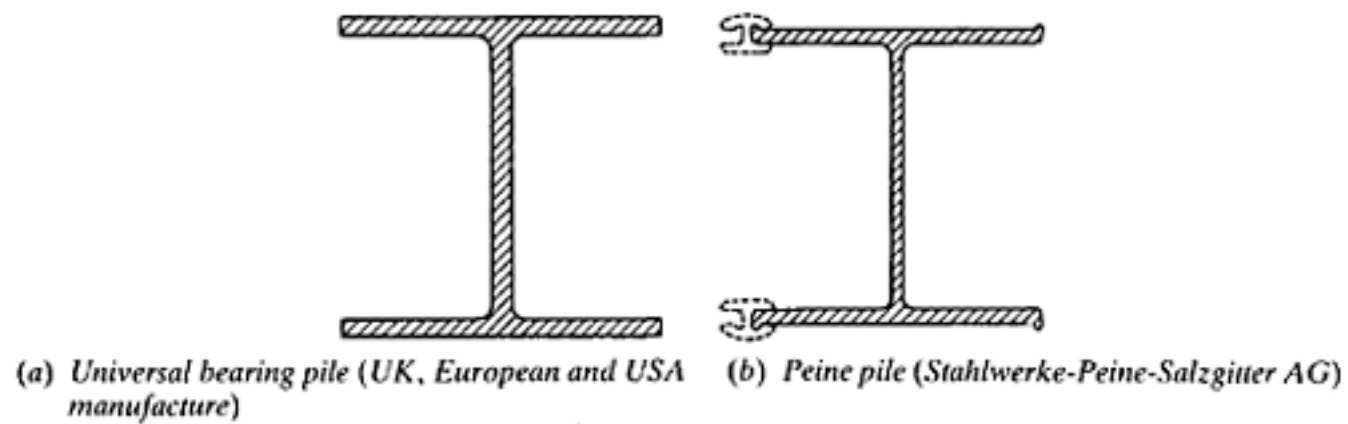


Fig. 2.21 Types of H-section steel piles

to be driven into soils containing obstructions without the tearing or buckling which can occur with thin steel shell piles.

### 2.2.5 Shoes for steel piles

No shoes or other strengthening devices at the toe are needed for tubular piles driven with open ends in easy to moderately easy driving conditions. Where open-ended piles have to be driven through moder-

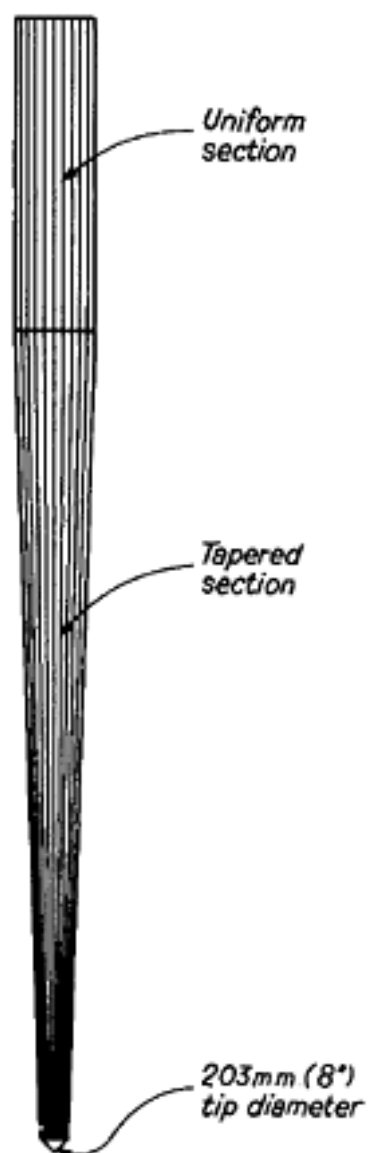


Fig. 2.22 Union Monotube pile (Union Metal Manufacturing Co.)

ately resistant layers to obtain deeper penetrations or where they have to be driven into weak rock, the toes should be strengthened by welding-on a steel ring. The internal ring (Figure 2.23a) may be used where it is necessary to develop the full external skin frictional resistance of the pile shaft. An external ring (Figure 2.23b) is useful for reducing the skin friction to enable end-bearing piles to be driven to a deep penetration, but the uplift resistance will be permanently reduced. Hard driving through strongly resistant layers or to seat a pile onto a rock may split or tear the ring shoe of the type shown in Figures 2.23a and 2.23b. For hard driving it is preferable to adopt a welded-on thick plate shoe designed so that the driving stresses are transferred to the parent pile over its full cross-sectional area (Figure 2.23c).

A shoe of this type can be stiffened further by cruciform steel plates (Figure 2.24a). Buckling and tearing of an external stiffening ring occurred when 610mm OD steel tube piles were driven into the sloping surface of strong limestone bedrock (Figure 2.24b).

Steel box piles can be similarly stiffened by plating unless they have a heavy wall thickness such that no additional strengthening at the toe is necessary. Steel tubular or box piles designed to be driven with closed ends can have a flat mild steel plate welded to the toe (Figure 2.25a) when they are terminated in soils or weak rocks. The flat plate can be stiffened by vertical plates set in a cruciform pattern. Where they are driven on to a sloping hard rock surface they can be provided with Oslo points as shown in Figure 2.25b.

Steel H-piles may have to be strengthened at the toe for situations where they are to be driven into strongly cemented soil layers, or soil containing cobbles and boulders. The strengthening may take the form of welding on steel angles (Figure 2.26a), or purpose-made devices such as the 'Pruyn Point' manufactured in USA by the Associated Pile and Fitting Corporation (Figure 2.26b) or the 'Strongshoe' and 'Jet shoe' manufactured in UK by Dawson Construction Plant Ltd.

## 2.2.6 Working stresses for steel piles

The requirements of various codes of practice for steel tubular, box and H-section piles are shown in Table 2.9. The American codes place a limit on the working stresses or on the maximum load which can be carried by a pile of any dimensions. This limitation is again made as a safeguard against excessively hard driving.

BS 8004 requires steel for piles to conform to BS 4360 Grades 43A, 50B 'or other grades to the approval of the engineer'. The selection of a grade of steel for a particular task depends on the environmental conditions as well as on the design working stresses. For piles wholly embedded in the ground, or for piles in river and marine structures which are not subjected to severe impact forces, particularly in tropical or temperate waters, a mild steel conforming to Grade 43A (minimum yield strength  $247\text{N/mm}^2$ ) or a high-tensile steel to Grade 50 (minimum yield strength  $355\text{N/mm}^2$ ) should be satisfactory. However, piles for deep-water platforms or berthing structures for large vessels (see Chapter 8) are subjected to high dynamic stresses from berthing impact and wave forces. In water at zero or sub-zero temperatures, there is a risk of brittle fracture under dynamic loading, and the effects of fatigue damage under large numbers of load repetitions and also of salt water corrosion need to be considered. Steels must be

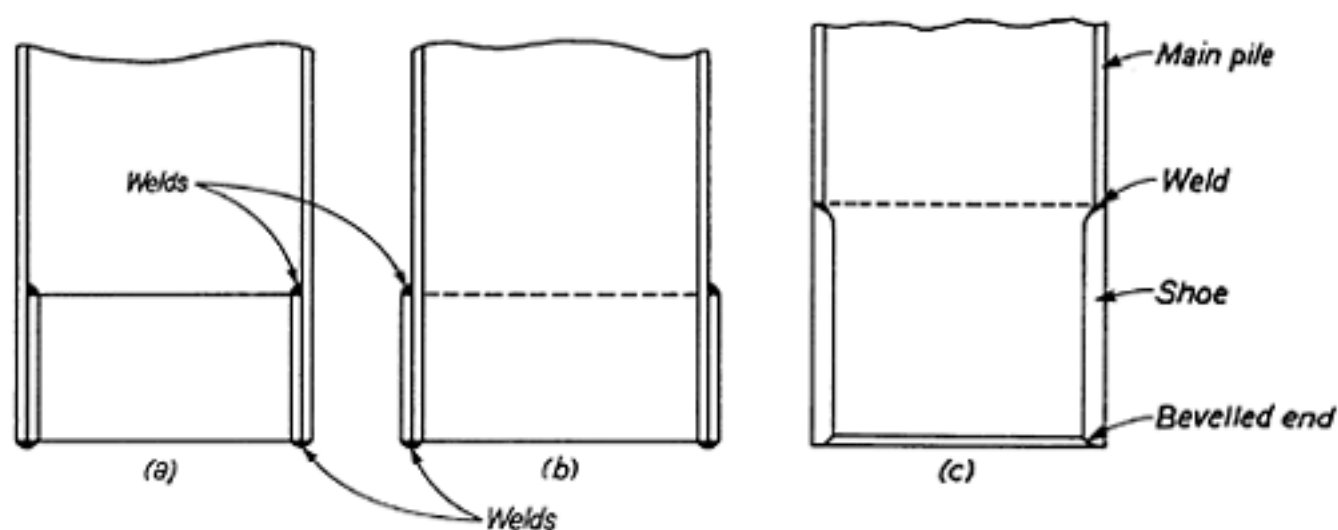
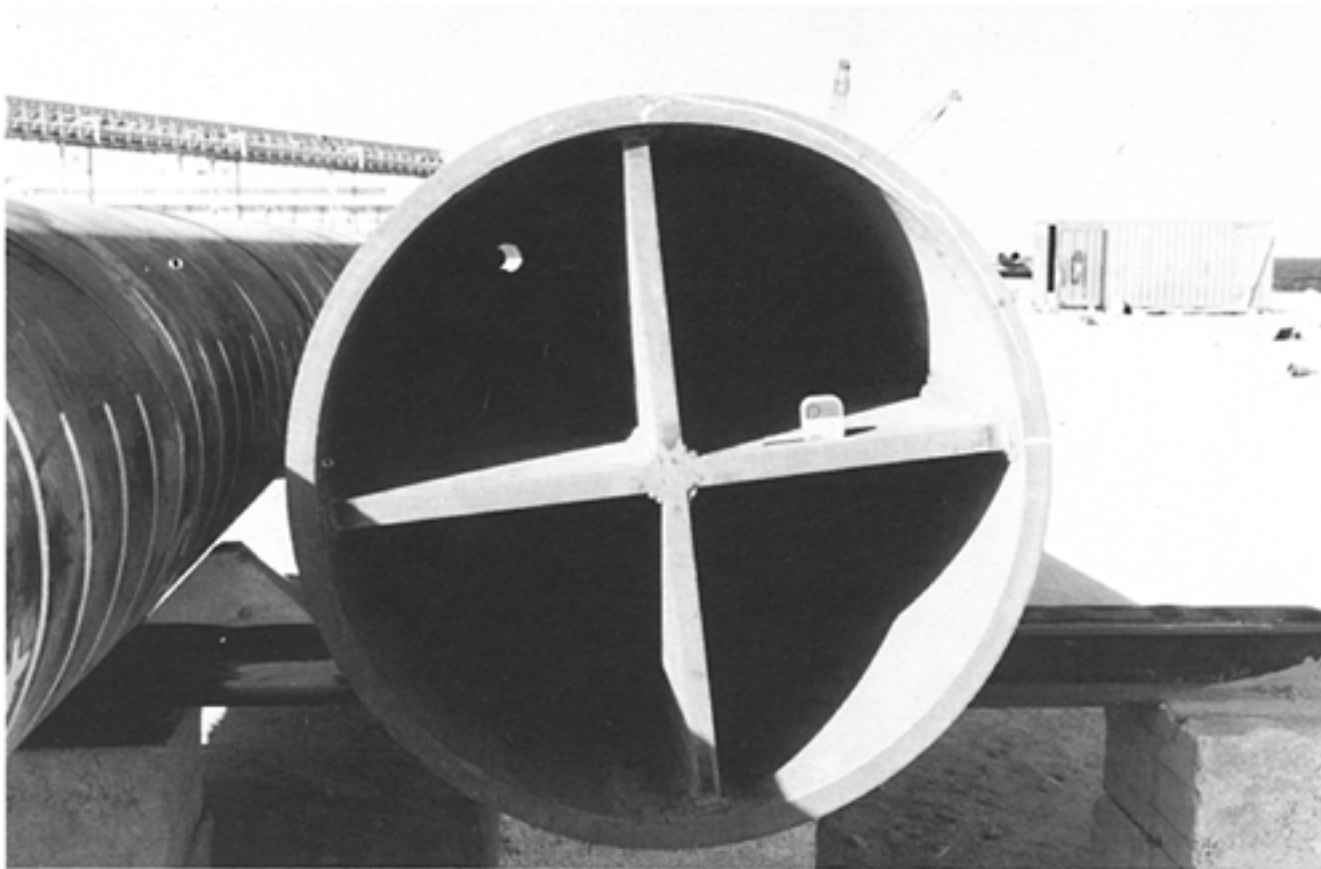


Fig. 2.23 Strengthening toe of steel tubular piles

- (a) Internal stiffening ring
- (b) External stiffening ring
- (c) Thick plate shoe



*Fig. 2.24a Strengthening shoe of tubular steel pile by cruciform plates*



*Fig. 2.24b Buckling and tearing of welded-on external stiffening ring to tubular steel pile driven on to sloping rock surface*

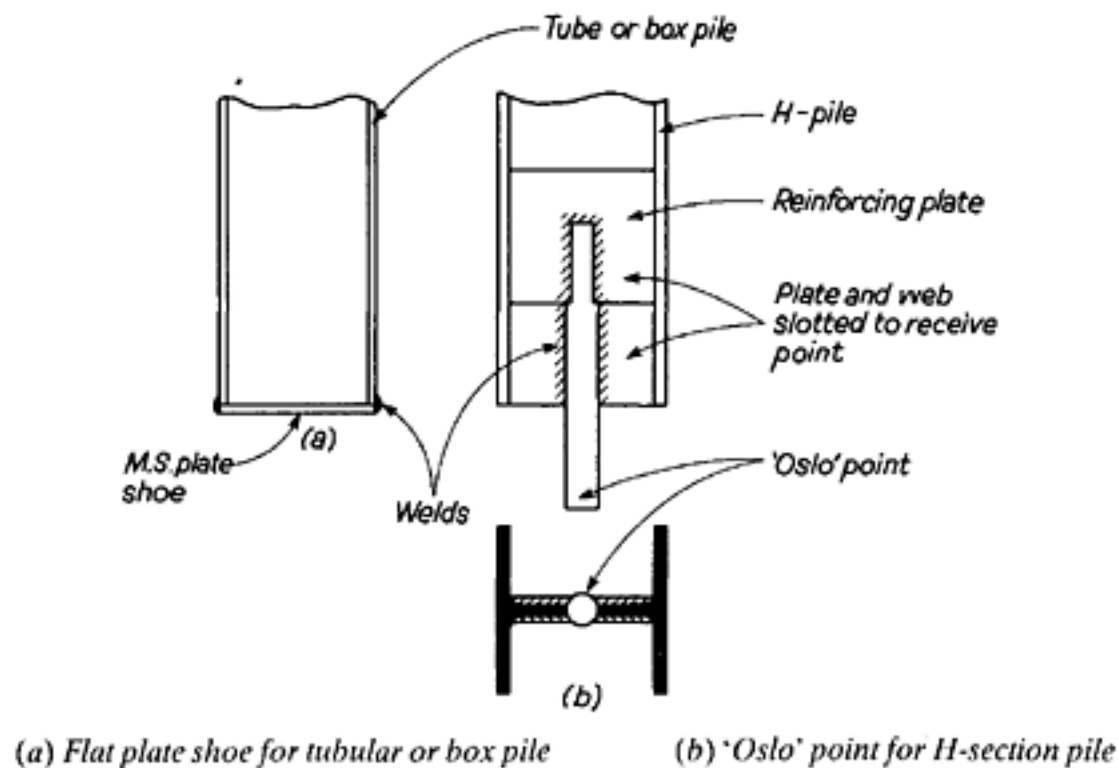


Fig. 2.25 Shoes for steel piles

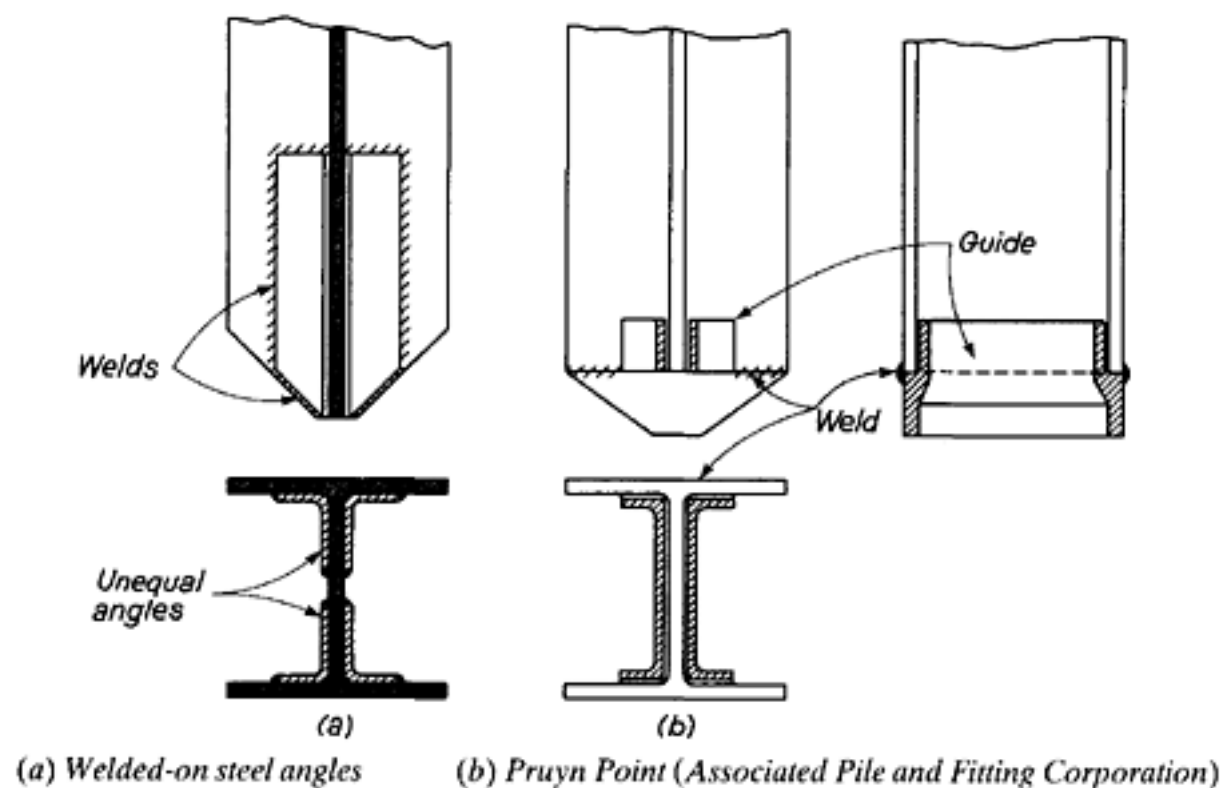


Fig. 2.26 Strengthening toe of H-section pile

selected to have a high impact value when tested at low temperatures. Steels conforming to the 50 or 55 grades in BS 4360 are required to have minimum average and minimum Charpy V impact values of 2.4m/N and 2.0m/N respectively when tested at 0°C. Piles or bracing members for deep-water structures may be required to be fabricated from plates  $\frac{1}{4}$  in or more in thickness. The steel for such plates requires a greater brittle fracture resistance at low temperatures and the above impact values are required to be met at -15°C. High-tensile alloy steel conforming to Grades 55 C or E in BS 4360 can meet these requirements.

## 2.3 Driven-and-cast-in-place displacement piles

### 2.3.1 General

Driven-and-cast-in-place piles are installed by driving to the desired penetration a heavy-section steel tube with its end closed. A reinforcing cage is next placed in a tube which is filled with concrete. The tube is withdrawn while placing the concrete or after it has been placed. In other types of pile, thin steel shells or precast concrete shells are driven by means of an internal mandrel, and concrete, with or without reinforcement, is placed in the permanent shells after withdrawing the mandrel.

Driven-and-cast-in-place piles have the principal advantage of being readily adjustable in length to

Table 2.9 Code of practice requirements for working stresses on steel piles

Country	Code	Working stress in compression		Remarks
		Related to yield strength	N/mm <sup>2</sup>	
United Kingdom	BS 8004	0.30f <sub>y</sub> <sup>(1)</sup> 0.50f <sub>y</sub> <sup>(2)</sup>	—	(1) Where safety factor on driving resistance is not greater than 2. (2) For jacked piles or where end-bearing piles are driven through relatively soft soils on to very dense granular soils or sound rock. Steel to conform to BS 4360.
Germany	DIN 4026	—	—	H-sections to conform to DIN 17100. Seamless steel tubes to conform to DIN 1629. Welded steel tubes to conform to DIN 17100.
USA	American Concrete Institute Recommendations <sup>(2.9)</sup>	0.35f <sub>y</sub>	87 (max)	Minimum thickness of pipe piles 2.5mm. Cross-sectional area of pipe piles to be at least 3% of gross cross-section. Pipe to ASTM A252–69.
	New York City (1985)	0.35f <sub>y</sub>	≥248 ≥248	Pipe piles min. thickness 3mm.  H-piles min thickness 10mm. Unless higher loads can be substantiated by a specific load test procedure, the basic maximum loads are as follows. Open-ended pipes bearing on medium to hard rock 2500kN for piles of 457mm OD or greater; 2000kN for 457mm or less OD. Closed-ended piles bearing on medium to hard rock 1500kN; open-ended pipes or H-piles bearing on soft rock 800kN; closed or open-ended piles or H-piles bearing on hardpan over rock 1000kN.
France	DTU13.2 (1978)	—	120	For building foundations using EN24–1 Steel.

suit the desired depth of penetration. Thus in the withdrawable-tube types the tube is driven only to the depth required by the ground conditions. In the shell types, the length of the pile can be easily adjusted by adding or taking away the short units which make up the complete shell. The withdrawable-tube piles are the most economical type of pile for land structures. In conditions favourable for their employment, where the required penetration depth is within the capability of the piling rig to pull out the tube, and there are no restrictions on ground heave or vibrations, they can be installed more cheaply than any other type of driven or bored pile. They also have the advantage, which is not enjoyed by all types of shell pile, of allowing an enlarged base to be formed at the toe. However, some codes of practice, notably that of New York City, forbid the use of a wholly uncased shaft for all forms of

driven-and-cast-in-place pile, where these are installed in soft to firm clays or in loose to medium-dense sands and materials such as uncompacted fill. These restrictions are the result of unfortunate experiences resulting from lifting of the concrete while pulling out the driving tube, and of squeezing ('waisting') the unset concrete in the pile shaft where this is formed in soft clays or peat. The firms who install these proprietary types of pile have adopted various techniques for avoiding these troubles, such as inserting permanent light-gauge steel shells before placing the concrete. However, such expedients increase the cost of the withdrawable-tube piles to the extent that their advantage in price over shell piles may be wholly or partially lost. The soundness of the uncased type of pile depends on the skill and integrity of the operatives manning the piling rig. If these factors can be assured there is no reason why uncased piles of this type cannot be used in most soil conditions, and they are extensively used in the widely ranging soil types found in the UK.

Piling rigs have not yet been developed to install driven-and-cast-in-place piles of the very large diameters which are possible with driven thick-walled steel tubes or bored-and-cast-in-place piles. Thus the working loads are limited to the light to medium range. Also the withdrawable-tube or thin-shell types are unsuitable for marine structures, but they can be employed in marine situations if they are extended above the sea bed as columns or piers in steel or precast concrete. Problems associated with ground heave when installing driven-and-cast-in-place piles in groups are discussed in 5.7.

### 2.3.2 Withdrawable-tube types

Descriptions of the various types of driven and cast-in-place piles are given in CIRIA report *Review of bearing pile types* (2.10). The methods of installing these piles are essentially the same. A piling rig consisting of a mast, leaders and winch on a track or roller-mounted frame (3.1) is used to support and guide the withdrawable tube. The latter is of heavy wall section and its lower end is closed by an expendable steel plate. The tube is driven from the top by a simple drop hammer or by a diesel or vibrating hammer. On reaching the required toe level, as predetermined by calculation or as determined by measurements of driving resistance, the hammer is lifted off and a reinforcing cage is lowered down the full length of the tube. A highly workable self-compacting concrete is then placed in the tube through a hopper, followed by raising the tube by a hoist rope operated from the pile frame. The tube may be filled completely with concrete before it is lifted or it may be lifted in stages depending on the risks of the concrete jamming in the tube. The length of the pile is limited by the ability of the rig to pull out the drive tube. This restricts the length to about 20m to 30m.

A variation of the above method is practised by Cementation Piling and Foundations Limited in their proprietary Franki pile. This employs an internal drop hammer acting on a plug of gravel at the bottom of the drive tube. The drive tube is carried down with the plug until the required toe level is reached when the tube is restrained from further penetration by rope tackle. Then the gravel plug and batches of dry concrete are hammered out to form a bulb or enlarged base to the pile. The reinforcing cage is then inserted, followed by placing a semi-dry concrete in batches as the drive tube is pulled out in stages. After each stage of withdrawal the concrete is compacted by the internal hammer (Figure 2.27). The operations of driving by internal hammer and concreting in stages are slower than the top driving method described above. Hence these techniques are used only when there are economic advantages, for example when the enlarged base adds appreciably to the bearing capacity of the pile.

In a further variation of the Franki technique, the gravel plug can be hammered out at several intermediate stages of driving to form a shell of compact material around the pile shaft. This technique is used in very soft clays which are liable to squeeze inwards when withdrawing the tube.

A full length reinforcing cage is always advisable in the driven-and-cast-in-place pile. It acts as a useful tell-tale against possible breaks in the integrity of the pile shaft caused by arching and lifting of the concrete as the tube is withdrawn. The problem of inward squeezing of soft clays and peats or of bulging of the shafts of piles from the pressure of fluid concrete in these soils is common to cast-in-place piles both of the driven and bored types. A method of overcoming this problem is to use a permanent light gauge steel lining tube to the pile shaft. However, great care is needed in withdrawing the drive tube to prevent the permanent liner being lifted with the tube. Even a small amount of lifting can cause transverse cracks in the pile shaft of sufficient width to result in excessive settlement of the pile head under the working load. The problem is particularly difficult in long piles when the flexible lining tube tends to snake and jam in the drive tube. Also where piles are driven in large groups, ground heave can lift the lining tubes off their seating on the unlined portion of the shaft. Snaking and jamming of the permanent liner can be avoided by using spacers such as rings of sponge rubber. In most cases the annulus left outside the permanent liner after pulling the drive tube will not close

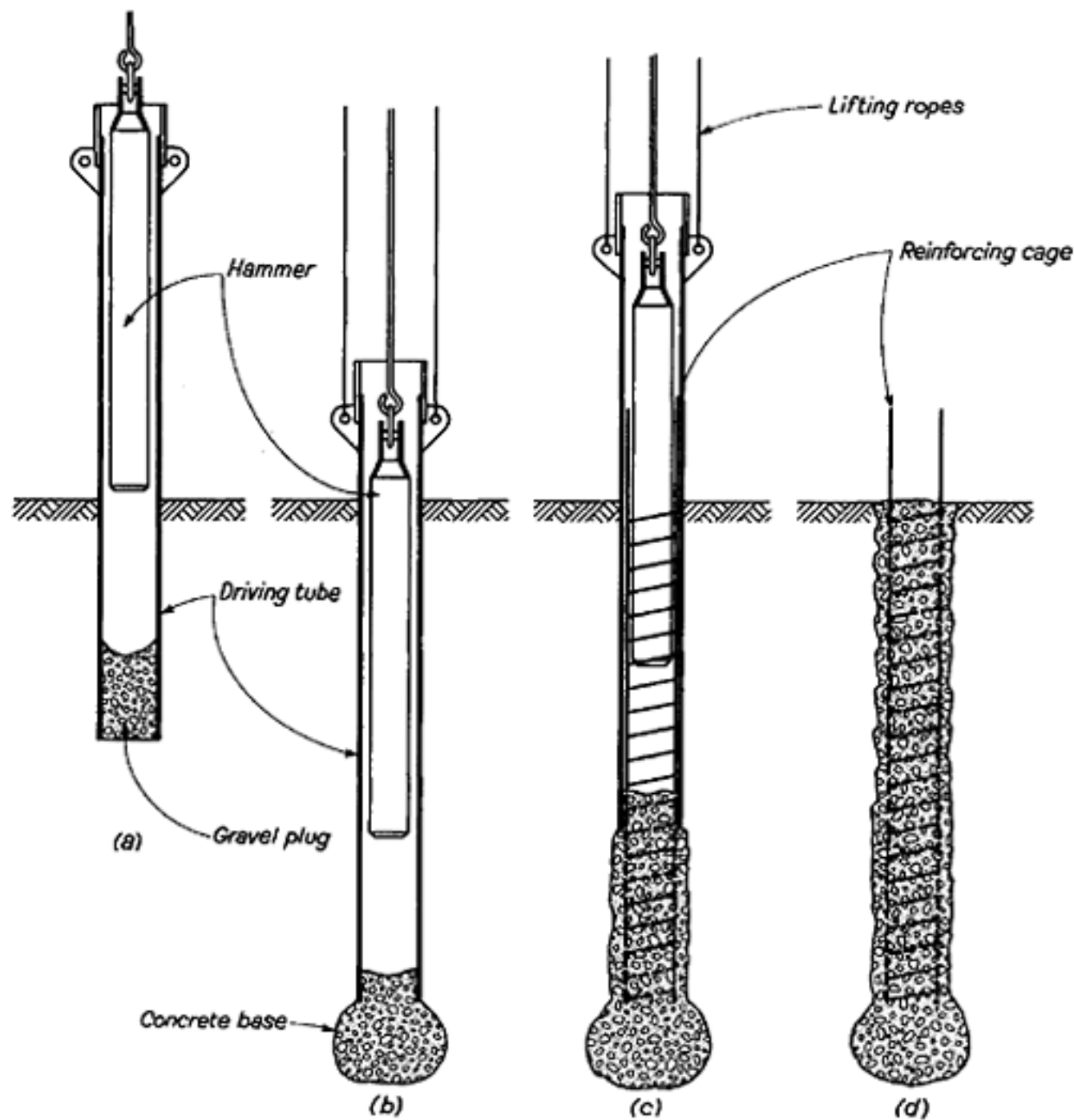


Fig. 2.27 Stages in installing a Franki pile

- (a) Driving piling tube
- (b) Placing concrete in piling tube
- (c) Compacting concrete in shaft
- (d) Completed pile

up. Hence there will be no skin frictional resistance available on the lined portion. This can be advantageous because dragdown forces in the zone of highly compressible soils and fill materials will be greatly reduced. However, the ability of the pile shaft to carry the working load as a column without lateral support below the pile cap should be checked. Problems concerned with the installation of driven and cast-in-place piles are discussed further in 3.4.5.

Allowable stresses on the shafts of these piles are influenced by the need to use easily workable self-compacting mixes with a slump in the range of 100mm to 175mm and to make allowances for possible imperfections in the concrete placed in unseen conditions. A cube compression strength in the range of  $21\text{N/mm}^2$  to  $30\text{N/mm}^2$  is usually adopted. BS 8004 limits the working stress of 25% of the 28-day cube strengths giving allowable stresses of  $5\text{N/mm}^2$  to  $7.5\text{N/mm}^2$ . For these values, allowable loads for piles of various shaft diameters are as follows:

Nominal shaft diameter (mm)	Allowable working load (kN)
300	350 to 500
350	450 to 700
400	600 to 900
450	800 to 1000
500	1000 to 1400
600	1400 to 2000

The higher ranges in the above table should be adopted with caution, particularly in difficult ground conditions.

The spacing of bars in the reinforcing cage should give ample space for the flow of concrete through

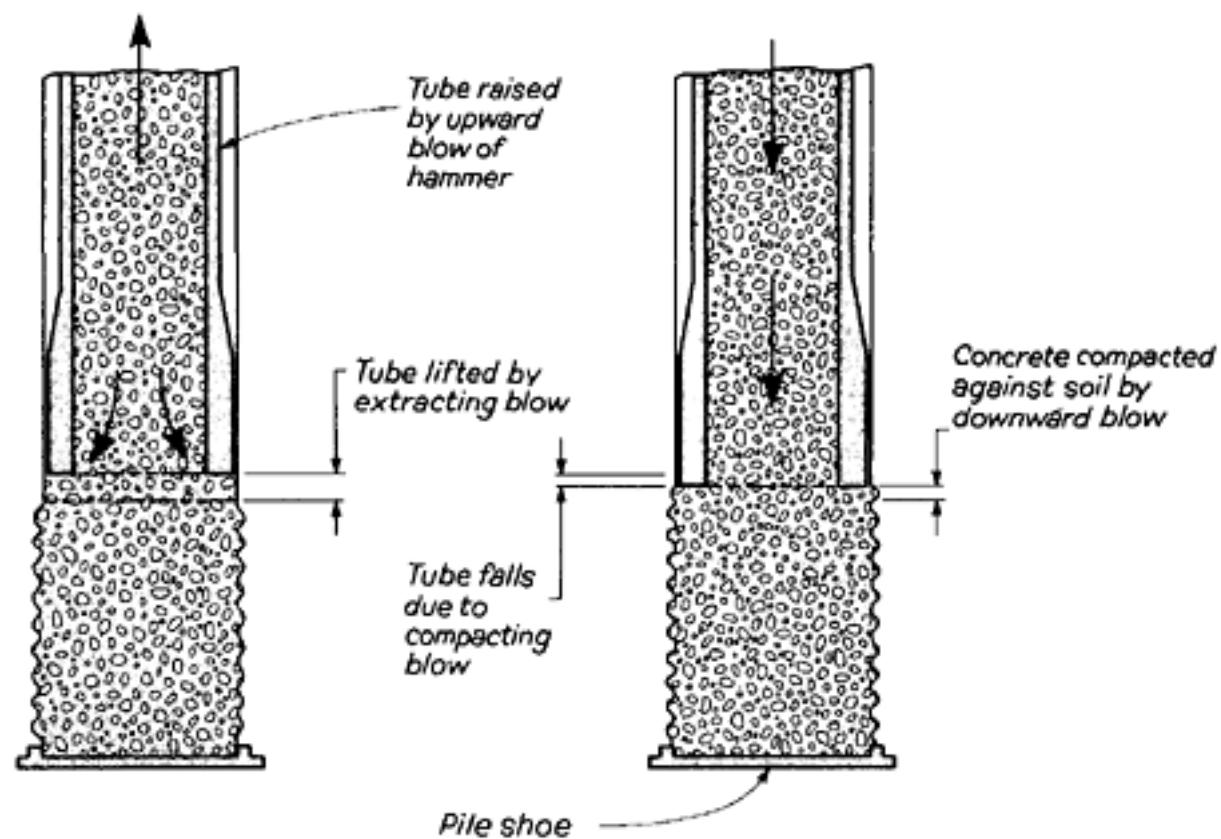


Fig. 2.28 Compacting concrete in Vibro pile

Maximum working loads are as follows.

Nominal shaft diameter (mm)	Nominal maximum working load (kN)
350	440
400	590
450	740
500	930
550	980
600	1500
715	2000

them. Bars of 5mm diameter in the form of a spiral or flat steel hoops used for lateral reinforcement should not be spaced at centres closer than 100mm.

The *Vibrex pile* installed in Holland by Verstraeten BV and in Belgium by Fundex PVBA employs a diesel or hydraulic hammer to drive the tube which is closed at the end by a loose plate. A vibrating unit, which is clamped to the upper end of the tube as it is driven down, is then employed to extract the tube after the concrete has been placed. A variation of the technique allows an enlarged base to be formed by using the hammer to drive out a charge of concrete at the lower end of the pile. The *Vibrex pile* is formed in shaft diameters of from 350 to 600mm and in lengths up to 38mm.

The *Fundex pile* installed by the same companies, is a form of screwpile. A helically-screwed drill point is held by a bayonet joint to the lower end of the piling tube. The latter is then rotated by a hydraulic motor on the piling frame and at the same time forced down by hydraulic rams. On reaching founding level, a reinforcing cage and concrete are placed in the tube which is then withdrawn. The *Tubex pile* also employs the screwed drill point, but the tubes are left in place for use in very soft clays when 'waisting' of the shaft must be avoided. The tube can be drilled down in short lengths, each length being welded to the one already in place. Thus the pile is suitable for installation in conditions of low headroom, for example for underpinning work.

The speciality of the *Vibro pile* is the method used to compact the concrete in the shaft by alternate upward and downward blows of a hammer on the driving tube. The upward blow of the hammer operates on links attached to lugs on top of the tube. This raises the tube and allows concrete to flow out. On the downward blow the concrete is compacted against the soil (Figure 2.28). The blows are made in rapid succession which keeps the concrete 'alive' and prevents its jamming in the tube. The *Vibro pile* is installed in the UK under the proprietary name 'Vibroform' by Piling Construction Ltd.

### 2.3.3 Shell types

Instead of steel lining tubes, *precast concrete* sections can be lowered down the temporary drive tube and bedded onto a layer of cement mortar placed on the shoe. The space around the precast concrete

sections can be filled with cement grout which is injected as the drive tube is pulled out. The *Positive pile* is an example of the technique. It was developed for use in the soft ground conditions in Singapore and has been used in Malaysia and Borneo. Types employing a *metal shell* generally consist of a permanent corrugated steel lining tube which is locked onto a steel plate or precast concrete shoe. The lining tube and shoe can be driven down by a collapsible mandrel which locks into the corrugations. Alternatively the shoe can be driven down by a temporary drive tube followed by placing the liner and locking it to the shoe, and then withdrawing the drive tube. The permanent liners are then filled with concrete and any necessary reinforcing steel. The feature of these piles is the provision for locking the lining tubes to the shoe, thus preventing uplift when the drive tube or mandrel is pulled out.

In France cased piles varying in diameter from 150mm to 500mm are installed by welding a steel plate to the base of the tubular section to project at least 40mm beyond the outer face of the steel. As the pile is driven down, a cement/sand mortar with a minimum cement content of  $500\text{kg/m}^3$  is injected into the annulus formed around the pile by the projecting plate through one or more pipes having their outlet a short distance above the end plate. The rate of injection of the mortar is adjusted by observing the flow of mortar from the annulus at the ground surface. The working load is designed to be carried by the steel section. A working stress of  $160\text{N/mm}^2$  is permitted by the French Code DTU 13.2 for EN24–1 steel. This is higher than the value given in [Table 2.9](#) for steel piles, because of the protection given to the steel by the surrounding mortar.

Steel H or box sections can be given mortar protection in a similar manner.

*Raymond Step-Taper Piles* (*Raymond Concrete Pile Co. USA*) consist of helically-corrugated light-steel shells made in 2.44m, 3.66m and 4.88m lengths, in diameters increasing progressively from 219mm to 441mm in nine increments. The shells are assembled on an internal mandrel with the smallest size 219mm at the bottom, and the assembly increasing in diameter and length as required by the design working loads ([Figure 2.29](#)). After driving is completed, the mandrel is withdrawn and the tapered shell pile is filled with concrete.

Where the required length of pile is greater than can be achieved by an assembly of standard units, the step-tapered pile can be extended in length by fixing a steel tube onto the assembly. The diameter of the tube can be varied to suit the requirements for carrying capacity.

The *West's Shell Pile* (*West's Piling and Construction Co.*) incorporates precast concrete shell units which are threaded onto an internal mandrel that carries a detachable precast concrete conical shoe at its lower end. The shells are joined by circumferential steel bands which are painted internally with

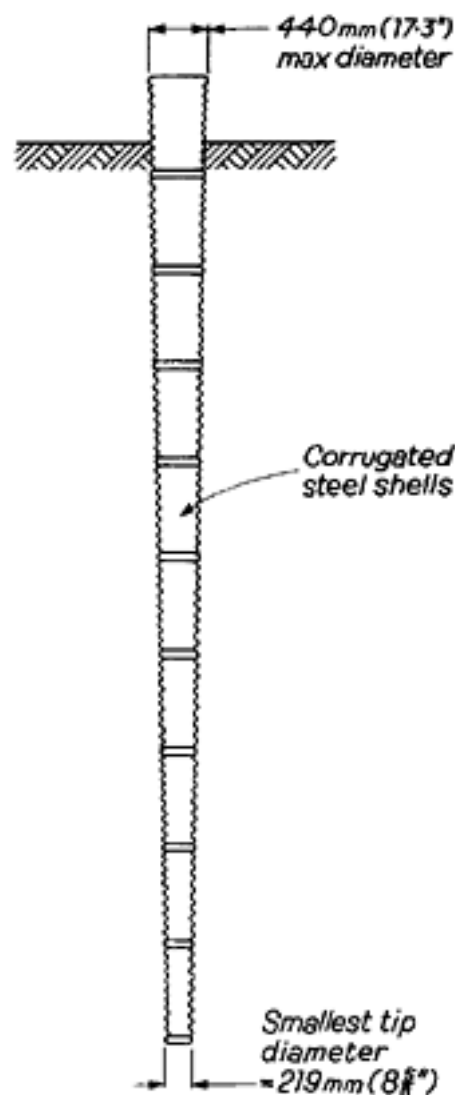


Fig. 2.29 The Raymond step taper pile

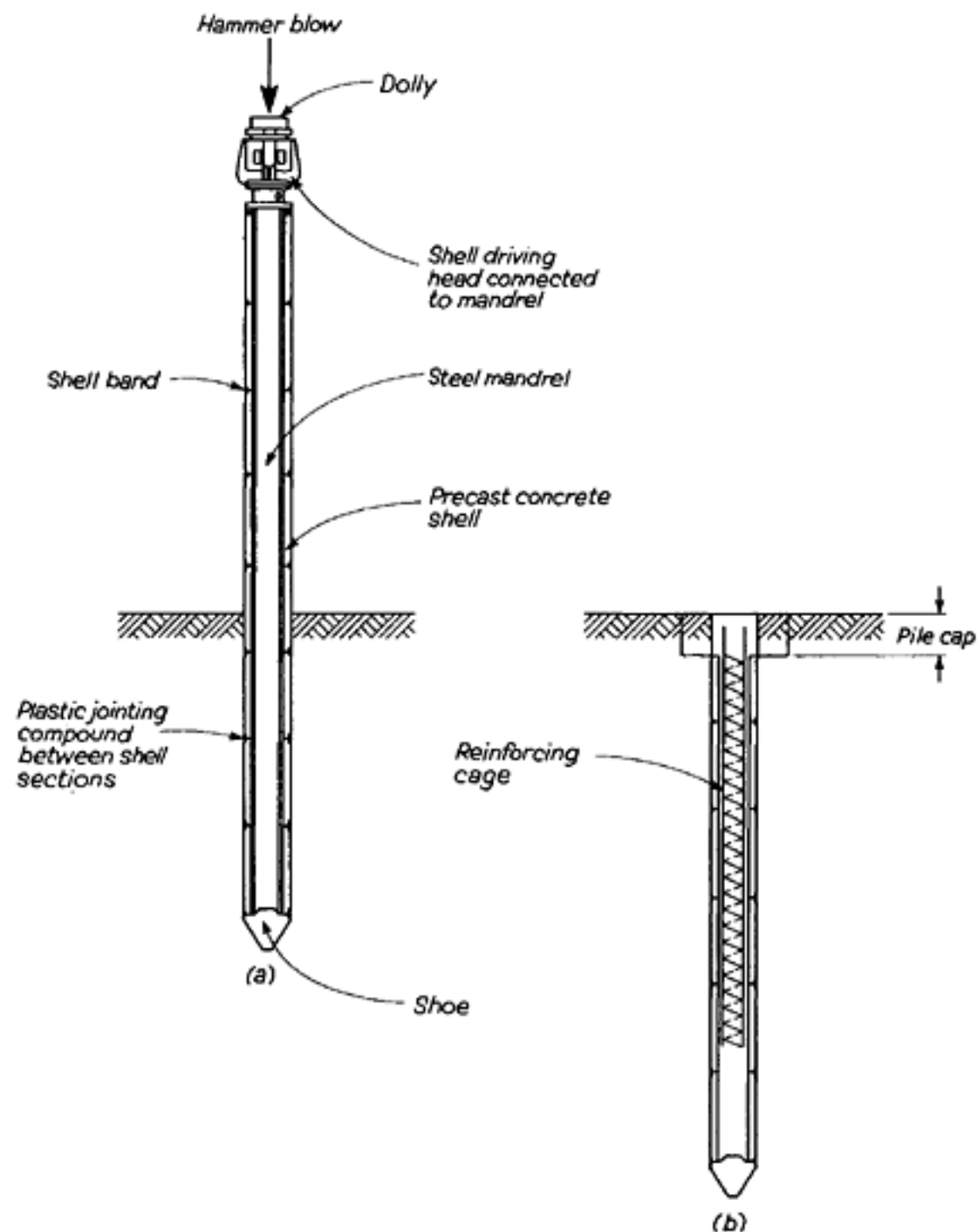


Fig. 2.30 Stages in installing a West's shell pile

(a) Driving shell units on mandrel

(b) Reinforcing cage installed before placing concrete in interior of shells

bitumen to make a watertight joint. On reaching founding level any surplus shells are removed, the mandrel is withdrawn and a reinforcing cage is lowered down the hollow shaft. Concrete is then placed to the required level. The driving head is designed so that the main hammer blow is delivered to the mandrel, but a cushioned blow is also made on the shells to ensure that they move down with the mandrel. The stages in driving the West's pile are shown in [Figure 2.30](#). The precast concrete shells are reinforced with polypropylene fibres to increase their resistance to impact during driving. The West's pile has the advantage of being readily adjustable in length by adding or removing the 914mm long shell units as required. Nominal maximum working loads for the five shell sizes are as follows.

External diameter (mm)	Core diameter (mm)	Nominal maximum working load (kN)
381	276	650
406	276	650
444	305	800
508	381	1200
533	381	1200

In conditions giving rise to ground heave (see [5.7](#)) the shells of piles already driven are liable to be lifted while driving adjacent piles. This may result in excessive settlement when the piles are subsequently loaded. Pre-boring may be necessary to overcome this difficulty. Also lifting or damage to the shells may occur if the mandrel deviates from a true line while driving past obstructions or onto a sloping bedrock surface.

### 2.3.4 Working stresses on driven-and-cast-in-place piles

It can be seen from [Section 2.3.3](#) that driven-and-cast-in-place piles encompass a wide variety of shapes, combinations of materials, and installation methods. A common feature of nearly all types is an interior filling of concrete placed in situ, which forms the main load-carrying component of the pile. Whether or not any load is allowed to be carried by the steel shell depends on its thickness and on the possibilities of corrosion or tearing of the shell. The requirements for working stresses on the core and shell or on the permanent driving tube are usually laid down in the various codes of practice (see [Table 2.10](#)).

These requirements can differ quite widely and sometimes rather illogically. It is usual to require a working stress on the concrete core which is lower than that allowed for precast concrete piles. This is because the codes of practice take into account possible deficiencies in workmanship during placing the concrete, or reductions in section of the pile shaft due to 'waisting' or buckling of the shells.

Shell piles are more widely used in the USA than elsewhere and most of the American codes require the shells to be at least 2.5mm thick before they can be permitted to carry a proportion of the load. Frequently a wall thickness of 3mm is required.

## 2.4 Replacement piles

### 2.4.1 General

Replacement piles are installed by first removing the soil by a drilling process, and then constructing the pile by placing concrete or some other structural element in the drilled hole. The simplest form of construction consists of drilling an unlined hole and filling it with concrete. However, complications may arise such as difficult ground conditions, the presence of ground water, or restricted access. Such complications have led to the development of specialist piling plant for drilling holes and handling lining tubes, but unlike the driven-and-cast-in-place piles, very few proprietary piling systems have been promoted. This is because the specialist drilling machines are available on sale or hire to any organization which may have occasion to use them. The resulting pile as formed in the ground is more or less the same no matter which machine, or method of using the machine, is employed. There have been proprietary systems such as the Prestcore pile, which incorporates precast units installed in the pile borehole, but these methods are largely obsolete.

There are two principal types of replacement pile. These are bored-and-cast-in-place piles, and drilled-in tubular (including caisson) piles. A general description of the two types now follows. Mechanical plant for installing the piles and methods of construction are described in [Section 3.3](#).

### 2.4.2 Bored-and-cast-in-place piles

In stable ground an unlined hole can be drilled by hand or mechanical auger. If reinforcement is required a light cage is then placed in the hole, followed by the concrete. In loose or water-bearing soils and in broken rocks casing is needed to support the sides of the borehole, this casing being withdrawn during or after placing the concrete. In stiff to hard clays and in weak rocks an enlarged base can be formed to increase the end-bearing resistance of the piles ([Figure 2.31](#)). The enlargement is formed by a rotating expanding tool, or by hand excavation in piles having a large shaft diameter. A sufficient cover of stable cohesive soil must be left over the top of the enlargement in order to avoid a 'run' of loose or weak soil into the unlined cavity. The German Code DIN 4014 requires a cover of at least 1m of stable soil over the top of the enlargement, as shown in [Figure 2.31](#).

Bored piles drilled by hand auger are limited in diameter to about 355mm and in depth to about 5m. They can be used for light buildings such as dwelling houses, but even for these light structures hand methods are used only in situations where mechanical augers, as described in 3.3.1, are not available.

Bored piles drilled by mechanical spiral-plate or bucket augers or by grabbing rigs can drill piles with a shaft diameter up to 7.3m, but it is usual to limit the maximum size to 2.13m diameter to suit the auger plant generally available. Boreholes up to 120m deep are possible with the larger rotary auger machines. Under-reaming tools can form enlarged bases in stable soils up to 7.3m in diameter. The size of enlarged bases formed by hand excavation is limited only by the practical considerations of supporting the sloping sides of the base ([Figure 2.31](#)). It is also possible to drive headings by hand between adjacent piles and to fill them with concrete to form a system of deep beam foundations ([Figure 2.32](#)). Rotary drilling equipment consisting of drill heads with multiple rock roller bits have been manufactured for drilling shafts up to 8m in diameter.

For reasons of economy and the need to develop skin friction on the shaft, it is the normal practice

Table 2.10 Working stresses or maximum working loads in compression on driven-and-cast-in-place piles

Country	Code	Shell	Structural core	Reinforcing steel	Concrete in shaft	Remarks
United Kingdom	BS 8004	—	—	—	$0.25u_w$	Concrete to have a minimum cement content of 300kg/m <sup>3</sup>
USA	American Concrete Institute Recommendations (1974)	—	$0.50f_y$ 172N/mm <sup>2</sup> max	$0.40f_y$ 206N/mm <sup>2</sup> max	$0.33u_w$ (unconfined) $0.40u_w$ (confined)*	* Steel shell confining concrete to be not more than 432mm in diameter. Shell to be 14g(US) or thicker, seamless or spiral welded, $f_y \nlessgtr 206\text{N/mm}^2$ , not exposed to corrosion and does not carry part of working load. Corrugated steel shells not considered as load-bearing
	New York (1985)	$0.35f_y$ <del><math>\nlessgtr 248\text{N/mm}^2</math></del>	—	$0.40f_y$ <del><math>\nlessgtr 206\text{N/mm}^2</math></del>	$0.25u_w$	Metal shells thinner than 3mm not to contribute to strength of pile section. Max working load not to exceed 1 500kN for bearing on intermediate to hard rock, 600kN for soft rock, 1000kN for hardpan overlying rock
Germany	DIN 4014	—	—	—	—	<div> <div>Pile diameter (mm)</div> <div>Max working load (kN)</div> <div>300 200</div> <div>350 250</div> <div>400 300</div> <div>500 400</div> </div> Minimum cement content to be 350kg/m <sup>3</sup>
France	DTU 13.2 (1978)	—	—	Min 5 bars not less than 12mm. Steel to be at least 0.5% of cross-section. Helical reinforcement spaced at not greater than 200mm pitch	5.5N/mm <sup>2</sup>	Minimum cement content 350kg/ m <sup>3</sup> , minimum cover to reinforcement 40mm. Working stresses can be increased to 6N/ mm <sup>2</sup> if tube is extracted by vibrator
Sweden	SBS-S23:6 (1968)	—	—	—	—	Concrete to have minimum crushing strength of 39N/mm <sup>2</sup>

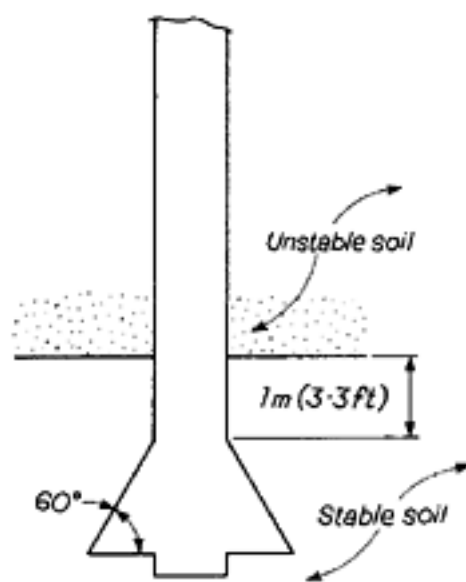


Fig. 2.31 Under-reamed base enlargement to a bored-and-cast-in-place pile

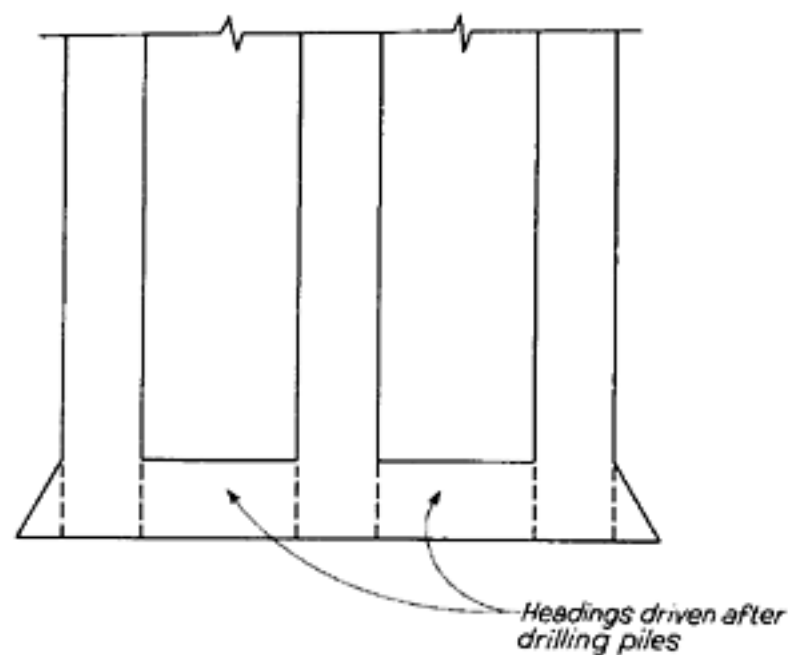


Fig. 2.32 Interconnecting bored-and-cast-in-place piles by hand-driven headings

to withdraw the casing during or after placing the concrete. As in the case of driven-and-cast-in-place piles this procedure requires care and conscientious workmanship by the operatives in order to prevent the concrete being lifted by the casing, and thus resulting in voids in the shaft or inclusions of collapsed soil.

The shafts or bored-and-cast-in-place piles are liable to 'necking' or 'waisting' in soft clays or peats. Sometimes a permanent casing of light spirally-welded metal is provided over the portion of the shaft within these soil types, but this measure can cause problems in installation (see [3.4.6](#)).

Reinforcement is not always needed in bored-and-cast-in-place piles unless uplift loads are to be carried (uplift may occur due to the swelling and shrinkage of clays). Reinforcement may also be needed in the upper part of the shaft to withstand bending moments caused by any eccentricity in the application of the load, or by bending moments transmitted from the ground beams (see [7.9](#)). However, it is often a wise precaution to use a full length reinforcing cage in piles where temporary support by casing is required over the whole pile depth. As noted in 2.3.2, the cage acts as a warning against the concrete lifting as the casing is extracted. The need to allow ample space between the bars for the flow of concrete is again emphasised.

Working stresses for concrete in bored-and-cast-in-place piles required by various codes of practice are shown in [Table 2.11](#).

*Continuous flight auger or auger injected piles*, generally known as CFA piles, are installed by drilling with a rotary continuous-flight auger to the required depth. In stable ground above the water table the auger is then removed and a sand-cement grout is pumped through a flexible pressure hose that has been fed down to the bottom of the unlined hole. This type of pile is referred to as cast-in-place. In unstable or water-bearing soils a flight auger is used with a hollow stem closed at the bottom by a plug. After reaching the final level a fairly fluid cement-sand mortar, or a concrete made with coarse aggregate not larger than 20mm, is pumped down the stem and fills the void as the auger is slowly withdrawn with or without rotation ([Figure 2.33](#)). Thus the walls of the borehole are continually supported by the spiral flights and the soil within them, and by the mortar as it is pumped in, the

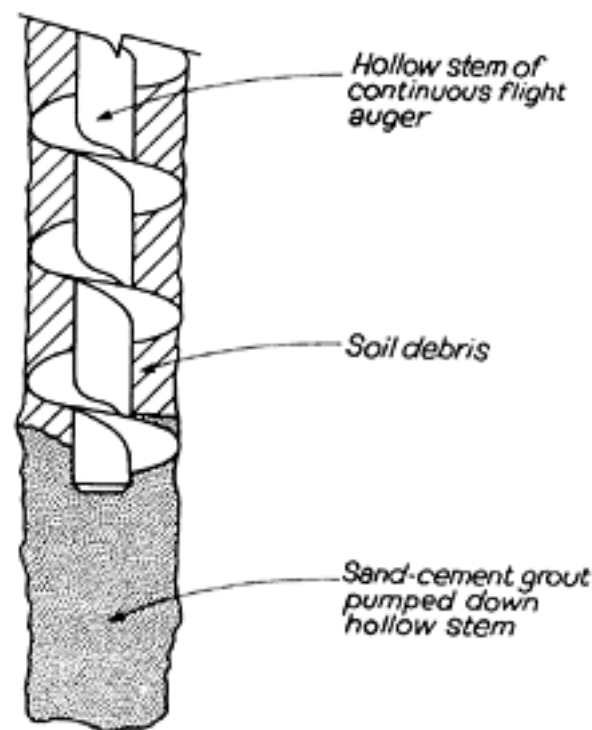


Fig. 2.33 Pumping grout to form an auger-injected pile

resulting pile having a cylindrical shaft. Reinforcing steel can be pushed into the fluid mortar to a depth of about 12m. Exceptionally, reinforcing cages up to 17m long were pushed down into the 30m long piles for the foundations of the approach viaducts to the Dartford Bridge. In some cases vibrators were used to assist penetration. With both types of pile the cement-sand mortar or concrete is mixed with a plasticizer to improve its 'pumpability', and an expanding agent is used to counter the shrinkage of the grout while it is setting and hardening. The shaft diameters range from the minipile sections (about 100mm) to 1m. Working loads are governed by code of practice relationships between permissible stress and compression strength of the sand-cement grout or fine concrete ([Table 2.11](#)).

In granular soils a hollow-stem auger can be used in conjunction with wing drill bits to mix the soil in place with a cement grout pumped down the stem. Continuous flight augers have been developed in Germany for drilling pile boreholes up to 1.5m in diameter.

The CFA pile has considerable advantages over the conventional bored pile in water-bearing and unstable soils. Temporary casing is not needed, and the problems of concreting underwater are avoided. The drilling operations are very quiet and vibrations are very low making the method suitable for urban locations. However, in spite of these considerable advantages the CFA pile depends for its integrity and load bearing capacity, as much as any other in-situ type of pile, on strict control of workmanship. This is particularly required where a high proportion of the load is carried in end bearing. Whereas with conventional bored piles the quality of the soil or rock at the toe can be checked by examining drill cuttings or by probing, in the case of the CFA pile a suitably resistant layer can only be assumed to be present by the increase in torque on the drill stem, as perceived by the driller or measured by a torque meter, or by instrumentation to compare the revolution rate of the drill stem with the rate of penetration of the auger. The drill cuttings from toe level do not reach the surface until after concrete or mortar injection has been completed. There are also doubts as to whether or not the injected material has flowed-out to a sufficient extent to cover the whole drilled area at the pile toe. For this reason it is advisable either to assume a base diameter smaller than that of the shaft or to adopt a conservative value for the allowable end bearing pressure.

The CFA pile is best suited for ground conditions where the majority of the working load is carried by skin friction, and the ground is free from large cobbles and boulders. Care is also needed to adjust the rate of withdrawing the auger to the rate of injecting the mortar or concrete. If the auger is raised too quickly the surrounding soil can collapse into cavities formed around the auger stem. The rate of raising the auger can be controlled by instruments monitoring the pressure and quantity of injected material. Because of the problems described above it is desirable to specify that CFA piling rigs should be provided with instruments to measure concrete flow rates and the torque on the drill stem, or the revolution/ penetration rate of the auger.

Further information on equipment and installation of CFA piles is given in a paper by Derbyshire([2.11](#)), and the report of a discussion at the Institution of Civil Engineers([2.12](#)).

### 2.4.3 Drilled-in tubular piles

The essential feature of the drilled-in tubular pile is the use of a tube with a medium to thick wall, which is capable of being rotated into the ground to the desired level and is left permanently in the

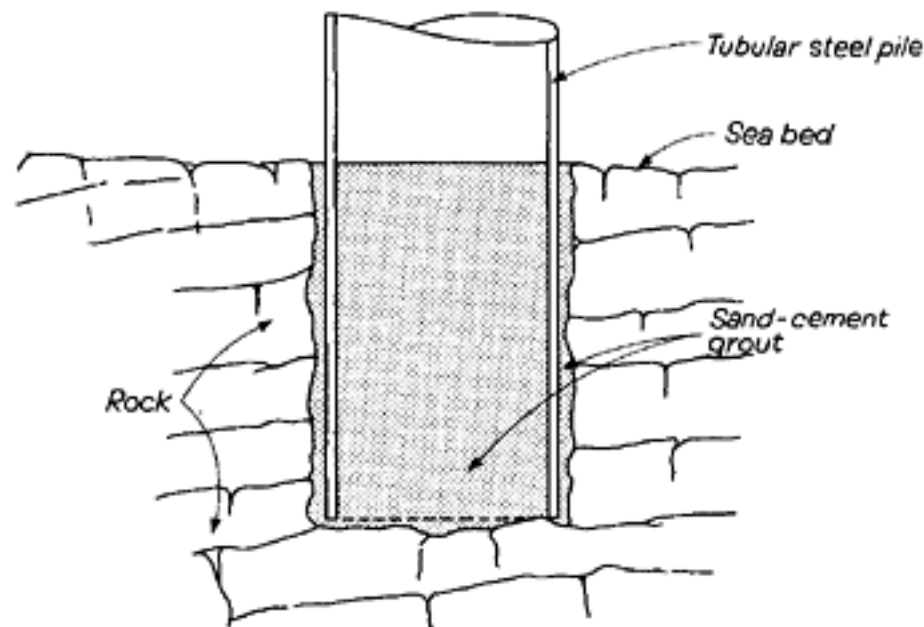
Table 2.11 Working stresses and maximum working loads in compression on bored-and-cast-in-place piles (including drilled-in tubes)

Country	Code	Tube	Structural core	Reinforcing steel	Concrete in shaft	Remarks									
United Kingdom	BS 8004	—	—	—	$0.25u_w$	Concrete to have a cement content not less than 300kg/m <sup>3</sup>									
USA	American Concrete Institute Recommendations (1974)	$0.35f_y$ 87N/mm <sup>2</sup> max	$0.50f_y$ 172N/mm <sup>2</sup> max	$0.40f_y$ 206N/mm <sup>2</sup> max	$0.33u_w$ (unconfined) $0.40u_w$ (confined)*	* Steel shell confining the concrete to be not greater than 432mm in diameter. Shell to be 14g (US) or thicker. Seamless or spiral welded; $f_y \nlessgtr$ : 206N/mm <sup>2</sup> . Not exposed to corrosion and does not carry part of working load. Corrugated steel shells not considered as load bearing									
	Chicago	—	—	—	$0.25u_w$	Where permanent lining tube is provided, maximum allowable stress is $0.3u_w + 1.5tf_y/D$ but not greater than $0.4u_w$ (where t is thickness and <i>D</i> is diameter of tube)									
	New York (1985)	$0.35f_y$ $\nlessgtr$ 248N/mm <sup>2</sup>	$0.50f_y$ $\nlessgtr$ 248N/mm <sup>2</sup>	$0.40f_y$ $\nlessgtr$ 206N/mm <sup>2</sup>	$0.25u_w$	Min thickness of tube to be 3mm before it can contribute to structure strength of pile. Max working load not to exceed 1 500kN for piles bearing on medium to hard rock, 800kN for bearing on soft rock and 1000kN for bearing on hardpan over rock. Max loads can be exceeded if substantiated by load tests. No upper limit for caisson piles with structural steel core. Uncased piles permitted only when borehole can be kept free of water during placement of concrete and sides and bottom can be inspected before placement									
Germany	DIN 4014					Min cement content 350kg/m <sup>3</sup> (400kg/m <sup>3</sup> for concrete placed under water). Min 5 reinforcing bars 14mm diameter. Longitudinal steel to be 0.8% of gross cross-sectional area. Cover 30mm (50mm in aggressive ground)  Max load on piles <table><tr><th></th><th>Without enlarged base</th><th>With enlarged base</th></tr><tr><td>Diameter (mm)</td><td>Max load (kN)</td><td>Size of base (mm)</td></tr><tr><td>300</td><td>200</td><td>600</td></tr></table>		Without enlarged base	With enlarged base	Diameter (mm)	Max load (kN)	Size of base (mm)	300	200	600
	Without enlarged base	With enlarged base													
Diameter (mm)	Max load (kN)	Size of base (mm)													
300	200	600													

						350      250      700 380 400      300      800 470 500      400      900 550 1000      650
France	DTU 13.2 (1978)	—	—	Min 5 bars not less than 12mm. Steel to be not less than 0.5% cross-section. Helical reinforcement spaced at not greater than 200mm pitch	5.5N/mm <sup>2</sup> 6N/mm <sup>2</sup> 5N/mm <sup>2</sup> 5.5N/mm <sup>2</sup> 5N/mm <sup>2</sup>	Concrete placed in dry hole Concrete placed in dry hole, casing extracted by vibrator Concrete placed underwater by tremie pipe Concrete placed underwater by tremie pipe, casing extracted by vibrator Concrete placed under bentonite
Sweden	SBS-S26:3 (1968)	—	—	—	—	Concrete to have minimum crushing strength of 39N/mm <sup>2</sup>

ground with or without an in-filling of concrete. Soil is removed from within the tube as it is rotated down, by various methods including grabbing, augering, and reverse circulation, as described in 3.3.4. The tube can be continuously rotated by a hydraulically-powered rotary table or be given a semi-rotary motion by means of a casing oscillator.

The drilled-in tubular pile is a useful method for penetrating ground containing boulders or other massive obstructions, heavy chisels being used to aid drilling. It is also used for founding in hard formations, where a 'rock socket' capable of resisting uplift and lateral forces can be obtained by drilling the tubes into the rock. In this respect the drilled-in tubular pile is a good type for forming berthing structures for large ships. These structures have to withstand high lateral and uplift loads for which a thick-walled tube is advantageous. In rock formations the resistance to these loads is provided by injecting a cement grout to fill the annulus between the outside of the tube and the rock forming the socket ([Figure 2.34](#)).



*Fig. 2.34 Drilled-in steel tubular pile with grouted annulus*

In the USA steel H-sections are lowered inside the drilled-in tubes and concrete is placed within the tubes to develop full end bearing on the pile and to ensure full interaction between tube, H-section 'core' and concrete. Because of the area of steel provided by the combined steel and concrete sections, very high loads can be carried by these 'caisson' piles where they are end bearing on a hard rock formation. Code of practice requirements for these and other forms of drilled-in tubular piles are included in [Table 2.11](#).

Steel tubular piles can be used for underpinning work by jacking them into the ground and cleaning out the soil as described in 9.2.2.

## 2.5 Composite piles

Various combinations of materials in driven piles or combinations of bored piles with driven piles, can be used to overcome problems resulting from particular site or ground conditions. The problem of the decay of timber piles above ground-water level has been mentioned in [Section 2.2.1](#). This can be overcome by driving a composite pile consisting of a precast concrete upper section in the zone above the lowest predicted ground-water level, which is joined to a lower timber section by a sleeved joint of the type shown in [Figure 2.3](#). The same method can be used to form piles of greater length than can be obtained using locally available timbers.

Alternatively a cased borehole may be drilled to below water level, a timber pile pitched in the casing and driven to the required depth, and the borehole then filled with concrete. Another variation of the precast concrete-timber composite pile consists of driving a hollow cylindrical precast pile to below water level, followed by cleaning out the soil and driving a timber pile down the interior.

In marine structures a composite pile can be driven that consists of a precast concrete upper section in the zone subject to the corrosive influence of sea-water and a steel H-pile below the soil line. The H-section can be driven deeply to develop the required uplift resistance from skin friction.

Generally, composite piles are not economical compared with those of uniform section, except as a means of increasing the use of timber piles in countries where this material is readily available. The joints between the different elements must be rigidly constructed to withstand bending and tensile stresses, and these joints add substantially to the cost of the pile. Where timber or steel piles are pitched and

driven at the bottom of drilled-in tubes, the operation of removing the soil and obtaining a clean interior in which to place concrete is tedious and is liable to provoke argument as to the standard of cleanliness required.

## 2.6 Minipiles and micropiles

Minipiles are defined in CIRIA report PGI(2.10) as piles having a diameter of less than 300mm. Generally they range in shaft diameter from 50 to 300mm, with working loads in the range of 50 to 500kN. The term ‘micro-pile’ is given to those in the lower range of diameter. They can be installed by a variety of methods. Some of these are:

- (i) Driving small-diameter steel tubes followed by injection of grout with or without withdrawal of the tubes,
- (ii) Driving thin wall shells in steel or reinforced concrete which are filled with concrete and left in place,
- (iii) Drilling holes by rotary auger, continuous flight auger, or percussion equipment followed by placing a reinforcing cage and in-situ concrete in a manner similar to conventional bored pile construction ([Section 2.4.2](#)).
- (iv) Lacking-down steel tubes, steel box-sections, or precast concrete sections. The sections may be jointed by sleeving or dowelling.

The principal use of minipiles is for installation in conditions of low headroom, such as underpinning work ([Section 9.2.2](#)), or for replacement of floors of buildings damaged by subsidence.

## 2.7 Factors governing choice of type of pile

The advantages and disadvantages of the various forms of pile described in 2.2 to 2.5 affect the choice of pile for any particular foundation project and these are summarized as follows:

### Driven displacement piles

#### *Advantages*

1. Material forming pile can be inspected for quality and soundness before driving.
2. Not liable to ‘squeezing’ or ‘necking’.
3. Construction operations not affected by ground water.
4. Projection above ground level advantageous to marine structures.
5. Can be driven in very long lengths.
6. Can be designed to withstand high bending and tensile stresses.

#### *Disadvantages*

1. Unjointed types cannot readily be varied in length to suit varying level of bearing stratum.
2. May break during driving, necessitating replacement piles.
3. May suffer unseen damage which reduces carrying capacity.
4. Uneconomical if cross-section is governed by stresses due to handling and driving rather than by compressive, tensile, or bending stresses caused by working conditions.
5. Noise and vibration due to driving may be unacceptable.
6. Displacement of soil during driving may lift adjacent piles or damage adjacent structures.
7. End enlargements, if provided, destroy or reduce skin friction over shaft length.
8. Cannot be driven in conditions of low headroom.

### Driven-and-cast-in-place displacement piles

#### *Advantages*

1. Length can easily be adjusted to suit varying level of bearing stratum.
2. Driving tube driven with closed end to exclude ground water.
3. Enlarged base possible.
4. Formation of enlarged base does not destroy or reduce shaft skin friction.
5. Material in pile not governed by handling or driving stresses.
6. Noise and vibration can be reduced in some types by driving with internal drop-hammer.

*Disadvantages*

1. Concrete in shaft liable to be defective in soft squeezing soils or in conditions of artesian water flow where withdrawable-tube types are used.
2. Concrete cannot be inspected after installation.
3. Length of some types limited by capacity of piling rig to pull out driving tube.
4. Displacement may damage fresh concrete in adjacent piles, or lift these piles, or damage adjacent structures.
5. Noise and vibration due to driving may be unacceptable.
6. Cannot be used in river or marine structures without special adaptation.
7. Cannot be driven with very large diameters.
8. End enlargements are of limited size in dense or very stiff soils.
9. When light steel sleeves are used in conjunction with withdrawable driving tube, skin friction on shaft will be destroyed or reduced.

**Bored-and-cast-in-place replacement piles***Advantages*

1. Length can readily be varied to suit variation in level of bearing stratum.
2. Soil or rock removed during boring can be inspected for comparison with site investigation data.
3. In-situ loading tests can be made in large-diameter pile boreholes, or penetration tests made in small boreholes.
4. Very large (up to 7.3m diameter) bases can be formed in favourable ground.
5. Drilling tools can break up boulders or other obstructions which cannot be penetrated by any form of displacement pile.
6. Material forming pile is not governed by handling or driving stresses.
7. Can be installed in very long lengths.
8. Can be installed without appreciable noise or vibration.
9. No ground heave.
10. Can be installed in conditions of low headroom.

*Disadvantages*

1. Concrete in shaft liable to squeezing or necking in soft soils where conventional types are used.
2. Special techniques needed for concreting in water-bearing soils.
3. Concrete cannot be inspected after installation.
4. Enlarged bases cannot be formed in cohesionless soils.
5. Cannot be extended above ground level without special adaptation.
6. Low end-bearing resistance in cohesionless soils due to loosening by conventional drilling operations.
7. Drilling a number of piles in group can cause loss of ground and settlement of adjacent structures.

**Choice of pile materials**

*Timber* is cheap relative to concrete or steel. It is light, easy to handle, and readily trimmed to the required length. It is very durable below ground-water level but is liable to decay above this level. In marine conditions softwoods and some hardwoods are attacked by wood-boring organisms. Timber piles are unsuitable for heavy working loads.

*Concrete* is adaptable for a wide range of pile types. It can be used in precast form in driven piles, or as insertion units in bored piles. Dense well-compacted good-quality concrete can withstand fairly hard driving and it is resistant to attack by aggressive substances in the soil, or in sea water or ground water. However, concrete in precast piles is liable to damage (possibly unseen) in hard driving conditions. Weak, honeycombed concrete in cast-in-place piles is liable to disintegration when aggressive substances are present in soils or in ground water.

*Steel* is more expensive than timber or concrete but this disadvantage may be outweighed by the ease of handling steel piles, by their ability to withstand hard driving, by their resilience and strength in bending, and their capability to carry heavy loads. Steel piles can be driven in very long lengths and cause little ground displacement. They are liable to corrosion above the soil line and in disturbed ground, and they require cathodic protection if a long life is desired in marine structures. Long steel piles of slender section may suffer damage by buckling if they deviate from their true alignment during driving.

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## CHAPTER 3

### Piling equipment and methods

There was a world-wide increase in the construction of heavy foundations in the period from 1950 to the 1970s as a result of developments in high office buildings, heavy industrial plants and shipyard facilities. The same period also brought the major developments of offshore oilfields. A high proportion of the heavy structures required for all such developments involved piled foundations, which brought about a great acceleration in the evolution of piling equipment. There were increases in the size and height of piling frames, in the weight and efficiency of hammers, and in the capacity of drilling machines to install piles of ever-increasing diameter and length. The development of higher-capacity machines of all types was accompanied by improvements in their mobility and speed of operation.

The development of piling equipment proceeded on different lines in various parts of the world, depending mainly on the influence of the local ground conditions. In Northern Europe the precast concrete pile continued to dominate the market and this led to the development of light and easily handled piling frames. These were used in conjunction with self-contained diesel hammers and winches, with the minimum of labour and without the need for auxiliary craneage, steam boilers or air compressors. The stiff clays of the mid-western states of America and the Great Lakes area of Canada favoured large-diameter bored piles, and mobile rotary drilling machines were developed for their installation. By contrast, the presence of hard rock at no great depth in the New York area favoured the continuing development of the relatively slender shell pile driven by an internal mandrel. The growth of the offshore oil industry in many parts of the world necessitated the development of an entirely new range of very heavy single-acting steam and hydraulically powered hammers designed for driving large-diameter steel piles guided by tubular-jacket structures. In the present day the increasing attention which is being given to noise abatement is influencing the design of pile hammers and the trend towards forms of pile that are installed by drilling methods rather than by hammering them into the ground.

Great Britain has a wide variety of soil types and the tendency has been to adopt a range of piling equipment selected from the best types developed in other parts of the world for their suitability for the soil conditions in any particular region. With the advances in the techniques of installing large-diameter bored piles, and the increasing popularity of these types for the foundations of heavy structures, it did appear at one stage that the capability of the bored pile to carry very heavy loads would outstrip that of the driven pile. However, with the stimulus provided by the construction of marine facilities for large tankers and deep-water oil-production platforms, the driven type of pile can now be installed in very large diameters that approach and in some cases exceed those of the larger bored piles.

The manufacturers of piling equipment and the range of types they produce are too numerous for all makes and sizes to be described in this chapter. The principal types of equipment in each category are described, but the reader should refer to manufacturers' handbooks for the full details of their dimensions and performance. The various items of equipment are usually capable of installing more than one of the many piling systems which are described in [Chapter 2](#). Installation methods of general application are described in the latter part of this chapter.

### 3.1 Equipment for driven piles

#### 3.1.1 Piling frames

The piling frame has the function of guiding the pile at its correct alignment from the stage of first pitching in position to its final penetration. It also carries the hammer and maintains it in position co-axially

with the pile. The essential parts of a piling frame are the *leaders* or *leads*, which are stiff members of solid, channel, box, or tubular section held by a lattice or tubular mast that is in turn supported at the base by a moveable carriage and the upper level by backstays. The latter can be adjusted in length by a telescopic screw device, or by hydraulic rams, to permit the leaders to be adjusted to a truly vertical position or to be raked forwards, backwards, or sideways. Where piling frames are mounted on elevated stagings, *extension leaders* can be bolted to the bottom of the main leaders in order to permit piles to be driven below the level of the base frame.

The *piling winch* is mounted on the base frame or carriage. This may be a double-drum winch with one rope for handling the hammer and one for lifting the pile. A three-drum winch with three sheaves at the head of the piling frame can lift the pile at two points using the outer sheaves, and the hammer by the central sheave. Some piling frames have multiple-drum winches which, in addition to lifting the pile and hammer, also carry out the duties of operating the travelling, slewing and raking gear on the rig.

The Menck tubular frame of German manufacture ([Figure 3.1](#)) has a single box-section leader to which the pile and hammer are held by bolted clamps. The Menck frame is made in five sizes with useful heights (under the hammer or helmet) ranging from 17.5m to 35m. These frames can drive at forward, backward and sideways rakes of 1 in 4, 1 in 1, and 1 in 6 respectively, and can carry hammers with a falling ram of up to 20 tonne in mass. Frames of this type are used most frequently for marine work where they are operated from stagings, jack-up platforms or pontoons.

The German Delmag piling frames ([Figures 3.2](#) and [3.3](#)) have tubular or latticed tubular leaders and stays with jaw attachments for the pile and hammer. They range in size from 7.5m to 18.4m of useful height. The largest size can lift a pile having a mass of 6 tonne.

In the USA the practice is to guide the pile between the leaders. The pile head is guided by a cap or helmet which has jaws on each side that engage with leaders. The hammer is similarly provided with jaws. The leaders are capable of adjustment in their relative positions to accommodate piles and hammers of various widths.

The Menck and Delmag frames shown in [Figures 3.1](#) to [3.3](#) are designed to be light and easy to erect but they rely on wheeled carriages for stability and movement across the site. British and American practice favours heavier crawler-mounted machines with their greater mobility on rough ground. The Swedish Akermanns rigs are also crawler-mounted. The Akermanns M14–5P ([Figure 3.4](#)) has a useful height of 21m. It has a box-section leader designed to operate with a 5-tonne drop hammer or a Delmag D22 diesel hammer. The leader can be adjusted by hydraulic rams to forward and backward rakes of 26° and 45° respectively.

The Junttan PM25 hydraulic piling rig, manufactured in Finland, has a box-section leader 22.5m high connected to the base crawler machine at the top by a pair of hydraulic arms and at the foot by a single hydraulic ram. These attachments enable the leader to be raked forwards and backwards by 18° (1:3) and 40° (1:1.2) respectively and lateral raking up to 12° (1:5). A hydraulic hammer of 10.1 tonne mass can be handled by this rig.

The Dawson DCP light tractor-mounted piling rig operates with a DCP/Krupp hydraulic hammer (see [Table 3.2](#)) or a vibratory hammer. The hydraulically-operated telescopic leaders are capable of raking and they can ‘crowd’ the pile with a force of 64kN. The rig is suitable for driving steel H-piles up to 12.5m long.

Piling frames can be designed for specific purposes. The American-designed ‘moon-beam’ frame ([Figure 3.5](#)) is used for driving raking piles in pairs for jetty structures. A flat angle of rake (1 in 2.4) is possible and the sideways rake prevents the piling frame from becoming locked to the pile on a rising tide, which can happen when driving from a pontoon-mounted rig with only a backward or forward rake.

### 3.1.2 Crane-supported (hanging) leaders

Although the complete piling rig with its base frame and leaders supported by a stayed mast provides the best means of ensuring stability and control of the alignment of the pile, there are many conditions which favour the use of leaders suspended from a standard crawler crane. Rigs of this type have largely supplanted the frame-mounted leaders for driving long piles on land in the UK and USA. The usual practice is to link the leaders by the head of the crane jib and to control their verticality or backward or forward rake by means of adjustable stays near the foot of the leaders. The latter bear on the ground through an enlarged foot which can be levelled by a screw jack. BSP International Foundations Ltd. TL series leaders ([Figure 3.6](#)) have heights of 19.0m and 21.9m and carry hammers of up to 3 tonne mass.

The 610mm and 835mm square section lattice leaders have a height to the cathead of 22.5 and 38m respectively, and can carry combined pile and hammer loads of 13 tonne and 21 tonne respectively.

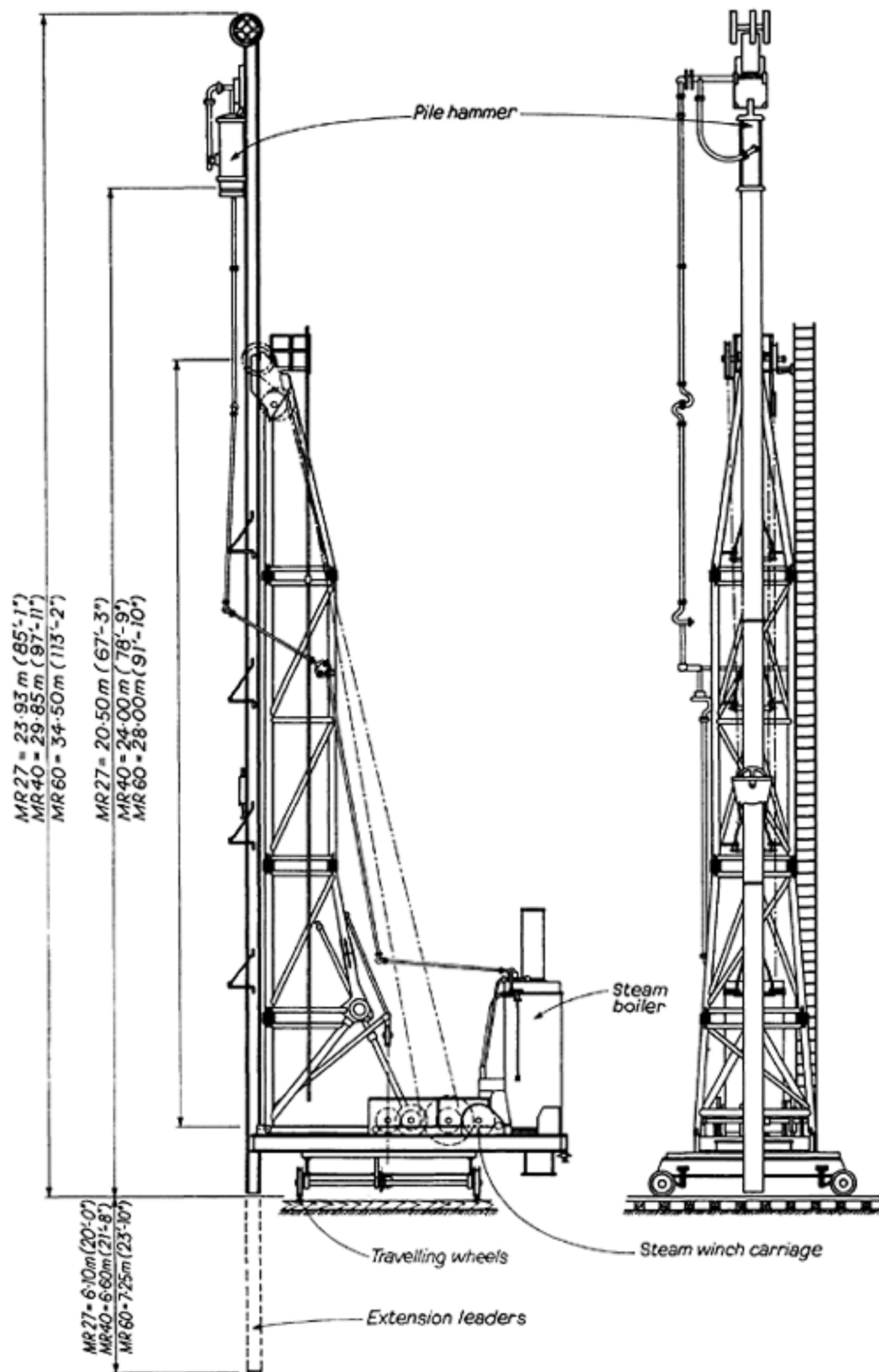


Fig. 3.1 Menck tubular piling frame

Backward and forward rakes of up to 1:3 are possible depending on the stability of the crawler crane.

There is a practical limit to the length of pile which can be driven by a given type of rig and this can sometimes cause problems when operating the rig in the conventional manner without the assistance of a separate crane to lift and pitch the pile. The conventional method consists of first dragging the pile in a horizontal position close to the piling rig. The hammer is already attached to the leader and drawn up to the cathead. The pile is then lifted into the leaders using a line from the cathead and secured by toggle bolts. The helmet, dolly and packing are then placed on the pile head ([Figure 3.21](#)) and the assembly is drawn up to the underside of the hammer. The carriage of the piling rig is then slewed round to bring the pile over to the intended position and the stay and angle of the crane jib are adjusted

to correct for vertically or to bring the pile to the intended rake. The problem is concerned with the available height beneath the hammer when it is initially drawn up to the cathead. Taking the example of leaders with a usable height of 20.5m in conjunction with a hammer with an overall length of 6.4m, after allowing a clearance of 1m between the lifting lug on the hammer to the cathead and about 0.4m for the pile helmet, the maximum length of pile which can be lifted into the leaders is about 12.7m. A somewhat longer pile could be handled if the leaders were of a type which allows vertical adjustment. Occasionally it may be advantageous to use leaders independent of any base machine. Thus if only two or three piles are to be driven, say as test piles before the main contract, the leaders can be guyed to ground anchors and operated in conjunction with a separate petrol or diesel winch. Guyed leaders are slow to erect and move, and they are thus not used where many piles are to be driven, except perhaps in the confines of a narrow trench bottom where a normal rig could not operate.

### 3.1.3 Trestle guides

Another method of supporting a pile during driving is to use guides in the form of a moveable trestle. The pile is held at two points, known as 'gates', and the trestle is designed to be moved from one pile or pile-group position to the next by crane (Figure 3.7). The hammer is supported only by the pile and is held in alignment with it by leg guides on the hammer extending over the upper part of the pile shaft. Because of flexure of the pile during driving there is a greater risk, especially with raking

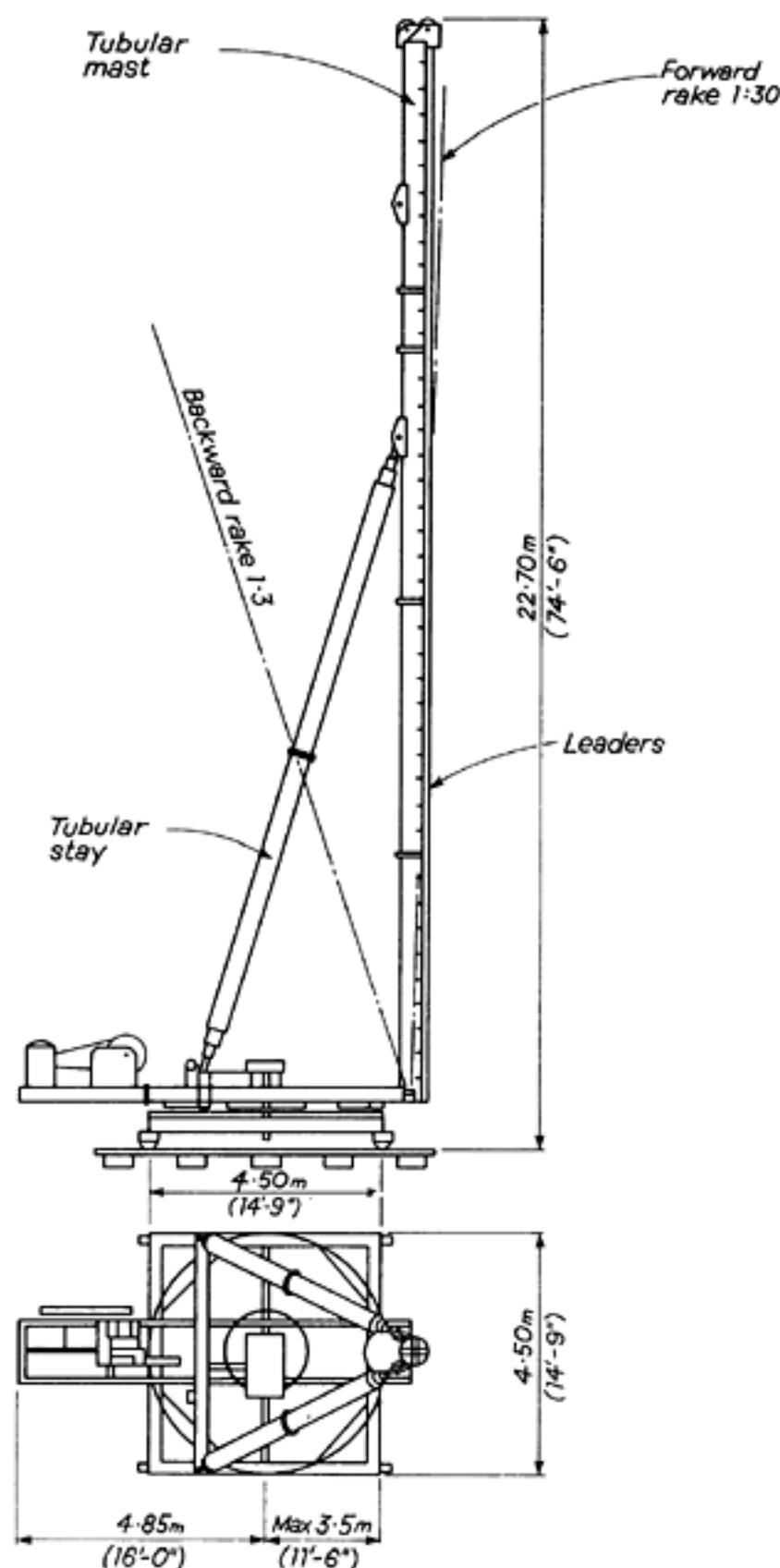


Fig. 3.2 Delmag GF.22 piling frame

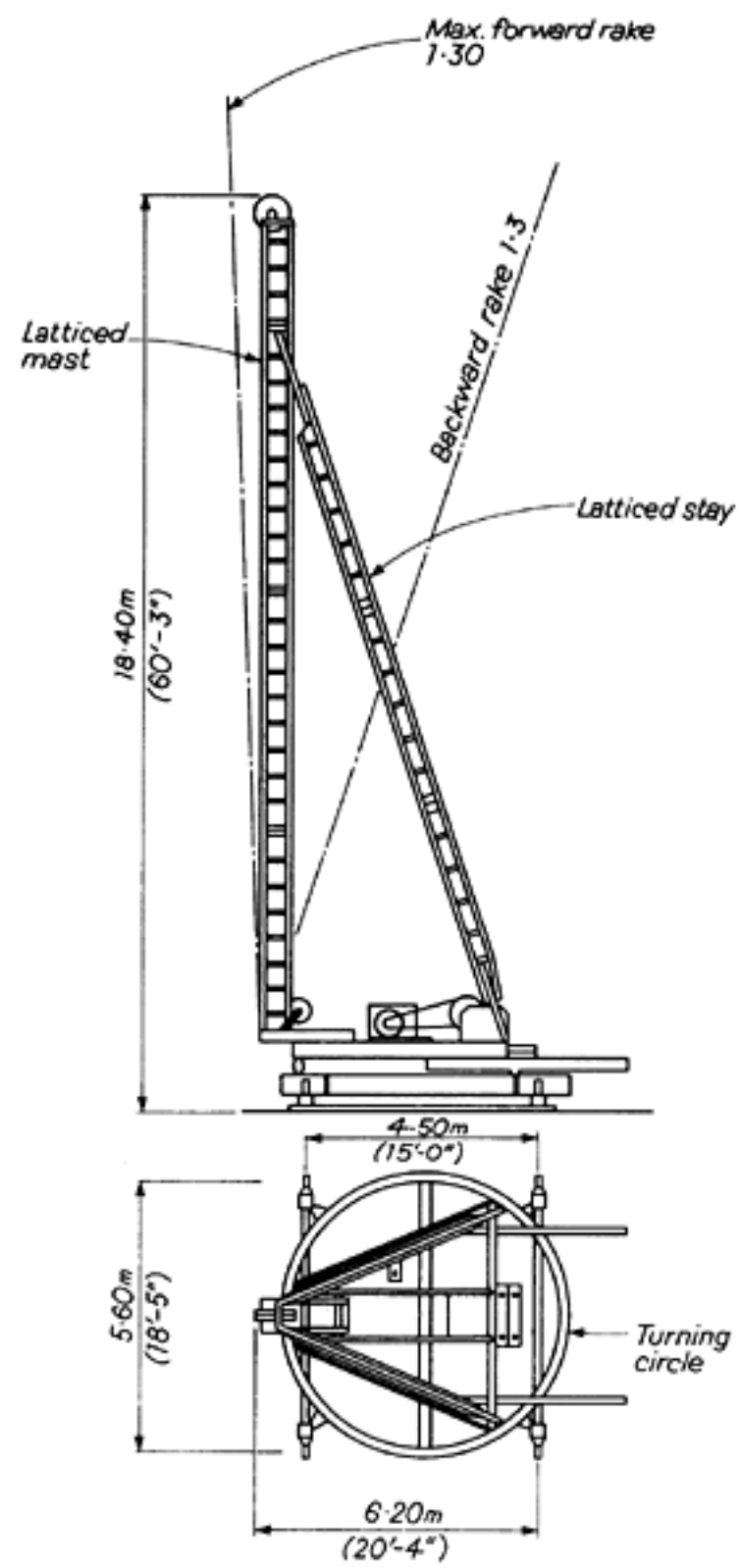


Fig. 3.3 Delmag G.17 piling frame

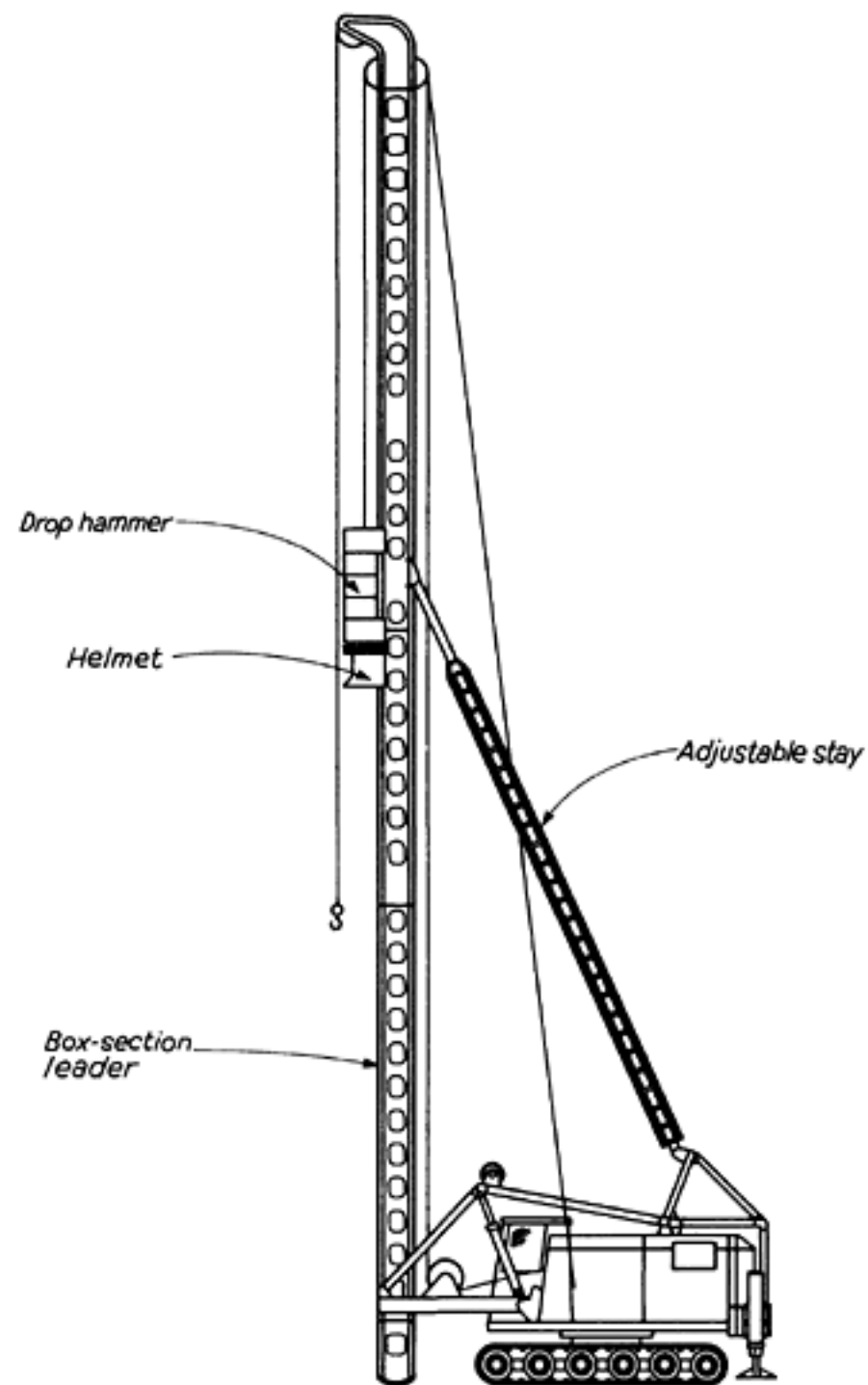


Fig. 3.4 Akermanns M.14-5P piling frame

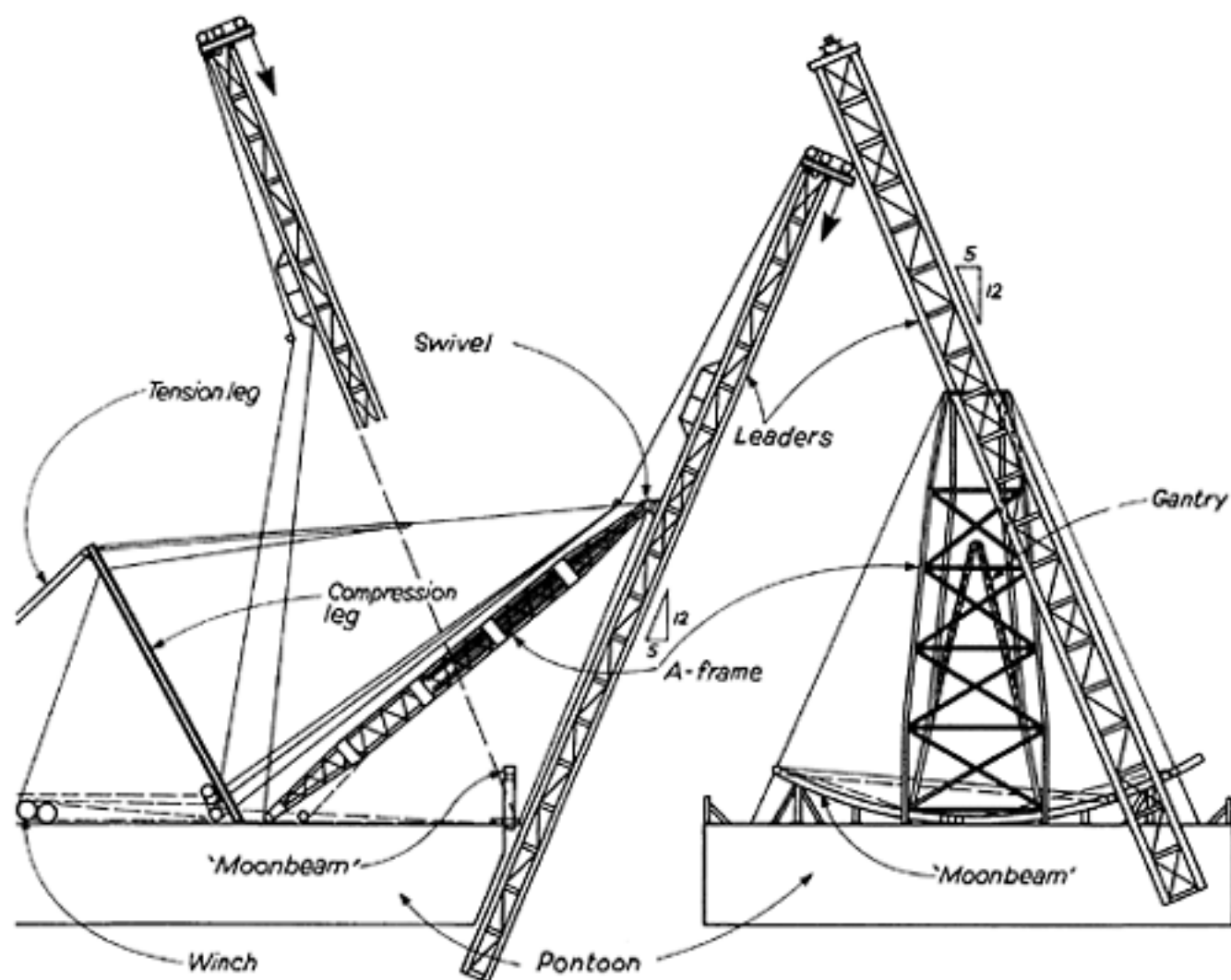
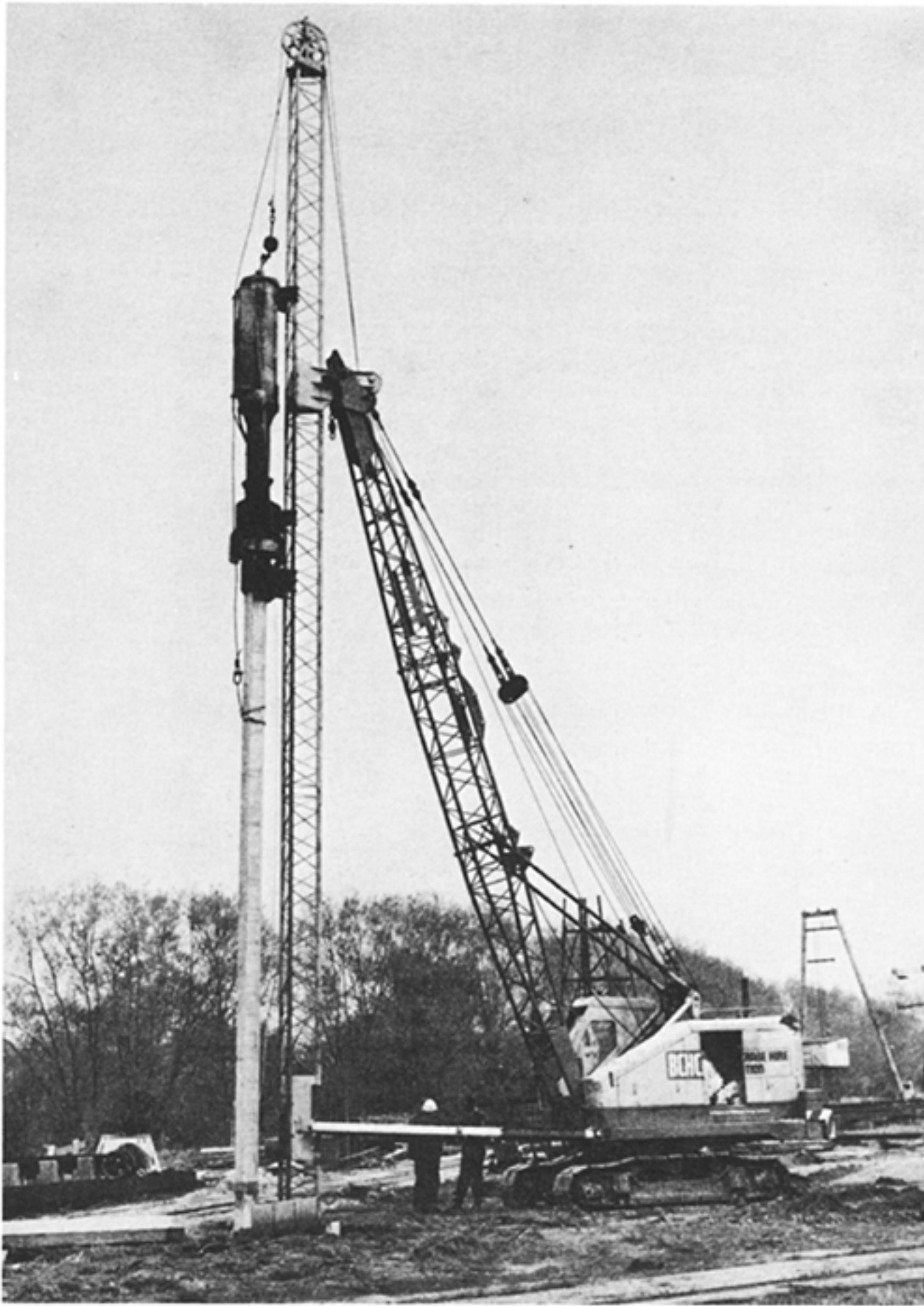


Fig. 3.5 'Moon-beam' piling frame



*Fig. 3.6 BSP International Foundations Ltd. TL type triangular hanging pile leaders (shown with BSP V.15 double-acting diesel hammer)*

piles, of the hammer losing its alignment with the pile during driving than in the case of piling frames which support and guide the hammer independently of the pile. For this reason the method of supporting the hammer on the pile in conjunction with trestle guides is usually confined to steel piles where there is less risk of damage to the pile head by eccentric blows. When driving long steel raking piles in guides it is necessary to check that the driving stresses combined with the bending stress caused by the weight of the hammer on the pile are within allowable limits.

Pile guides which are adjustable in position and direction to within very close limits are manufactured in Germany. Their principal use is for mounting on jack-up barges for marine piling operations. A travelling carriage or gantry is cantilevered from the side of the barge or spans between rail tracks on either side of the barge 'moon-pool'. The travelling gear is powered by electric motor and final positioning is by hydraulic rams. Hydraulically operated pile clamps or gates are mounted on the travelling carriage at two levels and are moved transversely by electric motor, again with final adjustment by

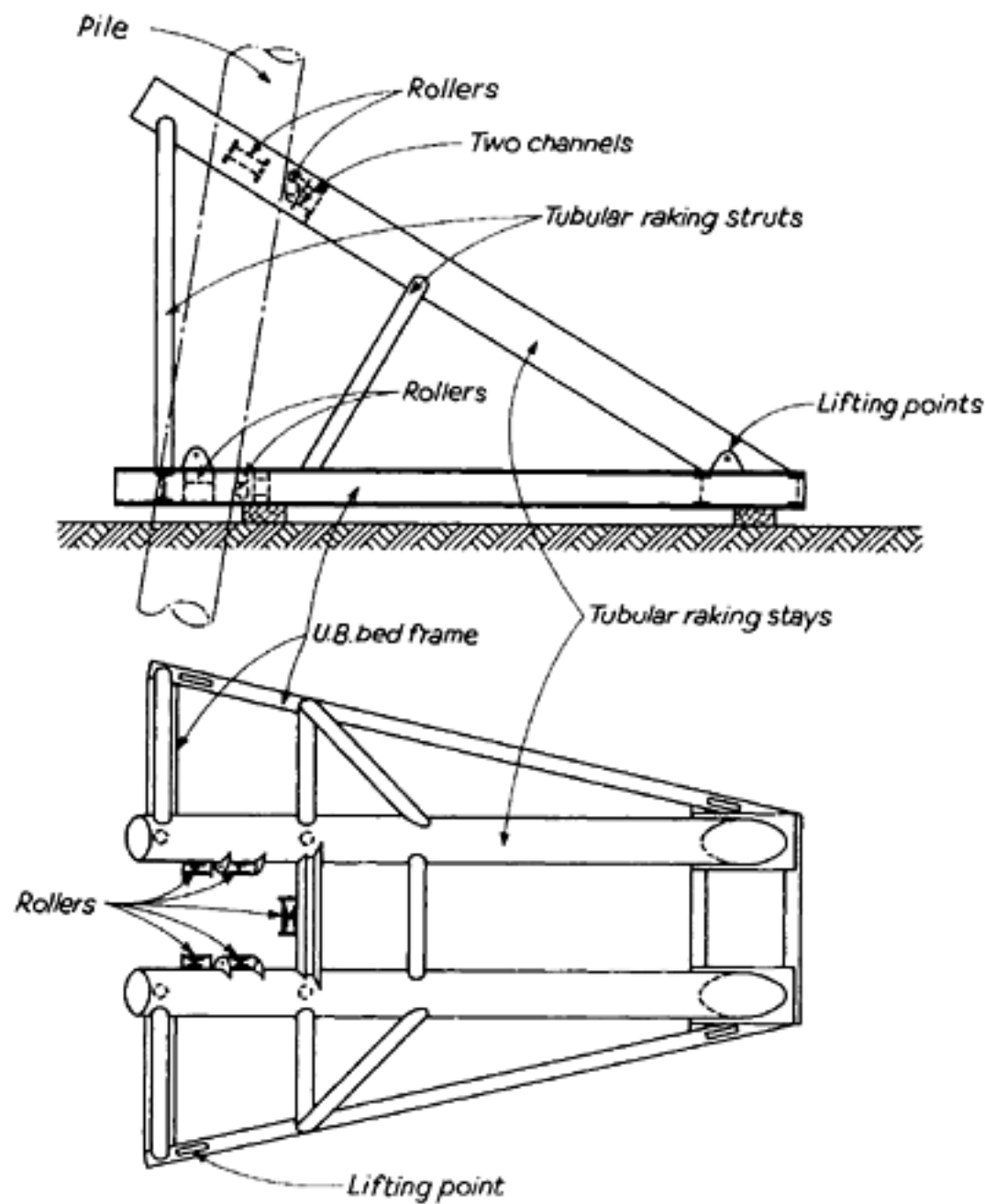


Fig. 3.7 Trestle guides for tubular raking pile

hydraulic rams allowing the piles to be guided either vertically or to raking positions. Guides provided by hydraulic clamps at 3m vertical intervals cantilevering from the side of a piling barge are shown in [Figure 3.8](#). The guide system was designed and operated by Seyzi Turkes-Feyzi Akkaya for driving 2.0m diameter steel tube piles for the foundations of the new Galata Bridge in Istanbul(3.1).

Trestle guides can be usefully employed for rows of piles that are driven at close centres simultaneously. The trestle shown in [Figure 3.9](#) was designed by George Wimpey and Co. for the wall foundations of Harland and Wolff's shipbuilding dock at Belfast(3.2). Three rows of five 356×368mm H-piles were pitched into the guides and were driven by a Delmag D22 hammer. Guides can be used in conjunction with piling frames for a two-stage driving operation, which may be required if the piles are too long to be accommodated by the available height of frame. Guides are used for the first stage of driving, the piles carrying the hammer which is placed and held by a crane. At this stage the pile is driven to a penetration that brings the head to the level from which it can be driven by the hammer suspended in the piling frame. The latter completes the second stage of driving to the final penetration ([Figure 3.10](#)).

### 3.1.4 Piling hammers

The simplest form of piling hammer is the *drop hammer*, which is guided by lugs or jaws sliding in the leaders and actuated by the lifting rope. The drop hammer consists of a solid mass or assemblies of forged steel, the total mass ranging from 1 tonne to 5 tonne. The striking speed is slower than in the case of single- or double-acting hammers, and when drop hammers are used to drive concrete piles there is a risk of damage to the pile if an excessively high drop of the hammer is adopted when the driving becomes difficult. There has been a revival of interest in the simple drop hammer because of its facility to be operated inside a sound-proofed box, so complying with noise abatement regulations (see [3.1.7](#)). Drop hammers are not used efficiently when operated from a pontoon-mounted piling frame working in open waters, since the height of the drop cannot be controlled when the pontoon is rising and falling on the waves. However, they can be used effectively in sheltered waters. The American Vulcan hammer, which has been designed to operate within the leaders, is shown in [Figure 3.11](#). The



*Fig. 3.8 Barge-mounted pile guides with hydraulically-operated clamps*

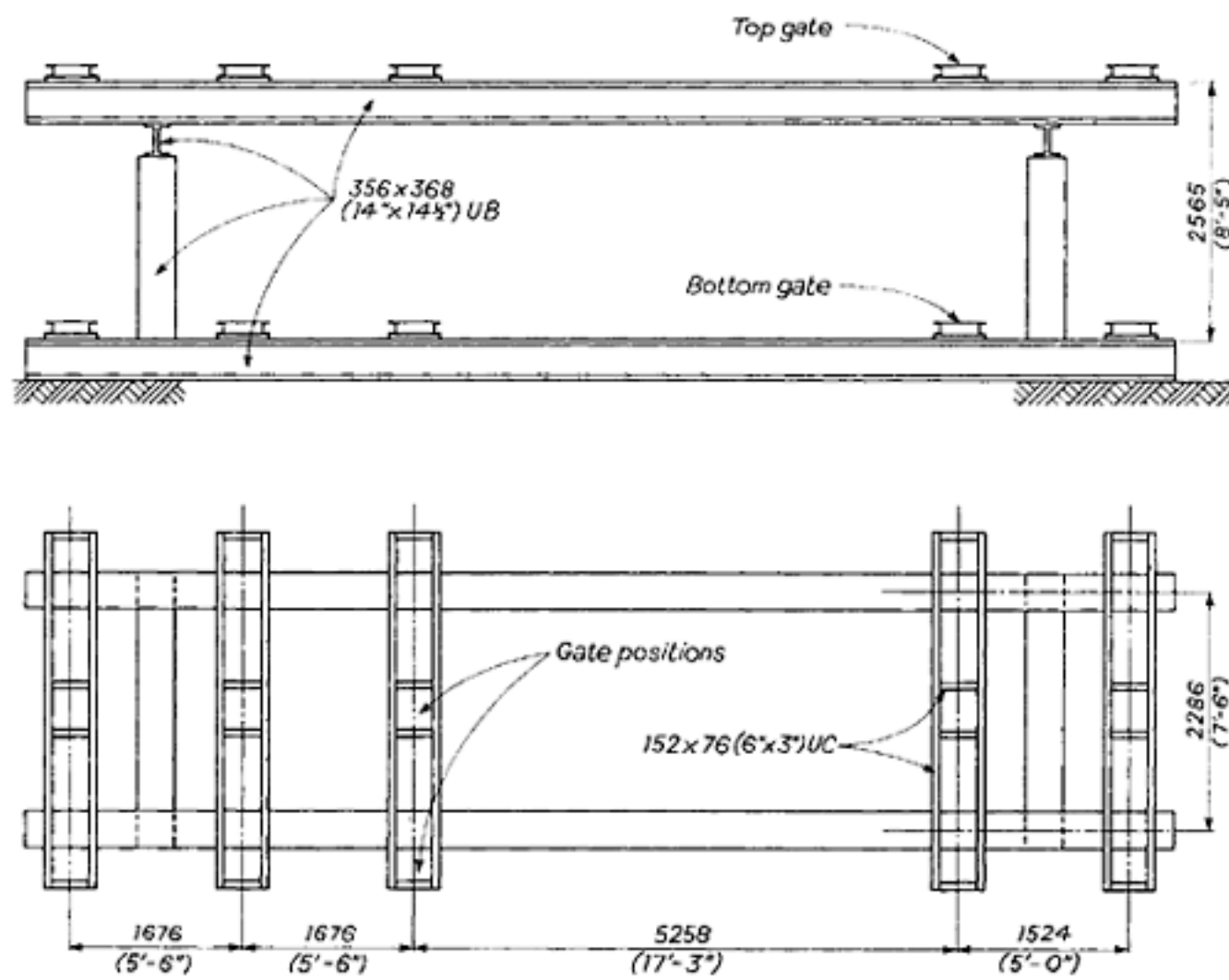


Fig. 3.9 Trestle guides for multiple vertical piles

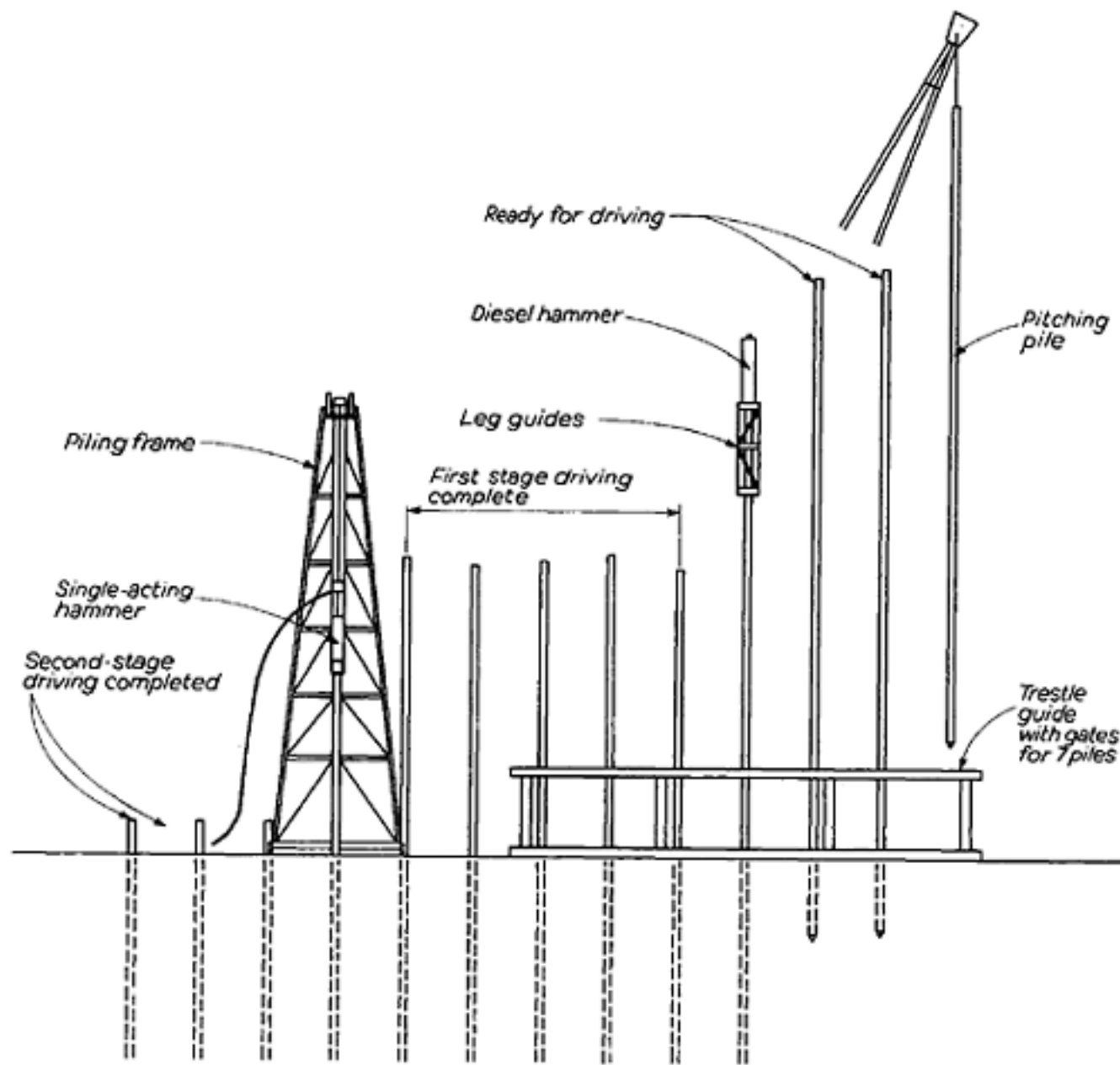


Fig. 3.10 Driving piles in stages in conjunction with trestle guides



Fig. 3.11 Vulcan drop hammer

Swedish Akermanns hammer ([Figure 3.12](#)) consists of an assembly of steel blocks, each with a mass of 1 tonne. Up to five blocks can be used to give a hammer of 5 tonne mass, with jaws to enable the assembly to operate in front of the Akermann box-section leader.

The *single-acting hammer* is operated by steam or compressed air, which lifts the ram and then allows it to fall by gravity. BSP single-acting hammers of the type shown in [Figure 3.13](#) range in mass from 2½ tonne to 15 tonne with a maximum height of fall of 1.37m. The single-acting hammer is best suited to driving timber or precast concrete piles, since the drop of each blow of the hammer is limited in height and is individually controlled by the operator. BSP International Foundations Ltd. has developed a solenoid system for controlling the drop of a single-acting hammer with the objective of accurate control with an infinitely variable stroke and the elimination of operator fatigue, the operator normally controlling the drop manually by a rope from the inlet valve. The single-acting hammer is also suitable for driving all types of pile in stiff to hard clays, where a heavy blow with a small drop is more efficient and less damaging to the pile than a large number of lighter blows. The steam or air supply for both single-acting and double-acting hammers should be at least 125% of the nominal consumption stated by the hammer manufacturer. The characteristics of the various types of single-acting hammer are shown in [Table 3.1](#).

The ram of *hydraulic hammers* is raised by hydraulic fluid under high pressure to a predetermined height, and then allowed to fall under gravity or is forced down onto the pile head. The BSP hydraulic hammer is shown in [Figure 3.14](#). A hydraulic actuator is activated by a solenoid-operated control valve which raises the piston rod. At the required stroke height the flow of the hydraulic fluid is cut off. Pressures within the actuator then equalize allowing the ram to decelerate as it approaches the top of its stroke. The hammer then falls freely under gravity and repositions the piston rod for the next stroke.

The basic ram weight is 3 tonne, and further 2 tonne segments can be added up to a total of 9 tonne. The drop height of the ram is between 0.2m and 1.2m with very close control of any specified height between these limits. The striking rate can be controlled manually or is automatic. In the latter case the striking rate is normally at 40 blows/minute at 1.2m drop.

BSP make heavier hydraulic hammers to special order and have manufactured one with a ram weight of 40 tonne ([Figure 3.15](#)). The characteristics of various makes of hydraulic hammer are listed in Table

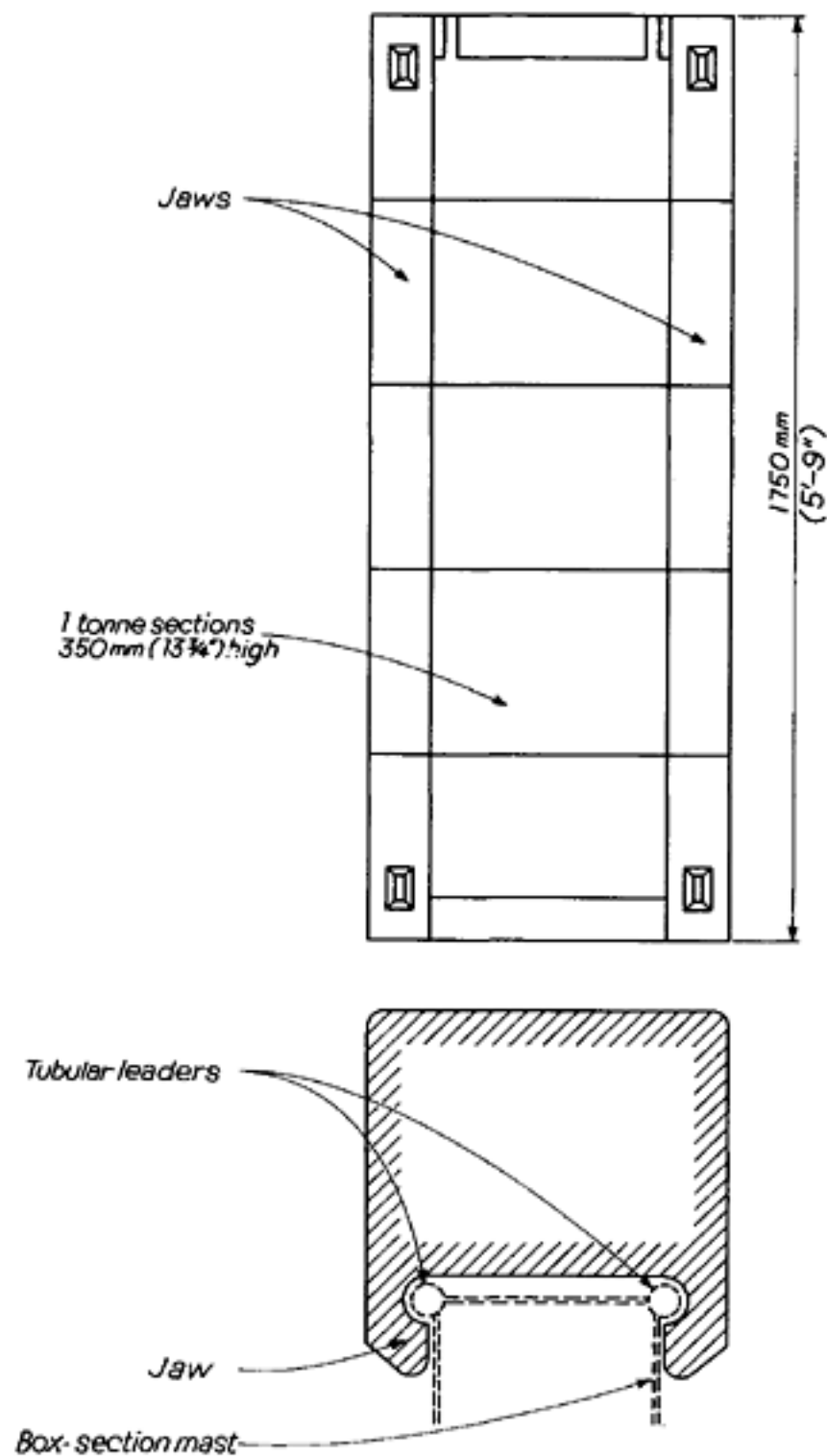


Fig. 3.12 Akermanns drop hammer

[3.2.](#) Generally these hammers have the advantage of being able to operate underwater, and because there is no exhaust they can be operated inside a soundproof box.

*Underwater hydraulic hammers* were developed specially for driving piles in deep water locations. The range of hammers manufactured by the Menck company is shown in [Table 3.2](#). The MHU 400 T hammer has driven a pile in 1000m depth of water. The MHU 3000 T with a ram weight of 180 tonne is the largest piling hammer ever constructed. The MHU hammers are designed to operate either as free-riding units mounted on the pile with a slack lifting line, or to reduce weight on the guides they can be suspended from the floating crane with a heave compensator to maintain constant tension in the lifting line. The power pack can be installed on the crane barge or platform or can be mounted on the hammer. Slender hammers can operate inside the pile or with a follower attached to the pile.

The '*Hydroblok*' hammer is a special type of hydraulic hammer developed by the Hollandsche Beton Group. It consists of a drop weight enclosed by a casing. The drop weight is in the form of a hollow cylinder incorporating a piston and an impact head, and nitrogen under high pressure forms a buffer between these two components. The drop weight is driven down hydraulically at a high striking rate, the driving force on the pile head being equal to the pressure of the nitrogen multiplied by the area of the impact head. The driving force can be regulated to suit the expected ground resistance. The nitrogen forming the buffer, cushions and sustains the blow on the pile head, thus preventing damage from high impact forces, and eliminating a tension wave in the pile (see [Section 7.3](#)).

*Double-acting (or differential-acting)* hammers are steam or air operated both on the upstroke and downstroke, and are designed to impart a rapid succession of small-stroke blows to the pile. The double-acting hammer exhausts the steam or air on both the up and down strokes. In the case of the differential-acting hammer, however, the cylinder is under equal pressure above and below the piston and is exhausted only on the upward stroke. The downward force is a combination of the weight of the ram and the difference in total force above and below the piston, the force being less below the piston because

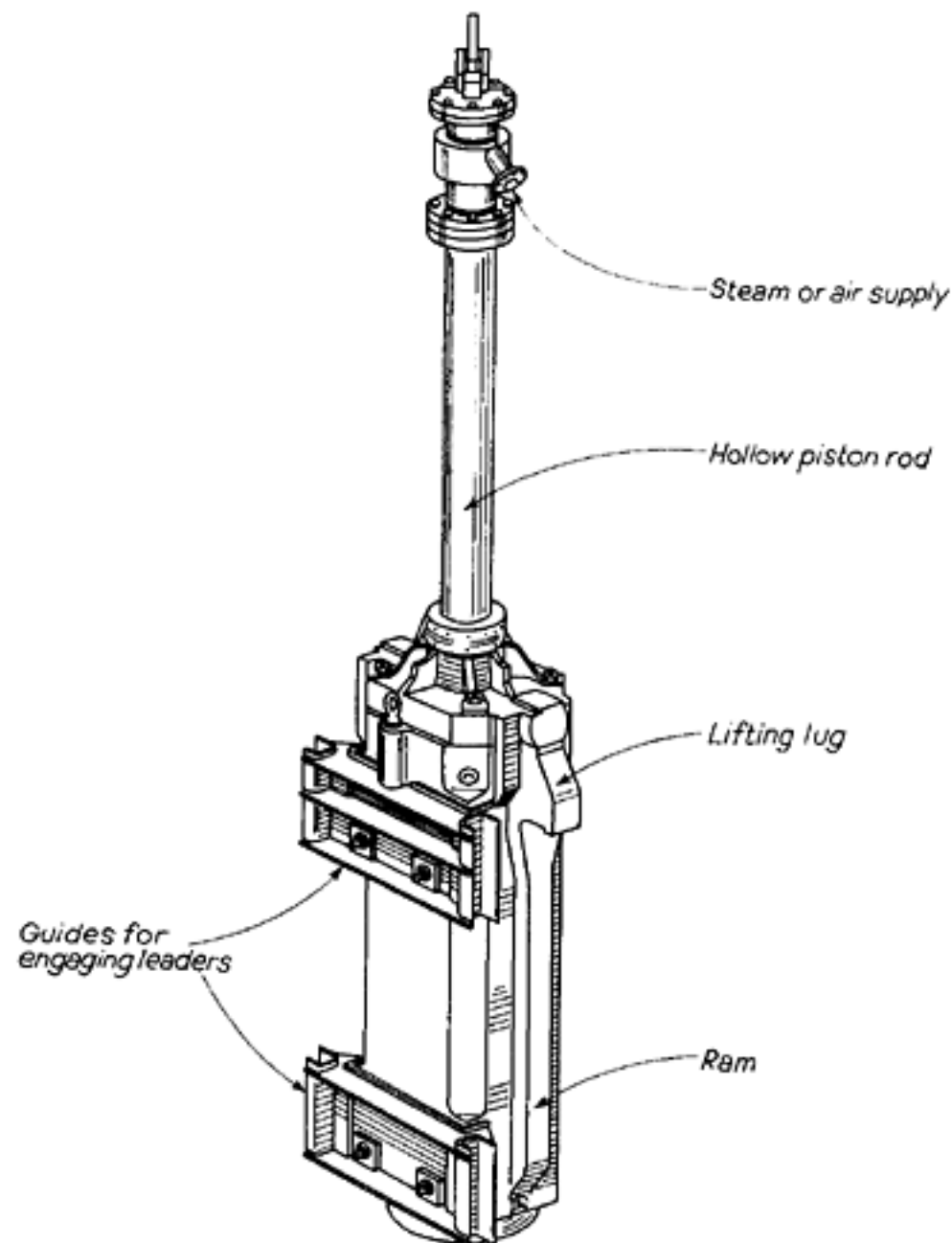


Fig. 3.13 BSP International Foundations Ltd. single-acting piling hammer

of the area occupied by the piston rod. These hammers are most effective in granular soils where they keep the ground ‘live’ and shake the pile into the ground, but they are not so effective in clays. Double-acting hammers have their main use in driving sheet piles and are not used for bearing piles in preference to diesel hammers. However, unlike the diesel hammer they can operate under water. The characteristics of the various makes are shown in [Table 3.3](#).

*Diesel hammers* are suitable for all types of ground except soft clays. They have the advantage of being self-contained without the need for separate power-packs, air compressors or steam-generators. They work most efficiently when driving into stiff to hard clays, and with their high striking rate and high energy per blow they are favoured for driving all types of bearing piles up to about 2.5m in diameter. The principle of the diesel hammer is that as the falling ram compresses air in the cylinder, diesel fuel is injected into the cylinder and this is atomized by the impact of the ram on the concave base. The impact ignites the fuel and the resulting explosion imparts an additional ‘kick’ to the pile, which is already moving downwards under the blow of the ram. Thus the blow is sustained and imparts energy over a longer period than the simple blow of a drop or single-acting hammer. The ram rebounds after the explosion and scavenges the burnt gases from the cylinder. The well-known Delmag hammer is shown in [Figure 3.16](#). BSP International Foundations Ltd. and Koehring-MKT manufacture double-acting diesel hammers with a striking rate of 80 to 100 blows/min compared with the rates of 40 to 60 blows/min attained by the comparable makes of single-acting diesel hammers. The characteristics of the various makes of diesel hammer are shown in [Table 3.4](#).

A difficulty arises in using the diesel hammer in soft clays or weak fills, since the pile yields to the blow of the ram and the impact is not always sufficient to atomize the fuel. The more resistant the ground, the higher the rebound of the ram, and hence the higher the energy of the blow. This can cause damage to precast concrete piles when driving through weak rocks containing strong bands. Although the height of drop can be controlled by adjusting by a rope-operated lever the amount of fuel injected, this control cannot cope with random hard layers met at varying depths, particularly when these are unexpected. The diesel hammer operates automatically and continuously at a given height of drop unless the lever is adjusted, whereas with the single-acting hammer every blow is controlled in height.



*Fig. 3.14 BSP International Foundations Ltd. hydraulic piling hammer*

### 3.1.5 Piling vibrators

Vibrators consisting of pairs of exciters rotating in opposite directions can be mounted on piles when their combined weight and vibrating energy cause the pile to sink down into the soil ([Figure 3.17](#)). Vibratory hammers operate most effectively when driving small displacement piles (H-sections or open-ended steel tubes), into loose to medium dense granular soils. Ideally a pile should be vibrated at or near to its natural frequency, which requires 100Hz for a 25m steel pile. Thus only the high-frequency vibrators are really effective for long piles([3.3](#)). Most types of vibrators operate in the low-frequency to medium-frequency range (i.e. 10 to 39Hz).

Vibrators are not very effective in firm clays and cannot drive piles deeply into stiff clays. They are frequently used in bored pile construction for sealing the borehole casing into a cohesive soil after predrilling through the granular overburden soils. After concreting the pile the vibrators are used to extract the casings and are quite efficient for this purpose in all soil types (see [3.4](#)).

Vibrators have an advantage over impact hammers in that the noise and shock wave of the hammer striking the anvil is eliminated. They also cause less damage to the pile and have a very fast rate of penetration in favourable ground. It is claimed that a rate of driving averaging 18m per minute may

Table 3.1 Characteristics of single-acting piling hammers\*

Maker	Type	Mass of ram (kg)	Maximum energy per blow (m/kg)	Maximum striking rate (blows/min)
BSP International Foundations Limited (United Kingdom)	—	2 540	3 425	50
	—	3 050	4 110	50
	—	4 060	5 480	50
	—	5 080	6 850	50
	—	6 110	8 220	50
	—	8 130	10 960	50
	—	10 160	13 700	50
	—	12 190	16 440	50
	—	15 240	20 550	50
	—	20 000	27 400	50
Koehring-MKT (United States of America)	S10	4 500	4 500	55
	S14	6 350	5 100	60
	S20	9 000	8 300	60
	OS40	18 100	16 600	55
	OS60	27 200	24 900	55
Menck (Germany)	MRBS850	8 600	12 900	45
	MRBS1 100	11 400	17 000	40
	MRBS1 800	17 500	26 300	44
	MRBS3 000	30 000	45 000	42
	MRBS3 900	39 400	70 900	36
	MRBS5 000	50 000	75 000	40
	MRBS6 000	60 000	105 000	34
	MRBS8 800	88 000	132 000	36
	MRBS12 500	125 000	219 000	36
Vulcan (United States of America)	2	1 360	1 000	70
	1	2 270	2 070	60
	06	2 950	2 690	60
	08	3 630	3 590	50
	010	4 540	4 490	50
	014	6 360	5 810	60
	016	7 260	6 740	60
	020	9 070	8 300	60
	030	13 610	12 440	55
	040	18 100	16 600	60
	060	27 200	24 890	62
	540	22 200	27 600	48
	560	23 800	41 460	45
	5 100	45 400	69 100	48
	5 250	113 400	172 750	38
	6 300	136 100	248 800	42

\*Note that the information given in in [Tables 3.1](#) to 3.5 does not necessarily represent the full range of equipment by each maker. The makers listed in these Tables should be contacted for full details.

be achieved in loose to medium-dense granular soils. If the electric generator used to power the exciter motors is mounted in a sound-proofed hut the vibrators can be used in urban areas with far lower risk of complaints arising due to noise and shock-

wave disturbance than when impact hammers are used. However, after starting the hammer the vibrations, as they increase to the operating frequency, may be in resonance with the natural frequency of nearby buildings. This can cause a short period of high amplitude vibrations which are quite alarming to the occupants. The same effect occurs when shutting down the hammer. Because of the limitation of the soil types in which they can be used, the complexity of the machinery, and its maintenance, vibrators are not used to the same extent as impact hammers for driving bearing piles. Vibrators are used to displace weak soils followed by placing crushed stone to form compacted columns in the ground. Used in this way the ‘Vibro-replacement’ process claims to strengthen the weak soils by compaction or consolidation. In the UK Keller Limited have extended the process whereby cement is added to the stone to provide a form of ‘pile’ capable of carrying light loading when taken down to a competent stratum.

Types of vibrators suitable for driving bearing piles are shown in [Table 3.5](#).

Table 3.2 Characteristics of hydraulic hammers

Maker	Type	Mass of ram (kg)	Maximum energy per blow (kN.m)	Striking rate at maximum stroke height (blows/min)
BSP International Foundations Limited* (United Kingdom)	HH3	3 000	36	46
	HH5	5 000	60	40
	HH7	7 000	84	31
	HH9	9 000	108	30
	HH14	11 000–16 000	132–192	38–30
	HH40	40 000	480	40
Banut* (Sweden)	—	3 000	15	30
	—	4 000	32	28
	—	5 000	40	24
	—	6 000	48	20
Dawson Construction Plant (United Kingdom)	DCP	2 000	4.5	800
Hollandsche Beton Group (Netherlands)	HBM500	4 300	100	40
	HBM850	10 000	180	40
	HBM1 500	20 000	300	40
	HBM3 000	63 000	1 000	40
IHC Hydrohammerbr (Netherlands)	S35	3 300	35	60
	S70	3 500	70	50
	S90	4 500	80	50
	S200	10 000	200	45
	S280	13 500	280	45
	S400	20 000	400	45
	S500	25 000	500	45
	S800	38 000	800	45
	S1 000	40 000	1 000	45
	S1 600	75 000	1 600	40
	S2 300	103 000	2 300	40
Menck (Germany)	MH48	2 500	48	40
	MH57	3 000	57	40
	MH68	3 500	68	40
	MH80	4 200	80	40
	MH96	5 000	96	40
	MH120	6 300	96	40
	MH145	7 500	145	40
	MH165	8 600	165	38
	MH195	10 000	195	38
	MHF5–12*	12 900	129	40
	MHU200 T	12 000	200	60
	MHU400 T	24 000	400	50
	MHU600 T	36 000	600	50
	MHU800 T	48 000	800	45
	MHU1 000 T	60 000	1 000	45

MHU1 700 T	102 000	1 700	40
MHU2 100 T	126 000	2 100	40
MHU3 000 T	180 000	3 000	35

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\* The BSP MHF and Banut hammers are free-fall types.

### 3.1.6 Selection of type of piling hammer

The selection of the most suitable type of hammer for a given task involves a consideration of the type and weight of the pile, and the characteristics of the ground into which the pile is to be driven. Single-acting and double-acting hammers hydraulic and diesel hammers are effective in all soil types and the selection of a particular hammer for the given duty is based on a consideration of the value of energy per blow, the striking rate, and the fuel consumption. The noise of the pile driving operation may in some circumstances be an important consideration in the selection of a hammer. This aspect is discussed in 3.1.7.

A knowledge of the value of energy per blow is required to assess whether or not a hammer of a given weight can drive the pile to the required penetration or ultimate resistance without the need for sustained hard driving or risk of damage to the pile or hammer. The safety of operatives can be



*Fig. 3.15 BSP International Foundations Ltd. 40-tonne hydraulic hammer*

endangered if sustained hard driving causes pieces of spalled concrete or mechanical components to fall from a height. The employment of a dynamic pile-driving formula can, with experience, provide a rough assessment of the ability of a hammer with a known rated energy value to achieve a specific ultimate pile resistance to the time of driving (see [1.4](#) and [7.3](#) for a further discussion of these formulae). However, the manufacturer's rated energy per blow is not always a reliable indication of the value to be used in a dynamic pile equation. The efficiency of a hammer can be very low if it is poorly maintained or improperly operated. Also the energy delivered by the hammer to the pile depends on the accuracy of alignment of the hammer, the type of packing inserted between the pile and the hammer, and on the condition of the packing material after a period of driving.

The increasing use of instruments to measure the stresses and acceleration at the head of a pile as it is being driven (see [Section 7.3](#)) has provided data on the efficiencies of a wide range of hammer types. Some typical values are:

Hammer type	Efficiency of hammer/cushioning system (%)
Hydraulic	65–90
Drop (winch-operated)	40–55
Diesel	20–80

Table 3.3 Characteristics of double-acting and differential-acting piling hammers

Maker	Type	Mass of ram (kg)	Maximum energy per blow (m/kg)	Maximum striking rate (blows/min)
BSP International Foundations Limited (United Kingdom)	500N	91	165	330
	600N	227	415	250
	700N	385	650	225
	900	726	1 210	145
	1 000	1 360	1 815	105
	1 100	2 270	2 650	95
Koehring MKT (United States of America)	9B3	726	1 210	145
	10B3	1 360	1 815	105
	11B3	2 270	2 650	95
Menck (Germany)	SB80	270	400	205
	SB120	390	600	175
	SB180	600	945	150
	SB270	870	1 410	130
	SB400	1 300	2 170	115
Raymond* (United States of America)	50CX	2 950	2 070	115–125
	65C	2 950	2 700	100–110
	80C	3 630	3 370	95–105
	125CX	6 800	5 620	105–115
	150C	6 800	6740	95–105
Vulcan* (United States of America)	18C	816	500	150
	30C	1 360	1 000	133
	50C	2 270	2 080	120
	65C	2 950	2 650	117
	80C	3 630	3370	111
	140C	6 350	4 970	103
	200C	9 070	6 930	98
	400C	18 140	15 660	100

\*Differential acting

The wide range in values for the diesel hammer reflects with greater sensitivity to the type of soil or rock into which the pile is driven and the need for good maintenance. Present-day practice is to base the selection of the hammer on a driveability analysis using the Smith wave equation (see 7.3) to produce curves of the type shown in [Figure 3.18](#). They show the results of an investigation into the feasibility of using a D100 diesel hammer to drive 2.0m OD by 20mm wall thickness steel tube piles through soft clay into a dense sandy gravel. The piles were to be driven with closed ends to overcome a calculated soil resistance of 17.5MN at the final penetration depth. [Figure 3.18](#) shows that a driving resistance (blow count) of 200 blows/250mm penetration would be required at this stage. This represents a rather severe condition. A blow count of 120 to 150 blows/250mm is regarded as a practical limit for sustained driving of diesel or hydraulic hammers. However 200 blows/250mm would be acceptable for fairly short periods of driving.

The American Petroleum Institute<sup>(3.4)</sup> defines refusal to driving as the point where the driving resistance exceeds either 300 blows per foot (248 blows/250mm) for 1.5 consecutive metres or 800 blows per foot (662 blows/250mm), for 0.3m penetration. [Figure 3.18](#) also shows the driving resistance curves for a 25-tonne drop hammer with drops of 1.5 or 2.0m to be used as a standby to achieve the required soil resistance if this could not be obtained by the diesel hammer.

Vibratory hammers are very effective in loose to medium-dense granular soils and the high rate of penetration of low-displacement steel piles driven by vibratory hammers may favour their selection for these conditions.

### 3.1.7 Noise control in pile driving

The control of noise in construction sites is a matter of increasing importance in the present drive to improve environmental conditions. Control of noise is necessary to protect the health of operatives on the sites and to eliminate, as far as possible, annoyance or hazards to the health of the general public<sup>(3.5)</sup>.

In Britain the BS Code of Practice 5228 recommends that no person should be exposed to a noise level of more than 90dBA for eight hours a day in a five-day week. It is recognized that the noise from

Table 3.4 Characteristics of diesel piling hammers

Maker	Type	Mass of ram (kg)	Maximum energy per blow (m/kg)	Maximum striking rate (blows/min)
Berminghammer (Canada)	B200	816	2 484	
	B225PI	1 361	4 036	
	B300PI	1 701	5 560	
	B3405PI	1 814	6 348	
	B4005PI	2 267	7 935	
	B4505PI	2 993	10 474	
	B5005PI	3 446	12 061	
	B5505PI	4 082	14 283	
	B23 D/A	1 270	3 174	
BSP International Foundations Limited (United Kingdom)	DE30C	1 360	3 731	47
	DE50C	2 260	6 219	47
	B15	1 500	3 793	80–100*
	B25	2 500	6 320	80–100*
	B35	3 500	8 850	80–100*
	B45	4 500	11 400	80–100*
Delmag (Germany)	D5	500	9 050	40–60
	D12	1 220	3 125	40–60
	D15	1 500	3 750	42–60
	D22/13	2 200	6 700	38–52
	D30/13	3 000	9 100–4 550	38–52
	D36/13	3 600	11 300–5 750	37–53
	D46/13	4 600	14 600–7 300	37–53
	D62/12	6 200	22 320–11 160	35–50
	D100/13	10 000	34 000–21 800	36–45
Hera (Netherlands)	H1 500	1 500	4 140	40–60
	H2 500	2 500	8 000	37–50
	H3 500	3 500	11 200	37–50
	H5 000	5 000	16 000	37–50
	HD7 500	7 500	24 000	37–50
Ishikawajima Harima (Japan)	J22	2 200	5 410	42–70
	J35	3 500	8 780	42–70
	J44	4 400	11 000	42–70
Kobe (Japan)	K13	1 300	3 700	40–60
	K25	2 500	7 500	39–60
	K35	3 500	10 500	39–60
	K45	4 500	13 500	39–60
	KB45	4 500	13 500	35–60
	KB60	6 000	16 000	35–60
	KB80	8 000	24 000	35–60
	K150	15 000	39 500	42–60
Mitsubishi (Japan)	M14	1 350	3 600	42–60
	M23	2 295	6 220	42–60

	M33	3 290	8 850	40–60
	M43	4 290	11 620	40–60
	MB70	7 185	18 950	38–60
Koehring-MKT (United States of America)	DE10	500	1 220	40–50
	DE20/B	910	2 300–1 600	40–50
	DE30/B	1 270	3 300–2 300	40–50
	DE40	1 810	4 400	40–50
	DE50/B	2 280	5 900–4 100	40–50
	DE70	3 175	8 700	40–50
	DE70/B	7 000	8 200–5 800	40–50
	DA35/B	1 270	3 100	48 <sup>†</sup> 82 <sup>*</sup>
	DA55/B	2 270	5 500	48 <sup>†</sup> 82 <sup>*</sup>
Link Belt (United States of America)	180	780	1 120	90–95 <sup>*</sup>
	312	1 750	2 080	100–105 <sup>*</sup>
	440	1 814	2 420	86–90 <sup>*</sup>
	520	2 300	3 640	80–84 <sup>*</sup>

Maker	Type	Mass of ram (kg)	Maximum energy per blow (m/kg)	Maximum striking rate (blows/min)
Vulcan (United States of America) and M.A.N.L.SA (Belgium)	1N100	1 360	3 400	50–60
	4N100	1 800	4 500	50–60
	3N100	2 400	6 000	50–60
I.C.E. (International Construction Equipment) b.v. (Netherlands)	180	782	1 770	90–95*
	440	1 814	4 300	88–92*
	520	2 300	6 200	80–84*
	640	2 730	7 800	74–77*
	1 070	4 540	14 600	64–70*

†Single-acting

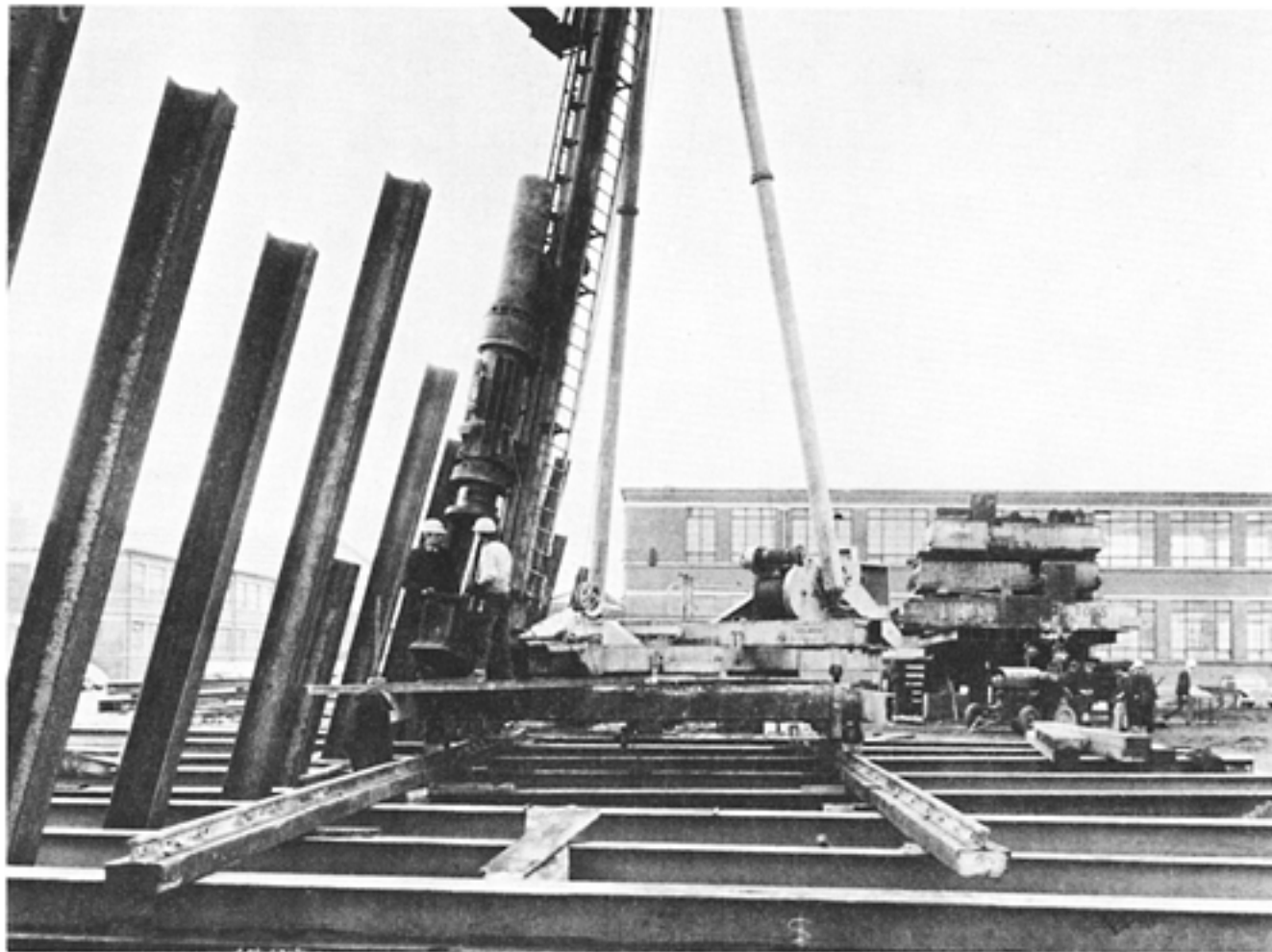
\*Double-acting

Table 3.5 Characteristics of pile-driving vibrators

Maker	Type	Frequency range (Hz)	Mass (kg)	Minimum power supply (KVA)
Bodine (United States of America)	—	up to 135	10 000	740
	—	up to 135	6 600	370
Dawson Construction Plant (United Kingdom)	DCP/Krupp	up to 50	1 500	70
Menck (Germany)	6.5–30	50	900	35
	22–30	50	2 200	125
	44–15/30	25–50	5 400	250
Müller (Germany)	MS-5H	27	1 080	54
	MS-10H	27	1 400	82
	MS-20H	28	3 200	115
	MS-25H	28	3 600	247
	MS-50H	27	5 800	430–560
	MS-100H	22	8 500	430–560
	MS-16E	25	2 000	—
	MS-40E	25	2 800	—
	MS-60E	25	7 200	—
	MS-60ET	25	7 500	—
Delmag (Germany)	PE1401	24.3	1 030	40
	PE2001	24.3	1 080	50
	PE3 001	24.3	1 930	125
	PE7 001	24.3	4 820	300
P.T.C. (Procédés Techniques de Construction) (France)	6H1	30	1 250	59
	13H1	27	1 860	110
	25H1	25	3 500	206
	25H2	27	3 500	260
	50H2	26	10 400	380
	60H1	27	10 400	490
	110H1	22	14 500	570
	110H2	22	12 100	570
I.C.E. (International Construction Equipment) bv. (Netherlands)	116	27	900	85

	216	27	900	135
	416	25	2 150	187
	815	25	2 170	354
	1 412	21	3 900	485
Tomen (Toyo Menka Kaisha) (Japan)	KM2-2 000A	18-21	2 565	49.6*
	VM2-4 000A	14-18	3 558	80.5*
	KM2-12 000	8-10	4 510	120.7*
	VM2-5 000	18-30	4 887	120.7*
	VM4-10 000	18	6 490	201.1*
	VM2-25 000	10	7 400	201.1*
Vibro Mac (Soil Mec) (Italy)	5	18-30	4 940	295
	12	10-16	6 100	295
Schenk (Germany)	DR60	17-39	7 200	250

\* Rated output of motor



*Fig. 3.16 Delmag D22 diesel hammer with helmet for driving steel H-piles*

pile-driving will exceed 90dBA but the operations are not continuous through the working day and the observed noise level can be converted to an equivalent sound level that takes into account the duration of the noise emission.

There is no legislation in Britain that lays down specific noise levels which must not be exceeded in areas accessible to the general public. Local authorities adopt their own standards of judging noise nuisance, and maximum day-time and night-time noise levels of 70dBA and 60dBA respectively are frequently stipulated for urban areas. The higher of these values can be compared with field observations of pile-driving noise obtained from a number of sources and shown in [Figure 3.19](#). Other information has shown that the attenuation of pile-driving impact noise to the 70dBA level from the noisiest of the hammers requires a distance of more than 1000m from the sound. Thus if a maximum sound level of 70dBA is stipulated by a local authority, it is necessary to adopt some means of controlling noise emission in order to protect the general public whose dwellings or place of work are closer to the construction operations<sup>(3.6)</sup>. One method of doing this is to enclose the hammer and pile with a sound-absorbent box. The Hoesch noise-abatement tower is formed of sandwiched steel plate/plastics construction and consists of an outer 2mm steel plate, a plastics layer 0.4mm thick, and an inner 1.5mm steel plate. The plates making up the box are jointed by a rubber insertion material, and the lid incorporates a sound-proofed air exhaust. A hinged door allows the pile and hammer to be pitched into the tower. The Hoesch tower reduced the noise from a Delmag D12 diesel hammer driving a sheet pile from 118 to 119dBA at 7m to 87 to 90dBA at the same distance.

A tower of similar construction is shown in [Figure 3.20](#). Shelbourne<sup>(3.7)</sup> described the use of the tower for driving 24m steel H-piles by means of a 3-tonne drop hammer. Sound level measurements of 60 to 70dBA were recorded 15m from the tower, compared with values of 100dBA before the noise abatement system was adopted.



*Fig. 3.17 Driving pile casing with Muller vibrator.*

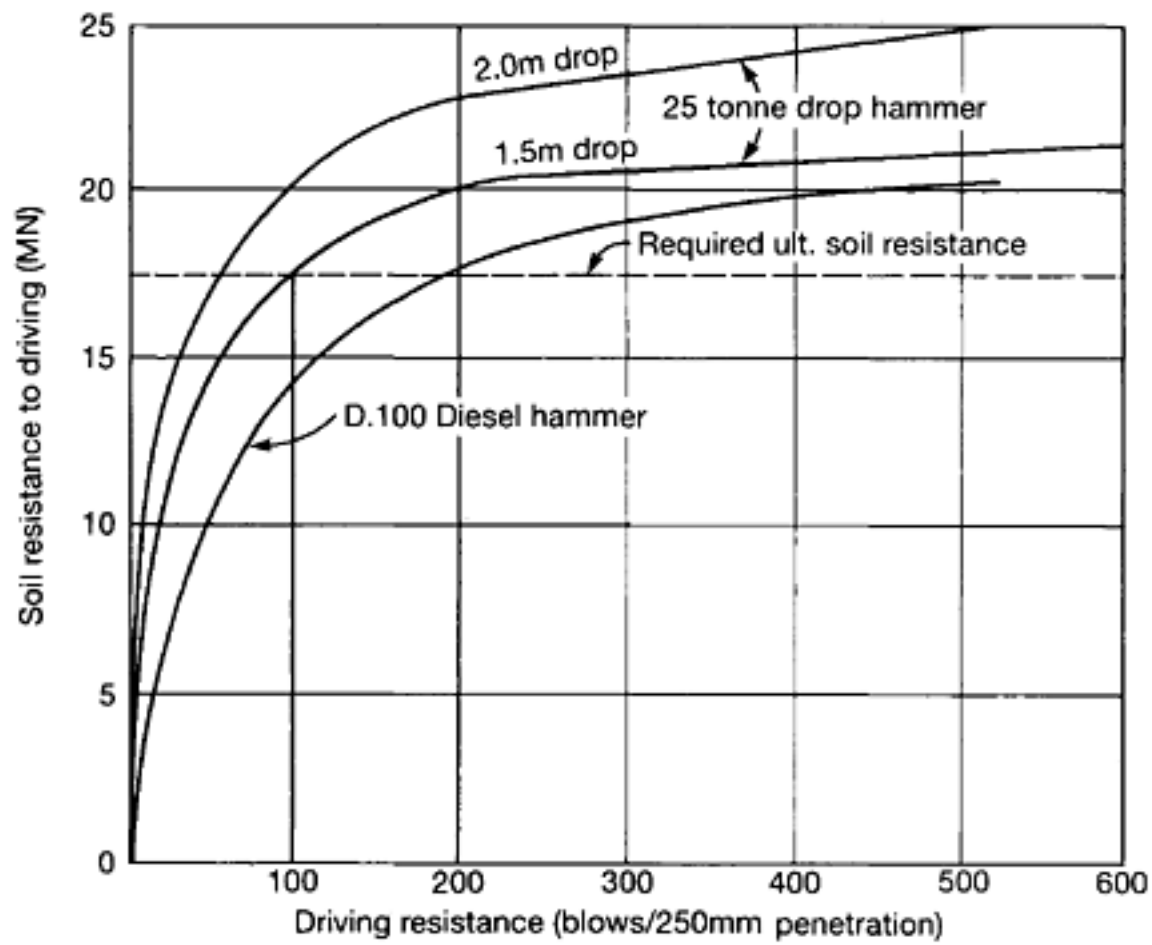


Fig. 3.18 Pile driveability curves

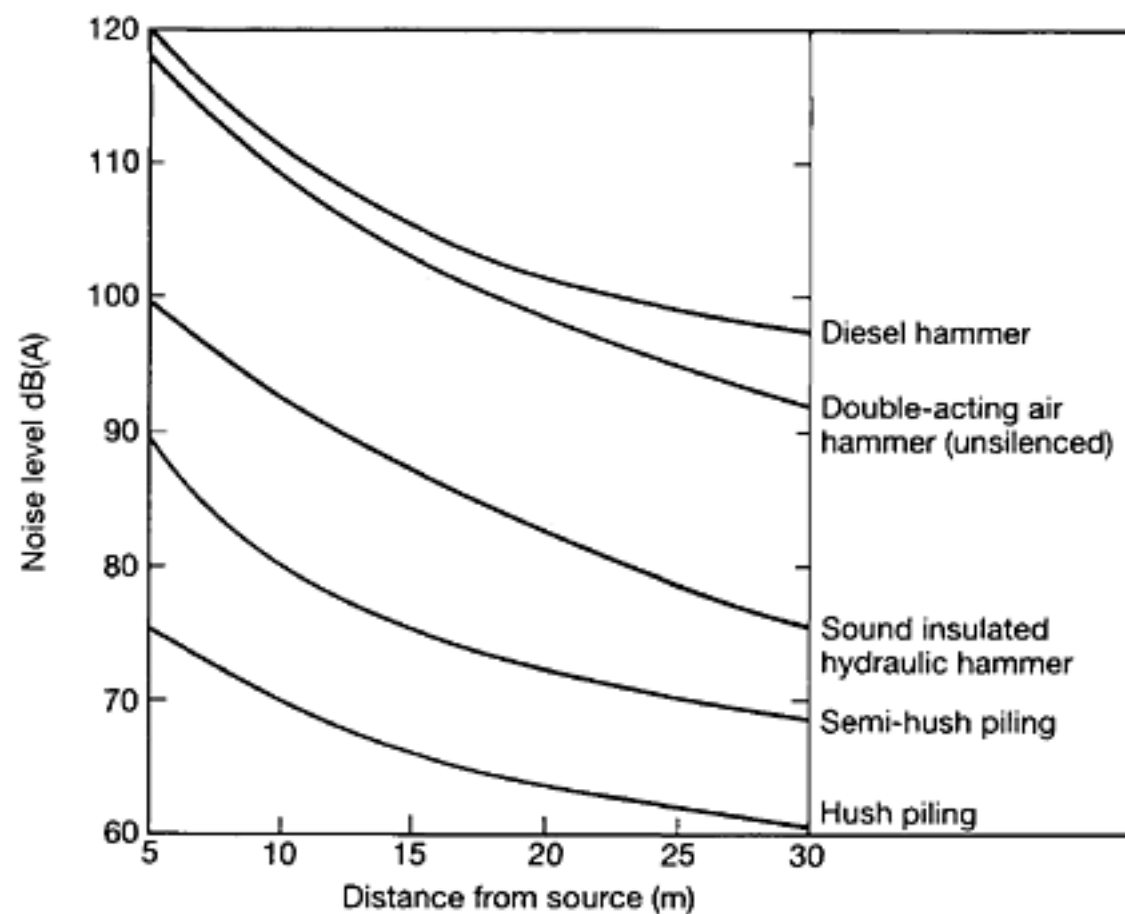


Fig. 3.19 Typical noise levels for various pile-driving techniques

Surrounding only the lower part of the hammer by a shroud is not particularly effective. A reduction of only 3 to 4dBA was obtained by shrouding a Delmag D22 hammer in this way. As noted in 3.1.4, the hydraulic hammer is a suitable type for enclosing in a sound-proof box.

### 3.1.8 Pile helmets and driving caps

When driving precast concrete piles, a helmet is placed over the pile head for the purpose of retaining in position a resilient 'dolly' or cap block that cushions the blow of the hammer and thus minimizes damage to the pile head. The dolly is placed in a recess in the top of the helmet ([Figure 3.21](#)). For easy driving conditions it can consist of an elm block, but for rather harder driving a block of hardwood such as oak, greenheart, pyinkado or hickory, is set in the helmet end-on to the grain. Plastics dollies



Fig. 3.20 Noise abatement tower used for 'Hush Piling' system

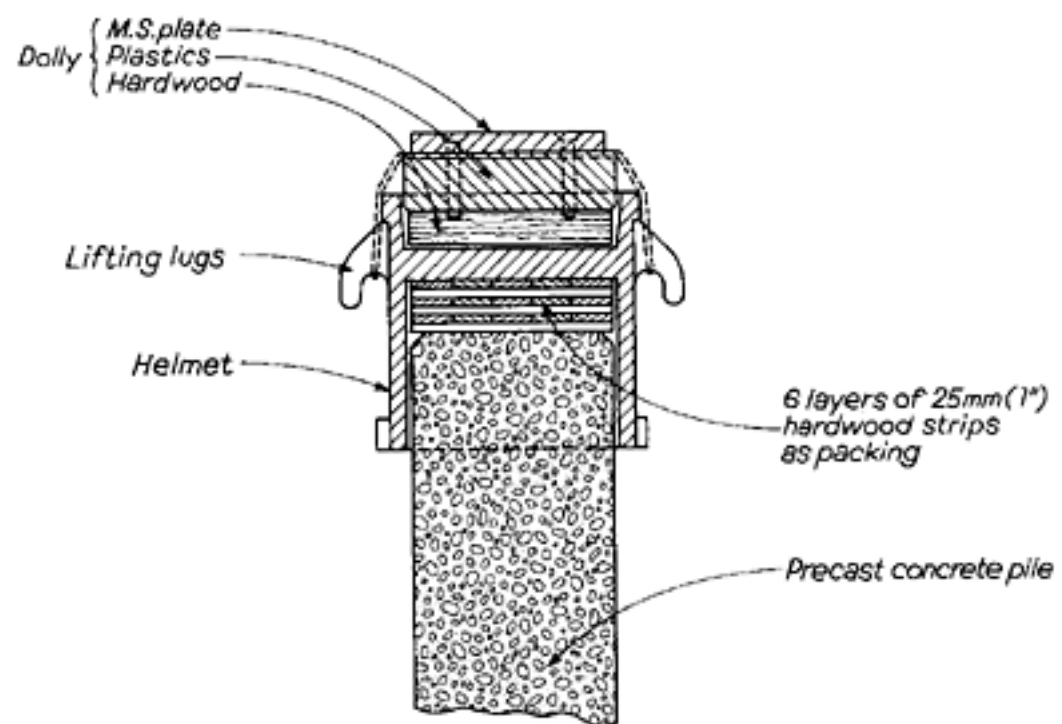


Fig. 3.21 Dolly and helmet for precast concrete pile

are the most serviceable for hard-driving concrete or steel piles. The Micarta dolly consists of a phenolic resin reinforced with laminations of cross-grain cotton canvas. Layers of these laminates can be bonded to aluminium plates, or placed between a top steel plate and a bottom hardwood pad. The helmet should not fit tightly onto the pile head but should allow for some rotation of the pile, which may occur as it strikes obstructions in the ground.

Packing is placed between the helmet and the pile head to cushion further the blow on the concrete. This packing can consist of coiled rope, hessian packing, thin timber sheets, coconut matting, wallboards, or asbestos fibre. The last-mentioned material has the advantage that it does not char when subjected to heat generated by prolonged driving. The packing must be inspected at intervals and renewed if it becomes heavily compressed and loses its resilience. Softwood packing should be renewed for every pile driven.

Williams<sup>(3.8)</sup> has described severe conditions for driving precast concrete piles at Uskmouth Power Station. He states that plastics dollies were used up to 40 times, compared with elm blocks which only lasted for a very few piles. The packing consisted of up to 125mm of sawdust in jute bags, covered with two dry cement sacks placed at right-angles to each other over the pile head.

Driving caps are used for the heads of steel piles but their function is more to protect the hammer from damage than to protect the pile. The undersides of the caps for driving box or H-section piles have projecting lugs to receive the head of the pile. Those for driving steel tubular piles ([Figure 3.22](#))

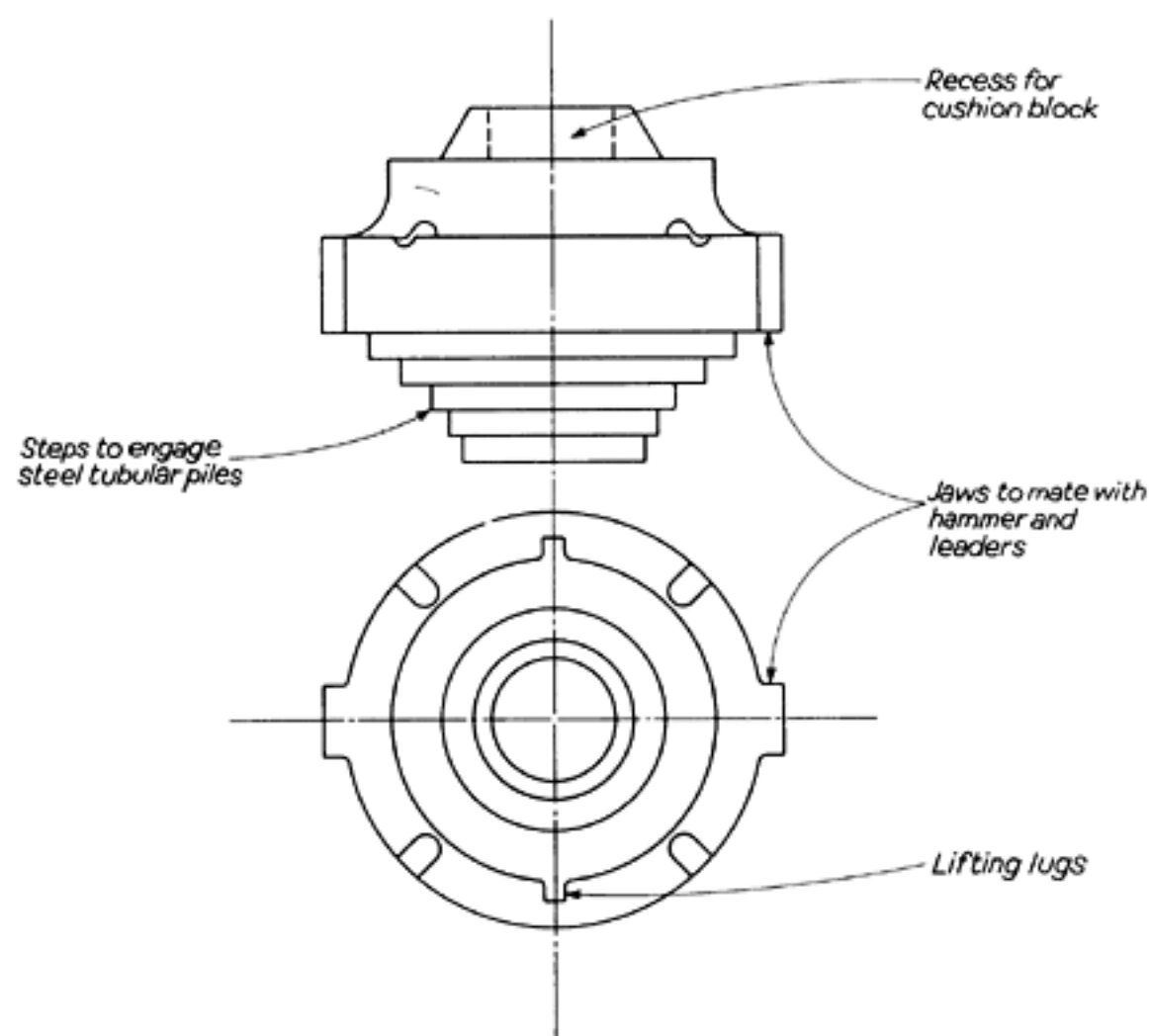


Fig. 3.22 Vulcan driving cap for steel tubular pile

have multiple projections that are designed to fit piles over a range of diameters. They include jaws to engage the mating hammers.

Plastic dollies of the Micarta type have a long life when driving steel piles to a deep penetration into weak rocks or soils containing cemented layers. However, for economy contractors often cushion the pile heads with scrap wire rope in the form of coils or in short pieces laid cross-wise in two layers. These are replaced frequently as resilience is lost after a period of sustained driving.

### 3.1.9 Jetting piles

Water jets can be used to displace granular soils from beneath the toe of a pile. The pile then sinks down into the hole formed by the jetting, so achieving penetration without the use of a hammer. Jetting is a useful means of achieving deep penetration into a sandy soil in conditions where driving a pile over the full penetration depth could severely damage it. Jetting is ineffective in firm to stiff clays, however, and when used in granular soils containing large gravel and cobbles the large particles cannot be lifted by the wash water. Nevertheless, the sand and smaller gravel are washed out and penetration

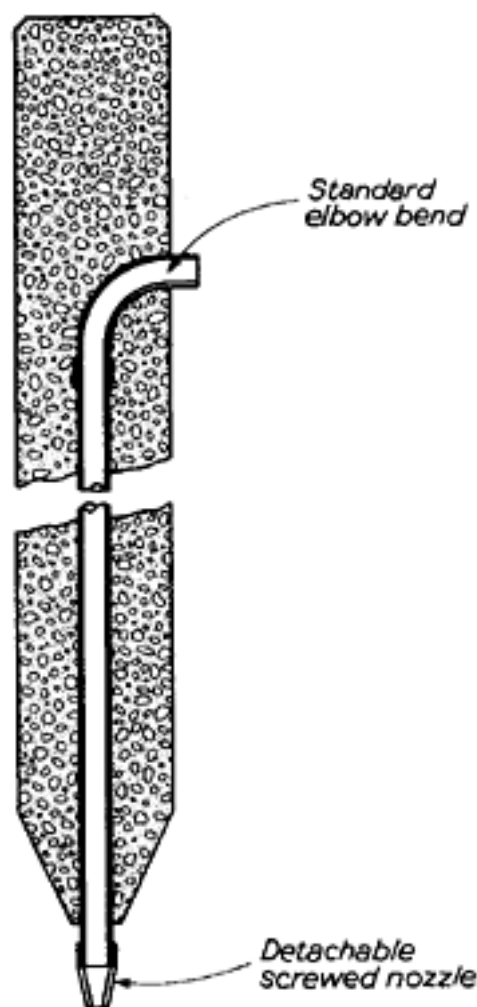


Fig. 3.23 Centrally-placed jetting pipe

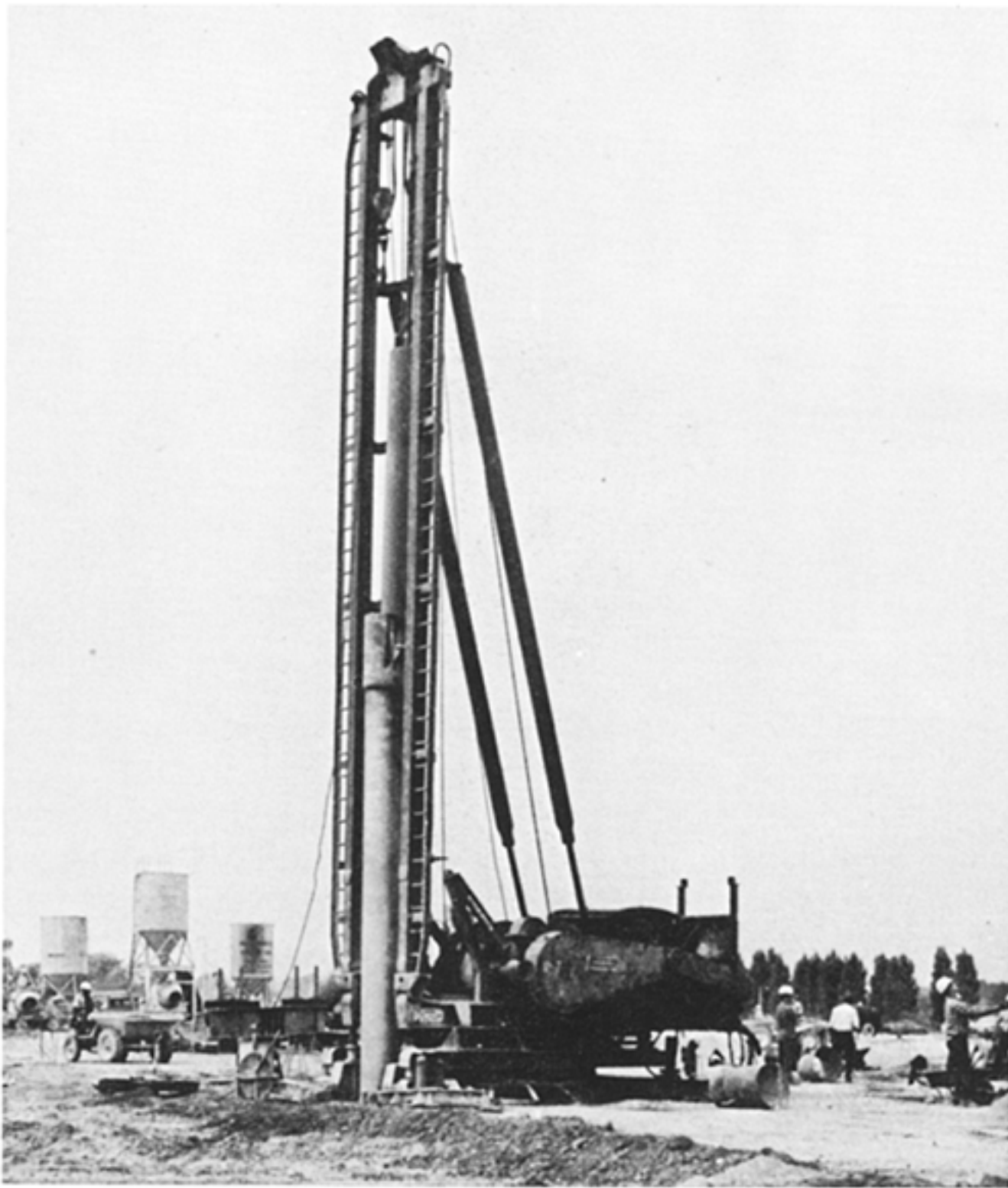
over a limited depth can be achieved by a combination of jetting and hammering. Air can be used for jetting instead of water, and a bentonite slurry can be also used if the resulting reduced skin friction is acceptable.

For jetting piles in clean granular soils a central jetting pipe is the most effective method, as this helps to prevent the pile from deviating off line. A 25 to 50mm nozzle should be used with a 50 to 75mm pipe ([Figure 3.23](#)). The quantity of water required for jetting a pile of 250 to 350mm in size ranges from 15 to 60 l/s for fine sands through to sandy gravels. A pressure at the pump of at least 5 bars is required. The central jetting pipe is connected to the pump by carrying it through the side of the pile near its head. This allows the pile to be driven down to a 'set' on to rock or some other bearing stratum immediately after shutting down the jetting pump.

A central jetting pipe is liable to blockage when driving through sandy soils layered with clays and the blockage cannot be cleared without pulling out the pile. A blockage can result in pile bursting if high jetting pressures are used. An independent jetting pipe worked down outside the pile can be used instead of a central pipe, but the time spent in rigging the pipe and extracting it can cause such delays to pile driving as to be hardly worth the trouble involved. Open-ended steel tubular piles and box piles can be jetted by an independent pipe worked down the centre of the pile, and H-piles can be similarly jetted by a pipe operated between the flanges. Large-diameter tubular piles can have a ring of peripheral jetting pipes, but the resulting pile fabrication costs are high. Gerwick([3.9](#)) has described the system for jetting 4m diameter tubular steel piles for a marine terminal at Cook Inlet, Alaska. Sixteen 100mm pipes were installed around the inner periphery of the pile. The nozzles were cut away at each side to direct the flow to the pile tip. Gerwick recommends that jetting nozzles should terminate about 150mm above the pile tip. He gives the following typical requirements for jetting large-diameter piles:

Jet pipe diameter	—40mm
Pressure	—20 bar (at pump)
Volume	—13l/s per jet pipe

The large volume of water used in jetting can cause problems by undermining the piling frame or adjacent foundations as it escapes towards the surface. It can also cause a loss of skin friction in adjacent piles in a group. Where skin friction must be developed in a granular soil the jetting should be stopped when the pile has reached a level of about one metre above the final penetration depth, the remaining penetration then being achieved by hammering the pile down. The jetting method is best suited to piles taken down through a granular overburden to end bearing on rock or some other material resistant to erosion by wash water.

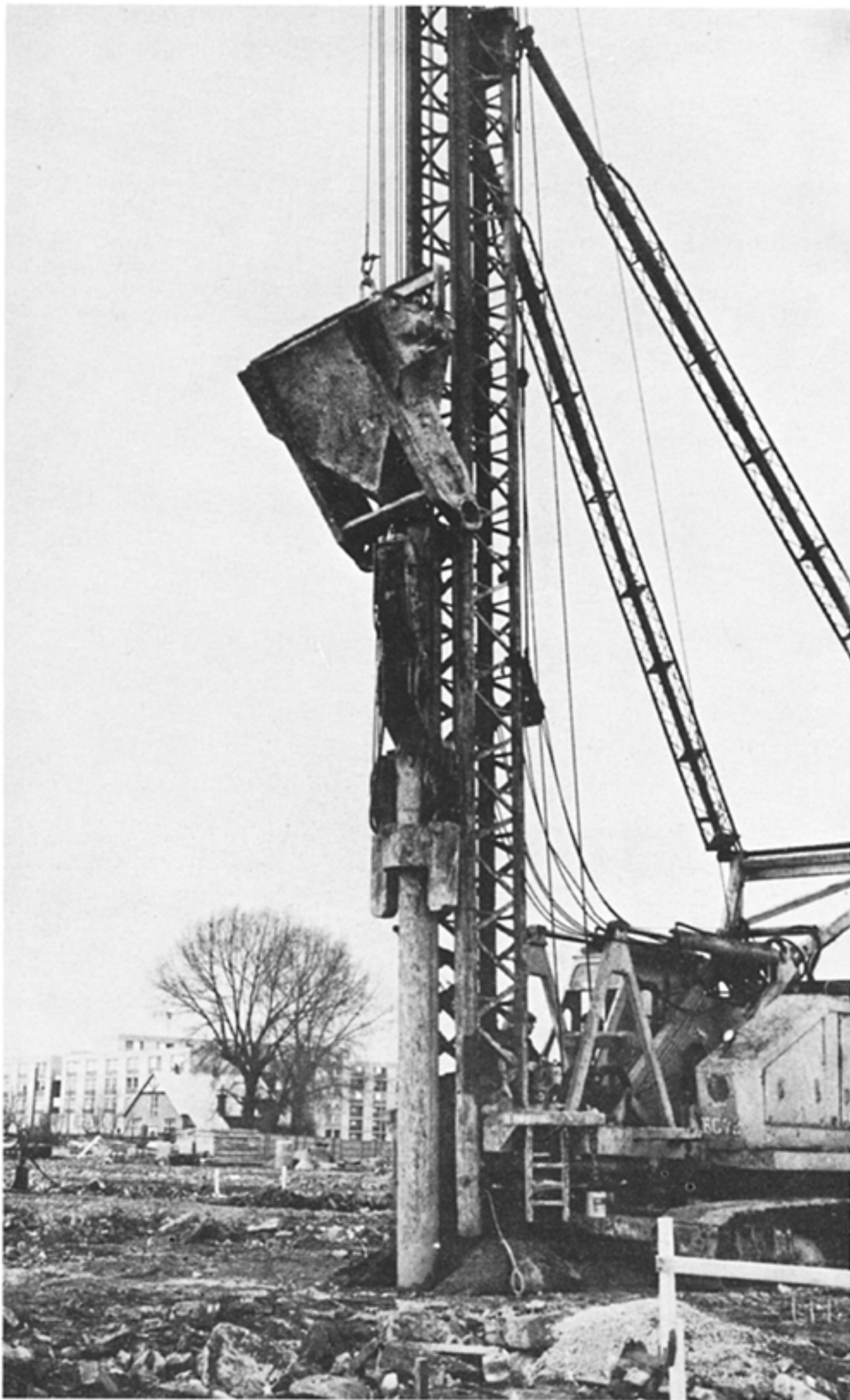


*Fig. 3.24 Franki pile-driving rig*

### **3.2 Equipment for installing driven-and-cast-in-place piles**

The rigs used to install driven-and-cast-in-place piles are similar in most respects to the types described in 3.1.1 to 3.1.3 but the firms who install proprietary types of pile usually make modifications to the rigs to suit their particular systems. The piling tubes are of heavy section, designed to be driven from the top by drop, single-acting, or diesel hammers, but the Franki ([Figure 3.24](#)) piles can be driven by an internal drop hammer. The leaders of the piling frames are often adapted to accommodate guides for a concreting skip ([Figure 3.25](#)).

Steel cased piles designed to be filled with concrete are driven more effectively by a hammer operating on the top, than by an internal drop hammer acting on a plug of concrete at the base. This is because a hammer blow acting on top of the pile causes the tube to expand and push out the soil at the instant of striking, followed by a contraction of the tube. This frees the tube from some of the skin friction as it moves downward under the momentum of the hammer. The flexure of the pile acting as a long strut also releases the skin friction at the moment of impact. However, when using an internal drop hammer, tension is induced in the upper part of the pile and the diameter contracts, followed by an expansion of the soil and an increase in skin friction as the pile moves downwards. Flexure along the piling tube does not occur when the hammer blow is at the base, and thus there is no reduction in skin friction from this cause. Tension caused by driving from the bottom can cause the circumferential cracking of reinforced concrete and thin-wall steel tubular piles.

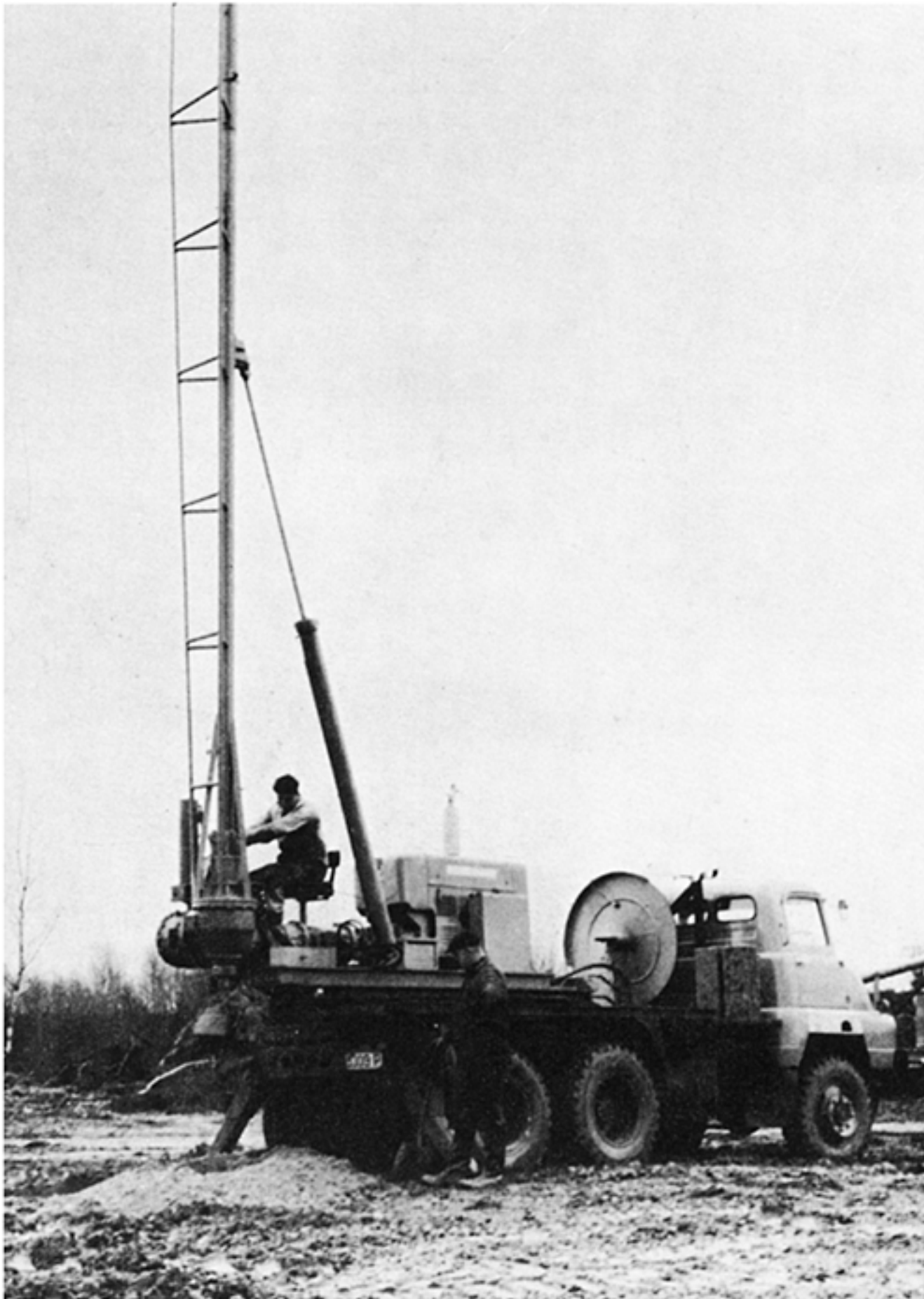


*Fig. 3.25 Discharging concrete into the driving tube of a GKN pile. Concreting skip traveling on pile frame leaders*

Top driving has another advantage in allowing the pile to be driven with an open end, thus greatly reducing the end-bearing resistance during driving, whereas the bottom-driven pile demands a solid plug at the pile base at all stages. In easy driving conditions bottom driving will give economy in the required thickness of the steel and considerable reduction in noise compared with top driving. However, great care is necessary to

avoid bursting of the tube by impact on the concrete when bottom driving through dense granular soil layers or into weak rocks containing bands of stronger rock. The concrete forming the plug should have a compacted height of not less than  $2\frac{1}{2}$  times the pile diameter. In calculating the quantity of concrete required, allowance should be made for a volume reduction of 20% to 25% of the uncompacted height. The 1:2:4 concrete should be very dry with a water : cement ratio not exceeding 0.25 by weight. A hard aggregate with a maximum size of 25mm should be used.

At least 10 initial blows should be given with hammer drops not exceeding 300mm then increasing gradually. The maximum height of drop should never exceed the maximum specified for the final set which is usually between 1.2m and 1.8m. Driving on a plug should not exceed a period of  $1\frac{1}{2}$  hours. After this time, fresh concrete should be added to a height of not less than the pile diameter and driving continued for a period of not more than  $1\frac{1}{2}$  hours before a further renewal. For prolonged hard driving it may be necessary to renew the plug every three-quarters of an hour.



*Fig. 3.26 'Highway' lorry-mounted auger drill*

### 3.3 Equipment for installing bored-and-cast-in-place piles

#### 3.3.1 Power augers

Power-driven rotary auger drills are suitable for installing bored piles in clay soils. A wide range of machines is available. The 'Highway' spiral plate auger ([Figure 3.26](#)) is a lorry-mounted machine which can drill holes up to 1370mm in diameter and to depths of up to 12.5m. The soil is removed from spiral plate augers by spinning them after withdrawal from the hole.



*Fig. 3.27 BSP International Foundations Ltd. 'Terradrill' Model TSA75 mounted on standard crawler base machine*

The range of 'Terradrill' machines ([Figure 3.27](#)), manufactured by BSP International Foundations Ltd. for attachment to standard crawler cranes, are capable of drilling from 254mm diameter boreholes to a depth of 26m for the smallest size to 3.5m diameter boreholes to depths of up to 100m with the largest machine (Model TCA110). These depths are achieved by using a triple telescoping kelly, but greater depths are possible when extension drill stems are used with the telescoping kelly. The Calweld drilling machines comprise lorry-mounted bucket drills and crawler-mounted plate augers or bucket drills. The lorry-mounted bucket drills range in size from the 100B with a 711mm diameter bucket capable of drilling to about 21m with the triple telescoping kelly or to 60m with extensions, to the 100B with a 1219mm bucket and reaming gear to drill shafts to 3.66m in diameter and to depths up to 40m with a triple telescoping kelly, and to 90m with extension drill stems. The crawler-mounted rigs range in size from the 55-CH with a bucket diameter of up to 1219mm and capable of drilling to 45m with a triple kelly, to the 200-CH capable of drilling with reamers up to 7.3m diameter shafts and to depths of up to 43m with a double kelly or to greater depths with extension stems. Raking piles with an inclination of up to 1 in 3 are possible with crawler-mounted rigs.

Various types of equipment are available for use with rotary augers. The standard and rock augers ([Figure 3.28a and b](#)) have scoop-bladed openings fitted with projecting teeth. The coring bucket is used to raise a solid core of rock ([Figure 3.28c](#)) and the bentonite bucket ([Figure 3.28d](#)) is designed to avoid scouring the mud cake which forms on the wall of the borehole. The buckets on the Calweld machines can be lifted through the ring drive gear and swung clear to discharge the soil. Grabs can also be operated from the kelly bar.

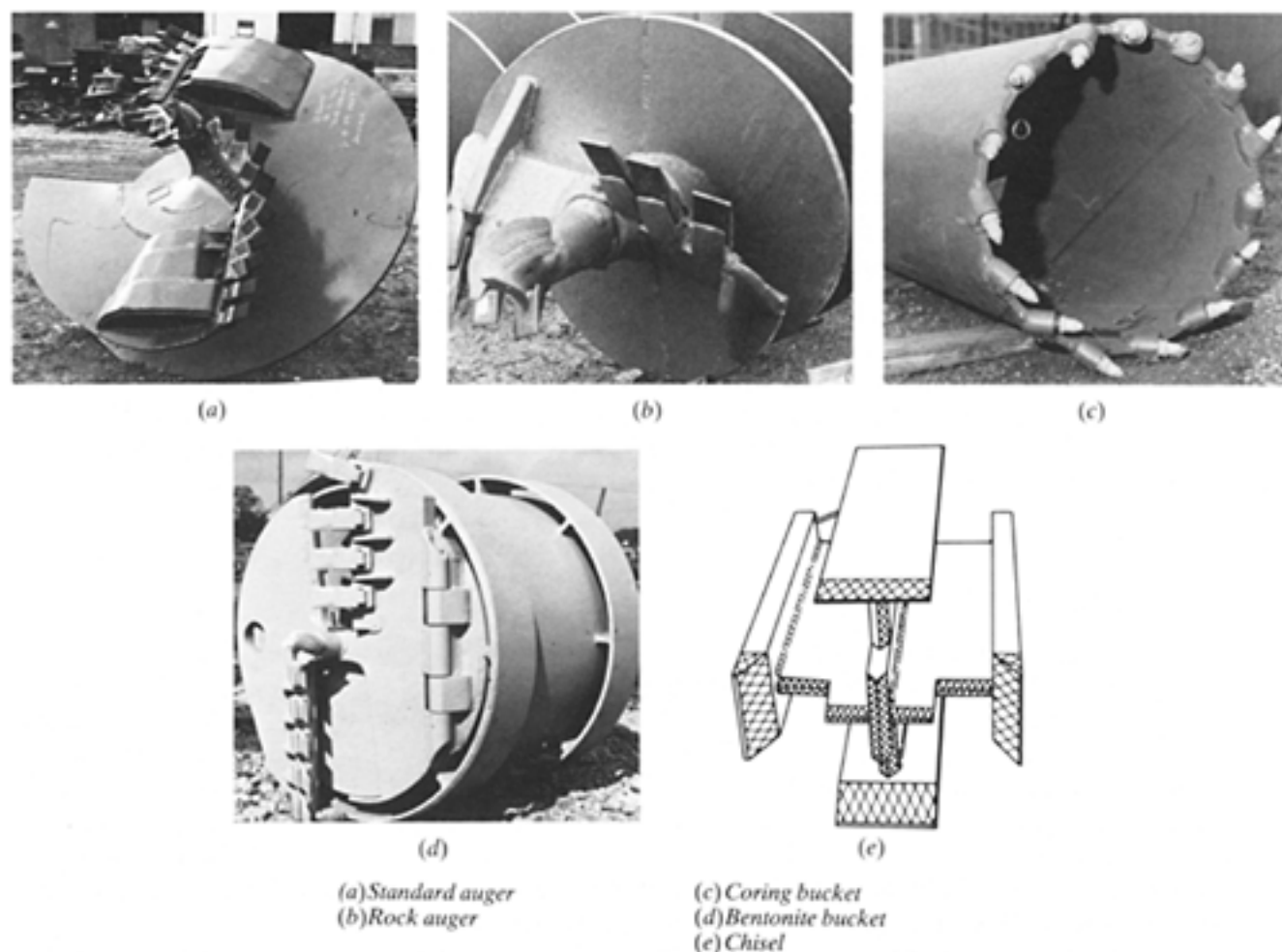


Fig. 3.28 Types of drilling tools

Enlarged or under-reamed bases can be cut by rotating a bellings-bucket within the previously drilled straight-sided shaft. The bottom-hinged bucket ([Figure 3.29a](#)) cuts to a hemispherical shape and because it is always cutting at the base it produces a clean and stable bottom. However, the shape is not so stable as the conical form produced by the top-hinged bucket ([Figure 3.29b](#) and [Figure 3.30](#)), and the bottom-hinged arms have a tendency to jam when raising the bucket. The arms of the top-hinged type are forced back when raising the bucket, but this type requires a separate cleaning-up operation of the base of the hole after completing the under-reaming. Belling buckets normally form enlargements

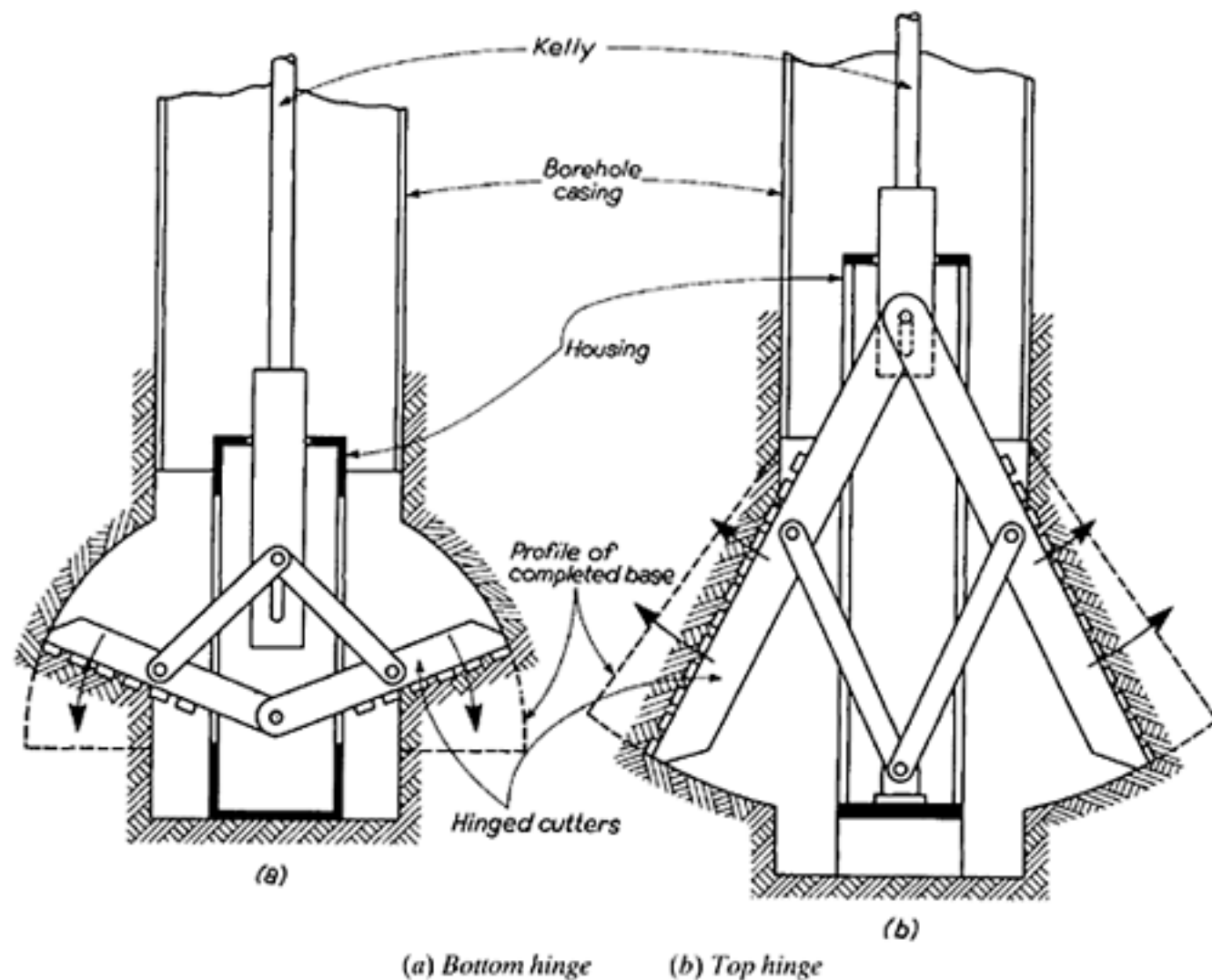


Fig. 3.29 Under-reaming tools

up to 3.7m in diameter but can excavate to a diameter of 6.1m with special attachments. Belling buckets require a shaft diameter of at least 0.76m to accommodate them.

The essential condition for the successful operation of a rotary auger rig is a cohesive soil which will stand without support until a temporary steel tubular liner is lowered down the completed hole or a cohesionless soil supported by a bentonite slurry. In these conditions fast drilling rates of up to 7m per hour are possible for the smaller shaft sizes. Methods of installing piles with these rigs are described in 3.4.6.

### 3.3.2 Grabbing rigs with casing oscillators

For drilling through sands, gravels, and loose rock formations, the pile boreholes may require continuous support by means of casing. For these conditions it is advantageous to use a casing oscillator which imparts a semi-rotating motion to the casing through clamps. Vertical rams attached to the clamps enable the casing to be forced down as the hole is deepened or raised as necessary. The semi-rotating motion is continuous, which prevents the casing becoming 'frozen' to the soil, and it is continued while extracting the casing after placing the concrete. The essential feature of pile boring with a casing oscillator is that the special double-wall casing is always allowed to drop to the full depth of the borehole. For this purpose the casing is jointed. The French Benoto pile casing has a male/female joint which is locked by inserting expanding plugs in holes around the periphery of the tubes.

Self-contained drilling rigs are manufactured that combine a casing oscillator with a mast for handling grabbing or auger drilling tools, and a chute for discharging the soil into a vehicle. The German Hochstrasser-Weise rig can drill cased holes to 55m vertically, or to 24m at a 1 in 4 rake. The minimum and maximum casing diameters are 600mm and 2400mm respectively. The German Bade oscillators are used for piles up to 2500mm in diameter. The French Benoto rig ([Figure 3.31](#)) normally drills holes to 1.0m in diameter and to depths of up to 30m. It can drill raked holes up to 1 in 5.

### 3.3.3 Continuous flight auger drilling rigs

A typical continuous flight auger rig is shown in [Figure 3.32](#). Comments on the operation of rigs of this type are given in 2.4.2.



*Fig. 3.30 Top-hinged under-reaming bucket*



*Fig. 3.31 Benoto piling rig showing discharge chute for spoil and casing oscillator at base of machine*

### **3.3.4 Reverse-circulation drilling rigs**

Reverse-circulation drilling rigs operate on the principle of the air-lift pump. Compressed air is injected near the base of the centrally-placed discharge pipe. The rising column of air and water lifts the soil which has been loosened by rotating cutters, and the casing tubes are also rotated to keep them freely moving in the soil as they sink down while the boring advances. The reverse-circulation rig manufactured by Alfred Wirth and Co. of Germany is shown in [Figure 3.33](#). The casing tubes and air-lift riser pipe are rotated together or separately by means of a hydraulic rotary table or power swivel. The riser pipe is maintained centrally in the casing by one or more stabilizers, and the soil boring is effected by rock roller bits on a cutter head. The diameters of the latter range from 0.76m to 8.0m. The Calweld reverse circulation rigs are manufactured to drill without reaming to diameters of up to 2.1m and to depths up to 230m.

Reverse-circulation rigs can drill at a fast rate in a wide range of ground conditions including weak rocks. They are most effective in granular soils and the large diameter of the air-lift pipes enables them to lift large gravel, cobbles, and small boulders when drilling in glacial soils, or in jointed rocks which are broken up by the rock roller bits. Under-reamed bases can be provided in stiff clays or weak rocks by means of a hydraulically-operated rotary enlarging tool mounted above the cutter head.

### **3.3.5 Tripod rigs**

Small-diameter piles with diameters of up to 600mm installed in soils which require continuous support by lining tubes are drilled by tripod rigs. The drilling is performed in clays by a clay-cutter, which is a simple tube with a sharpened cutting edge, the tube being driven down under the impact of a heavy drill stem. The soil which jams inside the tube is prised out by spade when the cutter is raised

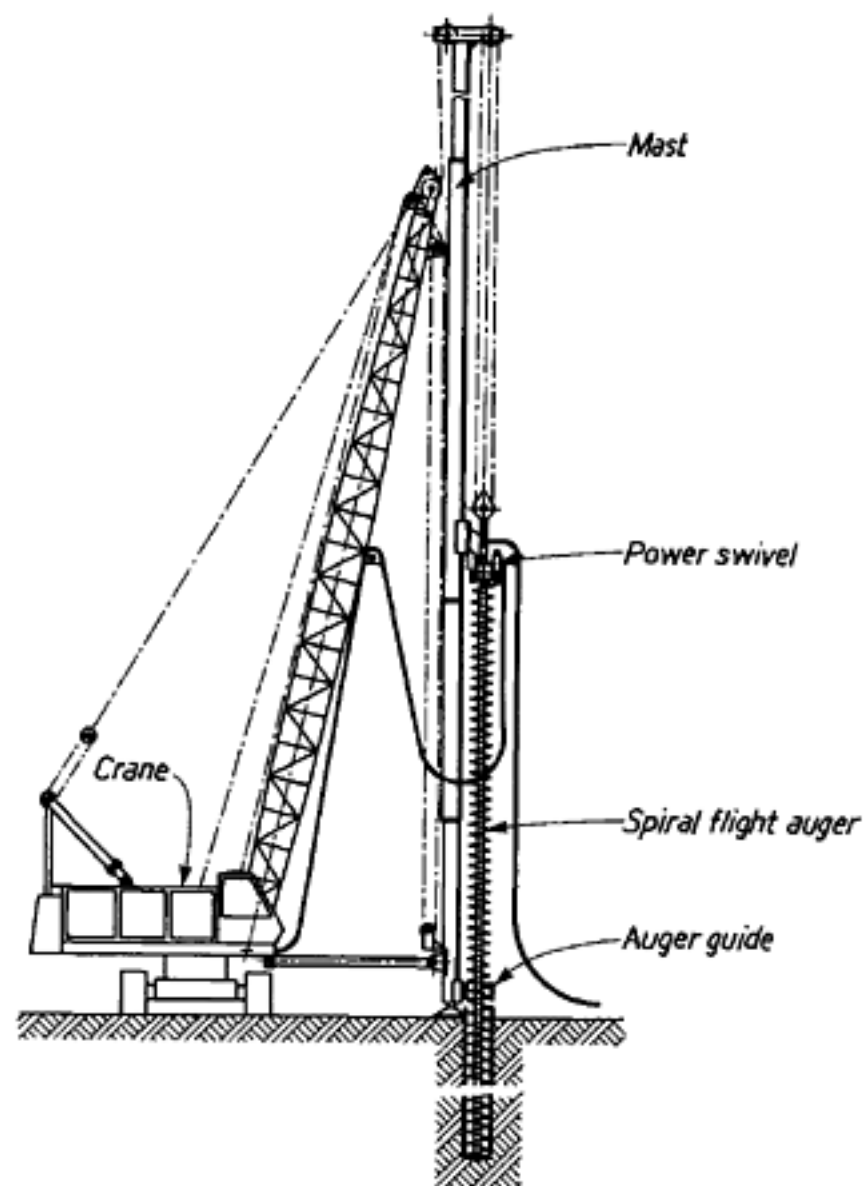
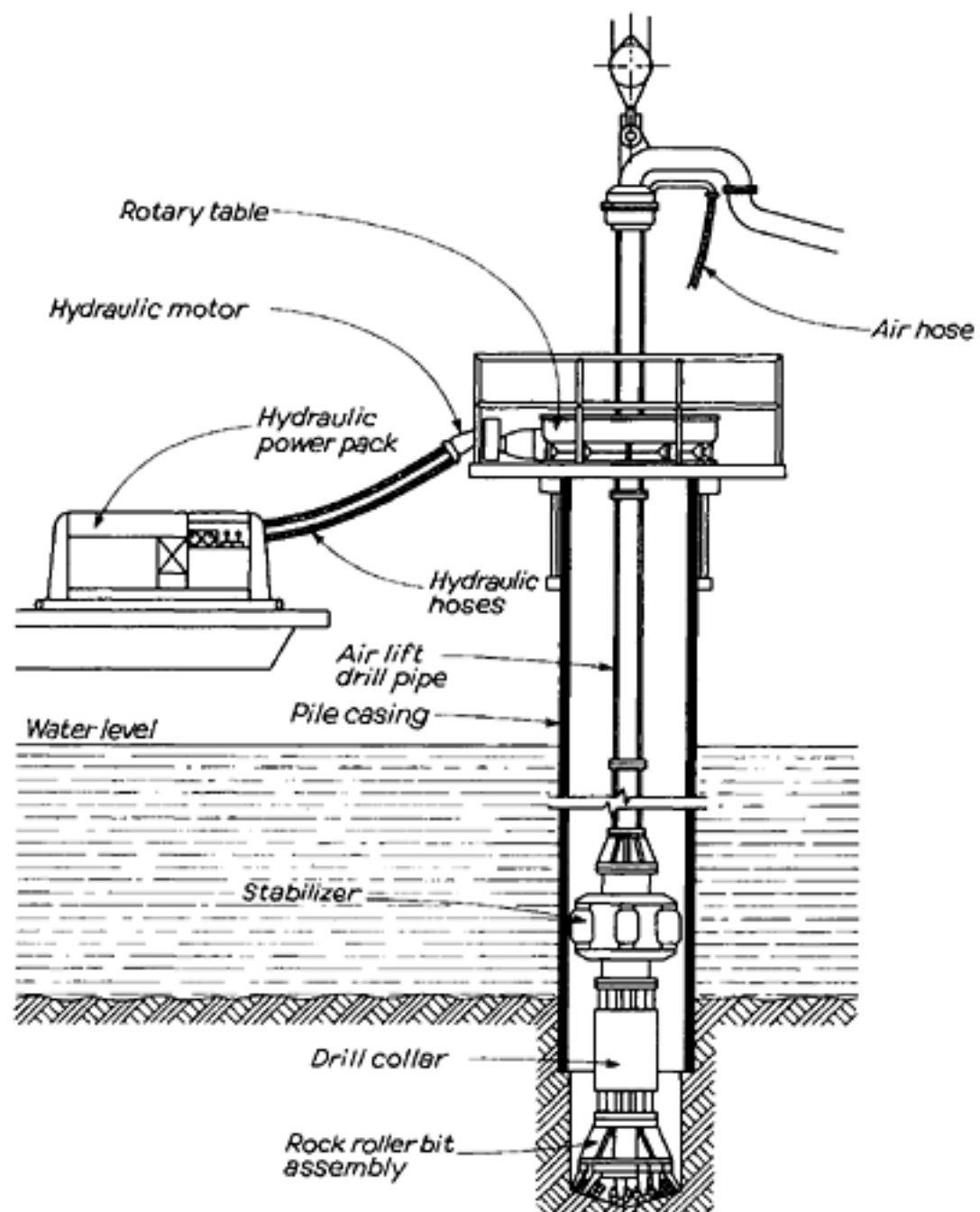


Fig. 3.32 The Wirth continuous flight auger rig



*Fig. 3.33 Wirth rotary table and rotating cutter*



*Fig. 3.34 Tripod piling rig installing bored piles under low headroom conditions*

to the surface. Drilling is effected in cohesionless soils by means of a baler or 'shell', which is again a simple tube with a cutting edge and flap valve to retain the soil, the soil being drawn into the baler by a suction action when the tool is raised and lowered. If no ground water is present in the pile borehole, water must be poured in, or a bentonite slurry may be used. This suction action inevitably causes loosening of the soil at the base of the pile borehole, thus reducing the base resistance ([see 4.3.6](#)). The loosening may be accompanied by settlement of the ground surface around the pile borehole. Rocks are drilled by chiselling and using a baler to raise the debris.

Tripod rigs are not as suitable as the spiral-plate auger types ([see 3.3.1](#)) for drilling small-diameter piles in clays, except in situations where low headroom or difficult access would prevent the deployment of lorry-mounted or tractor-mounted augers. A tripod rig working in conditions of low headroom is shown in [Figure 3.34](#). Methods of operating tripod rigs have been described by North-Lewis and Scott([3.10](#)).

### **3.3.6 Drilling for piles with bentonite slurry**

Lining tubes or casings to support the sides of pile boreholes are a requirement for most of the bored-pile installation methods using equipment described in [3.3.1](#) to [3.3.5](#). Even in stiff cohesive soils it is desirable to use casings for support since these soils are frequently fissured or may contain pockets of sand which can collapse into the bore-holes, resulting in accumulations of loose soil at the pile toe, or discontinuities in the shaft.

Casing can be avoided completely (except for a short length that is used as a guide at the top of the hole) by providing support to the pile borehole in the form of a slurry of bentonite clay. This clay, or a montmorillite clay with characteristics similar to bentonite, has the property of remaining in suspension in water to form a stiff ‘gel’ when allowed to become static. When agitated by stirring or pumping, however, it has a mobile fluid consistency. In a granular soil, the slurry penetrates the walls of the borehole and gels there to form a strong and stable ‘filter-cake’. In a clay soil there is no penetration of the slurry but the hydrostatic pressure of the fluid, which has a density of 1.04g/l prevents collapse where the soil is weakened by fissures.

When used in conjunction with auger or grab-type rigs the slurry is maintained in a state of agitation by the rotating or vertical motion of the drilling tools. When it becomes heavily contaminated with soil or diluted by ground water it can be replaced by pumping-in fresh or reconditioned slurry. It is used most efficiently in conjunction with reverse-circulation rigs (see 3.3.4). The slurry is pumped into the outer casing and the slurry-soil mixture that is discharged from the air-lift riser pipe is allowed to settle in lagoons to remove soil particles. It is then further cleaned in a cyclone, and chemicals to aid gelling are added before the reconditioned slurry is pumped into a holding tank and then returned to the pile borehole. When used for piling work on land or in river works the use of bentonite slurry causes disposal problems. Local authorities will not allow it to be discharged into sewers or watercourses, and the waste slurry must be removed by tanker vehicles.

Bentonite slurry is used in a simple and rather crude way in conjunction with rotary auger equipment when drilling pile boreholes through sands and gravels to obtain deeper penetration into stiff cohesive soils. The hole must be augered through the sands and gravels without support, and then the casing is lowered down. It is uneconomical to provide screwed joints in large-diameter lining tubes and all joints are made by welding. To save time and cost in welding, the holes are drilled to the maximum possible depth before installing the first length of casing. In these conditions support may be provided while drilling by means of a bentonite slurry. Where the depth of cohesionless soil is relatively small it is uneconomical to bring in high-speed mixers, slurry tanks, pumps and reconditioning plant for the normal employment of bentonite techniques. Instead, a few bags of the dry bentonite are dumped into the pile borehole and mixed with the ground water or by adding water, to form a crude slurry which is adequate to smear the wall of the borehole and give it the necessary short-term support. After drilling with this support through the granular overburden, the casing is lowered in one or more lengths and pushed down to seal it into the stiff cohesive soil below. The thrust is provided either by the hydraulically-operated ‘crowd’ mechanism on the kelly-bar of the drilling machine or by means of a vibrator (see 3.1.5) mounted on the casing. This technique is known as ‘mudding-in’ the casing.

The use of a bentonite slurry to aid drilling with or without temporary lining tubes may cause some difficulties when placing concrete in the pile. The nature of these problems and the means of overcoming them are described in 3.4.7, and the effects of a bentonite slurry on the skin friction and end-bearing resistance of piles are discussed in 4.2.3 and 4.3.6. Reese, O’Neill and Tourn<sup>(3.11)</sup> recommend a minimum diameter of 600mm for piles installed using slurry techniques, to avoid some of the problems associated with the method.

### 3.3.7 Base and skin grouting of bored and cast-in-place piles

When bored and cast-in-place piles are installed in granular soils, the drilling operation may loosen the soil surrounding the shaft and beneath the base of the pile borehole. Such loosening below the base can cause excessive working load settlements when the majority of the load is carried by end bearing. Base grouting is a means of restoring the original in-situ density and reducing settlements.

Bolognesi and Moretto<sup>(3.12)</sup> described the use of stage grouting to compress the soil beneath the toes of 1.00 to 2.00m bored piles supporting two bridges over the Parana River in Brazil, the piles being drilled with the aid of a bentonite slurry. The soil beneath the pile toes loosened by the drilling operations was subjected to a grouting pressure of up to 10MN/m<sup>2</sup>. The cement grout was introduced through a cylindrical metal basket pierced by a number of holes and filled with uniform gravel (Figure 3.35). The basket, with its upper surface covered by a rubber sheet, was lowered into the borehole suspended from the pile reinforcing cage. The pile was then concreted, followed by the injection of the grout into the basket through a 38mm pipe set in the concrete of the shaft. The uplift caused by the grouting pressure was usually resisted by the skin friction in the pile shaft, but in some cases the pile cap was constructed to provide additional dead-load resistance. Although Bolognesi and Moretto did not mention any weakening at the pile toe caused by the entrapment of bentonite slurry, as described by Reese *et al.*<sup>(3.11)</sup>, the stage-grouting technique would be a useful method of expelling any slurry from beneath the toe of a pile.

A similar method of compacting the soil beneath the base of a bored pile by means of grout pressure

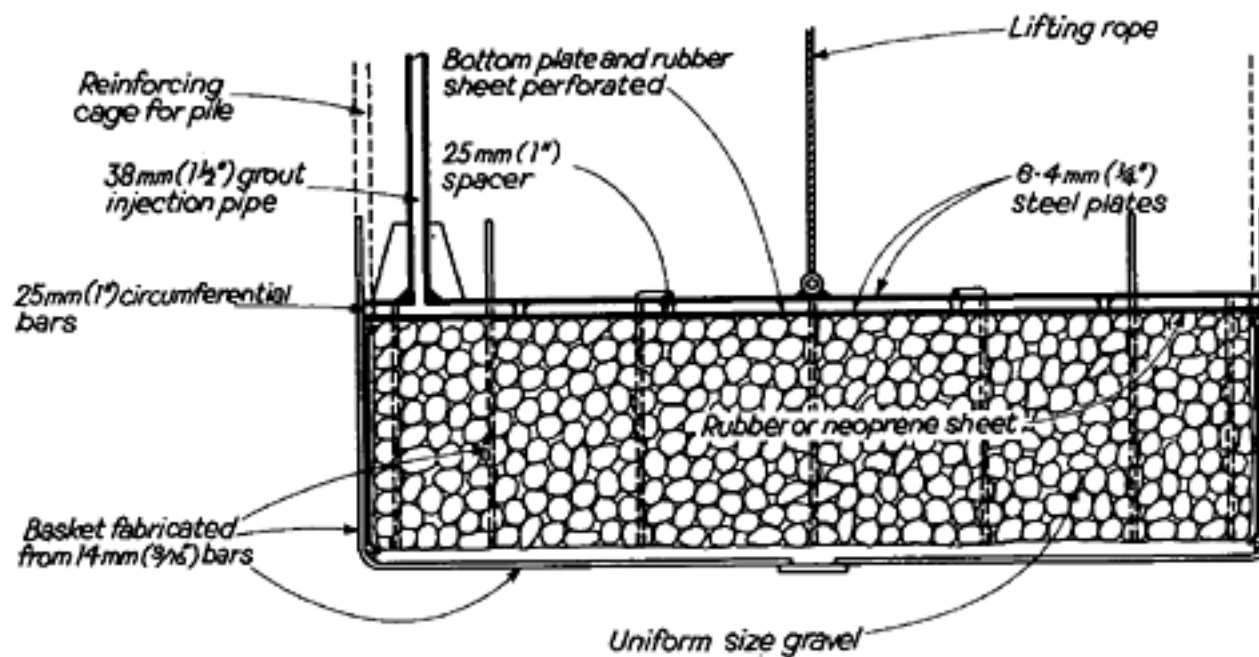


Fig. 3.35 Preloading cell for compressing loosened soil beneath base of bored piles by grouting (after Bolognesi and Moretto)<sup>(3.12)</sup>

is used in the Bauer piling system. After completing the drilling, which can be performed underwater in favourable conditions, the reinforcing cage is lowered to the bottom of the borehole. At the bottom of the cage the bars are welded to a circular plate. A flexible metal sheet covers the whole of the underside of this plate. A grout injection pipe is connected to the space between the plate and the sheet, and a peripheral ring of grout pipes is attached to the reinforcing bars for a predetermined height above the pile base. All grout pipes are extended to a pump and metering unit at the ground surface. The pile is then concreted. After a waiting period to allow the concrete to harden a cement grout is injected into the peripheral injection pipes with the object of bonding the lower part of the pile shaft to the surrounding soil. A further period of a few days is allowed for the grout to harden, then the space between the metal sheet and steel plate is injected with grout under high pressure. The uplift on the steel plate is resisted by the peripheral grout/soil bond stress on the shaft and the soil beneath the flexible sheet is thus compressed. The height of the peripheral grouting above the pile base depends on the required base pressure and hence on the design base resistance of the pile.

Direct injection of cement grout beneath the pile base was used to re-compress sand disturbed by drilling 1.2m diameter bored piles supporting an office building at Blackwall Yard, London. Yeats and O'Riordan<sup>(3.13)</sup> described the installation of a 38.2m deep test pile. The shaft was drilled by rotary auger under a bentonite slurry through the alluvium and stiff to hard clays of the London clay and Woolwich Reading formation into very dense Thanet Sands. The upper 31 metres of the shaft were supported by casing. After completing the drilling four separate grout tube assemblies as shown in [Figure 3.36](#) were lowered to the base of the borehole. The injection holes in the tubes were sleeved with rubber (tubes-à-manchette). The pile shafts were then concreted under bentonite, and 24 hours after this water was injected to crack the concrete surrounding the grout tubes. Base grouting commenced 15 days after concreting. The injections were undertaken in stages with pressures up to 60 bar and frequent checks to ensure the pile head did not lift by more than 1mm.

Similar base grouting techniques were used at six sites in the Docklands area of London beneath piles with diameters in the range of 0.75 to 1.5m<sup>(3.14)</sup>. Comparisons of the base-bearing pressures of piles with grouted and ungrouted bases are discussed in [Section 4.3.4](#).

### 3.4 Procedure in pile installation

Each class of pile employs its own basic type of equipment and hence the installation methods for the various types of pile in each class are the same. Typical methods are described below to illustrate the use of the equipment described in the preceding sections of this chapter. Particular emphasis is given to the precautions necessary if piles are to be installed without unseen breakage, discontinuities or other defects. The installation methods described in this section are applicable mainly to vertical piles. The installation of raking piles whether driven or bored is a difficult operation and is described in 3.4.10.

#### 3.4.1 Driving timber piles

Timber piles are driven by drop hammer or single-acting hammer after pitching them in a conventional piling frame, in crane-suspended leaders, or in trestle guides. The Swedish piling code requires the

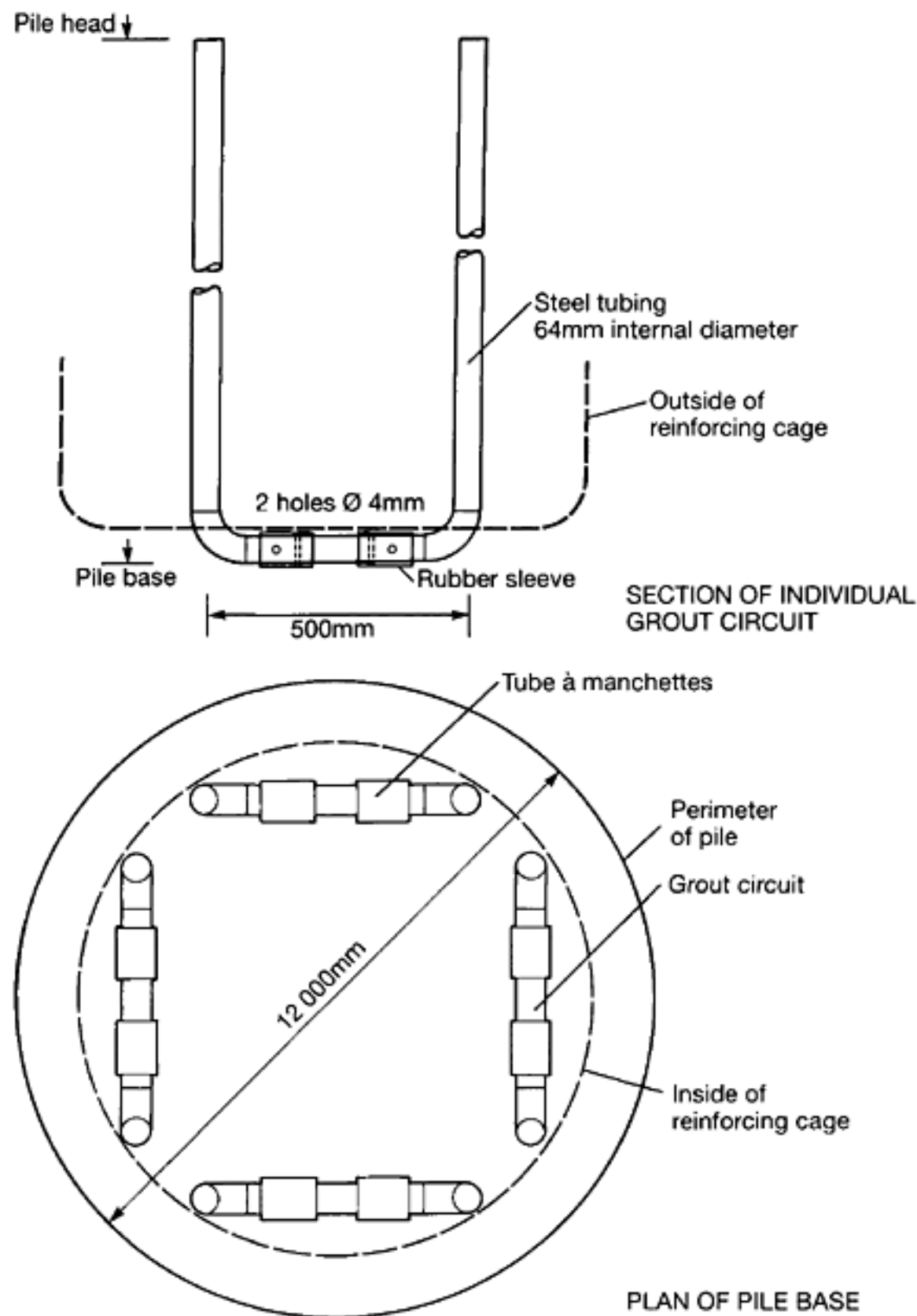


Fig. 3.36 Arrangement of circuits for base-grouting of piles (after Yeats and O'Riordan<sup>(3.13)</sup>)

drop hammer to weigh at least 1.5 times the weight of the pile and helmet with a minimum of 1 tonne. Diesel hammers, unless they are of the light type used for driving trench sheeting, are too powerful and are liable to cause splitting at the toe of the pile. The heads of squared piles are protected by a helmet of the type shown in [Figure 3.21](#). Round piles are driven with their heads protected by a steel hoop. A cap is used over the pile head and hoop, or packing can be placed directly on the head.

Care should be taken to prevent damage to the creosote protection by avoiding the use of slings or hooks which gouge the pile deeply. The damage caused by minor incisions is no more than the scratching caused by stones encountered while driving the piles.

### 3.4.2 Driving precast (including prestressed) concrete piles

The methods of handling the piles after casting and transporting them to the stacking area are described in 2.2.2. They must be lifted from the stacking positions only at the prescribed points. If designed to be lifted at the quarter or third points, they must not at any stage be allowed to rest on the ground on their end or head. Particular care should be taken to avoid over-stressing by impact if the piles are transported by road vehicles. Additional support points should be introduced if necessary. A helmet of the type shown in [Figure 3.21](#) and its packing are carefully centred on the pile, and the hammer position should be checked to ensure that it delivers a concentric blow. The hammer should preferably weigh not less than the pile. BS 8004 requires that the weight or power of the hammer should be sufficient to ensure a final penetration of about 5mm per blow unless rock has been reached. Damage

to the pile can be avoided by using the heaviest possible hammer and limiting the stroke. BS 8004 states that the stroke of a single-acting or drop hammer should be limited to 1.2m and preferably to not more than 1m. The Swedish piling code requires a drop hammer to weigh at least 3 tonne, except that 2-tonne hammers can be used for piles with a maximum length of 10m and a maximum load of 450kN, but a 4-tonne hammer should be used for long piles in compact materials. This code recommends that the drop of the hammer should be limited to 300 to 400mm in soft or loose soils to avoid damage by tensile stresses. The drop should be limited to 300mm when driving through compact granular soils.

The driving of the piles should be carefully watched, and binding by toggle bolts due to the pile rotating or moving off line should be eased. The drop of the hammer should be reduced if cracking occurs, and if necessary the hammer should be changed for a heavier one. After the completion of driving the pile heads should be prepared for bonding into the pile caps as described in 7.7. Hollow piles with a solid end may burst under the impact of the hammer if they become full of water, and holes should therefore be provided to drain off accumulated water. Where a soil plug is formed at the toe of an open-ended pile, water accumulation or arching of the soil within the pile may also result in bursting during driving. Further guidance is given in CIRIA Report PG8(3.15).

### 3.4.3 Driving steel piles

Because of their robustness steel piles can stand up to the high impact forces from a diesel hammer without damage other than the local distortion of the pile head and toe under hard driving. Open-ended tubular or box piles or H-piles can be driven to a limited penetration by a vibrator.

To achieve the required depth of penetration it is sometimes necessary to reduce the base resistance by removing the soil plug which forms at the bottom of an open-ended tubular or box pile. A sandy-soil plug can be removed by simple water jetting. A plug of clay or weak broken rock can be removed by lowering the air-lift device shown in [Figure 3.37](#) down the tube, the soil or broken rock in the plug being loosened by dropping or rotating the riser pipe. A reverse-circulation rig with a rotating cutter ([Figure 3.33](#)) is an efficient means of removing soil if justified by the number and size of the piles. Power augers of the type shown in [Figure 3.27](#) can only be used after the pile has been driven down to its final level where there is space for the crawler-mounted frame carrying the auger to be manoeuvred over the pile head. All the methods described above (except the power auger method) can be used to drill below the pile toe and so ease the driving resistance. However, drilling below the toe also reduces the skin friction and the method may have to be restricted to end-bearing piles. This aspect is discussed further in the section on piling for marine structures (8.3). Because of the delays involved in alternate drilling and driving operations, it is desirable that any drilling to ease the driving resistance should be restricted to only one operation on each pile.

Difficulties arise when it is necessary to place a plug of concrete at the toe of the cleaned-out pile to develop high end-bearing resistance, or to transfer uplift loads from the superstructure to the interior wall of the hollow pile through a reinforcing cage. In such cases a good bond must be developed between

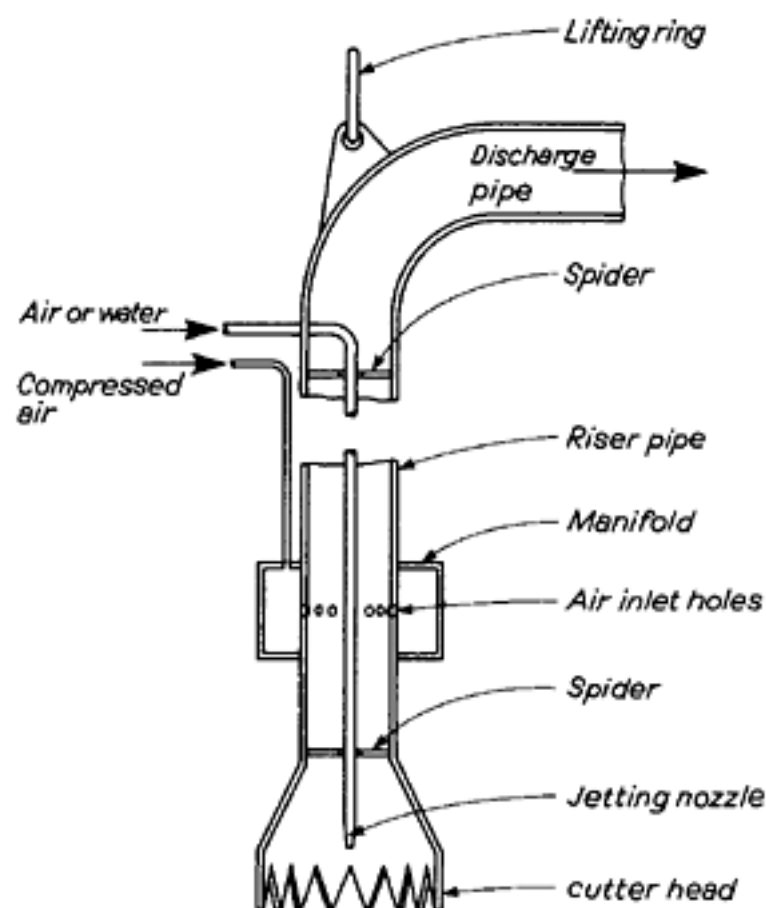


Fig. 3.37 Air-lift for cleaning-out soil from steel tubular piles

the concrete filling and the interior of the steel pile. This requires any adherent soil which remains after removing the soil plug to be cleaned off the pile wall. A sandy soil can be effectively removed by water jetting or by air-lifting, but an adherent clay may require high-pressure water jets to remove it. A rig can be used for this purpose that comprises a central air-lift pipe and a base plate with jetting nozzles around the periphery. The assembly can be half-rotated as necessary. However, the process is tediously slow since the jets tend to drill small-diameter holes in the clay. Equipment has yet to be devised which will quickly and effectively remove the clay adhering to the wall of a pile to a sufficient standard of cleanliness to achieve a good bond with a concrete plug. The procedure for placing the concrete plug in the cleaned-out pile or for completely filling a steel tubular or box pile is similar to that described below for shell piles.

#### 3.4.4 Driving and concreting steel shell piles

Steel shell piles are driven by drop hammers or single-acting hammers acting on the head of an internal mandrel or core which is collapsed to allow it to be withdrawn before placing the concrete. Problems arise with heave when driving shell piles in groups, and distortion or collapse of the shells when driving past obstructions. Shell piles have the advantage that the interior of the shell can be inspected before concrete is placed. This can be done with the aid of light reflected down the pile by a mirror, or by a narrow beam lamp. Distortion of the shells can be detected by lowering a lamp down to the toe. If it disappears wholly or partially then distortion has occurred. This can be corrected by pulling up the shells and redriving them or, in the case of tapered shells, by inserting and redriving a new tapered shell assembly. The problem of heave is discussed in 5.7 to 5.9.

Sometimes some leakage of ground water occurs through shells in quantities which do not justify replacing the damaged units. The water can be removed from the shells before placing the concrete by pumping (if the depth to the pile toe is within the suction lift of the available pump), by an air lift, or by baling. If, after removing the water, the depth of inflow is seen to be less than a few centimetres in five minutes the collected water can again be removed and concrete placed quickly to seal off the inflow. For higher rates of seepage the water should be allowed to fill the pile up to its rest level, and the concrete should then be placed by tremie-pipe as described in 3.4.7.

Concrete placed in 'dry' shell piles is merely dumped in by barrow or chute. It should be reasonably workable with a slump of 100 to 150mm to avoid arching as it drops down a tapered shell or onto the reinforcing cage. The cement content should be such as to comply with the code requirements shown in [Table 2.10](#), or with any special requirements for durability (see [10.3](#)). The American Concrete Institute<sup>(2.9)</sup> states that vibration due to driving adjacent piles has no detrimental effect on fresh concrete in shell piles. Therefore concreting can proceed immediately after driving the shell even though adjacent shells are being driven, provided there are no detrimental effects due to ground heave or relaxation (see [5.7](#)).

#### 3.4.5 The installation of withdrawable-tube types of driven-and-cast-in-place piles

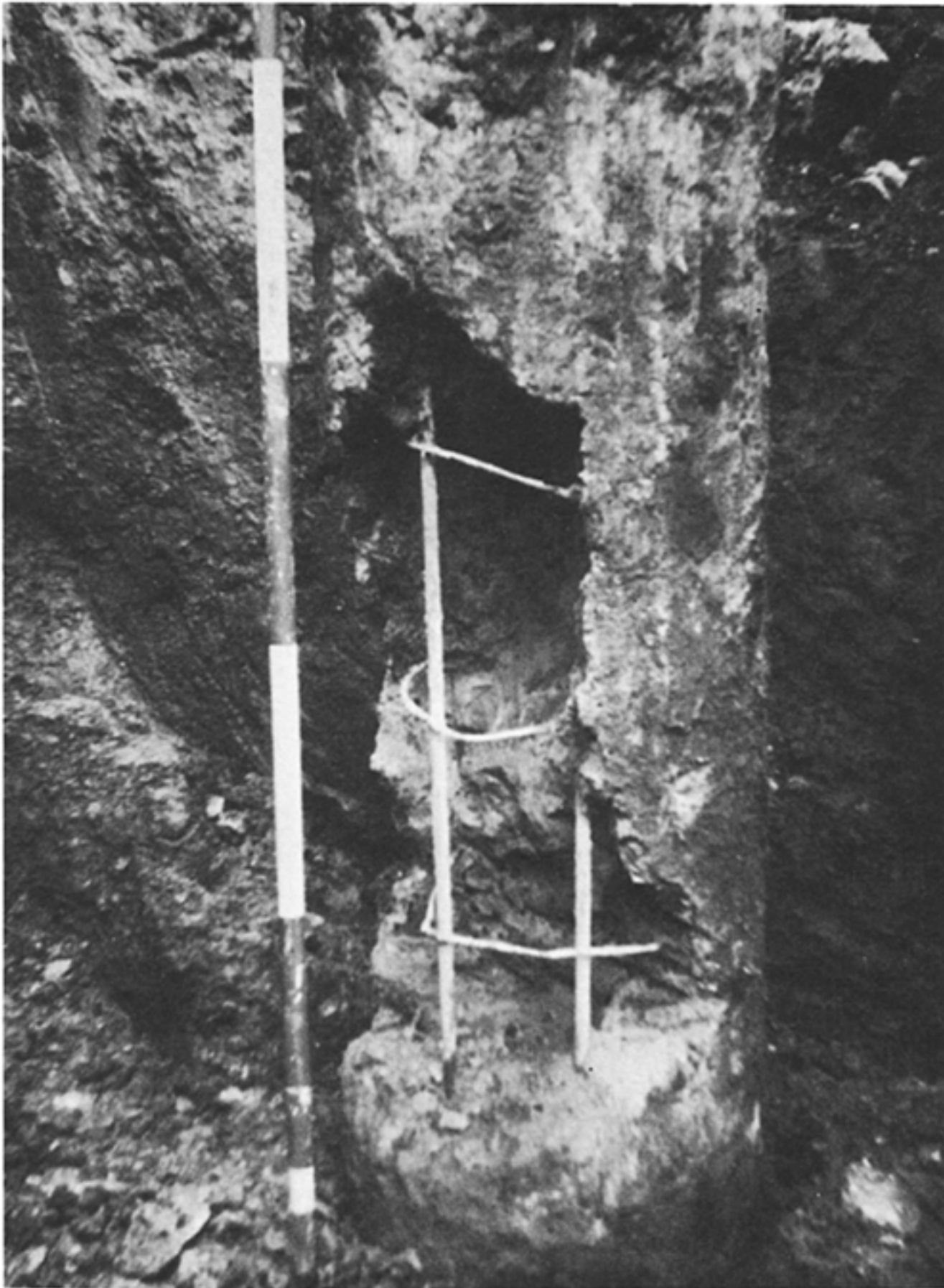
There are no standard procedures for installing driven-and-cast-in-place piles of the types which involve the driving and subsequent withdrawal of a casing tube. The methods for each type of pile are described in 2.3.2. The mix proportions and workability of the concrete depend on the type of pile. Where the concrete is compacted by internal drop hammer a mix is required that is drier than that which is suitable for compaction by vibrating the piling tube. The workability and mix proportions of the concrete should be left to the piling contractor, subject to compliance with the code requirements ([Table 2.10](#)) and the needs regarding durability (see [10.3](#)).

The procedures to be adopted for avoiding 'waisting' or 'necking' of the shaft, or the inclusion of silt pockets and laitance layers, are similar to those adopted for bored-and-cast-in-place piles and are described in the following section of this chapter. Precautions against the effects of ground heave are described in [Section 5.8](#). Because the casing tube is, in all cases, driven down for the full length of the pile, it is essential to ensure that the interior of the tube is free of any incrustations of hardened concrete. Even small incrustations can cause the concrete to arch and jam as the tube is withdrawn. If the reinforcing steel is lifted with the tube the pile shaft is probably defective and should be rejected. Further guidance is given in CIRIA Report PG8<sup>(3.15)</sup>.

#### 3.4.6 The installation of bored-and-cast-in-place piles by power auger equipment

The employment of a power auger for the drilling work in bored-and-cast-in-place piles pre-supposes that the soil is sufficiently cohesive to stand unsupported, at least for a short time. Any upper soft

or loose soil strata or water-bearing layers are ‘cased-off’ by drilling down a casing or pushing the tubes down into the pre-drilled hole by vibrator or the crowd mechanism on the kelly bar. If necessary, ‘mudding-in’ techniques are used at this stage (see [3.3.6](#)). After the auger has reached the deeper and stiffer cohesive soils, the borehole is taken down to its final depth without further support, until the stage is reached when a loosely-fitting tube is lowered down the completed hole. This loose liner may be required for safety purposes when inspecting the pile base before placing the concrete; or if an enlarged base is required, the lining prevents the clay collapsing around the shaft over the period of several hours or more required to drill the under-ream. The loose liner may not be needed for straight-sided piles in weak rocks, or in stable unfissured clays, where there is no risk of collapse before or during the placing of the concrete. However, if the clays are in any degree fissured there is a risk of the walls collapsing during concreting, and thus leading to defects of the type shown in [Figure 3.38](#). Lining tubes *must* be inserted in potentially unstable soils if a visual inspection is to be made of the pile base.



*Fig. 3.38 Defective shaft of bored pile caused by collapse of clay after lifting casing*

Favourable conditions for stability of the borehole are given by care in setting up the rig on a firm level base and attention to maintenance of verticality. Tilting of the rig or violent operation of the auger leads to misalignment and the need for corrective action by reaming the sides. The requirements for the safety of operatives should be rigorously followed (see British Standard 5573, Safety precautions in the construction of large diameter boreholes for piling and other purposes). Casings protecting open pile boreholes should extend above ground level and should be provided with a strong cover. A vibrator of the type described in 3.1.5 is a useful expedient for extracting the upper casings used to support soft clays or loose sand. Support may be needed for the upper part of an enlarged base if this is being formed by hand excavation. The support can be in the form of a 'spider', consisting of a number of hinged arms mounted on a ring. The assembly is lowered down the shaft with the arms in a near-vertical closed position. They are then lifted upwards and outwards and locked in position to form a cone which is pulled up against the clay surface.

The final cleaning-up operation before placing concrete in a bored pile consists of removing large crumbs of soil or trampled puddled clay from the pile base. Any lumps of clay adhering to the walls of the borehole or to the lining tubes should be cleaned off. The reinforcing cage can then be placed and concreting commenced. The time interval between the final cleaning-up and placing concrete should not exceed six hours. If there is any appreciable delay the depth of the pile bottom should be checked against the measured drilled depth before placing the concrete to ensure that no soil has fallen into the hole. If the reinforcing cage extends only part-way down the hole it should be suspended from the top of the pile shaft before commencing to place the concrete.

The concrete used in the pile base and shaft should be easily workable with a slump of 100 to 150mm. Such a mix is self-compacting and does not require ramming or vibrating. The mix proportions should be such as to ensure compliance with the requirements regarding strength and minimum cement content of the relevant code of practice ([Table 2.11](#)), or with any special requirements for durability (see [10.3](#)). A dry mix should be used for the first few charges of concrete if the pile base is wet, or if hand spreading is adopted in an enlarged base. The concrete in the shaft is fed through a hopper or chute placed centrally over the pile to direct it clear of the sides and the reinforcement. After completing concreting, the lining tubes are withdrawn. If a loose liner is used inside an upper casing, the former is lifted out as soon as the concrete extends above the base of the outer tube. Vibrators can be used to extract the casing. The quantity of concrete placed in the shaft should allow for the outward slumping which takes place to fill the space occupied by the tube and any overbreak of the soil outside it. At this final stage there is inevitably some laitance which has risen to the top of the concrete. The laitance may be diluted and contaminated with water and silt expelled from around the casing as the concrete slumps outwards to fill the gap. Thus the level of the concrete should be set high so that this weak laitance layer can be broken away before bonding the pile head onto its cap. The terms of the contract should make it clear whether or not this removal should be performed by the piling contractor.

The concrete in a pile shaft may be required to be terminated at some depth below ground level, *e.g.* when constructing from ground surface level, piles designed to support a basement floor. It is a matter of some experience to judge the level at which the concrete should be terminated and it is difficult to distinguish between fluid concrete and thick laitance when plumbing the level with a float. Fleming and Lane([3.16](#)) recommend the following tolerances for all conditions.

Concrete cast under water: +1.5 to +3m

Concrete cast in dry uncased holes: +75 to +300mm

Concrete cast in cased holes, the greater of

$$a \text{ +75mm to +300mm + cased length/15}$$

or

$$b \text{ +75mm to +300mm + [depth to casting level - 900mm/10]}$$

The 1988 edition of the Institution of Civil Engineers Specification for Piling specifies casting tolerances for three conditions of placing concrete in pile boreholes with and without temporary casing. The ground surface or piling platform level is defined as the 'commencing surface'. The three conditions all refer to a situation where the cut-off level is at a depth  $H$  below the commencing surface such that  $H$  is between 0.15 and 10m. The conditions are:

(a) Concrete placed in dry boreholes using temporary casing and without permanent lining: The casting tolerance in metres is specified to be  $0.3 + H/12 + C/8$ , where  $C$  is the length of temporary casing below the commencing surface.

(b) Concrete placed in dry boreholes within permanent tubes or permanent casings or where cut-off levels are in stable ground below the base of any casing: The casting tolerance in metres is specified to be  $0.3 + H/10$

(c) Concrete placed under water or a drilling fluid: The casting tolerance in metres is specified to be  $1.0 + H/12 + C/8$

The reader is referred to the ICE Specification for the various qualifications to the above tolerances. It will be noted that the casing length rather than the diameter is a factor which influences casting tolerances. This reflects the problems which occur when extracting the temporary casing.

The use of a permanent casing in the form of a light-gauge metal sleeve surrounding a pile shaft in soft clays or peats was described in 2.4.2. This sleeving cannot be used within a temporary lining tube where the latter has to be withdrawn in a long length by means of a vibrator or by jacking. This involves the risk of distortion or jamming of the sleeve, which is then lifted while raising the temporary tube with disastrous effects on the concrete in the pile shaft. The sleeve can be used within an outer temporary liner where the depth of soft clay is shallow, and it can be used in conjunction with a casing oscillator which keeps the outer tube free of any jamming by the sleeve. There are no problems of using the light-gauge sleeve where power auger drilling can be performed to produce a stable hole without employing a temporary outer lining tube.

Unfortunately, defects in a pile shaft of the type shown in [Figure 3.38](#) are by no means uncommon, even when placing a workable concrete in the dry open hole of a large-diameter bored pile. Defects can take the form of large unfilled voids, or pockets of clay and silt in the concrete. Some causes of these defects are listed below.

1. Encrustations of hardened concrete or soil on the inside of the lining tubes can cause the concrete to be lifted as the tubes are withdrawn, thus forming gaps in the concrete. *Remedy:* The tubes must be clean before they are lowered down the bore-hole.
2. The falling concrete may arch and jam across the lining tube or between the tubes and the reinforcement. *Remedy:* Use a concrete of sufficient workability to slump easily down the hole and fill all voids.
3. The falling concrete may jam between the reinforcing bars and not flow outwards to the walls of the borehole. *Remedy:* Ensure a generous space between the reinforcing bars. The cage should be stiff enough to prevent it twisting or buckling during handling and subsequent placing of concrete. Widely-spaced stiff hoops are preferable to helical binding. Check that the bars have not moved together before the cage is lowered down the hole.
4. Lumps of clay may fall from the walls of the borehole or lining tubes into the concrete as it is being placed. *Remedy:* Always use lining tubes if the soil around the borehole is potentially unstable and do not withdraw them prematurely. Ensure that adhering lumps of clay are cleaned off the tubes before they are inserted and after completing drilling.
5. Soft or loose soils may squeeze into the pile shaft from beneath the base of the lining tubes as they are withdrawn, forming a 'waisted' or 'necked' shaft. *Remedy:* Do not withdraw the casing until the placing of the concrete is complete. Check the volume of concrete placed against the theoretical volume and take remedial action (removal and replacement of the concrete) if there is a significant discrepancy.
6. If bentonite has been used for 'mudding-in', the hydrostatic pressure of the bentonite in the annulus, which is disturbed on lifting the casing, may be higher than that of the fluid concrete, thus causing the bentonite to flow into the concrete. This is a serious defect and is difficult to detect. It is particularly liable to happen if the concrete is terminated at some depth below the top of the 'mudded-in' casing. *Remedy:* Keep a careful watch on the level of the bentonite gel when the casing is lifted. Watch for any changes in level of the concrete surface and for the appearance of bentonite within the concrete. If inflow of the bentonite has occurred the defective concrete must be removed and replaced and the 'mudding-in' technique must be abandoned.
7. Infiltration of ground water may cause gaps, or honeycombing of the concrete. *Remedy:* Adopt the techniques for dealing with ground water in pile boreholes described in the following section.

Further guidance on the installation procedures is given in CIRIA Report PG2([3.17](#)).

### 3.4.7 Concreting pile shafts under water

Ground water in pile boreholes can cause serious difficulties when placing concrete in the shaft. A depth of inflow of only a few centimetres in, say, five minutes which has trickled down behind the lining tubes or has seeped into the pile base can be readily dealt with by baling or pumping it out and then placing dry concrete to seal the base against any further inflow. However, larger flows can cause progressive increases in the water content of the concrete, weakening it, and forming excess laitance.



Fig. 3.39 Defective shaft of bored pile caused by cement being washed out of unset concrete

A strong flow can even wash away the concrete completely. The defective piles shown in [Figure 3.39](#) were caused by the flow of water under an artesian head from a fissured rock on which the bored piles were bearing after the boreholes had been drilled through a soft clay overburden. The lined boreholes were pumped dry of water before the concrete was placed, but the subsequent ‘make’ of water was sufficiently strong to wash away some of the cement before the concrete has set. The remedial action in this case was to place dry concrete in bags at the base of the pile borehole and then to drive precast concrete sections into the bags.

In all cases of strong inflow the water must be allowed to rise to its normal rest level (or better, to be topped up above this level to stabilize the pile base) and then to concrete the pile using a *tremie pipe*.

Although a bottom-opening bucket is sometimes used instead of a tremie-pipe for placing concrete in pile boreholes, the author as a general rule condemns this practice. This is because the crane operator handling the bucket cannot tell, by the behaviour of the crane rope, whether or not he has lowered the bucket to the correct level into the fluid concrete before he releases the hinged flap. If he releases the bucket flap prematurely, the concrete will flow out through the water and the cement will be washed out. On the other hand, if he plunges the bucket too deeply it will disturb the concrete already placed when it is lifted out. The bottom-dumping bucket method has no advantage over the tremie pipe and the author would use it only if a pile were large enough for the lowering and dumping to be controlled by a diver.

A method sometimes used for concreting piles under water involves the insertion of a grout pipe to the bottom of the borehole. Clean coarse aggregate is then placed around the pipe, and the casing is lifted out. A cement-sand grout is next injected through the pipe, the grout rising up the borehole to fill the voids in the aggregate. This technique is known as ‘prepacked concrete’ but the author would not recommend it in preference to placing concrete by tremie pipe. This is because the water in a bored pile is rarely clean, and the silt stirred up by dumping the aggregate tends to get dispersed on to the surface of the stones. It is then displaced by the rising column of grout and tends to form layers or pockets of muddy laitance.

The procedure for drilling pile boreholes with support by a bentonite gel is described in 3.3.6 and in CIRIA Report PG3(3.18). Problems can be caused when placing concrete in a bentonite-filled hole. A tremie-pipe is used, and there must be a sufficient hydrostatic pressure of concrete in the pipe above bentonite level to overcome the external head of the slurry, to rupture the gel, and to overcome friction in the tremie pipe. Sometimes a dispersing agent is added to the bentonite to break down the gel before placing the concrete. Reese *et al.*(3.11) recommend using a clean trémie pipe having a diameter of at least 300mm and concrete with a slump of at least 150mm. Where the mud becomes flocculated and heavily charged with sand (*i. e.* has a specific gravity greater than 1.35 to 1.4) it should be replaced by a lighter mud before placing the concrete. Circumferential steel should be kept to a minimum. A flap valve should be used on the end of the tremie pipe rather than the paper bag or polyethylene ‘go-devil’. The concrete in the piled foundations for the Wuya Bridge, Nigeria(3.19) was placed under bentonite. The piles were 18 to 21m deep and a mud density of 1600kg/m<sup>3</sup> was necessary to prevent the sides collapsing. The concrete failed to displace the gel which was stiffened by the high ground temperatures and jamming occurred, especially when placing was suspended to remove each section of the tremie pipe. The problem was finally overcome by increasing the workability of the concrete by means of a plasticizer together with a retarder. The tremie pipe was lifted out as a single unit to avoid the delays in breaking the pipe joints.

### **3.4.8 The installation of bored-and-cast-in-place piles by grabbing, vibratory and reverse-circulation rigs**

The use of either grabbing, vibratory, ‘or reverse-circulation machines for drilling pile boreholes can involve continuous support by lining tubes which may or may not be withdrawn after placing the concrete. In all three methods the tubes may have to follow closely behind the drilling in order to prevent the collapse of the sides and the consequent weakening of skin friction. The boreholes must be kept topped up with water in order to avoid ‘blowing’ of the pile bottom as a result of the upward flow of the ground water. This is particularly necessary when drilling through water-bearing sand layers interbedded with impervious clays.

Grabbing in weak rocks can cause large accumulations of slurry in the boreholes which make it difficult to assess the required termination level of the pile in sound rock. The slurry should be removed from time to time by baling or by air-lift pump with a final cleaning-up before placing the concrete.

The techniques of placing concrete in ‘dry’ holes, or under water, are exactly the same as described in 3.4.6 and 7.

### **3.4.9 The installation of bored-and-cast-in-place piles by tripod rigs**

Pile boreholes in clays are drilled by a clay cutter operated from a tripod rig. Water should not be poured down the hole to soften a stiff clay, or used to aid removal of the clay from the cutter as this causes a reduction in skin friction. When drilling in cohesionless soils the lining tubes should follow closely behind the drilling to avoid overbreak, and the addition of water is needed to prevent ‘blowing’ and to facilitate the operation of the baler or shell.

Piles drilled by tripod rigs are relatively small in diameter, requiring extra care when placing the concrete as this is more likely to jam in the casing tubes when they are lifted. Curtis(3.20) suggests checking the concrete level by hanging a float on top of the concrete and comparing its measurement from the top of the tube with the amount of tube extracted. He also suggests that the position of the reinforcing cage should be checked by a ‘tell-tale’ wire and indicator. Problems can occur when placing concrete in raking piles. Internal ramming is impossible as the rammer catches on the reinforcing cage. A high slump concrete is necessary with special precautions being taken to prevent the reinforcement being lifted with the lining tubes.

### **3.4.10 The installation of raking piles**

The advantages of raking piles in resisting lateral loads are noted in Chapters 6 and 8. However, the installation of such piles may result in considerable practical difficulties, and they should not be employed without first considering the method of installation and the ground conditions. If the soil strata are such that the piles can be *driven* to the full penetration depth without the need to drill out a soil plug or to use jetting to aid driving, then it should be feasible to adopt raking piles up to a maximum rake of 1 horizontal to 2 vertical. However, the efficiency of the hammer is reduced due to the friction of the ram in the guides. It may therefore be necessary to use a more powerful hammer than that required for driving vertical piles to the same penetration depth.

The vertical load caused by the pile and hammer on the leaders of the piling frame must be taken into consideration. Also when driving piles by guides without the use of leaders the bending stresses caused by the weight of the hammer on the upper end of the pile must be added to the driving stresses and a check should be made to ensure that the combined stresses are within allowable limits.

The principal difficulties arise when it is necessary to drill ahead of an open-ended pile to clear boulders or other obstructions, using the methods described in 3.3.4. When the drill penetrates below the shoe of the pile tube it tends to drop by gravity and it is then likely to foul the shoe as it is pulled out to resume further driving. Similarly, under-reaming tools are liable to be jammed as they are withdrawn. The risks of fouling the drilling tool are less if the angle of rake is small (say 1 in 10 or less) and the drill string is adequately centralized within the piling tube. However, the drill must not be allowed to penetrate deeply below the toe of the pile. This results in frequent alternations of drilling and driving with consequent delays as the hammer is taken off to enter the drill, followed by delays in entering and coupling up the drill string, and then removing it before replacing the hammer.

Difficulties also arise when installing driven-and-cast-in-place piles by means of an internal drop hammer, due to the friction of the hammer on the inside face of the driving tube. Installers of these piles state that a rake not flatter than 1 in 3.7 is possible.

Power augers can drill for pile boreholes at angles of rake of up to 1 in 3 but when casing is necessary to support the pile borehole the same difficulties arise with the jamming of the bucket or auger beneath the toe of the casing. Generally, bored piles which require the use of a casing are installed by tripod rigs, and they cannot be installed at rakes flatter than about 1 in 3.

The American Concrete Institute<sup>(2.9)</sup> recommends using an over-sanded mix for placing concrete in raking pile shells or tubes. A concrete mix containing 480kg/m<sup>3</sup> of coarse aggregate with a slump of 100mm is recommended. This mix can be pumped down the raking tube.

### 3.4.11 Positional tolerances

It is impossible to install a pile, whether by driving, drilling or jacking, so that the head of the completed pile is always exactly in the intended position or that the axis of the pile is truly vertical or at the specified rake. Driven piles tend to move out of alignment during installation due to obstructions in the ground or the tilting of the piling frame leaders. Driving piles in groups can cause horizontal ground movements which deflect the piles. In the case of bored piles the auger can wander from the true position or the drilling rig may tilt due to the wheels or tracks sinking into soft ground. However, controlling the positions of piles is necessary since misalignment affects the design of pile caps and ground beams (see 7.8 and 7.9), and deviations from alignment may cause interference between adjacent piles in a group or dangerous concentrations of load at the toe (Figure 5.7). Accordingly, codes of practice specify tolerances in the position of pile heads or deviations from the vertical or intended rake. If these are exceeded, action is necessary either to redesign the pile caps as may be required or to install additional piles to keep the working loads within the allowable values.

Some codes of practice requirements are as follows:

*BS 8004*: Driven and cast-in-place, and bored and cast-in-place piles should not deviate by more than 1 in 75 from the vertical, or more than 75mm from their designed position at the level of the piling rig. Larger tolerances can be considered for work over water or raking piles. A deviation of up to 1 in 25 is permitted for bored piles drilled at rakes of up to 1 in 4.

*BS Code of Practice for Maritime Structures*: A deviation of up to 1 in 100 is permitted for vertical piles driven in sheltered waters or up to 1 in 75 for exposed sites. The deviation for raking piles should not exceed 1 in 30 from the specified rake for sheltered waters or 1 in 25 for exposed sites. The centre of piles at the junction with the superstructure should be within 75mm for piles driven on land or in sheltered waters. Where piles are driven through rubble slopes the code permits a positional tolerance of up to 100mm, and for access trestles and jetty heads a tolerance of 75mm to 150mm is allowed depending on the exposure conditions.

*Institution of Civil Engineers*<sup>(2.1)</sup>: Positional—Maximum deviation of centre point of pile to centre point on drawing not more than 75mm, but additional tolerance for pile cut-off below ground level. Verticality—pile to be made vertical within tolerance of 1 in 100 at commencement of installation. Maximum deviation of finished pile from the vertical is 1 in 75. For raking piles set pile to within 1 in 50 at commencement of installation. Maximum deviation of finished pile from the specified rake is 1 in 25 for piles raking up to 1:6 and 1 in 15 for piles raking more than 1:6. Relaxation permitted in exceptional circumstances subject to implications of this action.

*D.T.U. No 13.2 (France)*: A maximum positional tolerance of 150mm is permitted, but the code states that it is normally expected that driven piles and cased bored piles can be installed to a tolerance of

60mm. Ordinary bored piles, including auger piles, are expected to be installed to a tolerance of 120mm. The axis of an isolated pile must not deviate by more than 3%. The axis of a pile in a group must not deviate by more than 2% if the inclination of all piles in the groups is in the same direction.

*New York City Building Code:* If the axis deviates by more than 4% from plumb or the specified rake the foundation design shall be modified to resist the resulting vertical and lateral forces. A tolerance of 75mm for the designed location is permitted without a reduction in the load capacity of a group. Where piles deviate by more than this amount the true loading is determined by taking into account the eccentricities as observed from a survey of the actual pile head positions. If the calculated load on any pile is in excess of 110% of the allowable load capacity, a correction is made by installing additional piles or by other methods of redistributing pile loads to reduce the maximum pile load to not more than 110% of the allowable capacity.

*American Concrete Institute Recommendations:* The position of the pile head is to be within 75 to 150mm for the normal usage of piles beneath a structural slab. The axis may deviate by up to 10% of the pile length for completely embedded vertical piles or for all raking piles, provided the pile axis is driven straight. For vertical piles extending above the ground surface the maximum deviation is 2% of the pile length, except that 4% can be permitted if the resulting horizontal load can be taken by the pile-cap structure. For bent piles the allowable deviation is 2% to 4% of the pile length depending on the soil conditions and the type of bend (*e.g.* sharp or gentle). Severely bent piles must be evaluated by soil mechanics' calculations or checked by loading tests.

The significance of positional tolerance to piling beneath deep basements is noted in [Section 5.9](#).

### 3.5 Constructing piles in groups

So far only the installation of single piles has been discussed. The construction of groups of piles can have cumulative effects on the ground within and surrounding the pile group. These effects are occasionally beneficial but more frequently have deleterious effects on the load-settlement characteristics of the piles and can damage surrounding property. Precautions can be taken against these effects by the installation methods and sequence of construction adopted. Because the problems are more directly concerned with the bearing capacity and settlement of the group as a whole, rather than with the installation of the piles, they are discussed in 5.7 to 5.9.

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## CHAPTER 4

# Calculating the resistance of piles to compressive loads

### 4.1 General considerations

#### 4.1.1 The basic approach to the calculation of pile resistance

The numerous types of pile and the diversity in their methods of installation have been described in Chapters [2](#) and [3](#). Each different type and installation method disturbs the ground surrounding the pile in a different way. The influence of this disturbance on the skin friction and end-bearing resistance of piles has been briefly mentioned (see [Section 1.3](#)). This influence can improve or reduce the bearing capacity of the piles and thus a thorough understanding of how the piles are constructed is essential to the formulation of a practical method of calculating loading capacity.

The basic approach used in this chapter to calculate the resistance of piles to compressive loads is the ‘static’ or soil mechanics approach. In recent years much attention has been given by research workers to calculation methods based on ‘pure’ soil mechanics theory. They postulate that the skin friction on a pile shaft can be determined by a simple relationship between the coefficient of earth pressure ‘at rest’, the effective over-burden pressure and the drained angle of shearing resistance of the soil, but they recognize that the coefficient of earth pressure must be modified by a factor which takes into account the method of pile installation. Similarly they believe that the end-bearing resistance of a pile can be calculated by classical soil mechanics theory based on the undisturbed shearing resistance of the soil surrounding the pile toe. The importance of the settlement of the pile or pile groups at the working load is recognized and methods have been evolved to calculate this settlement, based on elastic theory and taking into account the transfer of load in skin friction from the pile to the soil.

The concepts of this research work commenced on quite simple lines, the two main groups, namely driven piles and bored piles, only being differentiated when considering pile behaviour. However, as the work progressed from the laboratory to the field, particularly in the study of the behaviour of instrumented full-scale piles, it was observed that there were very fundamental departures from classical soil mechanics theory, and the all-important effects of installation procedures on pile behaviour were realized. The installation of piles results in highly complex conditions developing at the pile-soil interface which are often quite unrelated to the original undisturbed state of the soil, or even to the fully remoulded state. The pore water pressures surrounding the pile can vary widely over periods of hours, days, months or years after installation, such that the simple relationships of skin friction to effective overburden pressure are unrealistic. Similarly when considering deformations of a pile group under its working load, any calculations of the transfer of load that are based on elastic theory which do not take account of soil disturbance for several diameters around the pile shaft and beneath the toe are quite meaningless.

Therefore while the author bases his approach to the calculation of pile carrying capacity on soil mechanics methods, this approach is simply an empirical one which relates known pile behaviour to simple soil properties such as relative density and undisturbed shearing strength. These can be regarded as index properties to which empirical coefficients can be applied to arrive at unit values for the skin-friction and end-bearing resistances.

Observations made on full-scale instrumented piles([4.1](#)) have so far only served to reveal the extreme complexities of the problems, and have shown that there is no simple fundamental design method. The empirical methods set out in this chapter have been proved by experience to be reliable for practical design of light to moderately heavy loadings on land-based or near-shore marine structures. Special

consideration using more complex design methods are required for heavily loaded marine structures in deep water. The engineer is often presented with inadequate information on the soil properties. He then has to decide whether to base his designs on conservative values with an appropriate safety factor without any check by load testing, or merely to use the design methods to give a preliminary guide to pile diameter and length and then to base the final designs on an extensive field testing programme with loading tests to failure. Such testing is always justified on a large-scale piling project. Proof-load testing as a means of checking workmanship is a separate consideration (see [Section 11.4](#)).

Where the effective overburden pressure is an important parameter for calculating the ultimate bearing capacity of piles (as is the case for granular soils) account must be taken of the effects of a rise in ground-water levels. This may be local or may be a general rise, due for example to seasonal flooding of a major river, or a long-term effect such as the predicted large general rise in ground-water levels in Greater London.

#### 4.1.2 The behaviour of a pile under load

For practical design purposes engineers must base their calculations of carrying capacity on the application of the load at a relatively short time after installation. The reliability of these calculations is assessed by a loading test which is again made at a relatively short time after installation. However, the effects of time on carrying capacity must be appreciated and these are discussed in 4.2.4 and 4.3.8.

When a pile is subjected to a progressively increasing compressive load at a rapid or moderately rapid rate of application, the resulting load-settlement curve is as shown in [Figure 4.1](#). Initially the pile-soil system behaves elastically. There is a straight-line relationship up to some point A on the curve and if the load is released at any stage up to this point the pile head will rebound to its original level. When the load is increased beyond point A there is yielding at, or close to, the pile-soil interface and slippage occurs until point B is reached, when the maximum skin friction on the pile shaft will have been mobilized. If the load is released at this stage the pile head will rebound to point C, the amount of 'permanent set' being the distance OC. The movement required to mobilize the maximum skin friction is quite small and is only of the order of 0.3 to 1% of the pile diameter. The base resistance of the pile requires a greater downward movement for its full mobilization, and the amount of movement depends on the diameter of the pile. It may be in the range of 10 to 20% of the base diameter. When the stage of full mobilization of the base resistance is reached (point D in [Figure 4.1](#)) the pile plunges downwards without any further increase of load, or small increases in load produce increasingly large settlements.

If strain gauges are installed at various points along the pile shaft from which the compressive load in the pile can be deduced at each level, the diagrams illustrated in [Figure 4.2](#) are obtained, which show the transfer of load from the pile to the soil at each stage of loading shown in [Figure 4.1](#). Thus when loaded to point A virtually the whole of the load is carried by skin friction on the pile shaft and there is little or no transfer of load to the toe of the pile ([Figure 4.2a](#)). When the load reaches point B the pile shaft is carrying its maximum skin friction and the pile toe will be carrying some load ([Figure 4.2b](#)). At Point D there is no further increase in the load transferred in skin friction but the base load will have reached its maximum value ([Figure 4.2c](#)).

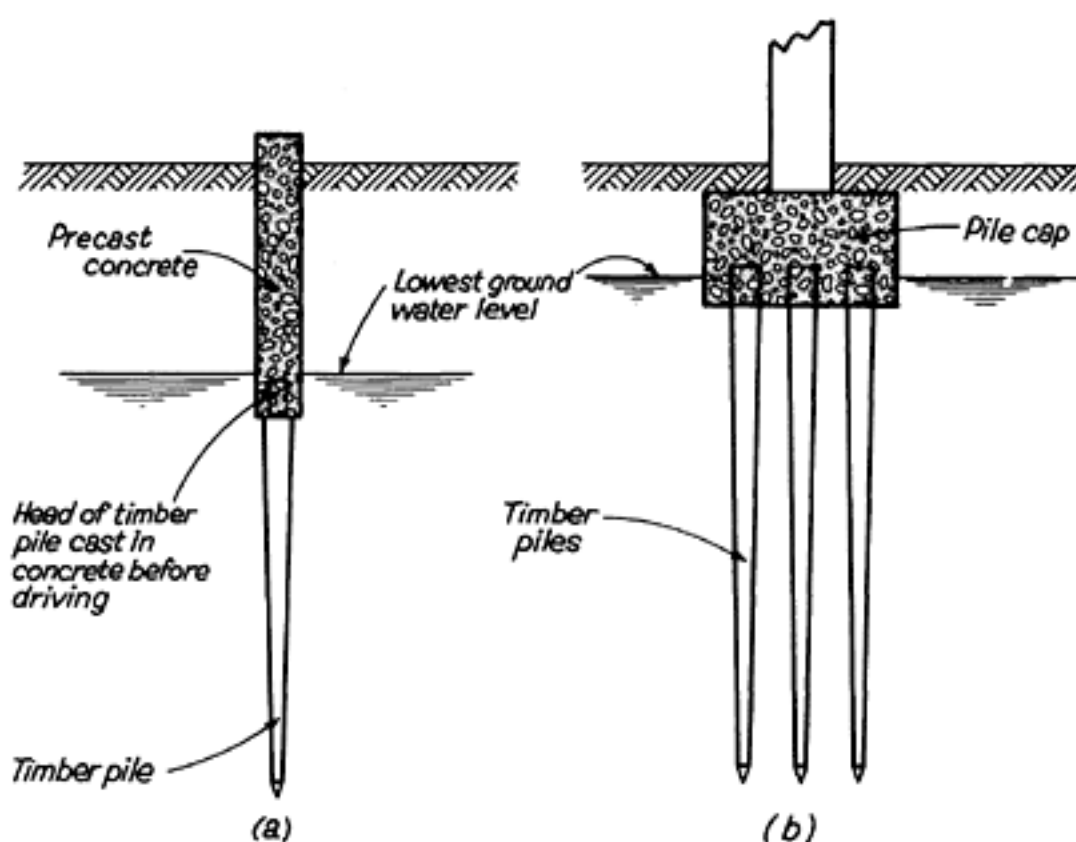


Fig. 4.1 Load-settlement curve for compressive load to failure on pile

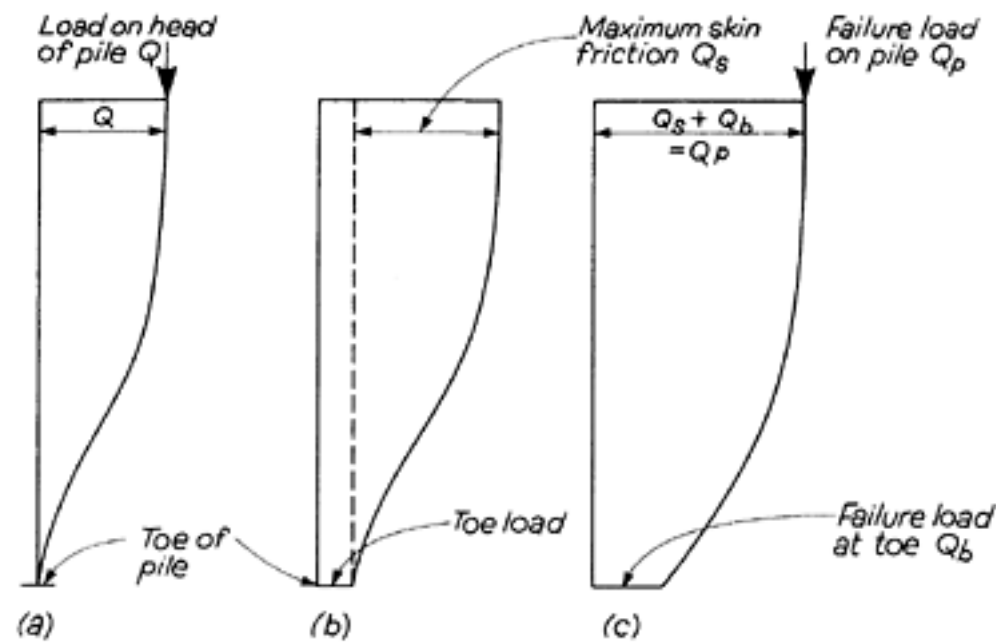


Fig. 4.2 Load transfer from head of pile to shaft

(a) At point A on load-settlement curve in [Figure 4.1](#)

(b) At point B on load-settlement curve in [Figure 4.1](#)

(c) At point D on load-settlement curve in [Figure 4.1](#)

The concept of the separate evaluation of shaft friction and base resistance forms the basis of all ‘static’ calculations of pile carrying capacity. The basic equation is

$$Q_p = Q_b + Q_s - W_p \quad \dots (4.1)$$

where  $Q_p$  is the ultimate resistance of the pile,  $Q_b$  is the ultimate resistance of the base,  $Q_s$  is the ultimate resistance of the shaft, and  $W_p$  is the weight of the pile.

The components  $Q_s$  and  $Q_b$  of the failure load  $Q_p$  are shown at the final loading stage in [Figure 4.2c](#). Usually the weight of the pile ( $W_p$ ) is small in relation to  $Q_p$  and this term is generally ignored in equation 4.1. However, it is necessary to provide for  $W_p$  in situations such as piling for marine structures in deep water, where a considerable length of pile extends above the sea bed.

Eurocode 7 expresses equation 4.1 in somewhat different terms.  $Q_p$  is denoted as  $Q$ , the *design* bearing capacity of the pile,

$Q_b = Q_{bk}/\gamma_B$  is the design base resistance and  $Q_s = Q_{sk}/\gamma_s$  is the design shaft resistance.  $Q_{bk}$  and  $Q_{sk}$  are further broken down and defined as

$$Q_{bk} = q_{bk} A_b \quad \dots (4.1a)$$

and

$$Q_{sk} = \sum_{i=1}^n q_{sik} A_{si} \quad \dots (4.1b)$$

where  $A_b$  is the nominal plan area of the base of the pile

$A_{si}$  is the nominal surface area of the pile in soil layer  $i$

$q_{bk}$  is the characteristic value of the resistance per unit area of the base

$q_{sik}$  is the characteristic value of the resistance per unit of the shaft in layer  $i$ .

$\gamma_b$  and  $\gamma_s$  are partial safety factors.  $\gamma_b$  is presently given in the draft Eurocode as 1.3, 1.6 and 1.45 for driven, bored and continuous flight auger piles respectively, and  $\gamma_s$  is given as 1.3 both for driven and bored piles. The characteristic values  $q_{bk}$  and  $q_{sik}$  are derived from calculation rules based on established correlations between the results of static load tests and the results of field or laboratory soil tests. The Code requires that the characteristic values  $q_{bk}$  and  $q_{sik}$  do not exceed the measured bearing capacities used to establish the correlation divided by 1.5 on average. This additional factor is used essentially to allow for uncertainties in the calculated method or scatter in the values on which the correlation is based.

#### **4.1.3 Definition of failure load**

The loading corresponding to point D on the load-settlement curve in [Figure 4.1](#) represents the ultimate resistance, or ultimate limit state, of the pile and is defined as the stage at which there is general shear failure of the soil or rock beneath the pile toe. However, this stage is of academic interest to the structural designer. A piled foundation has failed in its engineering function when the relative settlement between adjacent single piles or groups of piles causes intolerable distortion of the structural

framework, or damage to claddings and finishes. This stage may be represented by some point such as E on the load-settlement curve ([Figure 4.1](#)). Thus structural failure will have occurred at a load lower than the ultimate resistance of the pile. Various criteria of assessing failure loads on piles from the results of loading tests are listed in [Section 11.4](#).

#### 4.1.4 Allowable loads on piles

A perfect design method for calculating allowable loads on piles would be one which predicted the load-deformation curve through all stages from initial loading to the point of ultimate failure. From such a predicted curve the structural designer would be able to distribute the load on to the piles to keep the deformation of the structure within tolerable limits. The foundation engineer would be able to satisfy himself that there was an adequate safety factor on the ultimate resistance to provide a safeguard against accidental overloading of the piles, and to allow for variations in the properties of the soil. Unfortunately, such a design method has not been developed to an extent to which it can be generally applied to all types of piles. In the present state of knowledge the best that can be done in advance of loading or dynamic tests on full-scale trial piles, is to make loading tests on plates set at the bottom of boreholes or test pits. The load-settlement relationship for the plate is then used to predict the settlement of the pile base (see [Section 4.6](#)). Alternatively the base settlement can be calculated from a knowledge of the deformation properties of the soil or rock obtained from field or laboratory tests (see [Sections 4.6](#) and [4.7](#)). However, it is still necessary to calculate the relative proportions of load carried by the shaft and the base of the pile. There is no sound theoretical basis for such calculations since the mechanism of load-transfer from pile to soil at loads less than ultimate is affected by the installation method, in just the same way as the ultimate resistances in skin friction and end bearing depend on the installation method.

The usual approach to the problem of assessing allowable pile loads is to predict the *ultimate resistance* of the pile from a knowledge of the physical properties of the undisturbed soil, and then to apply an arbitrary safety factor to this value to obtain the allowable load. The value of the safety factor depends on the variability or otherwise of the soil properties, the confidence or lack of confidence of the engineer in the empirical methods of predicting the ultimate pile resistance, and the tolerable movement of the pile at the working load.

Experience of a very large number of loading tests taken to failure, made on piles of diameters up to about 600mm and of many types both in clays and sands, has shown that if a safety factor of 2.5 is applied, namely

$$\text{allowable load} = \frac{\text{ultimate pile resistance}}{2.5},$$

then the settlement at the working load is unlikely to exceed 10mm. However, because of the uncertainties already referred to in calculating the ultimate load it is necessary to make a loading test on a trial pile of the same type, length and size as that proposed for the permanent works, or to make several such tests to confirm that the settlement at the working load is acceptable. Preferably these tests should be taken to the stage of ultimate failure to establish the real safety factor. In the case of projects involving a very large number of piles, economies can be achieved by testing to failure piles of varying lengths or of different types in order to determine the most efficient combination of pile length, size and installation method. Omission of the preliminary test piling is acceptable only in cases where the engineer has previous experience of pile behaviour in similar soil conditions.

BS 8004 recommends a safety factor between 2 and 3 subject to various qualifications. Comparison of pile capacity calculations between conventional and Eurocode methods generally shows that the Eurocode factors produce an equivalent safety factor of 2 on the capacity calculated conventionally using average shear strengths to calculate skin friction and lower bound strengths to calculate base resistance.

Where piles are end bearing on a strong intact rock the concept of a safety factor against ultimate failure does not apply, since it is likely that the pile itself will fail as a structural unit before shearing failure of the rock beneath the pile toe occurs. The allowable loads are then governed by the safe working stress in compression and bending on the pile shaft (or the Eurocode regulations for the characteristic strength of the pile divided by the appropriate material factor) and the settlement of the pile due to elastic deformation and creep in the rock beneath the base of the pile, together with the elastic compression of the pile shaft.

When using the Eurocode 7 regulations the procedure is first to check that the allowable load does not exceed the ultimate limit-state of the pile material, soil or rock, and then to determine pile head settlements by calculation or from loading tests to check that the serviceability limit-state of the pile is not exceeded, where this state is defined as the load on the pile or pile group which does not cause damage to the supported structure, machinery or services by excessive deformation or other movements. The serviceability limit-state usually governs the design load.

## 4.2 Piles in cohesive soils

### 4.2.1 Driven displacement piles

When a pile is driven into a cohesive soil (for brevity this will hereafter be called a clay) the soil is displaced laterally and in an upward direction, initially to an extent equal to the volume of the pile entering the soil. The clay close to the pile surface is extensively remoulded and high pore-water pressures are developed. In a soft clay the high pore pressures may take weeks or months to dissipate. During this time the skin-friction and end-bearing resistance, in so far as they are related to the effective overburden pressure (the total overburden pressure minus the pore water pressure), are only slowly developed. The soft clay displaced by the pile shaft slumps back into full contact with the pile. The water expelled from the soil is driven back into the surrounding clay, resulting in a drier and somewhat stiffer material in contact with the shaft. As the pore-water pressures dissipate and the re-consolidation takes place the heaved ground surface subsides to near its original level.

The effects in a stiff clay are somewhat different. Lateral and upward displacement again occurs, but extensive cracking of the soil takes place in a radial direction around the pile. The clay surrounding the upper part of the pile breaks away from the shaft and may never regain contact with it. If the clay has a fissured structure the radial cracks around the pile propagate along the fissures to a considerable depth. Beneath the pile toe, the clay is extensively remoulded and the fissured structure destroyed. The high pore pressures developed in the zone close to the pile surface are rapidly dissipated into the surrounding crack system and negative pore pressures are set up due to the expansion of the soil. The latter may result in an initially high ultimate resistance which may be reduced to some extent as the negative pore pressures are dissipated and relaxation occurs in the soil which has been compressed beneath and surrounding the lower part of the pile.

The end-bearing resistance of the displacement pile (the term  $Q_b$  in equation 4.1) is calculated from the equation

$$Q_b = N_c c_b A_b \quad \dots(4.2)$$

where  $N_c$  is the bearing capacity factor,  $c_b$  is the characteristic undisturbed undrained cohesion at the pile toe, and  $A_b$  is the cross-sectional area of pile toe. The bearing capacity factor  $N_c$  is approximately equal to 9 provided that the pile has been driven at least to a depth of five diameters into the bearing stratum.  $N_c c_b$  is equal to  $q_{bk}$  in Eurocode terms (equation 4.1a),

and it should be factored as required by the code taking into account the influence of local zones of weak soil beneath the pile toe and the effects of pile installation. It is not strictly correct to take the undisturbed cohesion for  $c_b$  since remoulding has

taken place beneath the toe. However, the greater part of the failure surface in end bearing shown in [Figure 4.3](#) is in soil which has been only partly disturbed by the penetration of the pile. In a stiff fissured clay the gain in strength caused by remoulding is offset by the loss due to large displacement strains along a fissure plane. In the case of a soft and sensitive clay the full undisturbed cohesion should be taken only when the working load is applied to the pile after the clay has had time to regain its original shearing strength (*i.e.*, after full dissipation of pore pressures); the rate of gain in the carrying capacity of piles in soft clays is shown in [Figure 4.4](#). It may be noted that a period of a year is required for the full development of carrying capacity in the Scandinavian ‘quick’ clays. In any case the end-bearing resistance of a small-diameter pile in clay is only a small proportion of the total resistance and errors due to the incorrect assumption of cohesion on the failure surface are not of great significance.

In terms of ‘pure’ soil mechanics theory the *ultimate skin friction* on the pile shaft is related to the horizontal effective stress acting on the shaft and the effective remoulded angle of friction between the pile and the clay. Thus

$$\tau_s = \sigma'_h \tan \delta_r \quad \dots(4.3)$$

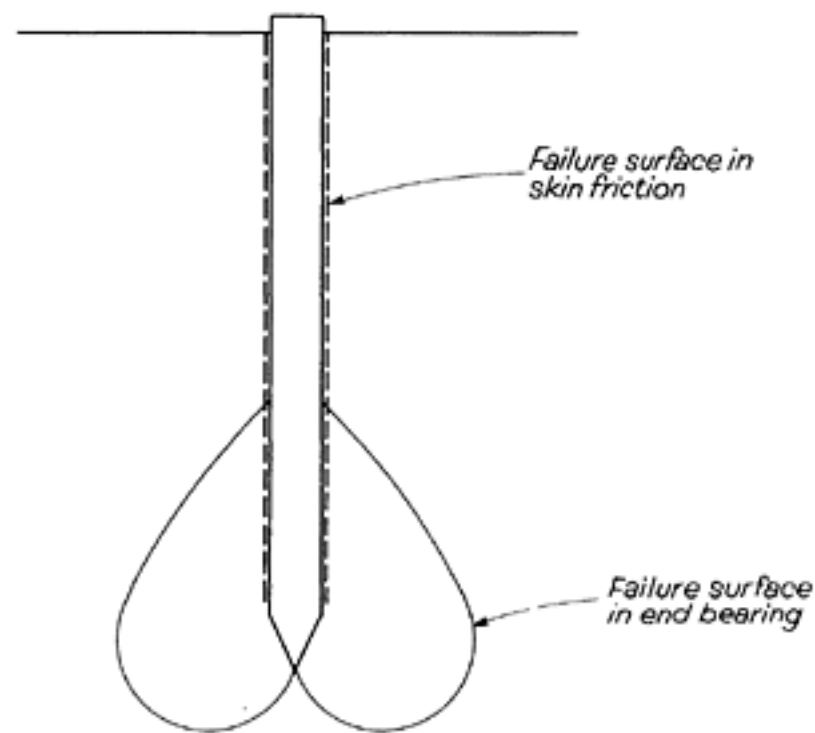


Fig. 4.3 Failure surfaces for compressive loading on piles

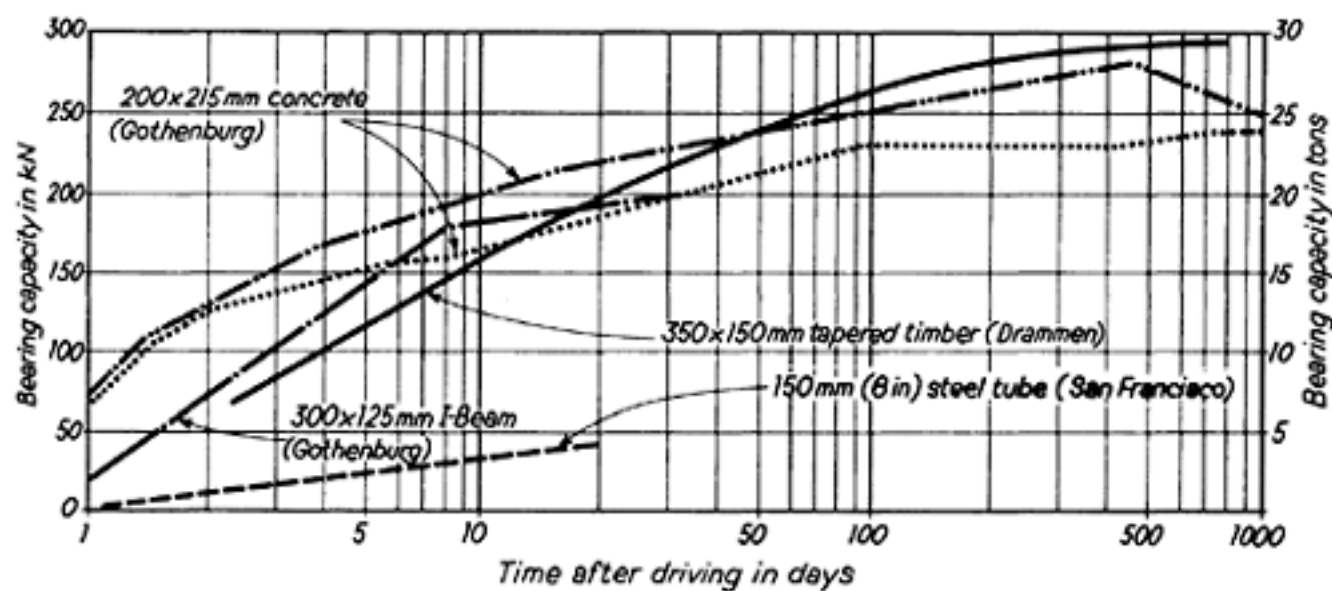


Fig. 4.4 Gain in carrying capacity with increasing time after driving of piles into soft clays

where  $\tau_s$  is the unit skin friction at any point,  $\sigma'_h$  is the horizontal effective stress, and  $\delta_r$  is the effective remoulded angle of friction.

A further simplifying assumption is made that  $\sigma'_h$  is proportional to the vertical effective overburden pressure,  $\sigma'_{vo}$ . Thus

$$\tau_s = K\sigma'_{vo} \tan \delta_r \quad \dots(4.4)$$

The value of  $K$  is constantly changing throughout the period of installation of the pile and its subsequent loading history. In the case of a driven pile in a stiff clay  $K$  is initially very high, as a result of the energy transmitted by the hammer blows required to displace the clay around the pile. However, at this time  $\sigma'_{ve}$  is very low or even negative due to the high pore-water pressures induced by the pile driving. In the case of a bored pile,  $K$  is low as the soil swells at the time of drilling the hole, but it increases as concrete is placed in the shaft. Because of these constantly changing values of  $K$ , and the varying pore pressures (and hence values of),  $\sigma'_{vo}$  'pure' soil mechanics methods cannot be applied to practical pile design without introducing empirical factors and simplified calculations to allow for these uncertainties. Semi-empirical methods based on these simplifications have been proposed by Burland<sup>(4.2)</sup> and Meyerhof<sup>(4.3)</sup>. In the case of piles which penetrate a relatively short distance into the bearing stratum of firm to stiff clay, that is piles carrying *light to moderate* loading, a sufficiently reliable method of calculating the ultimate skin friction,  $Q_s$ , on the pile shaft is to use the equation:

$$Q_s = \alpha \bar{c}_u A_s \quad \dots(4.5)$$

where  $a$  is an adhesion factor,  $\bar{c}_u$  is the characteristic or average undisturbed undrained cohesion of the soil surrounding the pile shaft, and  $A_s$  is the surface area of the pile shaft contributing to the support of the pile in skin friction (usually measured from the ground surface to the toe). When following Eurocode 7,  $\alpha \bar{c}_u$  is equivalent to  $q_{sik}$  used to obtain  $Q_{sk}$  which is factored to obtain the design shaft resistance  $Q_s$ .

The adhesion factor depends partly on the cohesive strength of the soil and partly on the nature of the soil above the bearing stratum of clay into which the piles are driven. The evaluation of the adhesion factor has been studied in some detail by the author. His early studies<sup>(4.4)</sup> showed a general trend towards a reduction in the adhesion factor from unity or higher than unity for very soft clays, to values as low as 0.2 for clays having a very stiff consistency. There was a wide scatter in the values over the full range of soil consistency and these seemed to be unrelated to the material forming the pile.

Much further light on the behaviour of piles driven into stiff clays was obtained in the research project undertaken by the author for the Construction Industry Research and Information Association (CIRIA) in 1969<sup>(4.5)</sup>. Steel tubular piles were driven into stiff to very stiff London clay and were subjected to loading tests at 1 month, 3 months and 1 year after driving. Some of the piles were then disinterred for a close examination of the soil surrounding the interface. This examination showed that the gap, which had formed around the pile as the soil was displaced by its entry, extended to a depth of 8 diameters and it had not closed up a year after driving. Between depths of 8 diameters and 14 to 16 diameters the clay was partly adhering to the pile surface, and below 16 diameters the clay was adhering tightly to the pile in the form of a dry skin 1 to 5mm in thickness which had been carried down by the pile. Thus in the lower part of the pile the failure was not between the pile and the clay, but between the skin and surrounding clay which had been heavily sheared and distorted. Strain gauges mounted on the pile to record how the load was transferred from the pile to the soil showed the distribution of load in [Figure 4.5](#). It may be noted that there was no transfer of load in the upper part of the pile, due to the presence of the gap. Most of the load was transferred in the lower part where the adhesion was as much as 20% greater than the undrained cohesion of the clay. The loss of adhesion for a short distance above the pile toe will also be noted. This is believed to be due to the formation of a tension crack in the soil at toe level. The soil immediately above the crack then tends to slump down causing a drag on the pile rather than contributing to its support.

Recent research by Bond and Jardine<sup>(4.5a)</sup> on extensively instrumented piles jacked into stiff London clay confirmed the author's findings on the nature of the soil disturbance very close to the pile. Negative pore pressures were induced in the clay close to the pile wall and positive pressures further away from the pile. Equalization of pore pressures after installation was very rapid occurring in a period of about 48 hours. There was no change in shaft friction capacity after the equalization period as observed by periodic first-time loading tests over a 31-month period.

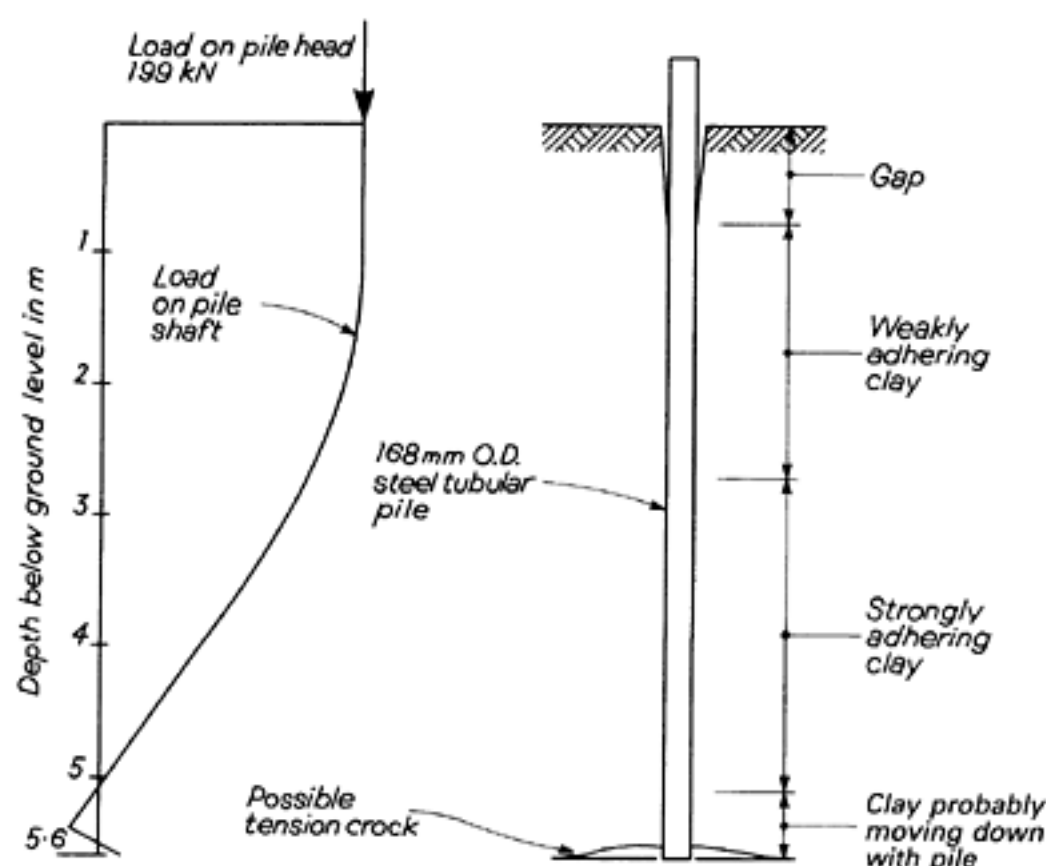


Fig. 4.5 Load transfer from pile to stiff clay at Stanmore<sup>(4.5)</sup>

The CIRIA project included a study of the effects of driving steel tubular piles through an overburden of soft clay and of sand into the stiff London clay. In both cases a skin was dragged down from the overburden soil for a distance of three pile diameters into the London clay (Figure 4.6). The effect of the soft clay skin was to reduce the adhesion factor within the stiff clay while the sand skin increased it. At levels deeper than three diameters the stiff London clay skin had been dragged down in the manner previously observed.

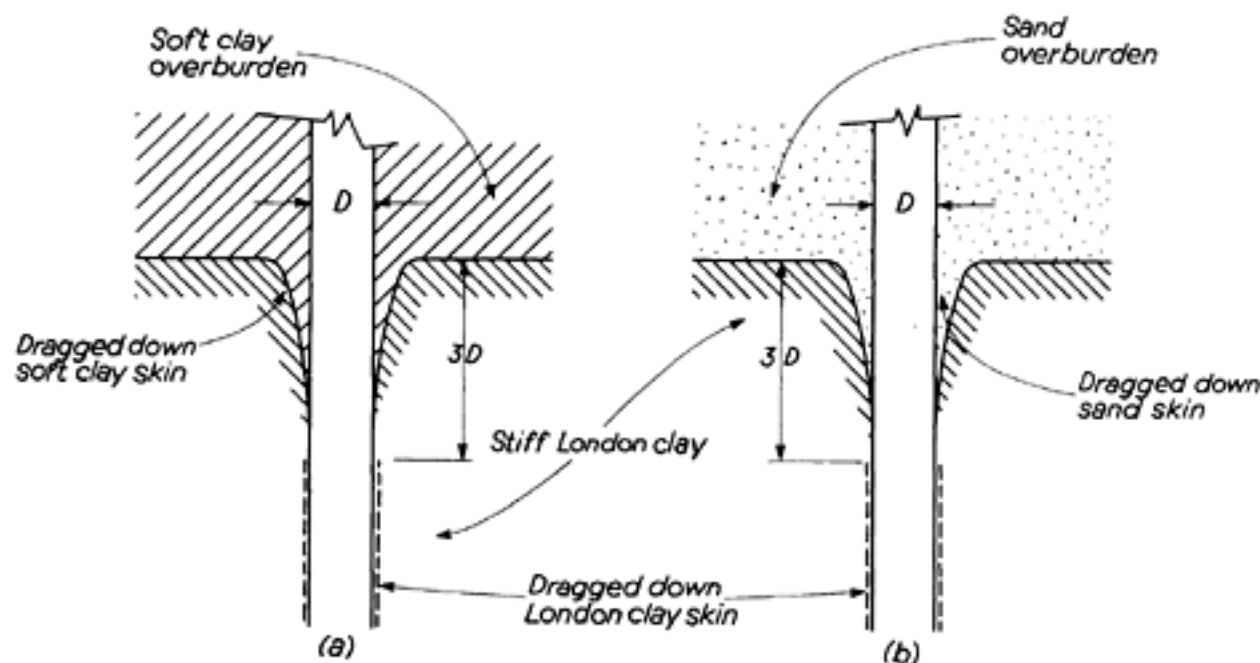


Fig. 4.6 Dragdown of overburden soil on pile shaft in clay

It was evident from this research that the penetration depth into the bearing stratum of stiff clay had an important effect on the adhesion factor. In the case of a short penetration into a stiff clay without other overlying strata the gap formed between the pile and the soil would occupy most of the penetration depth and the adhesion factor would be very low. Similarly, a short penetration into a stiff clay after driving through a soft clay overburden would result in the dragged-down soft clay skin occupying most of the pile surface and again resulting in a low adhesion factor, although the shearing strength of the clay skin is higher than that of the parent soft clay due to consolidation caused by pile displacement. However, when a pile is driven through a sand overburden to a short penetration into stiff clay the sand skin occupies most of the pile surface with a correspondingly high adhesion factor. The greater the depth of penetration into a stiff clay stratum the less will be the effect of the overburden conditions. When piles penetrate very deeply, the gap or the skin of soft clay or sand only occupies a small proportion of the embedded surface. However if the deep clay strata contain layers of sand or sandy gravel these act in the same manner as a layer of sand above the clay stratum and can be taken into account in assessing the skin friction.

The results of the author's research<sup>(4.4, 4.5, 4.6)</sup> have been embodied in the design curves for adhesion factors for displacement piles carrying light to moderate loading driven into stiff clays shown in Figure 4.7. Most of the data from which the curves were derived was obtained from loading tests on piles of solid section or closed-end tubular piles. The validity of these curves has been tested by comparing the predicted total carrying capacity with the observed ultimate carrying capacity for seventy-eight cases of pile loading tests taken to failure on sites where information on soil cohesion values was available. In all cases a bearing capacity factor ( $N_c$ ) of 9 was used to calculate the end-bearing resistance to compression loads. It can

be seen from Figure 4.8 that the adhesion factors obtained from the design curves in Figure 4.7 tend to give conservative results, particularly for piles driven through soils with sandy or gravelly overburdens. If a safety factor of 2 had been adopted to arrive at a safe working load, failure at the working load would have occurred in only two cases out of the 78. The selection of a suitable safety factor is discussed later in this chapter (Section 4.6). It must be emphasized that Figure 4.7 shows *design* curves which take into account all the uncertainties in the method of calculation and variations in the adhesion factor that are known to occur in identical piles on the same site. They do not give average failure conditions. Hence when following Eurocode regulations the ultimate unit skin friction need not necessarily be divided by a factor of 1.5 to obtain the characteristic value. The value calculated could be taken as  $Q_{sk}$  which is then factored to obtain the design skin friction  $Q_s$ .

The adhesion factors shown in Figure 4.7 are applicable to tapered piles, where the tendency is to

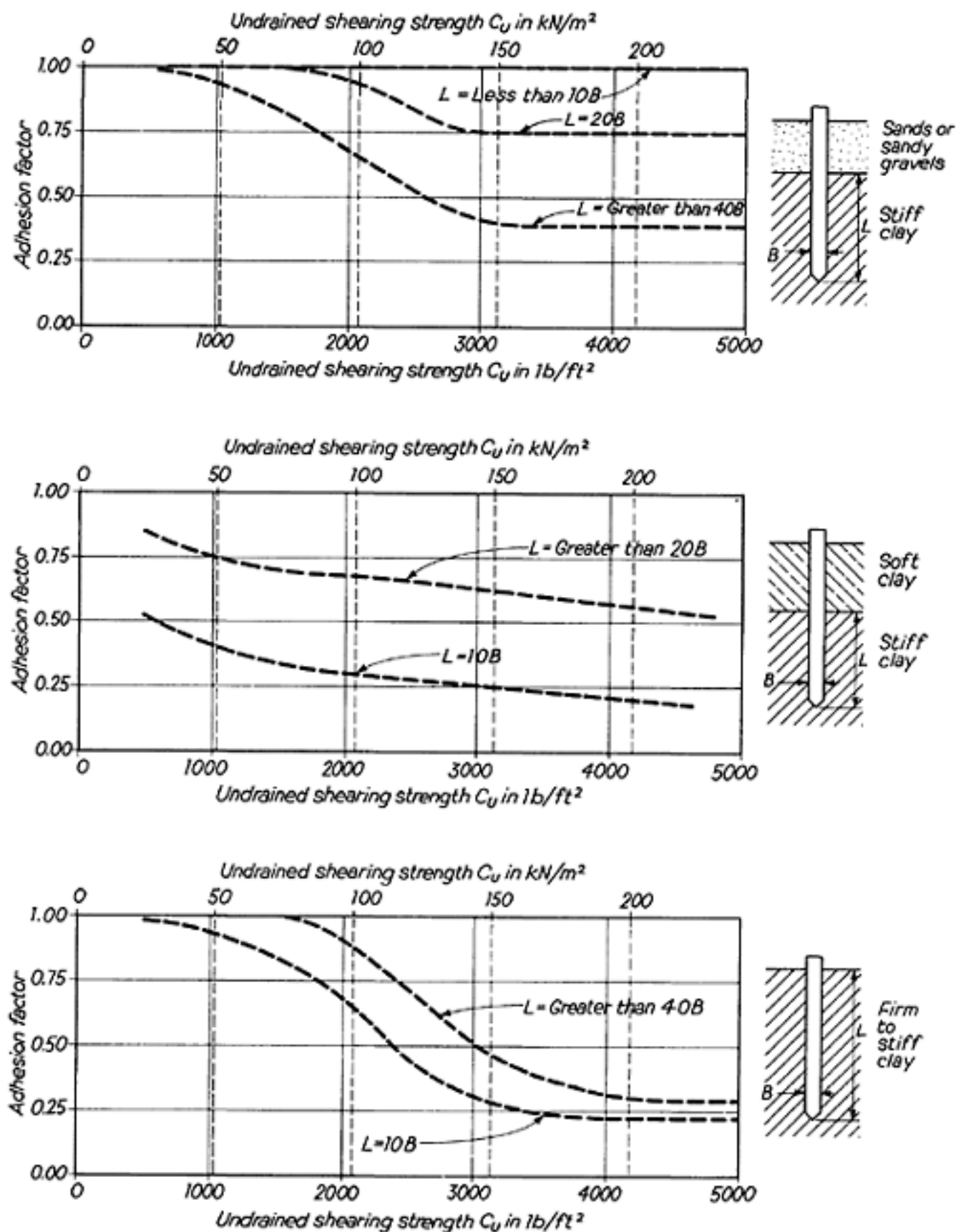


Fig. 4.7 Design curves for adhesion factors for piles driven into clay soils

close up the gap around the upper part of the shaft and to increase the consolidation of a dragged-down soft clay skin. However, the beneficial effects of a taper do not appear to be of great significance to the values of the adhesion factor, according to the published test results which are available for cohesive soils. Hence a shape factor has not been introduced in equation 4.5. The adhesion factors shown in [Figure 4.7](#) are not applicable to tubular piles driven with a projecting base plate or an external stiffening ring ([Figure 2.23b](#)) since the gap formed down the shaft in a stiff clay may destroy the skin friction. Different concepts apply to the determination of the skin friction where it is necessary to drive piles to a deep penetration into a stiff clay to obtain the required bearing capacity for compression or uplift loading. Research, mainly in the field of pile design for offshore structures, has shown that the mobilization of skin friction is influenced principally by two factors. These are the over-consolidation ratio of the clay and the slenderness (or aspect) ratio of the pile.

The over-consolidation ratio is defined as the ratio of the maximum previous vertical effective overburden pressure,  $\sigma'_{vc}$ , to the existing vertical effective overburden pressure,  $\sigma'_{vo}$ . For the purposes of pile design, Randolph and Wroth<sup>(4.7)</sup> have shown that it is convenient to represent the over-consolidation ratio by the simpler ratio of the undrained shear strength to the existing effective overburden pressure,  $c_u/\sigma'_{vo}$ .

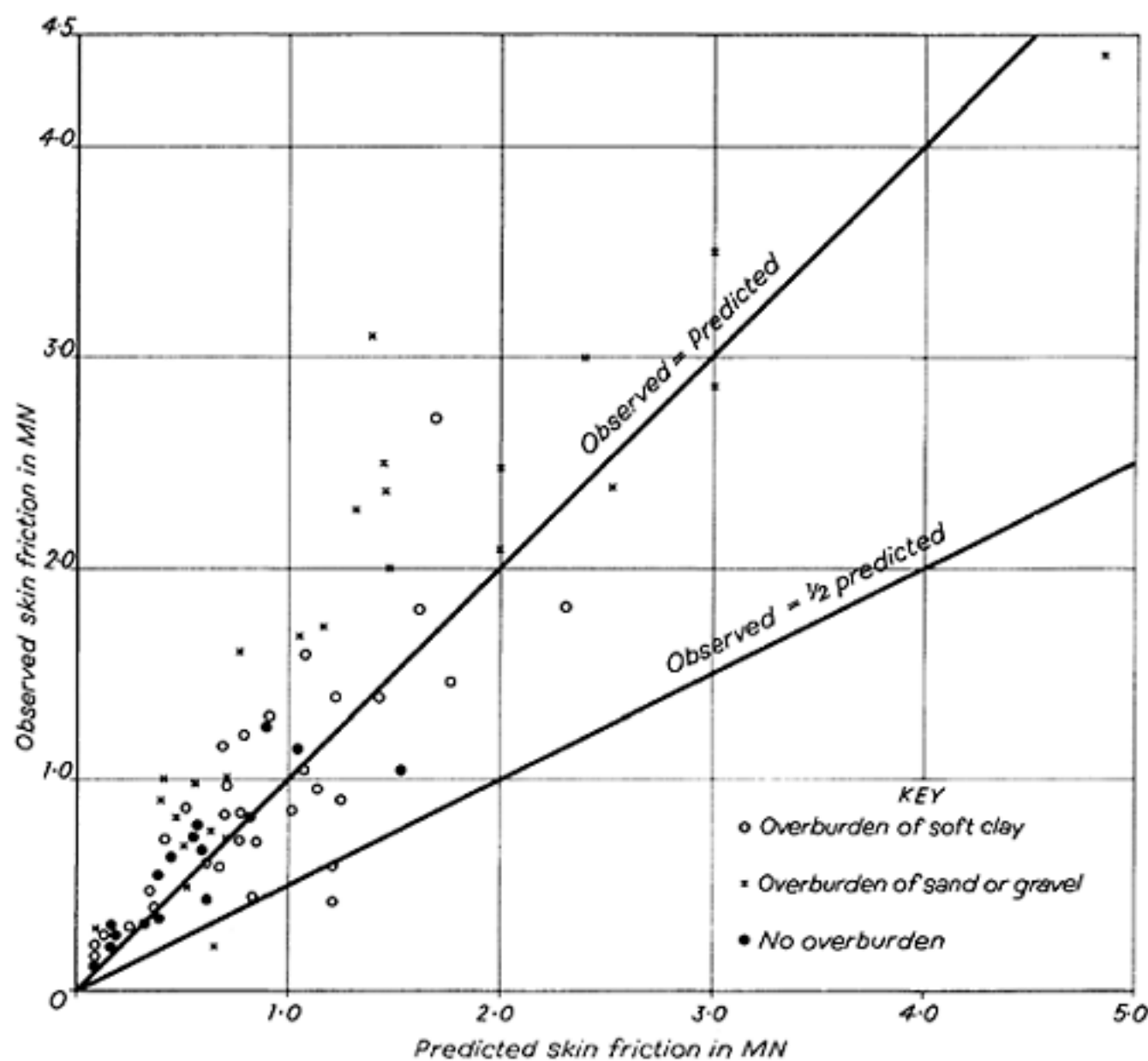


Fig. 4.8 Predicted against observed ultimate loads on piles driven into day soils

Randolph and Wroth showed that the  $c_u/\sigma'_{vo}$  ratio could be correlated with the adhesion factor,  $\alpha$ . A relationship between these two has been established by Semple and Rigden<sup>(4.8)</sup> from a review of a very large number of pile loading tests, the majority of them being on open-end piles either plugged with soil or concrete. This is shown in Figure 4.9a for the case of a rigid pile, and where the skin friction is calculated from the peak value of  $c_u$ . To allow for the flexibility and slenderness ratio of the pile it is necessary to reduce the values of  $\alpha_p$  by a length factor,  $F$ , as shown in Figure 4.9b. Thus total skin friction

$$Q_s = F\alpha_p \bar{c}_u A_s \quad \dots(4.6)$$

when following Eurocode 7,  $F\alpha_p \bar{c}_u$  is equivalent to the summation of  $q_{sik}$  along the shaft. Because  $F$  and  $\alpha_p$  were obtained by correlation between loading tests and average values of  $c_u$  the calculated shaft resistance should be divided by a factor of 1.5 to obtain  $Q_{sk}$  in equation 4.1b.

The slenderness ratio,  $L/B$ , influences the mobilization of skin friction in two ways. First, a slender pile can 'whip' or flutter during driving causing a gap around the pile at a shallow depth and reducing horizontal stress at the pile/soil interface at lower levels. The second influence is the slip at the interface when the shear stress at transfer from the pile to the soil exceeds the peak value of shear strength and passes into the lower residual strength. This is illustrated by the shear/strain curve of the simple shear box test on a clay. The peak shear strength is reached at a relatively small strain followed by the much lower residual strength at long strain.

It follows that when an axial load is applied to the head of a long flexible pile the relative movement between the pile and the clay at a shallow depth can be large enough to reach the stage of low post-peak strength at the interface. Near the pile toe the relative movement between the compressible pile and the compressible clay may not have reached the stage of mobilizing the peak shear strength. At some intermediate level the post-peak condition may have been reached but not the lowest residual condition. It is therefore evident that calculation of the skin friction resistance from the results of the peak undrained shear strength, as obtained from unconfined or triaxial compression tests in the laboratory, may overestimate the available friction resistance of long piles. The length factors shown in Figure 4.9b are stated by Semple and Rigden to allow both for

the flutter effects and the residual or part-residual shear strength

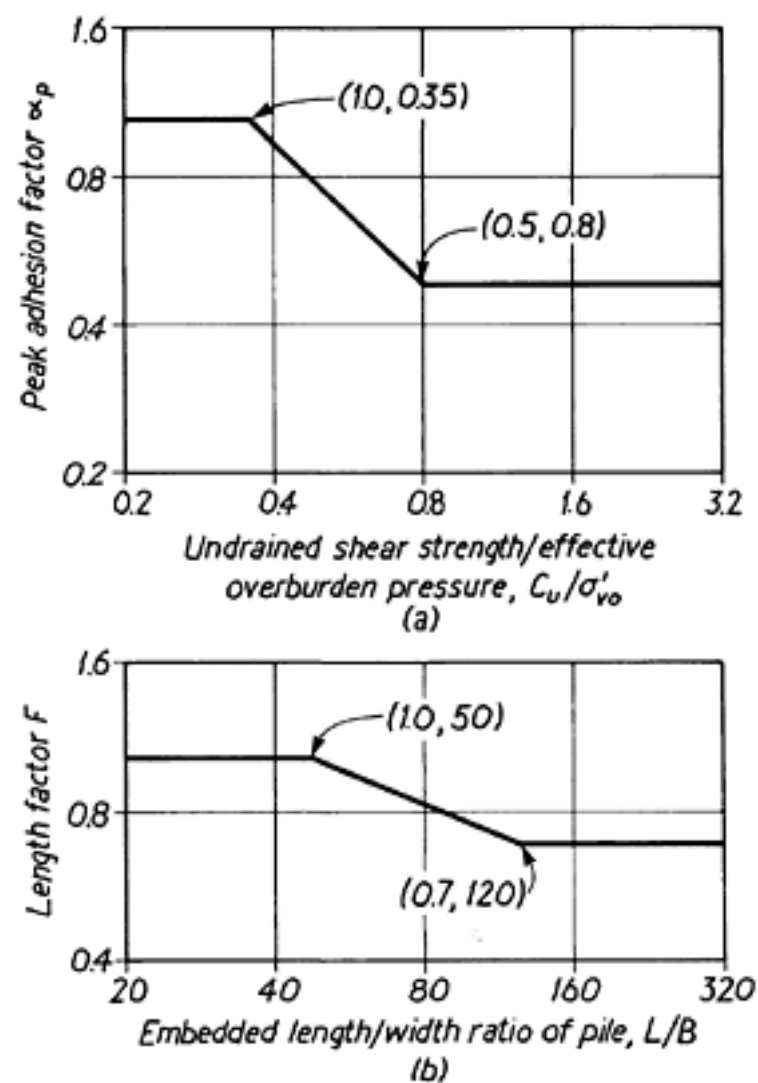


Fig. 4.9 Adhesion factors for piles driven to deep penetration into clays (after Semple and Rigden 4.8)

- (a) Peak adhesion factor vs. shear strength/effective overburden pressure  
 (b) Length factor

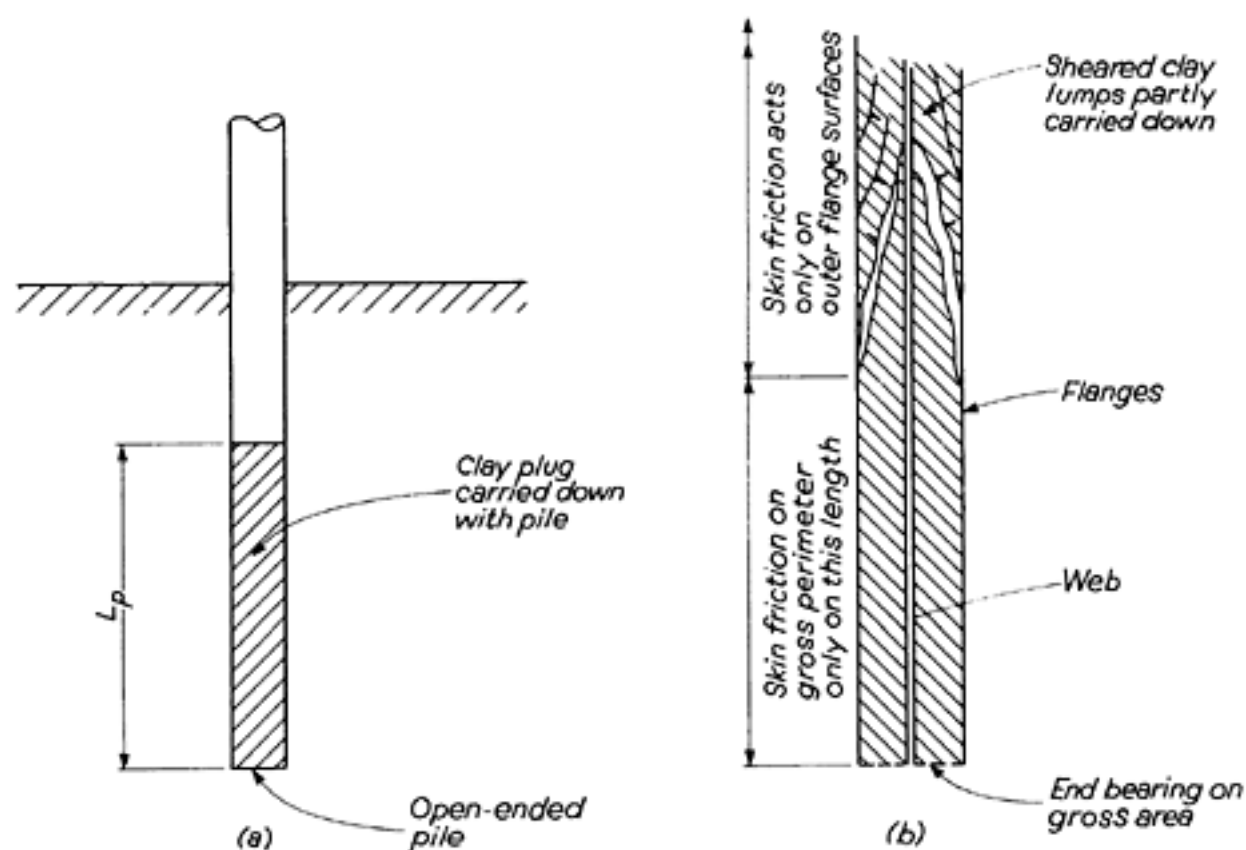


Fig. 4.10 Formation of soil plug at toe of small-displacement piles

- (a) Open-ended tube  
 (b) H-section

conditions at the interface. The effect of these conditions on the settlement of single piles is discussed in 4.6.

In marine structures where piles may be subjected to uplift and lateral forces caused by wave action or the impact of berthing ships, it is frequently necessary to drive the piles to much greater depths than those necessary to obtain the required resistance to axial compression loading only. To avoid premature refusal at depths which are insufficient to obtain the required uplift or lateral resistance, tubular piles are frequently driven with open ends. At the early stages of driving soil

enters the pile when the pile is said to be 'coring'. As driving continues skin friction will build up between the interior soil and the pile wall. This soil is acted on by inertial forces resulting from the blows of the hammer. At some stage the inertial forces on the core plus the internal skin friction will exceed the bearing capacity of the soil at the pile toe calculated on the cross-sectional area of the open end. The plug is then carried down

by the pile as shown in [Figure 4.10a](#). However, on further driving and when subjected to the working load, the pile with its soil plug does not behave in the same way as one driven to its full penetration with the tip closed by a steel plate or concrete plug. This is because the soil around and beneath the open end is not displaced, and consolidated to the same extent as that beneath a solid end pile.

Stevens<sup>(4.8a)</sup> has shown that the length  $L_p$  in [Figure 4.10a](#) when the pile is changing from the coring to the plugging mode can be calculated from the equation

$$L_p = \frac{2.25c_{ub}}{f_s + 0.25\gamma D(1 - \bar{a})} \quad \dots(4.7)$$

where  $D$  is the plug diameter

$\gamma$  is the submerged weight of the soil

$f_s$  is the unit skin friction between the plug and the pile during driving

• is the average plug acceleration in g's

The term • can be obtained from dynamic tests made at the time of driving using the equipment described in [Section 7.3](#).

Comparative tests on open end and closed end piles were made by Rigden, *et al.* <sup>(4.9)</sup>. The two piles were 457mm steel tubes driven to a penetration of 9m into stiff glacial till in Yorkshire. A clay plug was formed in the open end pile and carried down to occupy 40% of the final penetration depth. However, the failure loads of the clay-plugged and steel plate closed piles were 1160kN and 1400kN respectively. Evaluation of the ultimate skin friction and base resistances showed that the external skin friction on the open end piles was 20% less than that on the closed end piles.

Accordingly it is recommended that where field measurements show that a clay plug is carried down, the total ultimate bearing capacity should be calculated as the sum of the external skin friction (obtained from equation 4.6 and [Figure 4.7](#)) multiplied by a factor of 0.8, and the ultimate base resistance,  $Q_b$ , obtained from equation 4.2 multiplied by a factor of 0.5.

Where an internal stiffening ring is provided at the toe of a steel pile the base resistance should be calculated only on the net cross-sectional area of the steel. Attempts to clean out the core of soil from within the pile and replace it by a plug of concrete or cement/sand grout are often ineffective due to the difficulty of removing the strongly adherent clay skin to provide an effective bond to the pile surface. Also on large diameter piles the radial shrinkage of the concrete or grout plug can weaken the bond with the pile. As already noted the majority of the pile tests used to derive the relationships in [Figure 4.9](#) were made on open-end piles plugged with soil or concrete. Hence the skin friction derived from them already incorporates the effect of the open end.

Plug formation between the flanges and web of an H-section pile is problematical. The possible plug formation at the toe of an H-pile is shown in [Figure 4.10b](#). The mode of formation of a dragged-down soft clay or sand skin has not been studied.

The author has observed a gap around all flange and web surfaces of H-piles driven into stiff glacial till. An H-pile is not a good type to select if it is desired to develop skin friction and end-bearing resistance in a stiff clay. The author recommends calculating the skin friction on the outer flange surfaces only, but plugging can be allowed for by calculating the end-bearing resistance on the gross cross-sectional area of the pile. Because of the conservative assumptions of skin friction and the relatively low proportion of the load carried in end-bearing the calculated resistance need not be reduced by the factor of 0.5 as recommended for tubular piles.

## 4.2.2 Driven-and-cast-in-place displacement piles

The end-bearing resistance of driven-and-cast-in-place piles terminated in clay can be calculated from equation 4.2. Where the piles have an enlarged base formed by hammering out a plug of gravel or dry concrete, the area  $A_b$  should be calculated

from the estimated diameter of the base. It is difficult, if not impossible, for the engineer to make this estimate in advance of the site operations since the contractor installing these proprietary piles makes his own decision on whether to adopt a fairly shallow penetration and hammer out a large base in a moderately stiff clay, or whether to drive deeper to gain skin friction, but at the expense of making a smaller base in the deeper and stiffer clay. In a hard clay it may be impracticable to obtain any worthwhile enlargement over the nominal shaft diameter. In any case, the base may have to be taken to a certain minimum depth to ensure that settlements of the pile group are not exceeded (see [5.2.2](#)). The decision as to this minimum length must be taken by the engineer or be approved by him.

The conditions for predicting skin friction on the shaft are different from those with driven pre-formed piles in some important aspects. The effect on the soil of driving the piling tube with its end closed by a plug is exactly the same as with a steel tubular pile; the clay is remoulded, sheared and distorted,

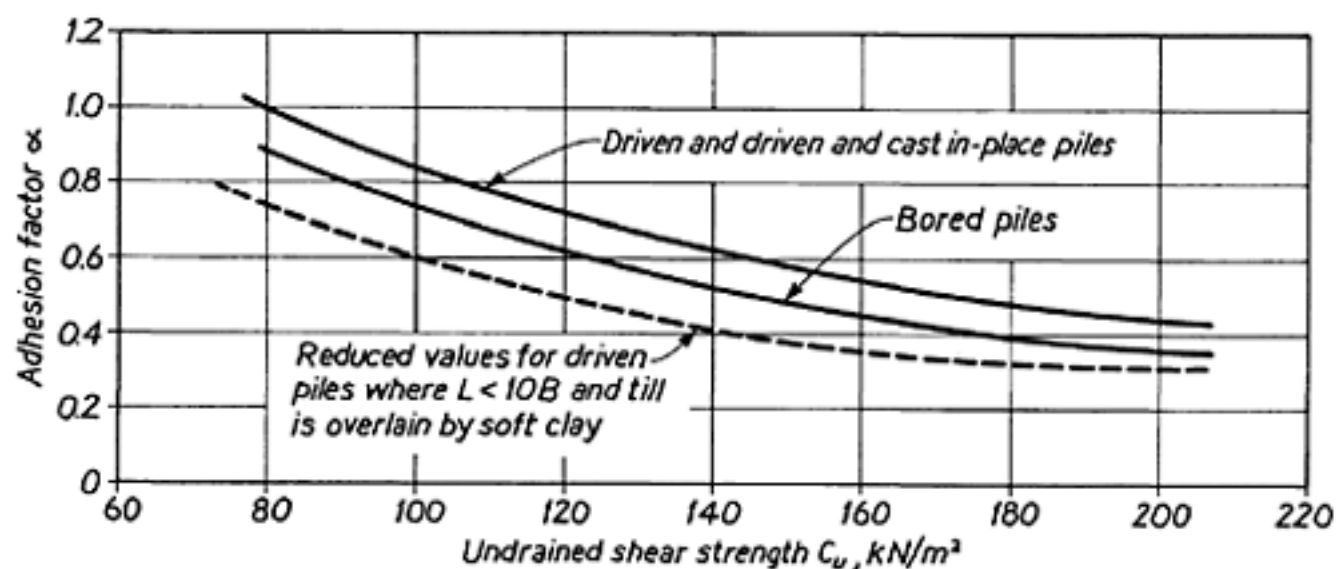


Fig. 4.11 Adhesion factors for piles in glacial till (after Weltman and Healy<sup>(4.10)</sup>)

giving the same conditions at the pile-soil interface as with the driven pre-formed pile. The clay has no chance to swell before the concrete is placed and the residual radial horizontal stress in the soil closes up any incipient gap caused by shrinkage of the concrete. Also the gap which may form around the upper part of the driving tube (or down the full length of the driving tube if an enlarged detachable shoe is used to close its base) becomes filled with concrete. The tube, while being driven, drags down a skin of soft clay or sandy soil for a few diameters into the stiff clay and it is quite likely that this skin will remain interposed between the concrete and the soil, *i.e.*, the skin is not entirely pulled out by adhering to the tube. However, in one important aspect there is a difference between the driven and the driven-and-cast-in-place pile in that water migrates from the unset concrete into the clay and softens it for a limited radial distance. This aspect is discussed in greater detail in the following [section \(4.2.3\)](#). Thus the adhesion factor for a driven-and-cast-in-place pile in a stiff clay may be slightly less than that for a driven pile in corresponding soil conditions. It will probably be greater over the length in a soft clay, however, since the concrete slumps outwards as the tube is withdrawn, producing an increase in effective shaft diameter.

The results of a number of loading tests on driven and driven and cast-in-place piles in glacial till have been reviewed by Weltman and Healy<sup>(4.10)</sup>. There appeared to be little difference in the  $\alpha$ — $c_u$  relationship for either type of pile. They

produced the design curves shown in [Figure 4.11](#) for the two types of driven pile including a curve for piles driven a short penetration into stiff glacial till overlain by soft clay. Their review also included a study of the skin friction on bored piles in glacial till.

The curves were based on average values of  $c_u$ . Hence when following Eurocode 7 the values of  $Q_{sk}$  in equation 4.1b should be divided by the factor of 1.5 to obtain the design shaft resistance.

### 4.2.3 Bored-and-cast-in-place non-displacement piles

The installation of bored piles using the equipment and methods described in [Sections 3.3.1 to 3.3.6](#) and [3.4.6](#) causes changes in the properties of the soil on the walls of the pile borehole which have a significant effect on the skin-frictional resistance of the piles. The effect of drilling is to cause a relief of lateral pressure on the walls of the hole. This results in swelling of the clay and there is a migration of pore water towards the exposed clay face. If the borehole intersects water-filled fissures or pockets of silt the water will trickle down the hole and form a slurry with the clay as the drilling tools are lowered down or raised from the hole. Water can also soften the clay if it trickles down from imperfectly sealed-off water-bearing strata above the clay, or if hose pipes are carelessly used at ground level to remove clay adhering to the drilling tools.

The effect of drilling is always to cause softening of the clay. If bentonite is used to support the sides of the borehole, softening of the clay due to relief of lateral pressure on the walls of the hole will still take place, but flow of water from any fissures will not occur. There is also a risk of entrapment of pockets of bentonite in places where overbreak has been caused by the rotary drilling operation. This would be particularly liable to occur in a stiff fissured clay.

After placing concrete in the pile borehole, water migrates from the unset concrete into the clay, causing further softening of the soil. The rise in moisture content due to the combined effects of drilling and placing concrete was observed by Meyerhof and Murdoch<sup>(4.11)</sup>, who measured an increase of 4% in the water content of London clay close to the interface with the concrete. The increase extended for a distance of 76mm from the interface.

This softening affects only the skin friction on the pile shaft. The soil within the zone of rupture beneath and surrounding the pile base ([Figure 4.3](#)) remains unaffected for all practical purposes and the end-bearing resistance  $Q_b$  can be calculated from

equation 4.2, the value of the bearing capacity factor  $N_c$  again being 9. However, Whitaker and Cooke([4.12](#)) showed that the

fissured structure of London clay had some significance on the end-bearing resistance of large bored piles, and they suggested that if a bearing capacity factor of 9 is adopted the characteristic shearing strength should be taken along the lower range of the graph of shearing strength against depth. If bentonite is used the effects of any entrapment of slurry beneath the pile base as described by Reese *et al.*([3.11](#)) should be allowed for by an appropriate reduction in end-bearing resistance.

The effect of the softening on the skin friction of bored piles in London clay was studied by Skempton([4.13](#)), who showed that the adhesion factor in equation 4.5 ranged from 0.3 to 0.6 for a number of loading test results. He recommended a value of 0.45 for normal conditions where drilling and placing concrete followed a reasonably rapid sequence. However, for short piles, where a large proportion of the shaft may be in heavily fissured clay, Skempton recommended the lower value of 0.3. Skempton observed that the actual unit skin friction mobilized in London clay did not exceed  $100\text{kN/m}^2$ , and this value should be taken as an upper limit when the unit skin friction is calculated from 0.3 or 0.45 times the average undisturbed cohesion. Alternatively, the curve for bored piles in [Figure 4.11](#) can be used to obtain the adhesion factor for very stiff to hard clays.

The author recommends that the same value of 0.3 should be used for small-diameter bored piles where there may be a long delay between drilling and placing the concrete, for example where piles are drilled in the morning and the borehole is left unlined awaiting the arrival of the ready-mixed concrete truck at the end of the day. The factor of 0.3 should also be used for large bored piles with enlarged bases which may involve a long delay between first drilling and finally concreting the shaft, giving a long period for the swelling and softening of the clay on the sides of the shaft. It is believed that differences in the method of drilling, such as between the scoring or gouging of a plate auger and the smoothing of a bucket auger, can also cause differences in skin friction. However, the effects of soil swelling and water from the concrete are likely to be of much greater significance in controlling the adhesion factor.

Fleming and Sliwinski([4.14](#)) reported no difference in the adhesion factor between bored piles drilled into clays in bentonite-filled holes and dry holes. In spite of this evidence it must be pointed out that if the use of a bentonite slurry to support an unlined hole in clay does not reduce the skin friction this must mean that the rising column of concrete placed by tremie pipe beneath the slurry has the effect of sweeping the slurry completely off the wall of the borehole. It is difficult to conceive that this happens in all cases; therefore the adhesion factor  $\alpha$  recommended for London clay, or for other clays in [Figure 4.11](#), should be reduced by 0.8 to allow for the use of bentonite unless a higher value can be demonstrated conclusively by loading tests.

In clays other than London clay, where there is no information from loading tests or publications, the adhesion factors shown in the curve for bored piles in glacial till ([Figure 4.11](#)) can be used as a guide to pile design. The calculated pile capacity should be confirmed by field loading tests. When using Eurocode 7 the ultimate base resistance and skin friction values should be factored in the same manner as described for driven piles in [Section 4.2.1](#).

When enlarged bases are provided on bored piles in a fissured clay there may be a loss of adhesion

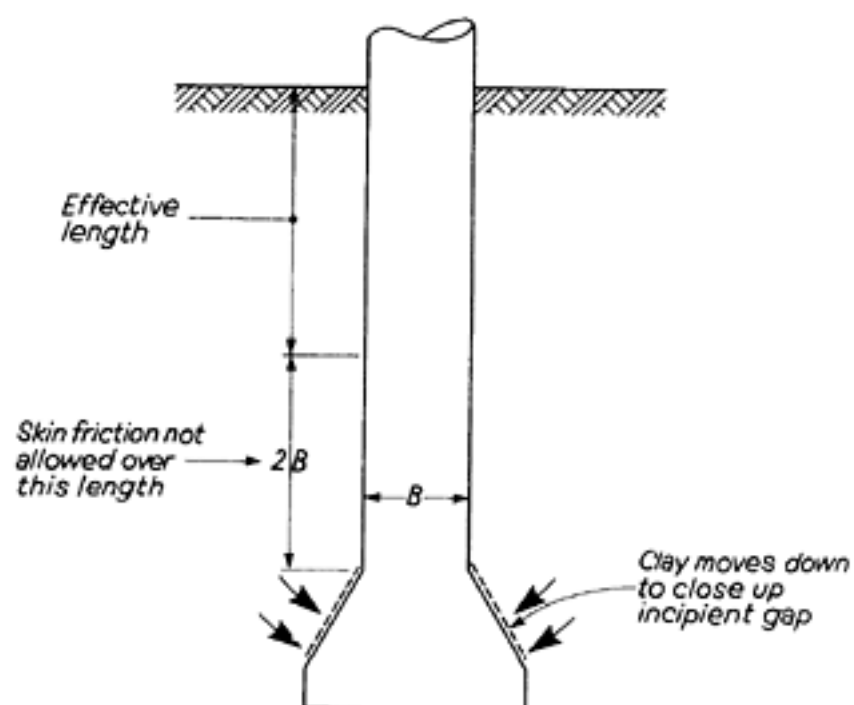


Fig. 4.12 Effective shaft length for calculating skin friction on shaft of under-reamed pile

over part of the pile shaft in cases where appreciable settlements of the pile base are allowed to occur. The effect of such movements is to open a gap between the conical surface of the base and the overlying clay. The latter then slumps downwards to close the gap and this causes a 'drag-down' on the pile shaft. Arching prevents slumping of the full thickness of clay from the ground surface to the pile base. It is regarded as over-cautious to add the possible drag-down force to the working load on the pile, but nevertheless it may be prudent to disregard the supporting action on the pile of skin friction over a height of two shaft diameters above the pile base, as shown in [Figure 4.12](#).

Disregarding skin friction over a height of two shaft diameters and taking an adhesion factor of 0.3 for the skin friction on the remaining length may make a pile with an enlarged base an unattractive proposition in many cases when compared with one with a straight shaft. However the enlarged-base pile is economical if the presence of a very stiff or hard stratum permits the whole of the working load to be carried in end bearing. Enlarged bases may also be a necessity to avoid drilling down to or through a water-bearing layer in an otherwise impervious clay.

Piles for marine structures are sometimes installed by driving a steel tube to a limited penetration below sea bed, followed by drilling-out the soil plug then continuing the drilled hole without further support by the pile tube. A bentonite slurry is sometimes used to support the borehole. On reaching the design penetration depth a smaller diameter steel tube insert pile is lowered to the bottom of the borehole and a cement-sand grout is pumped-in to fill the annulus around the insert pile. The grout is injected either through a small-diameter pipe or is pumped directly down the insert pile.

Kraft and Lyons<sup>(4.15)</sup> have shown that the adhesion factor used to calculate the skin friction on the grout/clay interface is of the same order as that used for the design of conventional bored and cast-in-place concrete piles. Where bentonite is used as the drilling fluid a reduction factor should be adopted as discussed above.

A considerable increase in the adhesion factor can be obtained if grout is injected under pressure at the pile/soil interface after a waiting period of 24 hours or more. Jones and Turner<sup>(4.16)</sup> report a two- to threefold increase in adhesion factor when post-grouting was undertaken around the shafts of 150mm diameter micropiles in London clay. However, the feasibility of achieving such increases should be checked by loading tests before using them for design purposes. The post-grouting technique is used as a first step around the shafts of bored piles where the Bauer process of base grouting is used as described in [Section 3.3.7](#).

#### 4.2.4 The effects of time on pile resistance in clays

Because the methods of installing piles of all types have such an important effect on the shaft friction it must be expected that with time after installation there will be further changes in the state of the clay around the pile, leading to an increase or reduction in shaft friction. The considerable increase in resistance of piles driven into soft sensitive clays due to the effects of re-consolidation have already been noted in 4.2.1.

Bjerrum<sup>(4.17)</sup> has reported on the effects of time on the skin friction of piles driven into soft clays. He observed that if a pile is subjected to a sustained load over a long period the shearing stress in the clay next to the pile is carried partly in effective friction and partly in effective cohesion. This results in a downward creep of the pile until such time as the frictional resistance of the clay is mobilized to a degree sufficient to carry the full shearing stress. If insufficient frictional resistance is available the pile will continue to creep downwards. However, the effect of long-period loading is to increase the effective skin friction and cohesion as a result of the consolidation of the clay. It must therefore be expected that if a pile has an adequate safety factor as shown by a conventional short-term loading test, the effect of the permanent (*i.e.* long-term) working load will be to increase the safety factor with time. However, Bjerrum further noted that if the load was applied at a very slow rate there was a considerable reduction in the skin friction that could be mobilized. He reported a reduction of 50% in the adhesion provided by a soft clay in Mexico City when the loading rate was reduced from 10mm per minute to 0.001mm per minute, and a similar reduction in soft clay in Gothenburg resulting from a reduction in loading rate from 1 to 0.001mm per minute. These effects must be taken into account in assessing the required safety factor if a pile is required to mobilize a substantial proportion of the working load in skin friction in a soft clay.

No observations have been published on the effects of sustained loading on piles driven in stiff clays, but there may be a reduction in resistance with time. Surface water can enter the gap and radial cracks around the upper part of the pile caused by the entry of displacement piles, and this results in a general softening of the soil in the fissure system surrounding the pile. The migration of water from the setting and hardening concrete into the clay surrounding a bored pile is again a slow process but there is some evidence of a reverse movement from the soil into the hardened concrete<sup>(4.18)</sup>. Some collected data

on reductions in resistance with time for loading tests made at a rapid rate of application on piles in stiff clays are as follows.

Type of pile	Type of clay	Change in resistance	Reference
Driven precast concrete	London	Decrease of 10 to 20% at 9 months over first test at 1 month	Meyerhof and Murdock(4.11)
Driven precast concrete	Aarhus (Septarian)	Decrease of 10 to 20% at 3 months over first test at 1 month	Ballisager(4.19)
Driven steel tube	London	Decrease of 4 to 25% at 1 year over first test at 1 month	Tomlinson(4.5)

It is important to note that the same pile was tested twice to give the reduction shown above. Loading tests on stiff clays often yield load-settlement curves of the shape shown in [Figure 11.13b](#) ([Section 11.4.2](#)). Thus the second test made after a time interval may merely reflect the lower ‘long-strain’ skin friction which has not recovered to the original peak value at the time of the second test. From the above data it is concluded that the fairly small changes in pile resistance for periods of up to one year are of little significance compared with other uncertain effects. An increase should be allowed only in the case of soft clays sensitive to remoulding.

### 4.3 Piles in cohesionless soil

#### 4.3.1 General

The classic formulae for calculating the resistance of piles in cohesionless soils follow the same form as equation 4.1. Expressed in the parameters of a cohesionless soil ( $c_u = 0$ ), the total pile resistance is given by the expression

$$Q_p = N_q \sigma'_{vo} A_b + \frac{1}{2} K_s \sigma'_{vo} \tan \delta A_s \quad \dots(4.8)$$

where  $\sigma'_{vo}$  is the effective overburden pressure at pile base level,  $N_q$  is the bearing capacity factor,  $A_b$  is the area of the base of the pile,  $K_s$  is a coefficient of horizontal soil stress which depends on the relative density and state of consolidation of the soil, the volume displacement of the pile, the material of the pile, and its shape,  $\delta$  is the characteristic or average value of the angle of friction between pile and soil, depending on whether or not the Eurocode rules are followed, and  $A_s$  is the area of shaft in contact with the soil. The factors  $N_q$  and  $K_s$  are empirical and based on correlations with static loading tests,  $\delta$  is obtained from empirical correlations with field tests. Hence when using Eurocode 7 the characteristic values of  $Q_{bk}$  and  $Q_{sk}$  should be factored to obtain the design bearing capacity  $Q$ . Table 2b of the code recommends a partial material factor of 1.2 to 1.25 for the parameter  $\tan \phi$ . The same factor can be used for  $\tan \delta$  in equation 4.8. The factor  $N_q$  depends on the ratio of the depth of penetration of the pile to its diameter and on the angle of shearing resistance  $\phi$  of the soil. The latter is normally obtained from the results on tests made in-situ (see [11.1.4](#)). The relationship between the standard penetration resistance  $N$  and  $\phi$ , as established by Peck, Hanson and Thornburn(4.20), and between the limiting static cone resistance,  $q_c$  and  $\phi$  as established by Durgunoglu and Mitchell(4.21), are shown in [Figures 4.13](#), and [4.14](#) respectively.

From tests made on instrumented full scale piles, Vesic(4.22) showed that the increase of base resistance with increasing depth was not linear as might be implied from equation 4.8, but that *rate* of increase actually decreased with increasing depth. For practical design purposes it has been assumed that the increase is linear for pile penetrations of between 10 and 20 diameters, and that below these depths the unit base resistance has been assumed to be at a constant value. This simple design approach was adequate for ordinary foundation work where the penetration depths of closed end piles were not usually much greater than 10 to 20 diameters. At these depths practical refusal was usually met with driving piles into medium dense to dense cohesionless soils.

However, the use of piled foundations for offshore petroleum production platforms has necessitated driving hollow cylindrical piles with open ends to very great depths below the sea bed to obtain resistance in skin friction to uplift loading. The assumption of a constant unit base resistance below a penetration depth of 10 to 20 diameters has been shown to be over-

conservative. It can be demonstrated by theoretical analysis, and proved by field experience, that the base resistance does not remain constant or reduce with depth. However, the *rate of increase* does increase with depth in a soil deposit of uniform density.

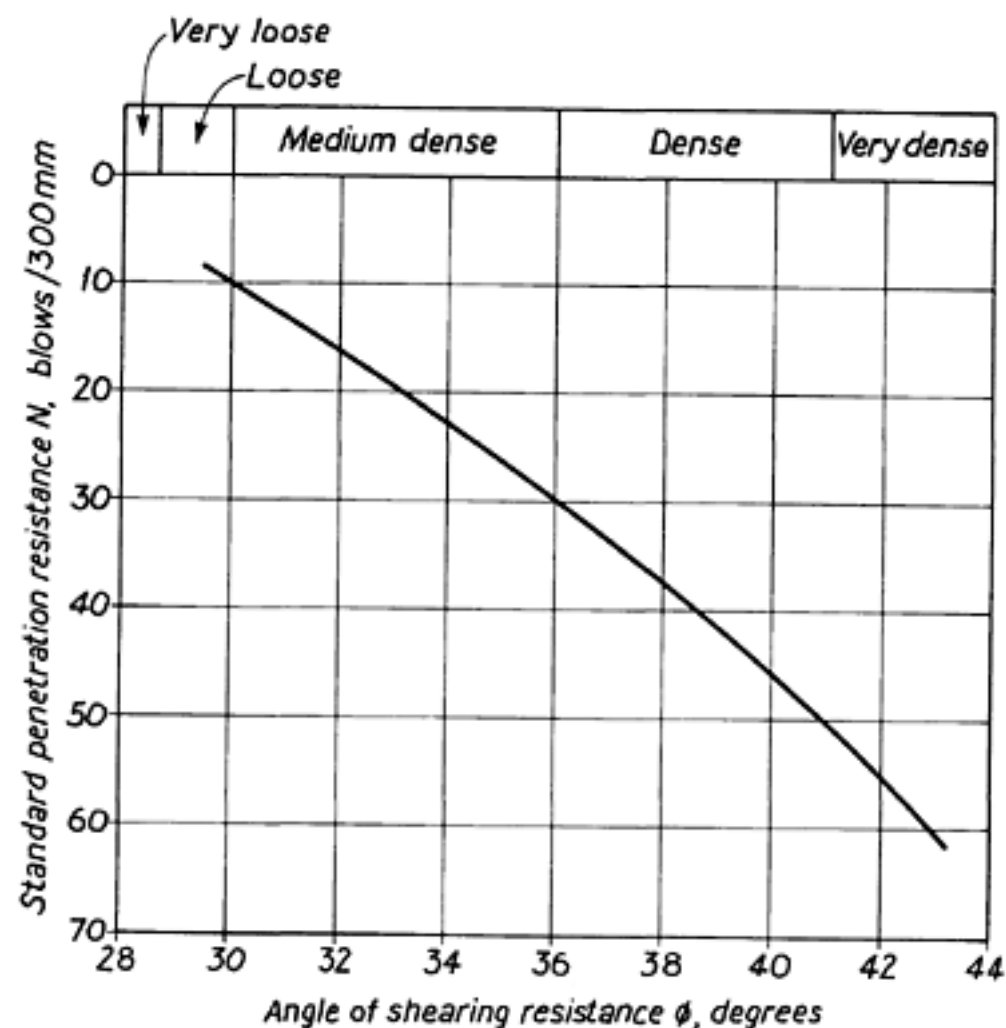


Fig. 4.13 Relationship between standard penetration test  $N$ -values and angle of shearing resistance (after Peck, Hanson and Thornburn<sup>(4.20)</sup>)

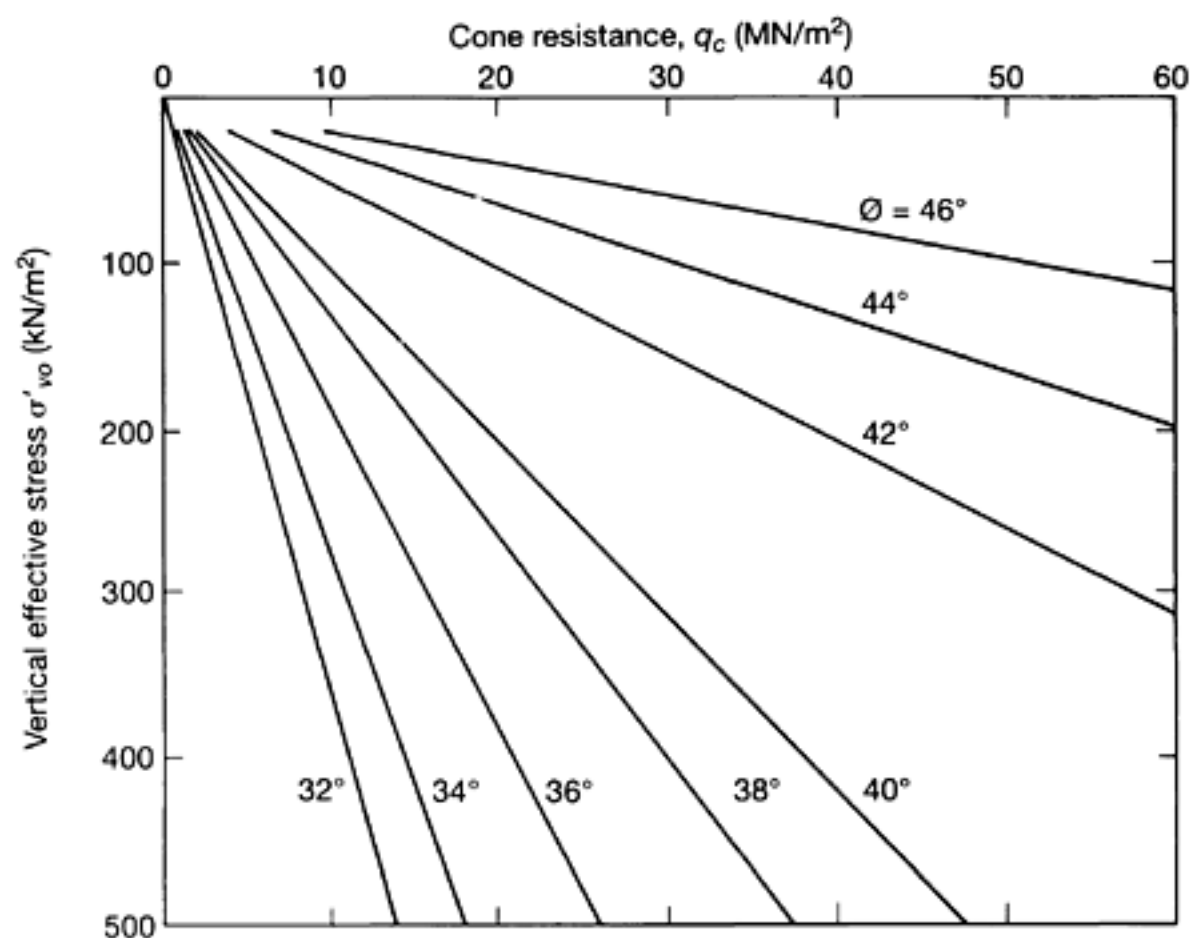


Fig. 4.14 Relationship between angle of shearing resistance and cone resistance for an uncemented, normally-consolidated quartz sand (after Durgonoglu and Mitchell<sup>(4.21)</sup>)

In further important research work, Vesic<sup>(4.23)</sup> analysed the failure pattern below the base of a pile in which a highly compressed conical wedge of soil forms beneath the base as the pile is driven or pushed down. In a loose soil the wedge moves down without producing other definable failure surfaces. In a dense soil the wedge pushes the radial shear zone into the surrounding plastic zone. The failure pattern can be analysed in terms of the expansion of a spherical cavity into an infinite soil mass behaving as an ideal elastic-plastic medium characterized by the soil strength parameters  $c$  and  $\phi$ , a deformation modulus  $E$ , the Poisson's ratio and a volume change parameter  $\Delta$ . The latter represents the average volumetric strain in the plastic zone surrounding the cavity.

The analytical work of Vesic and its application to practical pile design can be found in reference 4.23. After his untimely

death, Vesic’s concepts were reviewed and extended by Kulhawy. The results

of his researchers are summarised in a review paper<sup>(4.24)</sup>. References should be made to this paper for a description of the stages in the analysis which led to the following simplified equation for the case of a square or circular pile embedded in a cohesionless soil (the drained loading condition).

$$\text{Ultimate unit base resistance} = q_b = 0.3 B_\gamma N_\gamma \zeta_{\gamma r} + \sigma'_{vo} N_q \zeta_{qr} \zeta_{qs} \zeta_{qd} \quad \dots(4.9)$$

where $B$	is the pile width or diameter
$\gamma$	is the density of the soil
$N_\gamma$ and $N_q$	are bearing capacity factors
$\zeta_{\gamma r}$ and $\zeta_{qr}$	are rigidity factors
$\zeta_{qs}$	is a shape factor
$\zeta_{qd}$	is a depth factor

The rigidity, shape and depth factors are all related to the angle of shearing resistance,  $\phi$ . Kulhawy related these dimensional factors in combination with the bearing capacity factors  $N_\gamma$  and  $N_q$  to  $\phi$  as shown in [Figure 4.15](#). For simplicity these

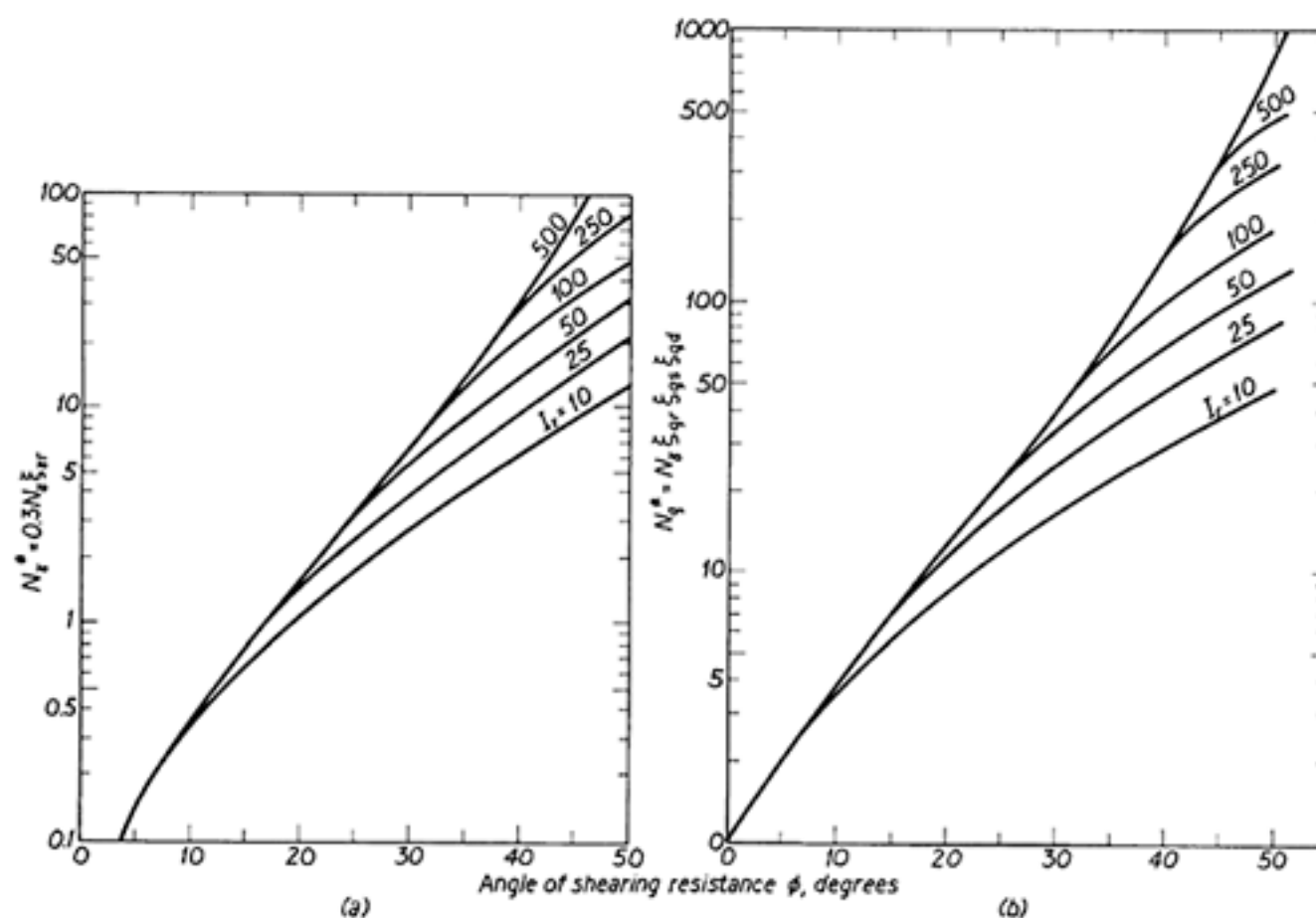
combinations are referred to as  $N_\gamma^*$  and  $N_q^*$ . It should be noted that the relationships between  $N_\gamma^*$ , and  $N_q^*$  vary  $\phi$  with the rigidity index,  $I_r$ , of the soil where

$$I_r = \frac{E}{2(1+\nu)\sigma'_{vo} \tan \phi} \quad \dots(4.10)$$

The deformation modulus,  $E$ , can be obtained by laboratory tests or from empirical relationships using results of field test such as the standard penetration test ([Figure 5.18](#)). Pressuremeter or Camkometer equipment can be used to obtain shear modulus values by in situ testing (see [Chapter 11](#)), from which  $E$  can be derived. Where static cone penetration tests have been made the deformation modulus can be obtained from [Figure 5.19](#). The value of the Poisson's ratio,  $\nu$ , is obtained from published figures. For example typical values are 0.2 for loose sands and 0.35 for dense sands.

Kulhawy<sup>(4.24)</sup> has given the following equation for determining the rigidity index and pile base level in loose and dense sands,

$$I_r = \frac{30}{\sqrt{\sigma'_{vo} \tan (28^\circ \text{ to } 26^\circ)}} \text{ for loose sands} \quad \dots(4.11)$$



*Fig. 4.15 Bearing capacity factors of  $N_\gamma$  and  $N_q$  for deep foundations in drained loading (after Kulhawy<sup>(4.24)</sup>)*

*(a) values of  $N_\gamma$*

*(b) values of  $N_q$*

and

$$I_r = \frac{110}{\sqrt{\sigma'_{vo}} \tan(40^\circ \text{ to } 37^\circ)} \text{ for dense sands} \quad \dots(4.12)$$

where the effective overburden pressure  $\sigma'_{vo}$  is expressed in  $\text{ton}/\text{ft}^2$

The first term in equation 4.9 is small in relation to the second. For penetrations deeper than five pile diameters it is typically less than 10% of the second term. Therefore for deep penetrations the first term can be neglected. This was done by Kulhawy who calculated the ultimate base resistance for very loose and very dense sands in dry and saturated conditions (that is, in the absence of groundwater and piles wholly below groundwater level) for a range of depths down to a penetration of 30m. Unit weights of  $18.1\text{kN}/\text{m}^3$  and  $19.7\text{kN}/\text{m}^3$  were used for the dry loose and dense sands respectively. These values shown in [Figure 4.16](#) may be used for preliminary design purposes in uniform sand deposits. For densities between very loose and very dense the base resistance values can be obtained by linear interpolation.

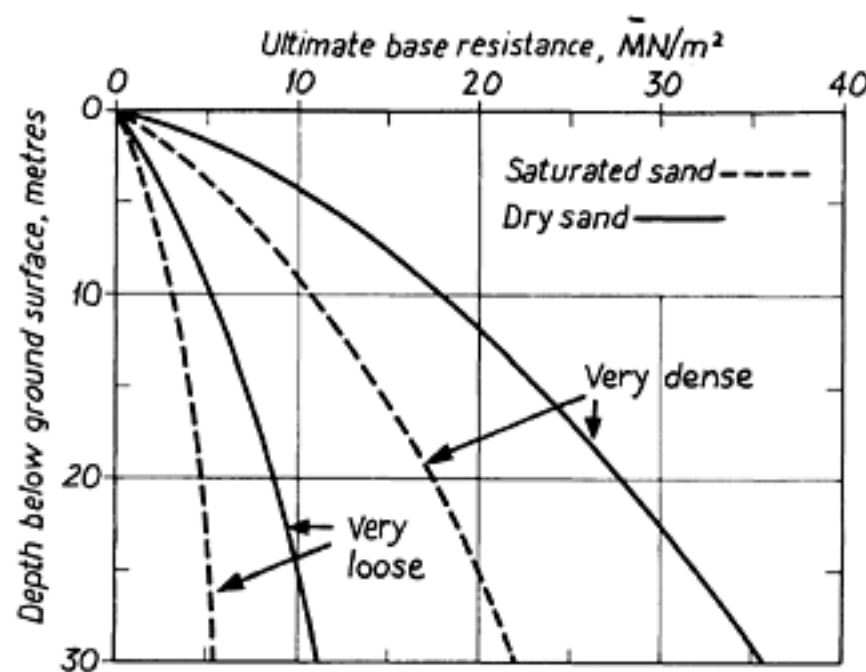


Fig. 4.16 Approximate ultimate base resistance for foundations in sand (after Kulhawy<sup>(4.24)</sup>)

[Figure 4.15b](#) shows that  $N_q^*$  values decrease with decreasing values of the rigidity index. The latter decreases with increasing depth of the pile base (equations 4.11 and 4.12). Also the  $\phi$  values decrease with increasing confining pressure and hence with increase of depth. This explains the trend to the reduction in the rate of increase of base resistance with depth referred to previously.

Reduction in the bearing capacity factor  $N_q$  with increase in penetration depths was also shown by Berezantsev *et al.*<sup>(4.25)</sup>.

Their values of  $N_q$  related to  $\phi$  and depth/width ratios as shown in [Figure 4.17](#). Ultimate base resistance values using these

factors have been calculated for a closed end pile of 1220mm diameter driven into a loose sand having a uniform unit submerged weight of  $7.85\text{kN}/\text{m}^3$  in [Figure 4.18a](#). The angle of shearing resistance of the sand has been assumed to decrease from  $30^\circ$  at the soil surface to  $28^\circ$  at 30m depth. It will be seen that the Berezantsev  $N_q$  values gave lower base resistance

than those of Kulhawy.

A similar comparison was made for the 1220mm pile driven into a dense sand having a uniform unit submerged density of  $10.8\text{kN}/\text{m}^3$ . The angle of shearing resistance was assumed to decrease from  $40^\circ$  at the soil surface to  $37^\circ$  at 30m. [Figure 4.18b](#) shows that the Kulhawy base resistance values in this case were lower than those of Berezantsev. The penetration depths in [Figure 4.18b](#) have been limited to 20m. This is because the pile capacity as determined by the base resistance alone exceeds the value to which the pile can be driven without causing excessive compression stress in the pile shaft. For example taking a heavy section tubular pile with a wall thickness of 25mm in high yield steel and limiting the compression stress to twice the value given by the allowable working stress of 0.3 times the yield stress, the ultimate pile load is 9.7MN. This exceeded at 12m and 20m penetration using the Berezantsev and Kulhawy factors respectively. The high base resistances which can be obtained in dense sands often make it impossible to drive piles for marine structures to a sufficient depth to obtain the required resistances to uplift and lateral loading. This necessitates using open end piles, possibly with a diaphragm across the pile at a calculated height above the toe as described in 2.2.4.

The second term in equation 4.8 is used for calculating the skin friction on the pile shaft. The value of  $K_s$  is critical to the

evaluation of the skin friction and is the most difficult to determine reliably because

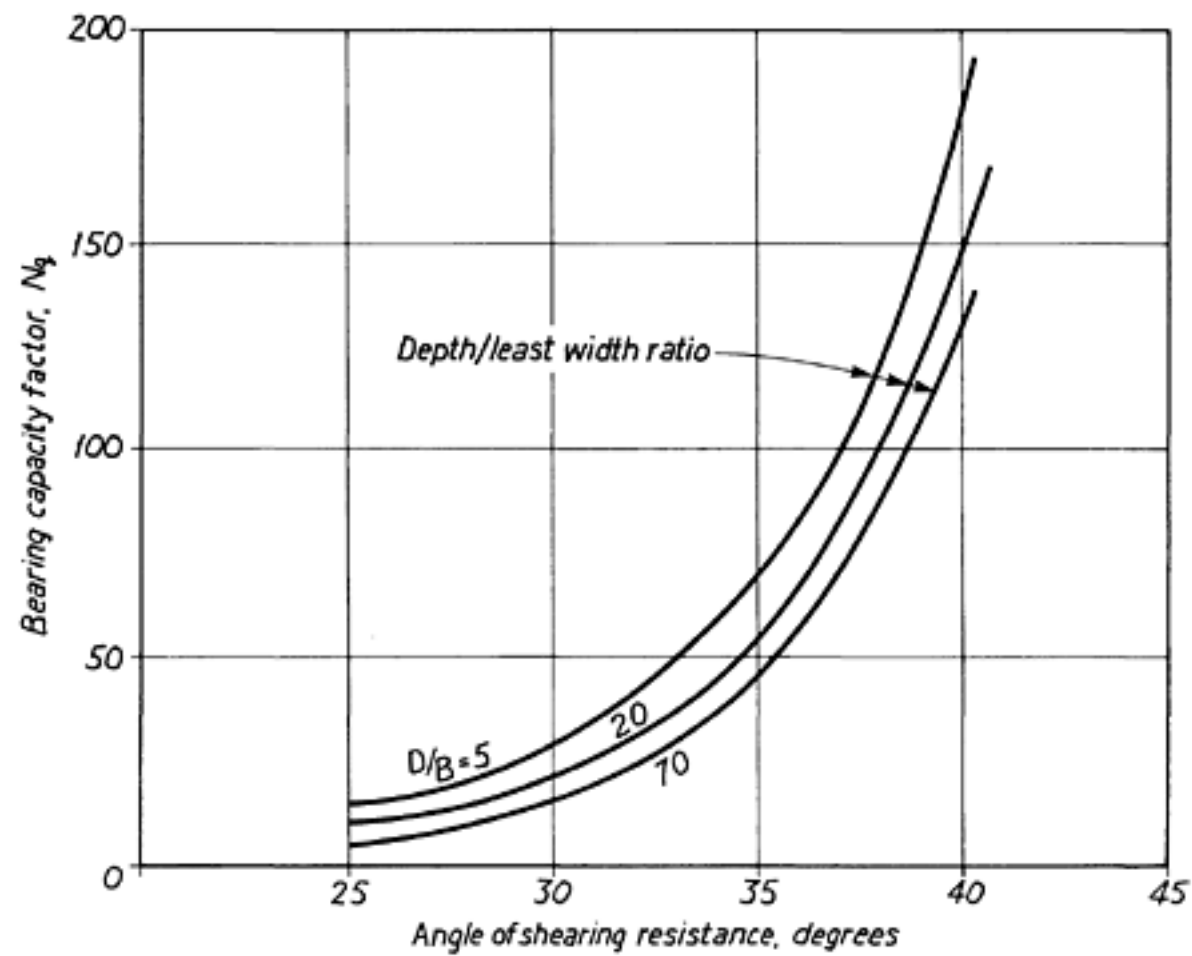


Fig. 4.17 Bearing capacity factors of Berezantsev et al. (4.25)

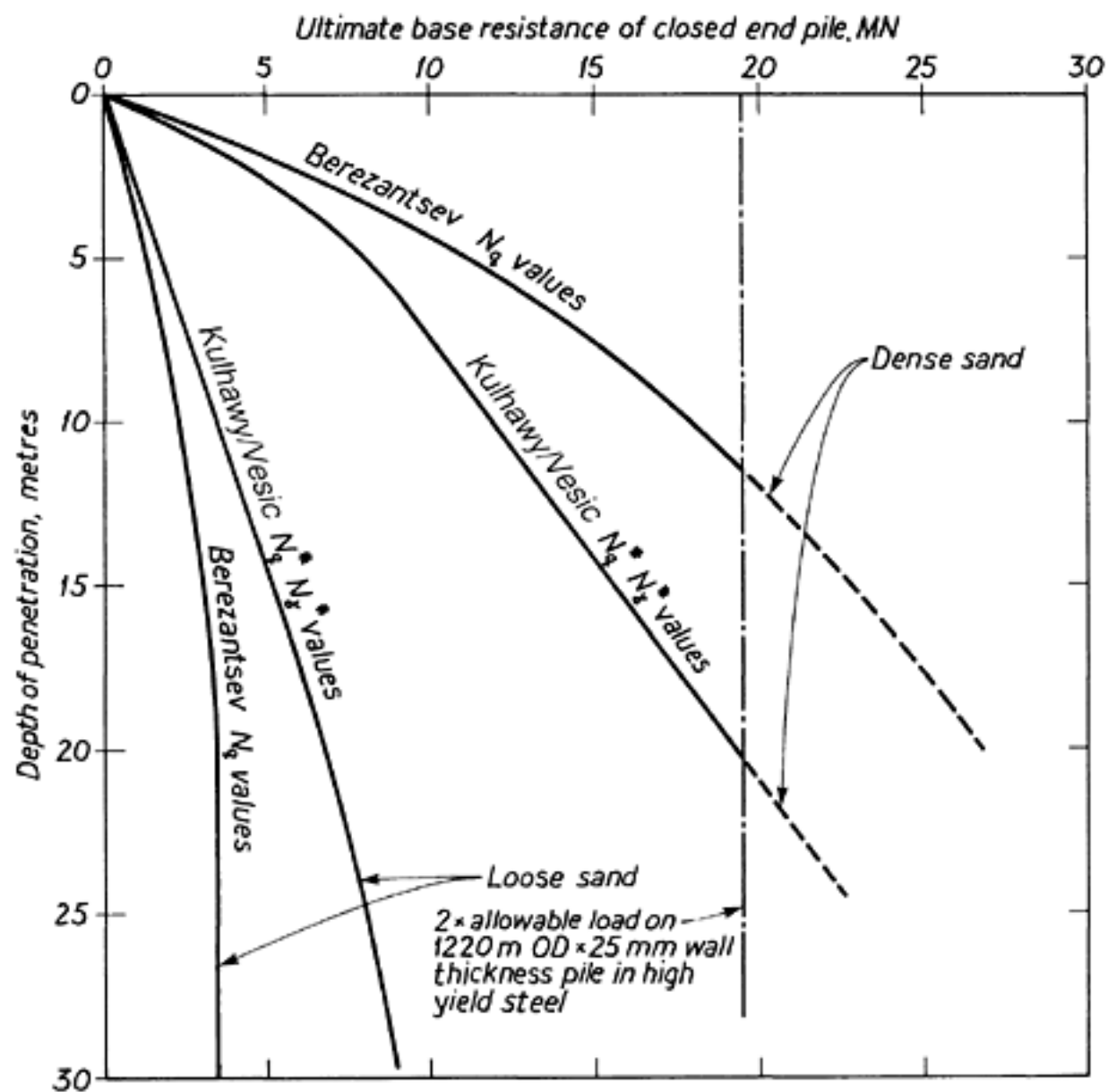


Fig. 4.18 Base resistance vs. penetration depth for 1220mm dia. closed-end pile driven into sand  
(a) loose sand (b) dense sand

it is dependent on the stress history of the soil and the changes which take place during installation of the pile. In the case of driven piles displacement of the soil increases the horizontal soil stress from the original  $K_o$  value. Drilling for bored piles

can loosen a dense sand, and thereby reduce the horizontal stress.

In a normally consolidated sand the in situ  $K_o$  is constant with depth and the value of  $K_s$  is then modified only by the

installation process. Kulhawy<sup>(4.24)</sup> gives the following values of  $K_s$  related to  $K_o$ , as shown in [Table 4.1](#).

Table 4.1 Values of the coefficient of horizontal soil stress,  $K_s$ 

Installation method	$K_s/K_o$
Driven piles, large displacement	1 to 2
Driven piles, small displacement	0.75 to 1.25
Bored and cast-in-place piles	0.70 to 1
Jetted piles	0.50 to 0.7

Typical values of  $K_o$  for a normally consolidated sand are:

Relative density	$K_o$
Loose	0.5
Medium-dense	0.45
Dense	0.35

If the cohesionless soil deposit is over-consolidated, that is to say, if it has been subjected to an overburden pressure at some previous time in its history, and the pressure then removed by erosion, the  $K_o$  values can be very much higher, of the order of

1 to 2 or more. It is possible to determine whether or not a cohesionless soil deposit is over-consolidated by reference to its geological history or by testing in the field using standard penetration tests or static cone tests. Normally consolidated soils show low penetration values at the surface increasing roughly linearly with depth. Over-consolidated soils show high values at shallow depths, sometimes decreasing at the lower levels.

Kulhawy<sup>(4.24)</sup> points out that some over-consolidation is the rule rather than the exception in most soil deposits. This taken in conjunction with the decrease in  $\phi$  value with increasing overburden pressure can result in calculated values of unit skin friction decreasing with depth, thus leading to the commonly used design rule that the unit skin friction remains at a constant value below a penetration depth of 10 to 20 diameters.

Kulhawy related the angle of friction,  $\delta$ , between the pile surface and the soil to the average effective angle of shearing resistance,  $\bar{\phi}$ , over the length of the pile shaft as shown in [Table 4.2](#).

Table 4.2 Values of the angle of pile to soil friction for various interface conditions

Pile/soil interface condition	Angle of pile/soil friction, $\delta$
Smooth (coated) steel/sand	$0.5\bar{\phi}$ to $0.7\bar{\phi}$
Rough (corrugated) steel/sand	$0.7\bar{\phi}$ to $0.9\bar{\phi}$
Precast concrete/sand	$\left. \begin{array}{l} 0.8\bar{\phi} \text{ to } 1.0\bar{\phi} \\ 1.0\bar{\phi} \end{array} \right\}$
Cast-in-place concrete/sand	
Timber/sand	$0.8\bar{\phi}$ to $0.9\bar{\phi}$

Published records of observed skin friction values obtained from tests on instrumented piles or from pull-out tests have shown that the *average* skin friction along the pile shaft does not greatly exceed 100kN/m<sup>2</sup>. The author uses this figure as peak value for skin friction on straight sided piles. Tapered piles can mobilize much higher skin friction because of the higher horizontal stress at the pile/soil interface. Nordlund<sup>(4.26)</sup> indicates that the unit skin friction is increased by a factor of at least 1.5 for an angle of taper of 0.5 degrees (approximately 1%). The effects of cyclic loading on the skin friction resistance of driven piles in cohesionless soils are discussed in 6.2.2.

Typical curves for the cumulative skin friction, base resistance and total resistance mobilized with increasing depth for a precast concrete pile driven into a medium dense sand are shown in [Figure 4.19](#).

### 4.3.2 Driven piles in cohesionless soils

Driving piles into loose cohesionless soils densifies the soil around the pile shaft and beneath the base. Increase in shaft

friction can be allowed by using the higher values of  $K_s$  related to  $K_o$  from [Table 4.1](#). However, it is not usual to allow any increase in the  $\phi$  values and hence the bearing capacity factors  $N_q$  or  $N_q^*$  caused by soil compaction beneath the pile toe.

The reduction in the rate of increase in end bearing resistance with increasing depth has been noted above. A further reduction is given when piles are driven into soils consisting of weak friable particles such as calcareous soils consisting of carbonate particles derived from disintegrated corals and shells. The soil tends to degrade under

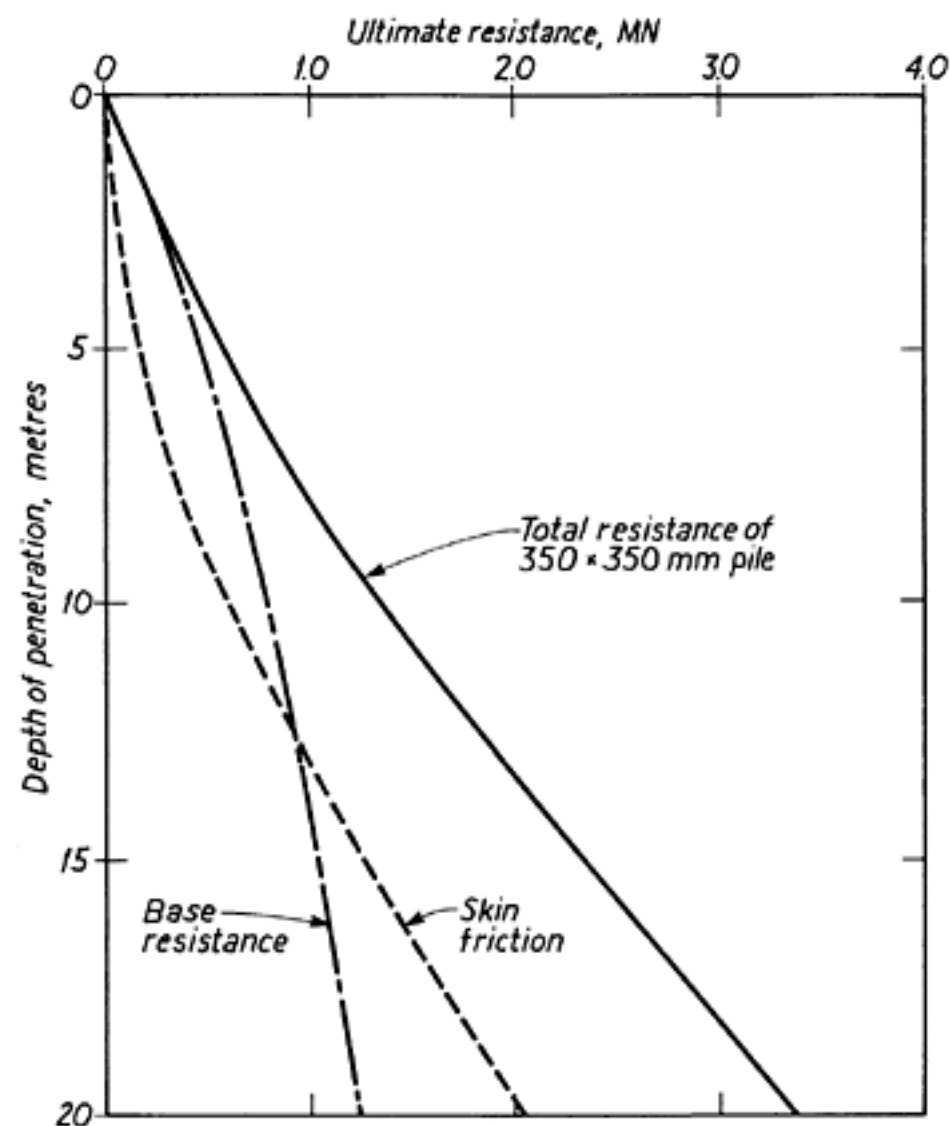


Fig. 4.19 Typical curves showing development of cumulative skin friction, base resistance and total resistance for a precast concrete pile driven into a medium-dense sand (groundwater level at ground level).

the impact of hammer blows to a silt-sized material with a marked reduction in the angle of shearing resistance. Because of these factors the few published records for driven piles which have been observed from instrumented tests have not shown values of the ultimate base resistance much higher than  $11 \text{ MN/m}^2$ . The author uses this figure for closed-end piles as a practical peak value for ordinary design purposes but recognises that higher resistances up to a peak of  $22 \text{ MN/m}^2$  may be possible when driving a pile into a dense soil consisting of hard angular particles. Such high values should not be adopted for design purposes unless proved by loading tests.

Figure 4.18b shows that the base resistance of a closed-end pile driven into a dense sand can reach the maximum compressive stress to which the pile can be subjected during driving at a relatively short penetration. The total skin friction for short penetrations is a small proportion only of the total pile resistance. Therefore, if the peak base resistance of  $11 \text{ MN/m}^2$  is used for design there is no advantage in attempting to drive piles deeply into medium-dense to dense soils with the risk of pile breakage in order to gain a small increase in shaft friction. On the other hand the proportion of skin friction on a pile driven into a loose sand is quite high relative to the base resistance. In the case of the  $1220 \text{ mm}$  closed-end pile shown in Figure 4.18a, the total skin friction at  $30 \text{ m}$  penetration is roughly equal to or is about 30% of the base resistance depending on whether the bearing capacity factors of Berezantsev or Kulhawy are used to determine the latter. Hence deep driving into a loose sand deposit may be the only means of using economically the allowable load on the pile shaft.

H-section piles are not economical for carrying high compression loading when driven into cohesionless soils. Plugging of the sand does not occur in the area between the web and flanges. The base resistance is low because of the small cross-sectional area. Accordingly the pile must be driven deeply to obtain worthwhile skin friction. The latter is calculated on the total surface of the web and flanges in contact with the soil. At Nigg in Scotland soil displacements of only a few centimetres were observed on each side of the flanges of H-piles driven about  $15 \text{ m}$  into silty sand, indicating that no plugging had occurred over the full depth of the pile shaft. The base resistance of these piles can be increased by welding short stubs or wings (see Figure 2.18a) at the toe. Some skin friction is lost on the portion of the shaft above these base enlargements.

The maximum working stress on proprietary types of precast concrete jointed piles is in the range of  $10 \text{ MN/m}^2$  to  $17 \text{ MN/m}^2$ . Therefore if the peak design ultimate resistance of  $11 \text{ MN/m}^2$  is adopted

the piles will have to develop substantial skin friction to enable the maximum working load to be utilised. This is feasible in loose to medium-dense sands ([Figure 4.19](#)) but impracticable in dense sands or medium-dense to dense sandy gravels. In the latter case peak base resistance values higher than  $11\text{ MN/m}^2$  may be feasible, particularly in flint gravels.

### 4.3.3 Piles with open-ends driven into cohesionless soils

It was noted in [Section 4.3.1](#) that it is frequently necessary to drive piles supporting off-shore petroleum production platforms to a very great depth below the sea bed in order to obtain the required resistance to uplift loading by skin friction. Driving tubular piles with open ends is usually necessary to achieve the required penetration depth. Driving is relatively easy, even through dense soils, because with each blow of the hammer the overall pile diameter increases slightly thereby pushing the soil away from the shaft. When the hammer is operating with a rapid succession of blows the soil does not return to full contact with the pile. A partial gap is found around each side of the pile wall allowing the pile to slip down. Flexure of the pile in the stick-up length above sea bed also causes low resistance to penetration.

At some stage during driving a plug of soil tends to form at the pile toe after which the plug is carried down with the pile. At this stage the base resistance increases sharply from that provided by the net cross-sectional area of the pile shoe to some proportion (not 100%) of the gross cross-sectional area.

The stage when a soil plug forms is uncertain; it may form and then yield as denser soil layers are penetrated. It was noted in [Section 2.2.4](#) that 1067mm steel tube piles showed little indication of a plug moving down with the pile when they were driven to a depth of 22.6m through loose becoming medium dense to dense silty sands and gravels in Cromarty Firth. No plugging, even at great penetration depths, may occur in uncemented or weakly cemented calcareous soils. Dutt *et al.* ([4.27](#)) described experiences when driving 1.55m diameter steel piles with open ends into carbonate soils derived from coral detritus. The piles fell freely to a depth of 21m below sea bed when tapped by a hammer with an 18-tonne ram. At 73m the driving resistance was only 15 blows/0.3m.

Generally the pile design for offshore gas and petroleum platforms is required by certifying authorities to conform to the recommendations of the American Petroleum Institute ([3.4](#)) For the case of open-end piles API recommend that the total pile resistance should be taken as the sum of the external skin friction, the end-bearing on the pile wall annulus, and the total internal skin friction; or the end-bearing resistance of the plug, whichever is less. For piles considered to be solidly plugged the end-bearing is assumed to act over the entire cross-sectional area of the pile. These are somewhat unrealistic concepts. In order to mobilize total plug resistance in internal skin friction the relative pile/soil movement at the top of the plug must be  $\frac{1}{2}\%$  of the pile diameter. Thus with a large-diameter pile and a long plug a considerable movement at the toe will be needed to mobilize peak skin friction resistance over the whole length.

Another uncertain factor is the ability of the soil plug to achieve resistance to yielding by arching action across the pile interior in order to provide a resistance equivalent to that of a solid end pile. Research has shown that the arching capacity is related principally to the diameter of the pile. Clearly it is unrelated to the in-situ density of the soil below the pile toe because the soil forming the plug is compacted by the pile driving. Kishida and Isemoto ([4.28](#)) set up steel cylinders of five different diameters ranging from 300 to 1000mm. They were filled with dry sand 'rained-in' to give plugs of lengths up to four diameters. The force required to push up the sand plug by a ram acting on the base of the cylinder was measured for each diameter and plug length. The results are shown in summary form in [Figure 4.20](#). They were compared with analyses made by a finite element method and it was found possible to reproduce the laboratory results with reasonably close agreement. Kishida and Isemoto concluded that

1. The sand within two pile diameters from the bottom of the plug was heavily compacted by the force from the ram.
2. The resistance to the force was provided mainly by frictional resistance to the plug arching over a length of two pile diameters.
3. The sand above two pile diameters was scarcely compacted, but the pressure of this overlying sand added substantially to the force required to push up the plug.
4. The ultimate unit force decreased with increase of cylinder diameter.

[Figure 4.21](#) shows the observed end-bearing resistance of open-end piles plugged with sand at six sites. At all the sites the piles terminated in dense or very dense sands or gravelly sands. The results in [Figure 4.21](#) cannot be compared directly with those shown in [Figure 4.20](#) because the plugs were formed in different ways—in the research work by static force, and at the six sites by driving the piles. There is no apparent trend of decreasing end-bearing resistance with increase of pile diameter or with reduction

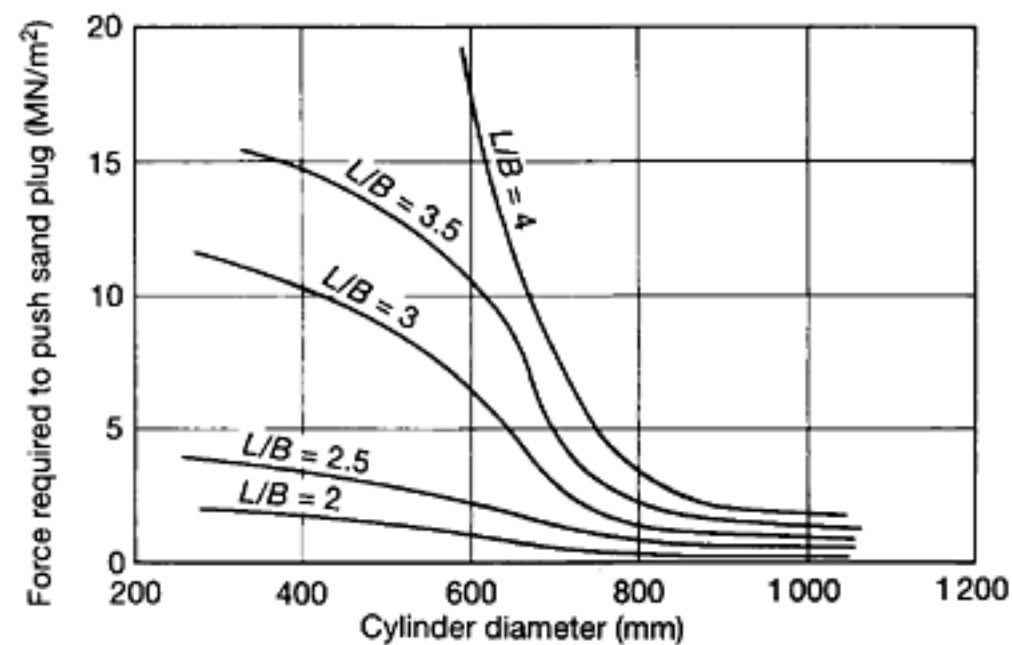


Fig. 4.20 The effect of cylinder diameter on force required to push up sand plug (after Kishida and Isemoto<sup>(4.28)</sup>)

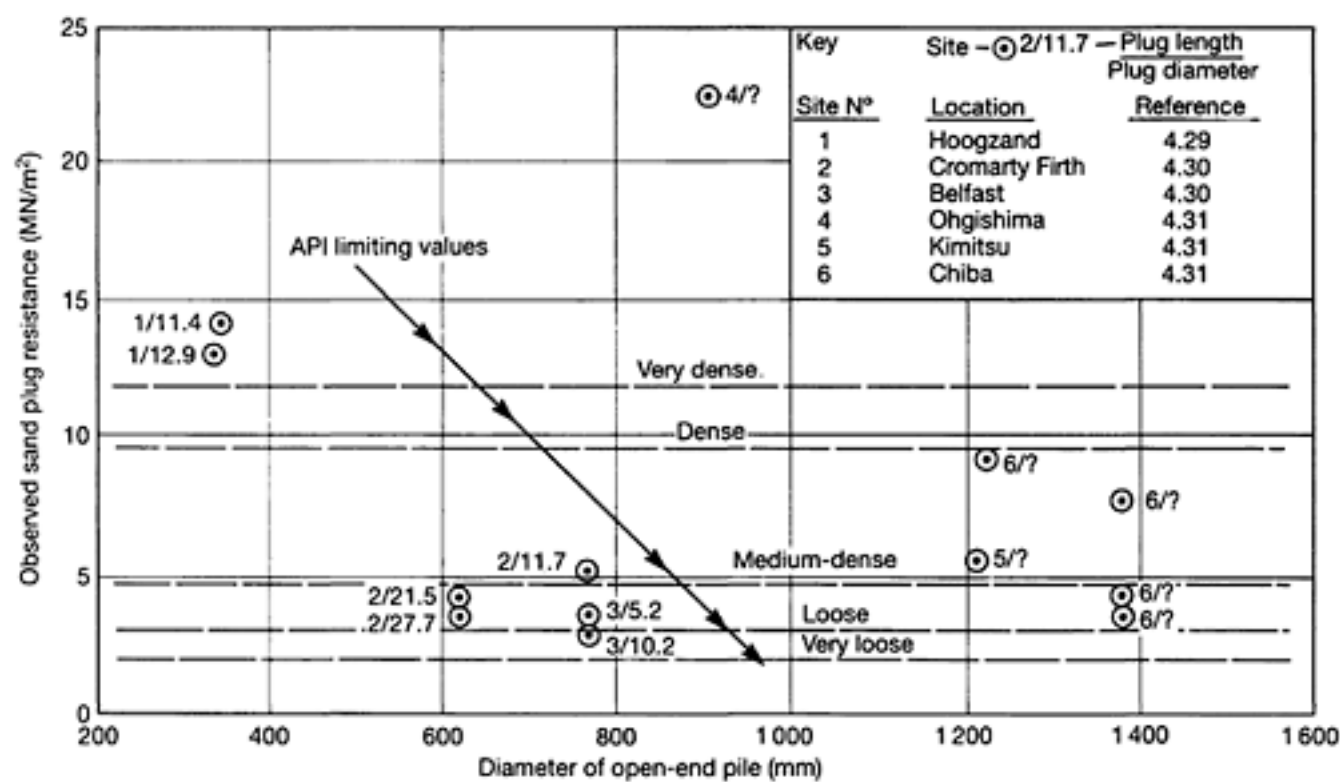


Fig. 4.21 Observed ultimate base resistance of open end steel tube piles plugged with sand (Based on a private communication from D.W.Might.)

in the ratio of plug length to diameter. Because the piles were driven into dense or very dense soils it is evident that the yielding occurred within the plug, not by failure of the soil below the plug.

Figure 4.21 makes it clear that if the API limiting values of base resistance had been used for design the end-bearing capacity of the piles would have been grossly over-estimated. The results suggest a limiting value of  $5 \text{ MN/m}^2$  for open-end piles irrespective of diameter or of the density of the soils into which they are driven. This value should be used in conjunction with a safety factor of 2.5.

Measures to increase the base resistance of open-end piles driven into cohesionless soils are described in Section 8.3.

#### 4.3.4 Grouted driven piles

The problem of low values of unit skin friction caused by the degradation of friable soil particles when driving piles into uncemented or weakly-cemented calcareous soils was mentioned in Section 4.3.2. A process of injecting grout under pressure into the interface between the exterior of the shaft and the surrounding soil after completion of driving a tubular steel pile has been developed by the French company Solmarine. Barthelemy *et al.*<sup>(4.32)</sup> claimed that the unit skin friction on piles driven into calcareous soils at offshore locations in Australia was increased by ten times or more.

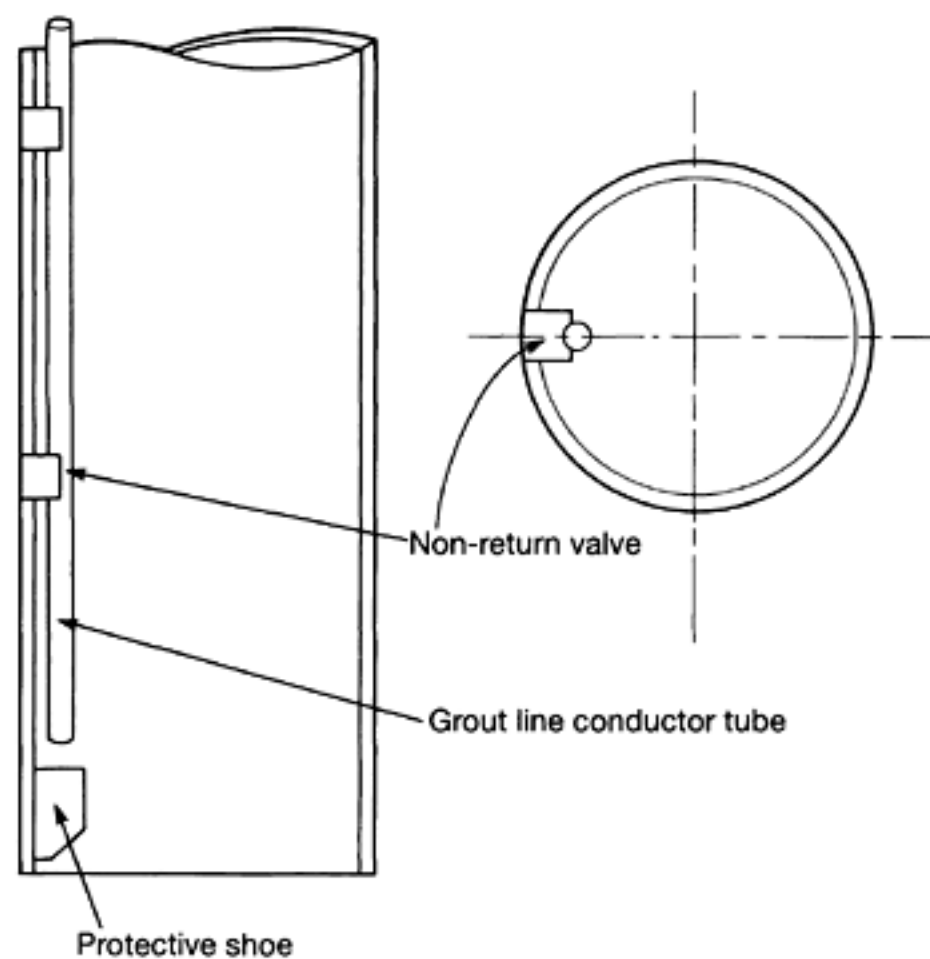


Fig. 4.22 Equipment for skin grouting around tubular pile

The process utilizes the tube-à-manchette device for injecting and re-injecting grout under pressure selectively at any predetermined level. The grouting arrangements are shown in [Figure 4.22](#). The grout is injected through small openings spaced at selected vertical intervals down the pile wall. The holes are covered by a rubber sheet such that back-flow of the grout cannot occur. Access to these non-return valves is made by a 50 to 100mm diameter conductor tube extending from pile head to toe level. To inject grout at any selected point the grout line, with a double packer to close off the tube above and below the non-return valve, is lowered down the tube and the packers are inflated. The grout is injected and the grout line is then moved to the next valve position. A return line allows grout to be circulated back to the mixing plant at the surface. The aim is to obtain, as far as practicable, a uniform skin of grout around the pile shaft.

#### 4.3.5 Driven-and-cast-in-place piles in cohesionless soils

Both the base resistance and shaft friction of driven-and-cast-in-situ piles can be calculated in the same way as described for driven piles in the preceding section. The installation of driven-and-cast-in-situ types does not loosen the soil beneath the base in any way, and if there is some loosening of the soil around the shaft as the driving tube is pulled out the original state of density is restored, if not exceeded, as the concrete is rammed or vibrated into place while pulling out the tube. Loosening around the shaft must be allowed for if no positive means are provided for this ramming or vibrating. The provision of an enlarged base adds considerably to the end-bearing resistance of these piles in loose to medium-dense sands and gravels. The gain is not so marked where the base is formed in dense soils, since the enlargement will not greatly exceed the shaft diameter and, in any case, full utilization of the end-bearing resistance may not be possible because of the need to keep the compressive stress on the pile shaft within safe limits (see [Table 2.10](#)). The Code of Practice, BS 8004, allows a maximum working stress of 0.25 times the cube compression strength. If the latter is taken as 25MN/m<sup>2</sup>, the allowable stress in the pile shaft at the working load is 6.25MN/m<sup>2</sup>. Therefore only a minor base enlargement is required if the peak ultimate base resistance of 11MN/m<sup>2</sup> is adopted.

#### 4.3.6 Bored-and-cast-in-place piles in cohesionless soils

If drilling for the piles is undertaken by baler (see [3.3.5](#)) or by grabbing under water there is considerable loosening of the soil beneath the pile toe as the soil is drawn or slumps towards these tools. This causes a marked reduction in end-bearing resistance and skin friction, since both these components must then be calculated on the basis of a low relative density ( $\phi = 28$  to  $30^\circ$ ). Only if the piles are drilled by power auger or reverse-circulation methods in conjunction with a bentonite slurry or by drilling under water using a base grouting technique as described in 3.3.7 can the end-bearing resistance be calculated

on the angle of shearing resistance of the undisturbed soil. However, the effects of entrapping slurry beneath the pile toe<sup>(3.11)</sup> must be considered. Loading tests should be made to prove that the bentonite technique will give a satisfactory end-bearing resistance. If there are indications that the entrapment of slurry beneath the toe cannot be avoided, the appropriate reduction in resistance should be made. Fleming and Sliwinski<sup>(4.14)</sup> suggest that the shaft friction on bored piles, as calculated from a coefficient of friction and the effective lateral pressure, should be reduced by 10 to 30% if a bentonite slurry is used for drilling in a cohesionless soil. Tests reported by Broms and Hill<sup>(4.33)</sup> showed that the skin friction on steel shell step-tapered piles inserted and driven into a hole pre-drilled by using bentonite slurry was only 35% of the value observed with holes pre-drilled without slurry. In contrast to this observation, however, the satisfactory use of bentonite techniques for constructing bored piles in sand has been indicated by the research of Reese *et al.*<sup>(3.11)</sup>.

The effects of loosening of the soil by conventional drilling techniques on the skin-frictional and base resistances of a bored pile in a dense sand is well illustrated by the comparative loading tests shown in Figure 4.22(a). Bored piles having a nominal shaft diameter of 483mm and a driven precast concrete shell pile (West's pile) with a shaft diameter of 508mm were installed through peat and loose fine sand into dense sand. The bored piles with toe levels at 4.6 and 9.1m failed at 220 and 350kN respectively, while the single precast concrete pile which was only 4m long carried a 750kN test load with negligible settlement.

#### 4.3.7 The use of in-situ tests to predict the ultimate resistance of piles in cohesionless soils

It has been noted that the major component of the ultimate resistance of piles in dense cohesionless soils is the base resistance. However, Figures 4.15 and 4.18 show that the values of  $N_q$  are very sensitive

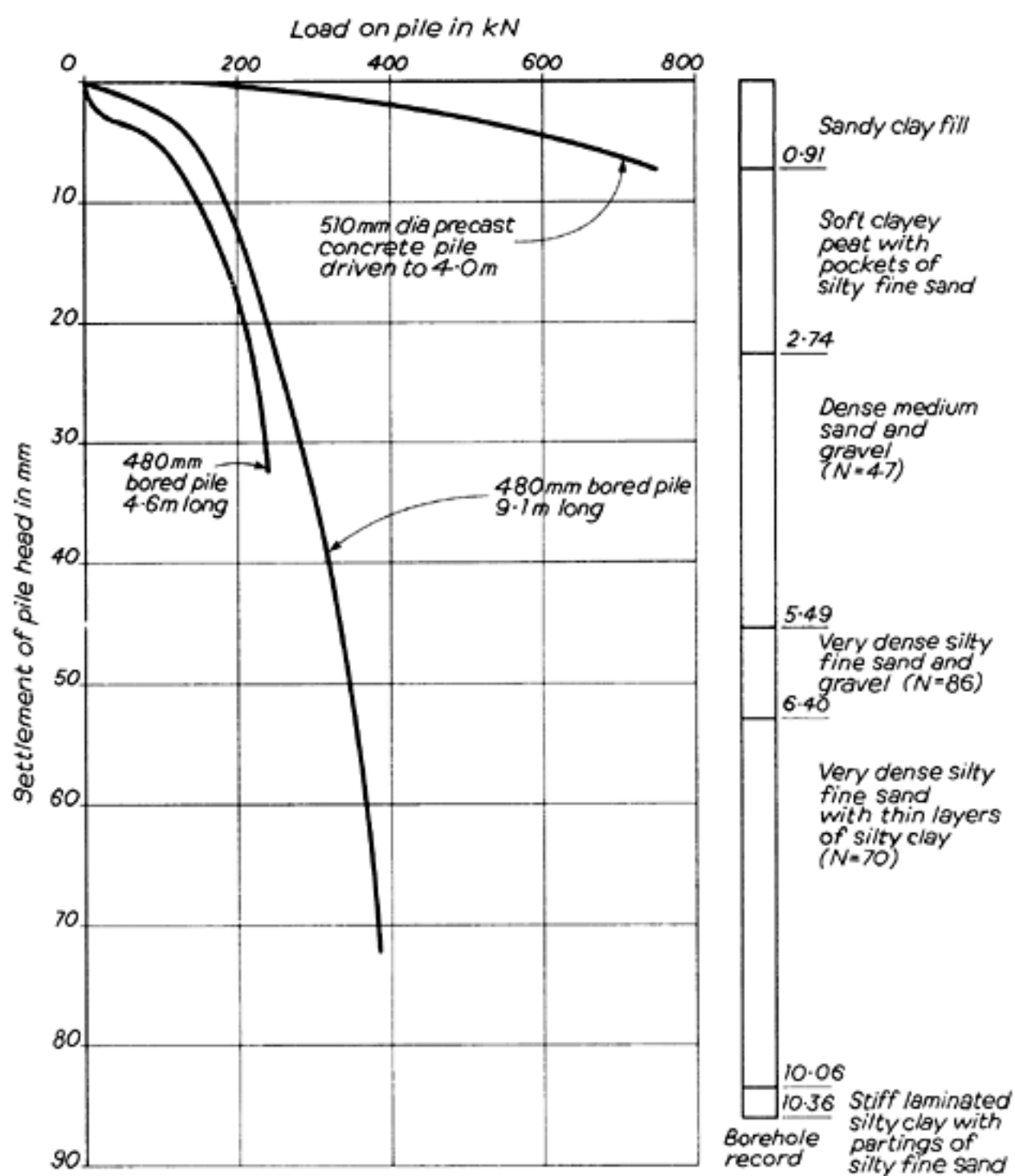


Fig. 4.22(a) Comparison of compressive resistance of driven piles and bored-and-cast-in-situ piles in dense to very dense cohesionless soils

to the values of the angle of shearing resistance of the soils. These values are obtained from in-situ tests made in boreholes, and if the boring method has loosened the soil, which can happen if incorrect techniques are used (see 11.1.4), then the base resistance of any form of driven pile is grossly underestimated. It is very unlikely that the boring method will compact the soil, and thus any over-estimation of the shearing resistance is unlikely.

A reliable method of predicting the skin friction and base resistance of driven and driven-and-cast-in-place piles is to use the static cone penetrometer (Dutch cone) at the site investigation stage (see 11.1.4). This equipment produces a curve of cone penetration resistance with depth (Figure 4.49). Extensive experience with pile predictions based on the cone penetrometer in Holland has produced a set of design rules which have been summarised by Meigh(4.34).

Although most engineers in Holland and others elsewhere base skin friction values on the measured local sleeve friction ( $f_s$ ), the author prefers to use established empirical correlations between unit skin friction and cone resistance ( $q_c$ ). This is because the cone resistance values are more sensitive to variations in soil density than the sleeve friction and identification of the soil type from the ratio of  $q_c$  to  $f_s$  is not always clear-cut. Empirical relationships of pile skin friction to cone resistance are shown in Table 4.3.

Table 4.3 Relationships between pile skin friction and cone resistance (after Meigh(4.34))

Pile type	Ultimate unit skin friction
Timber	$0.012 q_c$
Precast concrete	$0.012 q_c$
Precast concrete enlarged base*	$0.018 q_c$
Steel displacement	$0.012 q_c$
Open-ended steel tube†	$0.008 q_c$
Open-ended steel tube driven into fine to medium sand	0.0033

\* Applicable only to piles driven in dense groups otherwise use 0.003 where shaft size is less than enlarged base.

† Also applicable to H-section piles.

A limiting value of 0.12MN/m<sup>2</sup> is used for the ultimate skin friction. The values shown in Table 4.3 are applicable to piles under static compression loading and a safety factor of 2.5 is used for  $q_c$  values obtained from the electrical cone and 3.0 for the mechanical cone (see Chapter 11). A somewhat higher safety factor would be used for piles subjected to cyclic compression loading to allow for degradation of the assumed siliceous sand (see Section 6.2.2 for piles carrying uplift loading).

Cone-resistance values should not be used to determine the skin friction to the shafts of bored piles. This is because of the loosening of the soil caused by drilling as described in the preceding section.

The end-bearing resistance of piles is calculated from the relationship

$$q_{ub} = \bar{q}_c \quad \dots(4.13)$$

where  $\bar{q}_c$  is the average cone resistance within the zone influenced by stresses imposed by the toe of the pile. This average value can be obtained by plotting the variation of  $q_c$  against depth for all tests made within a given area. An average curve is

then drawn through the plots either visually or using a statistical method. The *allowable* base pressure is then determined from the value of the average curve at pile toe level divided by the appropriate safety factor (Figure 4.23a). The value of the safety factor will depend on the scatter of results. It is normally 2.5 but it is a good practice to draw a lower bound line through the lower cone-resistance values, ignoring sharp peak depressions provided that these are not clay bands in a sand deposit. The allowable base pressure selected from the average curve should have a small safety factor when calculated from the lower bound  $q_c$  at the toe level (Figure 4.23a).

The method generally used in the Netherlands is to take the average cone resistance ( $\bar{q}_{c-1}$ ) over a depth of up to four pile

diameters below the pile toe, and the average  $(\bar{q}_{c-2})$  eight pile diameters above the toe as described by Meigh(4.34). The ultimate base resistance is then

$$q_{ub} = \frac{\bar{q}_{c-1} + \bar{q}_{c-2}}{2} \quad \dots(4.14)$$

The shape of the cone-resistance diagram is studied before selecting the range of depth below the pile to obtain  $\bar{q}_{c-1}$ . Where the  $q_c$  increases continuously to a depth of  $4D$  below the toe, the average value

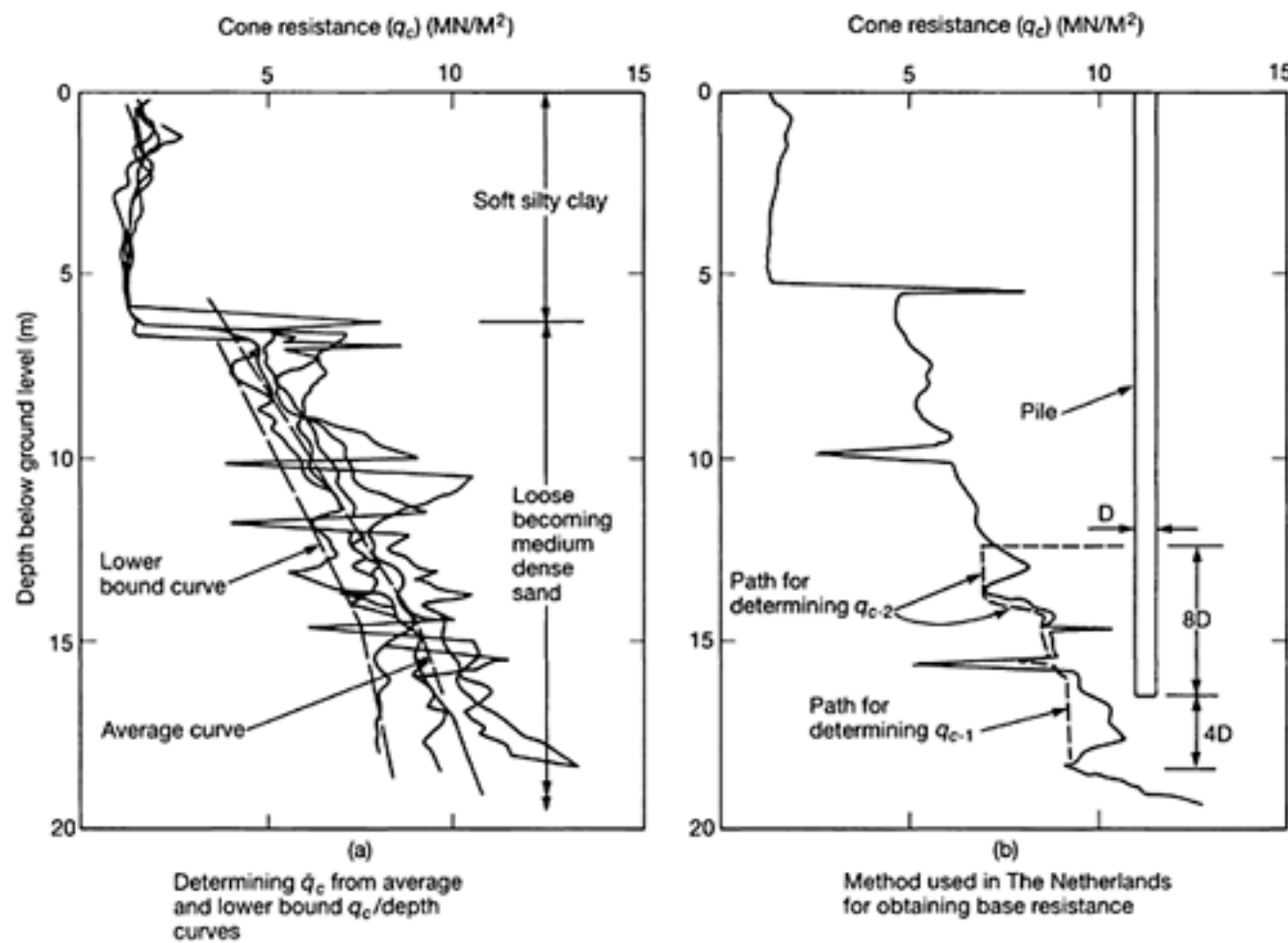


Fig. 4.23 Use of static cone penetration tests (CPT) to obtain design values of average cone resistance ( $\bar{q}_c$ ) in cohesionless soils

of  $\bar{q}_{c-1}$  is obtained only over a depth of  $0.7D$ ). If there is a sudden decrease in resistance between  $0.7D$  and  $4D$  the lowest value in this range should be selected for  $\bar{q}_{c-1}$  (Figure 4.23b). To obtain  $\bar{q}_{c-2}$  the diagram is followed in an upward direction and the envelope is drawn only over those values which are decreasing or remain constant at the value at the pile toe. Minor peak depressions are again ignored provided that they do not represent clay bands; values of  $q_c$  higher than  $30\text{MN/m}^2$  are disregarded over the  $4D$ – $8D$  range. Safety factors generally used in the Netherlands in conjunction with the ‘ $4D$ – $8D$ ’ method to obtain the allowable pile load are given by te Kamp(4.35) as

Timber	1.7
Precast concrete, straight shaft	2.0
Precast concrete, enlarged shaft	2.5

An upper limit is placed on the value of the ultimate base resistance obtained by either of the methods shown in Figure 4.23. Upper limiting values depend on the particle-size distribution and over-consolidation ratio and are shown in Figure 4.24. Values of the ultimate skin friction obtained from Table 4.3 and base resistance from equation 4.13 are empirically derived and correspond to  $q_{sik}$  and  $q_{bk}$  in Eurocode 7. Hence values of  $Q_{sk}$  and  $Q_{bk}$  obtained from them should be factored to obtain the design values of  $Q_s$  and  $Q_b$ .

Cone-resistance values cannot be used to obtain the end-bearing resistance of bored and cast-in-place piles because of the loosening of the soil caused by drilling as described in the preceding chapter.

A further factor must be considered when calculating pile skin friction and end-bearing resistance from CPT data. This is the effect on changes in overburden pressure on the  $q_c$  (and also local friction) values at any given level. Changes in overburden

pressure can result from excavation, scour of a river or sea bed, or the loading of the ground surface by placing fill. The direct relationship between  $q_c$  and overburden pressure is evident from Figure 4.14. Taking the case of a normally-

consolidated sand deposit the ratio of the vertical to horizontal effective stress is denoted by the coefficient for horizontal earth pressure at rest ( $K_{onc}$ ). When the vertical effective stress  $\sigma'_{vnc}$  is reduced by excavation to a new value  $\sigma'_{vn}$  the value of

$K_{onc}$  is not reduced in the ratio of  $\sigma'_{\text{out}}$  to  $\sigma'_{\text{in}}$  but it is reduced to some value

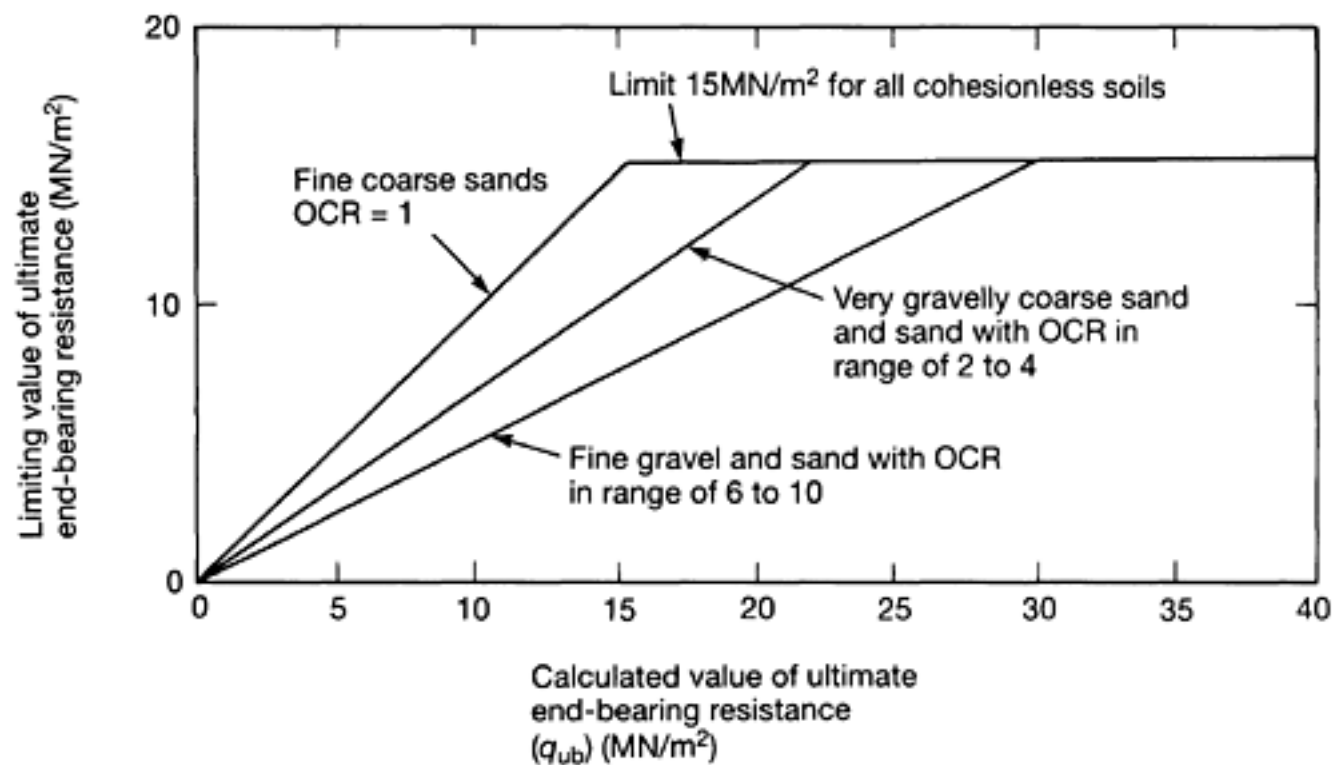


Fig. 4.24 Limiting values of pile end-bearing resistance for solid end-piles (after te Kamp<sup>(4.35)</sup>)

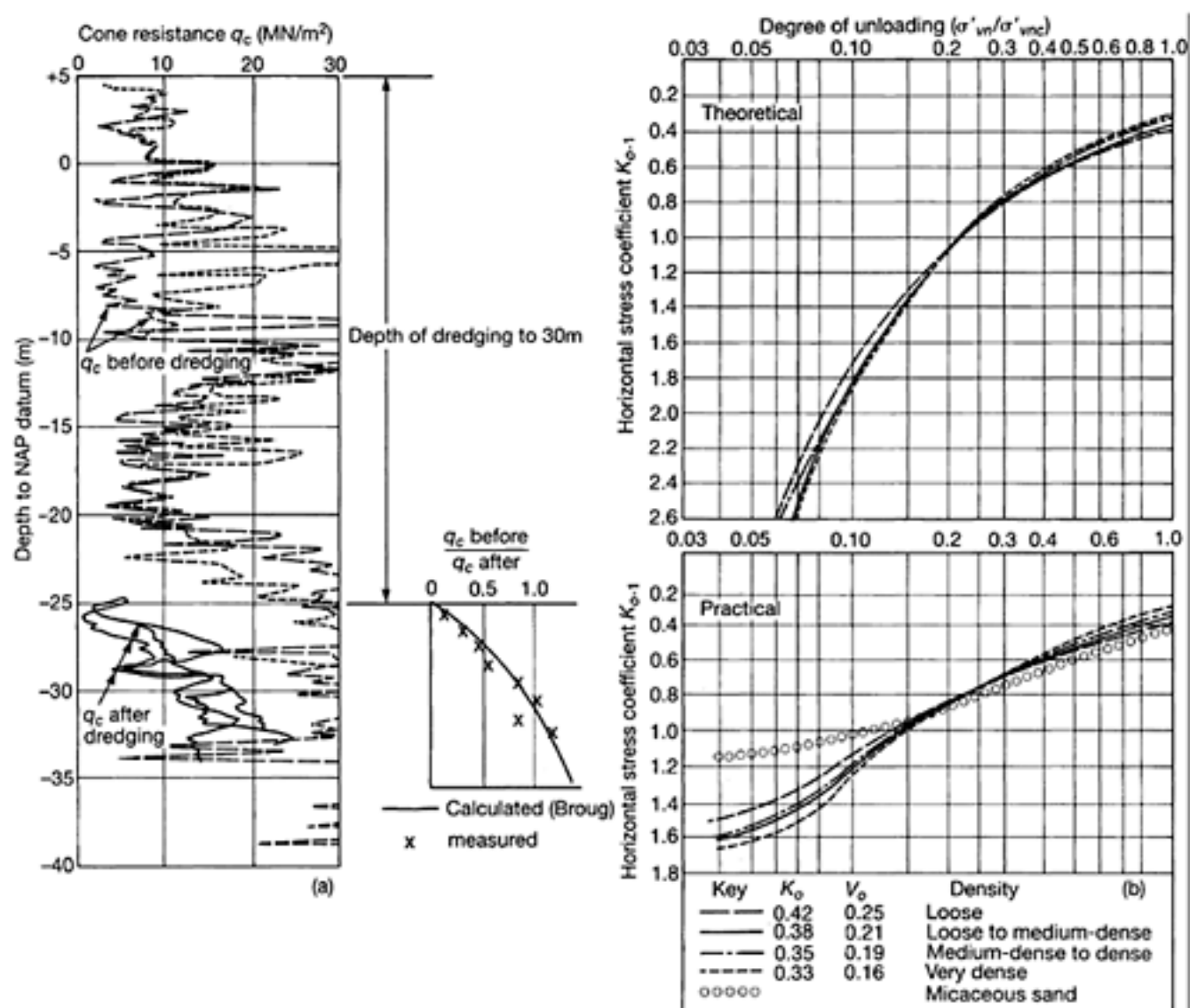


Fig. 4.25 (a) Cone resistance vs depth before and after dredging sand (after Gijt and Brassinga<sup>(4.36)</sup>). (b) Reduction of horizontal stress coefficient  $K_o$  due to reduction in vertical effective stress (after Broug<sup>(4.37)</sup>)

$K_{o-1}$  depending on the degree of unloading ( $\sigma'_{vm}/\sigma'_{vnc}$ ). The degree of unloading is a function of depth. The effects are most marked in the shallow layers below the new excavation level becoming less marked with increasing depth when the overburden pressure changes become proportionately smaller. Small reductions in overburden pressure cause only elastic movements in the assembly of soil particles. Larger

reductions cause plastic yielding of the assembly and a proportionate reduction of horizontal pressures. Broug<sup>(4.37)</sup> has shown that the threshold value for the change from elastic to elastoplastic behaviour of the soil assembly occurs when the degree of unloading becomes less than 0.4.

The effect of unloading on cone resistance values was shown by de Gijt and Brassinga<sup>(4.36)</sup>. [Figure 4.25a](#) shows  $q_c$ /depth

plots before and after dredging to a depth of 30m in the normally-consolidated alluvial sands of the River Maas in connection with an extension to the Euroterminal in the Netherlands. Large reductions in overburden pressure within the zone 10m below the new harbour bed caused the reduction in cone resistance shown in [Figure 4.25a](#). The difference between the observed new cone resistance and the mean line predicted from the Broug curves ([Figure 4.25b](#)) did not exceed 5%.

The effects are most marked where the soil deposits contain weak particles such as micaceous or carbonate sands. Broug<sup>(4.37)</sup> described field tests and laboratory experiments on sands containing 2 to 5% of micaceous particles. These studies were made in connection with the design of piled foundations for the Jamuna River bridge in Bangladesh where scour depths of 30 to 35m occur at times of major floods.

The basis of evaluating the change in cone resistance due to overburden changes is the effect of horizontal stress on the resistance to penetration of the cone. The horizontal stress depends on the density of the soil mass, the mineralogy of the particles and their configuration in the assembly. The ratio between the new and original horizontal pressure governs the degree of change in  $q_c$ . The new cone resistance is given by the equation

$$q_{cn} = \frac{K_{o-1} \times \sigma'_{vn}}{K_{onc} \times \sigma'_{vnc}} \times q_{cv} \quad \dots(4.41a)$$

where  $q_{cn}$  is the new cone resistance after excavation

$K_{o-1}$  is the coefficient of horizontal stress after excavation

$K_{onc}$  is the coefficient of horizontal stress before excavation (in normally-consolidated sand  $K_{onc} = 0.3$  to  $0.4$ )

$\sigma'_{vn}$  is the vertical effective stress after excavation

$\sigma'_{vnc}$  is the vertical effective stress before excavation

$q_{cv}$  is the original cone resistance before excavation.

Values of  $K_{o-1}$  can be obtained from [Figure 4.25b](#) for various degrees of unloading as represented by  $\sigma'_{vn}/\sigma'_{vnc}$ . These values

were obtained by Broug<sup>(4.37)</sup> from cyclic oedometer tests on quartz sand (Monterey), compacted to a range of densities and from Ko-triaxial tests on micaceous quartz sand from the Jamuna River. A quartz sand, with strong particles, will show a relatively small reduction of the horizontal stress coefficient and hence in the cone resistance. Conversely, micaceous sand, with quartz particles partly separated by the weak mica, will show a relatively high reduction of the horizontal stress coefficient and hence in the cone resistance.

Two sets of curves are shown on [Figure 4.25b](#). The theoretical values of  $K_{o-1}$  were obtained from analytical studies,

introducing a non-linear stress-strain relationship. Laboratory test results showed, for both sand types, that for degrees of unloading less than 0.4 the  $K_{o-1}$  values deviated from the theoretical due to induced internal failure of the soil mass. The test

results led to the establishment of the practical values of  $K_{o-1}$  in [Figure 4.25b](#).

Even the assumption that any soil mass would be in normally consolidated condition prior to unloading, defined by stress ratio  $K_{onc}$ , has resulted in an excellent agreement between predicted and monitored cone resistance after unloading. This

means that calculation of  $q_c$  can be carried out with sufficient accuracy with the assumption of a normally-consolidated

condition, as defined by the stress ratio  $K_{onc}$  before unloading.

The relationship established for Dutch soil conditions, shown in equation 4.13, is not necessarily applicable to cohesionless soils everywhere. The yielding and rupture of the soil caused by pushing a cone into the ground are different from those resulting from driving a pile by hammer followed by static loading. The work of Vesic<sup>(4.23)</sup> has shown the importance of the state of preconsolidation and mineralogy of the soil grains in any correlation of in-situ conditions with pile resistance. It may be fortuitous that the static cone resistance in Holland (and Belgium) was found to be equal to the pile base resistance.

Elsewhere Gregersen *et al.* [\(4.38\)](#) found the pile base resistance to be only one-half of the cone resistance for loose medium to coarse sands in Norway, and Gruteman *et al.* [\(4.39\)](#) reported that a factor of 0.75 is applied to the cone resistance to obtain the ultimate base resistance of piles in silty sands in Russia.

It will have been noted that the static cone penetration test, which measures the resistance of the *undisturbed* soil, is used as a measure of the resistance to penetration of a pile into a soil which has been compacted by the pile driving. Heijnen [\(4.40\)](#) measured the cone resistance of a loose to medium-dense

silty fine sand before and after installing driven-and-cast-in-situ piles. The increase in resistance at various distances from the 1m diameter enlarged base caused by the pile driving was as follows.

Distance from pile axis (m)	Increase in static cone resistance (%)
1	50 to 100
2	About 33
3.5	Negligible

In spite of the considerable increase in resistance close to the pile base, the ultimate resistance of the latter was in fact accurately predicted by the cone resistance value of the *undisturbed* soil by using equation 4.13. This indicates that the effect of compaction both in driven and driven-and-cast-in-situ piles is already allowed for when using this equation.

Field trials to correlate the static cone resistance with pile loading tests are necessary in any locality where there is no previous experience to establish the relationship between the two. In the absence of such tests the base resistance should be taken as one-half of the static cone resistance with the application of a factor of safety of 2.5 to obtain the allowable unit pressure on the base of the pile. Experience has shown that if a safety factor of 2.5 is applied to the ultimate base resistance as calculated from the cone resistance the settlement at the working load is unlikely to exceed 10mm for piles of base widths up to about 500mm. For larger base widths it is desirable to check that pile head settlements resulting from the design end-bearing pressure are within tolerable limits. Pile head settlements can be calculated using the methods described in [Section 4.6](#).

#### 4.3.8 Time effects for piles in cohesionless soils

It was noted in [Section 4.2.4](#) that a marked increase in the bearing capacity of piles driven into soft clays could occur after a period of months or years after completion of driving, whereas there may be a reduction in capacity of up to 20% for piles driven into stiff clays. It is not usual to allow for increase in reduction in bearing capacity of either driven or bored piles in cohesionless soils but the engineer should be aware of a possible *reduction* in capacity where piles are driven into fine sands and silts.

Peck *et al.* ([4.20](#)) stated that ‘If the fine sand or silt is dense, it may be highly resistant to penetration of piles because of the tendency for dilatancy and the development of negative pore pressures (Art. 4.7) during the shearing displacements associated with insertion of the piles. Analysis of the driving records by means of the wave equation may indicate high dynamic capacity but instead of freeze, large relaxations may occur.’

An example of this phenomenon was provided by the experiences of driving large diameter tubular steel piles into dense sandy clayey silts for the foundations of the new Galata Bridge in Istanbul ([4.30](#)). The relaxation in capacity of the 2m OD piles in terms of blows per 250mm penetration is shown in [Figure 4.26](#). The magnitude of the reduction in driving resistance was not related to the period of time between cessation and resumption of driving. It is likely that most of the reduction occurred within a period of 24 hours after completing a stage of driving. The widely varying time periods shown in [Figure 4.26](#) were due to the operational movements of the piling barge from one pile location or group to another.

Correlation of blow count figures with tests made with the dynamic pile analyser ([Section 7.3](#)) showed a markedly smaller reduction in dynamic soil resistance than indicated by the reduction in blow count after the delay period.

These experiences emphasize the need to make re-driving tests after a minimum period of 24 hours has elapsed after completing the initial drive. Loading tests should not be made on piles in cohesionless soils until at least seven days after driving. Where piles are driven into laminated fine sands, silts and clays, special preliminary trial piling should be undertaken to investigate time effects on driving resistance. These trials should include tests with the pile driving analyser.

#### 4.4 Piles in soils intermediate between sands and clays

Where piles are installed in sandy clays or clayey sands which are sufficiently permeable to allow dissipation of excess pore pressure caused by application of load to the pile, the base resistance can be calculated for the case of drained loading using equation 4.9. The angle of shearing resistance used for obtaining the bearing capacity factors  $N_q^*$  and  $N_\gamma^*$  should be the effective angle  $\phi$  obtained from unconsolidated

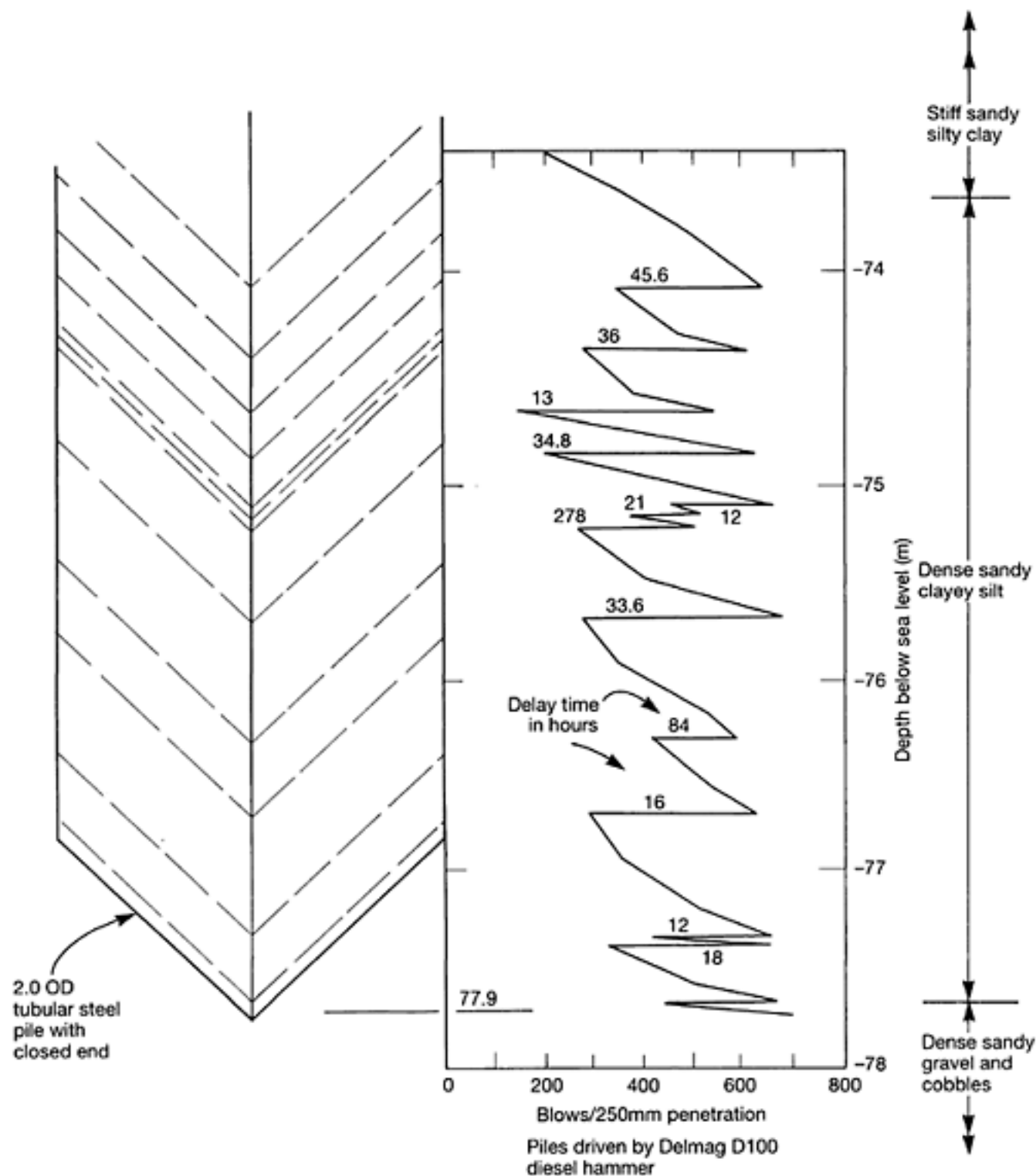


Fig. 4.26 Driving resistance over final 4.5m of penetration for 2.0m tubular steel pile showing reduction in driving resistance after various delay periods, New Galata Bridge, Istanbul

drained triaxial compression tests. Alternatively, the effective strength parameters can be used in conjunction with Terzaghi's general equation for the bearing capacity of shallow foundations:

$$q_b = 1.3 c' N_c + \sigma'_{vo} (N_q - 1) + 0.4 \gamma B N_\gamma \quad \dots(4.15)$$

Values of the bearing capacity factors  $N_c$  and  $N_q$  are given in [Figure 4.27](#). In a uniform soil deposit this equation gives a

linear relationship for the increase of base resistance with depth. Therefore the base resistance should not exceed the peak value of 11MN/m<sup>2</sup> unless pile loading tests show that higher ultimate values can be obtained.

The effective overburden pressure,  $\sigma'_{vo}$ , in equation 4.15 is the total overburden pressure minus the pore water pressure at the pile toe level. Where the latter is below groundwater level, the submerged density should be used for  $\gamma$  in the second term.

The skin friction should be calculated from the second term in equation 4.8, again using the drained angle,  $\phi'$ , to obtain the friction angle,  $\delta$ . It is important to distinguish between uniform  $c-\phi$  soils and layered  $c$  and  $\phi$  soils, as sometimes the layering is not detected in a poorly executed soil investigation.

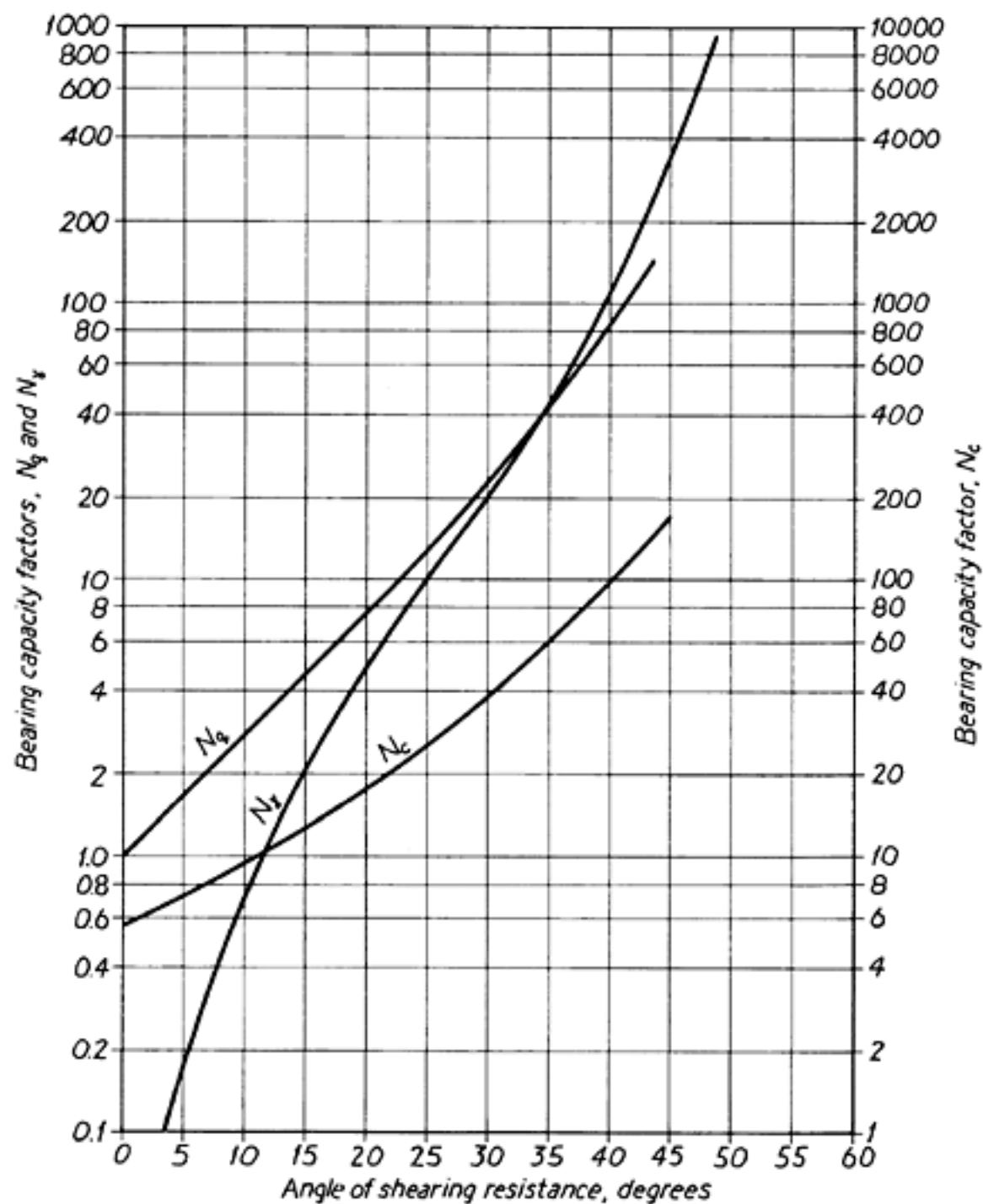


Fig. 4.27 Terzaghi's bearing capacity factors,  $N_c$ ,  $N_q$  and  $N_\gamma$  for shallow foundations (to be used in conjunction with equation 4.15)

#### 4.5 Piles in layered cohesive and cohesionless soils

It will be appreciated from Sections 4.2 and 4.3 that piles in cohesive soils have a relatively high skin friction and a low end-bearing resistance, and in cohesionless soils the reverse is the case. Therefore when piles are installed in layered soils the location of the pile toe is of great importance. The first essential is to obtain a reliable picture of the depth and lateral extent of the soil layers. This can be done by making in-situ tests with static or dynamic cone test equipment (see Section 11.1.4), correlated by an adequate number of boreholes. If it is desired to utilize the potentially high end-bearing resistance provided by a dense sand or gravel layer, the variation in thickness of the layer should be determined and its continuity across the site should be reliably established. The bearing stratum should not be in the form of isolated lenses or pockets of varying thickness and lateral extent.

Where driven or driven-and-cast-in-situ piles are to be installed, problems can arise when piles are driven to an arbitrary 'set' to a level close to the base of the bearing stratum, with the consequent risk of a breakthrough to the weaker clay layer when the piles are subjected to their working load (Figure 4.28a). In this respect the driven-and-cast-in-place pile with an enlarged base is advantageous, as the bulb can be hammered out close to the top of the bearing stratum (Figure 4.28b). The end-bearing resistance can be calculated conservatively on the assumption that the pile always terminates within or just above the cohesive soil layer, *i.e.*, by basing the resistance on that provided by the latter layer. This is the only possible solution for sites where the soils are thinly-bedded, and there is no marked change in driving resistance through the various layers. However, this solution can be uneconomical for sites where a dense cohesionless soil layer has been adequately explored to establish its thickness and continuity. A method of calculating the base resistance of a pile located in a thick stiff or dense layer underlain by a weak stratum has been established by Meyerhof<sup>(4.3)</sup>. In Figure 4.29 the unit base resistance of the pile is given by the equation:

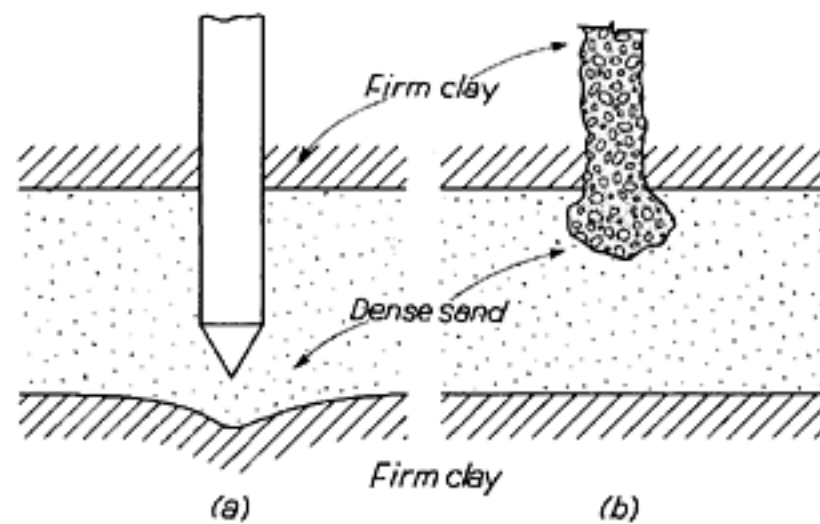


Fig. 4.28 Pile driven to end bearing into relatively thin dense soil layer

(a) Driven pile

(b) Driven-and-cast-in-place pile

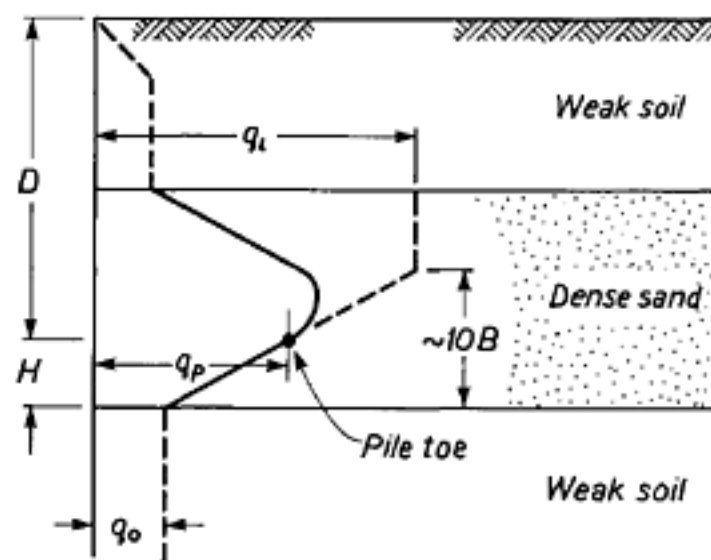


Fig. 4.29 End bearing resistance of piles in layered soils

$$q_b = q_o + \frac{q_l - q_o}{10B} H \leq q_l \quad \dots(4.16)$$

where  $q_o$  is the ultimate base resistance in the lower weak layer

$q_l$  is the ultimate base resistance in the upper stiff or dense stratum

$H$  is the distance from the pile toe to the base of the upper layer

$B$  is the width of the pile at the toe

The author adopted the following procedure for the piled foundations of British Coal's bulk-handling plant at Immingham, where a layer of fairly dense sandy gravel was shown to exist at a depth of about 14.6m below ground level. The thickness of the gravel varied between 0.75 and 1.5m and it lay between thick deposits of firm to stiff boulder clay. The end-bearing resistance in the gravel of the 508mm diameter driven-and-cast-in-situ piles was more than 3000kN as derived from loading tests to obtain separate evaluations of skin friction and base resistance. It was calculated that if the toe of the pile reached a level at which it was nearly breaking through to the underlying clay, the end-bearing resistance would then fall to 1000kN and the safety factor of the pile would be reduced to 1.2 at the working load of 800kN. This safety factor was inadequate, and it was then necessary to drive the pile some 3.6m deeper to mobilize additional skin friction so as to raise the safety factor to a satisfactory value. A careful record was made to compare the driving resistance of piles driven completely through the gravel to a deeper penetration and those terminating on the gravel layer ([Figure 4.30](#)). An evaluation of this record led to the establishment of the following rules.

1. When the driving resistance in the gravel increased rapidly from 20mm per blow to 5mm per blow for a complete 300mm of driving it was judged that the pile was properly seated in the gravel stratum.
2. The pile was then required to be driven a further 75mm without any reduction in the driving resistance.
3. If the resistance was not maintained at 5mm per blow, it was judged that the gravel layer was thin at that point, and the pile was liable to break through to the clay. Therefore, the pile had

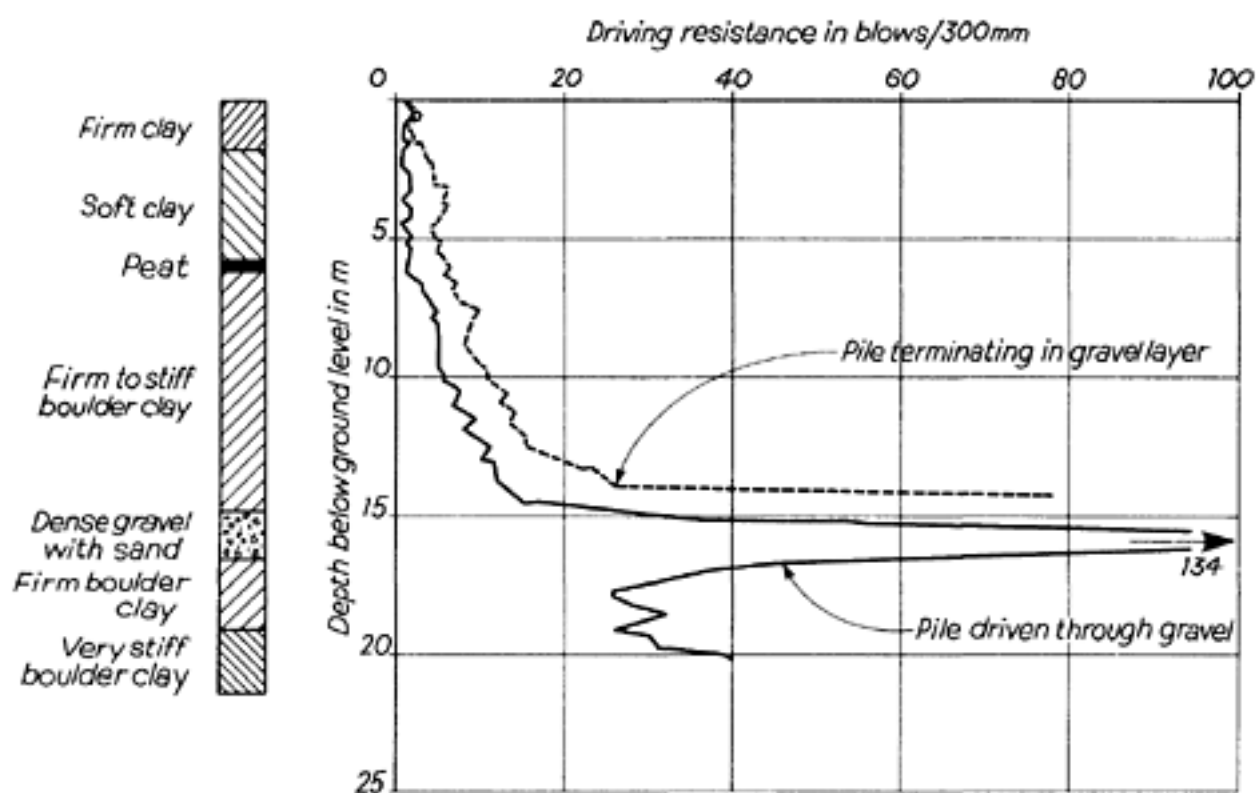


Fig. 4.30 Resistance to driven-and-cast-in-place piles provided by a thin layer of dense sand and gravel at Immingham

to be driven further to a total penetration of 20m, which was about 3 to 4m below the base of the gravel, to obtain the required additional skin-frictional resistance.

The effects of driving piles in groups onto a resistant layer underlain by a weaker compressible layer must be considered in relation to the settlement of the group. This aspect is discussed in [Chapter 5](#).

#### 4.6 The settlement of the single pile at the working load for piles in soil

It is necessary to divide the calculated ultimate resistance of the pile (or the ultimate resistance derived from load testing) by a safety factor to obtain the design working load on the pile. A safety factor is required for the following reasons.

1. To provide for natural variations in the strength and compressibility of the soil.
2. To provide for uncertainties in the calculation method used.
3. To ensure that the working stresses on the material forming the pile shaft are within the safe limits.
4. To ensure that the total settlement(s) of the single isolated pile or the group of piles are within tolerable limits.
5. To ensure that the differential settlements between adjacent piles or within groups of piles are within tolerable limits.

The need for a safety factor to cover the uncertainties in the calculation methods will have been evident from the earlier part of this chapter, and in this respect it is a 'factor of ignorance' rather than a safety factor. With regard to reason 4 above, the load-settlement curves obtained from a very large number of loading tests in a variety of soil types, both on displacement and non-displacement piles, have shown that for piles of small to medium (up to 600mm) diameter, the settlement under the working load will not exceed 10mm if the safety factor is not lower than 2.5. This is reassuring and avoids the necessity of attempting to calculate settlements on individual piles that are based on the compressibility of the soils. A settlement at the working load not exceeding 10mm is satisfactory for most building and civil engineering structures provided that the group settlement is not excessive.

However, for piles larger than 600mm in diameter the problem of the settlement of the individual pile under the working load becomes increasingly severe with the increase in diameter, requiring a separate evaluation of the skin friction and base load. The question of the correct safety factor then becomes entirely the consideration of the permissible settlement. The load-settlement relationships for the two components of skin friction and base resistance and for the total resistance of a large-diameter pile in a stiff clay are shown in [Figure 4.31](#). The maximum shaft resistance is mobilized at a settlement of only 10mm but the base resistance requires a settlement of nearly 150mm for it to become fully mobilized. At this stage the pile has reached the point of ultimate resistance at a failure load of 4.2MN. A safety factor of 2 on this condition gives a working load of 2.1MN, under which the settlement of

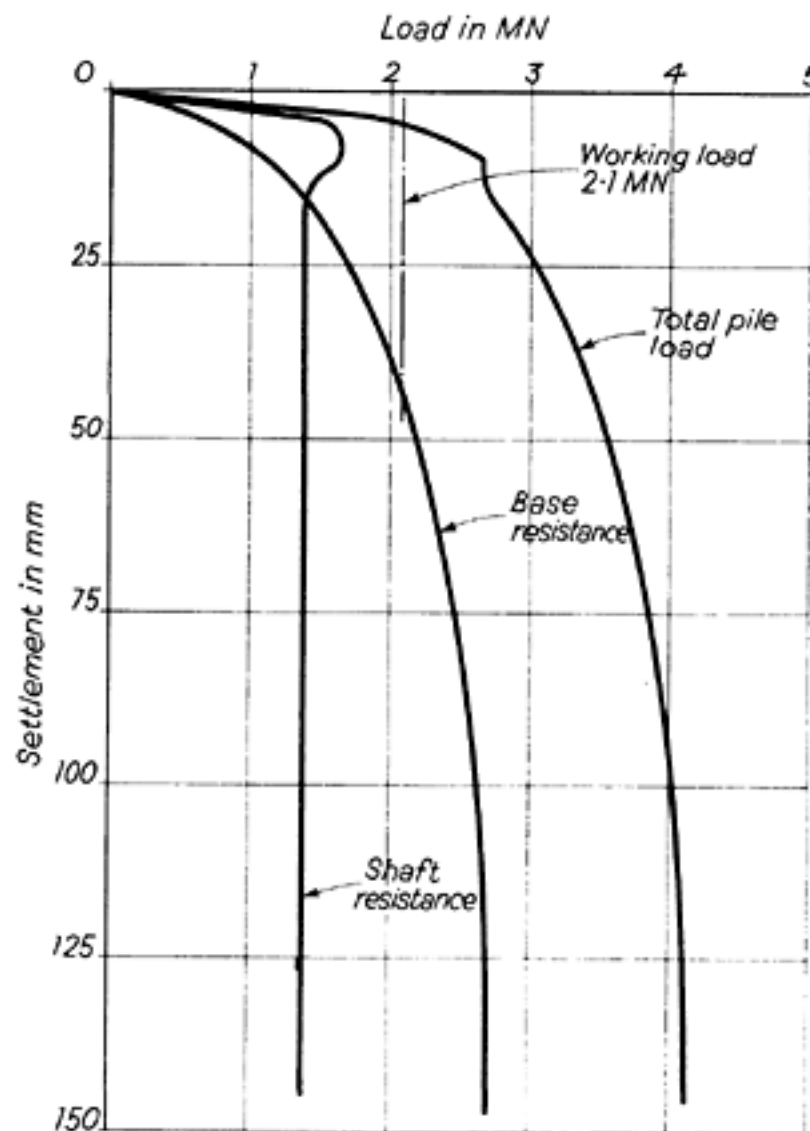


Fig. 4.31 Load-settlement relationships for large-diameter bored piles in stiff clay

the pile will be nearly 5mm. This is well within the settlement which can be tolerated by ordinary building structures. The full shaft resistance will have been mobilized at the working load, but only 22% of the ultimate base resistance will have been brought into play. For economy in pile design the settlement at the working load should approach the limit which is acceptable to the structural designer, and this usually involves mobilizing the full shaft resistance.

It is desirable at this stage to introduce the concept of partial safety factors on the ultimate shaft and base loads. Burland *et al.* (4.41) have presented a simple stability criterion for bored piles in clay which states that if an overall load factor of 2 is stipulated, together with a minimum factor of safety in end bearing of 3, then the maximum safe load on the pile is the lesser of the two expressions  $(\frac{1}{2}Q_p)$  and  $(Q_s + \frac{1}{3}Q_b)$ , where  $Q_p$  is the ultimate load on the whole pile,  $Q_s$  is the ultimate load on the shaft, and  $Q_b$  is the ultimate load on the base.

Burland *et al.* state that the first expression is nearly always dominant for straight-sided piles and for long piles with comparatively small under-reams, whereas the second expression often controls piles with large under-reamed bases. Satisfaction of the above criteria does not necessarily mean that the settlement at working load will be tolerable. Experience based on loading tests on piles in similar soil conditions may give a guide to the order of settlement that may be expected. If there is no such experience available, then it may be necessary to undertake loading tests on full-scale piles. This is very costly for large piles and a more economical procedure is to estimate values from the results of loading tests made on circular plates at the bottom of the pile boreholes, or in trial shafts.

Burland *et al.* (4.41) plotted the settlement of test plates divided by the plate diameter ( $p_i/B$ ) against the plate bearing pressure divided by the ultimate bearing capacity for the soil beneath the plate (*i.e.*,  $q/q_f$ ) and obtained a curve of the type shown in Figure 4.32. If the safety factor on the end-bearing load is greater than 3, the expression for this curve is

$$p_i/B = K \times q/q_f \quad \dots(4.17)$$

When plate bearing tests are made to failure, the curve can be plotted and, provided that the base safety factor is greater than 3, the settlement of the pile base  $p_i$  can be obtained for any desired value of  $B$ .

The procedure used to estimate the settlement of a circular pile is as follows.

1. Obtain  $q_f$  from the failure load given by the plate bearing test.

2. Check  $q_f$  against the value obtained by multiplying the shearing strength by the appropriate bearing capacity factor  $N_c$ ,  
*i.e.*,  $qf$  should equal  $N_c \times cb$ .

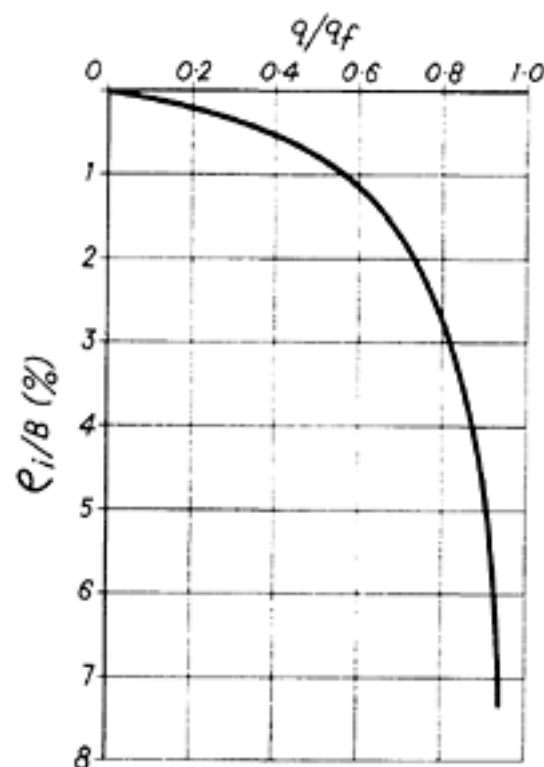


Fig. 4.32 Elastic settlements of bored piles in London clay at Moorfields (after Burland, Butler and Dunican<sup>(4.41)</sup>)

3. Knowing  $q_f$ , calculate the end-bearing resistance  $Q_b$  of the pile from  $Q_b = A_b \times q_f$ .
4. Obtain the safe end-bearing load on the pile from  $W_b = Q_b/F$ , where  $F$  is a safety factor greater than 3.
5. Obtain  $q$  from  $q = W_b / \frac{1}{4}\pi B^2$  and hence determine  $q/q_f$ .
6. From a curve of the type shown in [Figure 4.32](#), read off  $\rho_i/B$  for the value of  $q/q_f$  and hence obtain  $\rho_i$  (the settlement of the pile base).

Merely increasing the size of the base by providing an under-ream will not reduce the base settlement, and if the settlement is excessive it should be reduced by one or more of the following measures.

1. Reduce the working load on the pile.
2. Reduce the load on the base by increasing the shaft resistance, *i.e.*, by increasing the shaft diameter.
3. Increase the length of the shaft to mobilize greater shaft friction, and to take the base down to deeper and less-compressible soil.

For piles in London clay,  $K$  in equation 4.17 has usually been found to lie between 0.01 and 0.02. If no plate bearing tests are made the adoption of the higher value provides a conservative estimate of settlement. Having estimated the settlement of the individual pile using the above procedure it is still necessary to consider the settlement of the pile group as a whole (see [Chapter 5](#)).

The greater the length of the pile the greater is the pile head settlement. From their analyses of a large number of load/settlement curves, Weltman and Healy<sup>(4.10)</sup> established a simple relationship for the settlement of straight shaft bored and cast-in-place piles in glacial till. The relationship given below assumed a pile diameter not greater than 600mm, a working stress on the pile shaft of about 3MN/m<sup>2</sup>, a length to diameter ratio of 10 or more, and stiff to hard glacial till with undrained shear strengths in excess of 100kN/m<sup>2</sup>. The pile head settlement is given by:

$$\rho = \frac{l_m}{4} \text{ in millimetres} \quad \dots(4.18)$$

where  $l_m$  is the length of embedment in glacial till in metres.

Precast concrete piles and some types of cast-in-place piles are designed to carry working loads with shaft stresses much higher than 3MN/m<sup>2</sup>. In such cases the settlement should be calculated from equation 4.18 assuming a stress of 3MN/m<sup>2</sup>. The settlement should then be increased *pro rata* to the designed working stress.

The above methods of Burland *et al.*, and Weltman and Healy, were developed specifically for piling in London clay and glacial till respectively and were based on the results of field loading tests made at a standard rate of loading as specified by the Institution of Civil Engineers ([Section 11.4](#)) using the maintained loading procedure. More generally the pile settlements can be calculated if the load carried by skin friction and the load transferred to the base at the working load can be reliably

estimated. The

pile head settlement is then given by the sum of the elastic shortening of the shaft and the compression of the soil beneath the base as follows:

$$\rho = \frac{(W_s + 2W_b)L}{2A_s E_p} + \frac{\pi}{4} \cdot \frac{W_b}{A_b} \cdot \frac{B(1 - \nu^2)I_p}{E_b} \quad \dots(4.19)$$

where $W_s$ and $W_b$	are the loads on the pile shaft and base respectively
$L$	is the shaft length
$A_s$ and $A_b$	are the cross-sectional area of the shaft and base respectively
$E_p$	is the elastic modulus of the pile material
$B$	is the pile width
$\nu$	is the Poisson's ratio of the soil
$I_p$	is an influence factor related to the ratio of $L/R$
$E_b$	the deformation modulus of the soil beneath the pile base

For a Poisson's ratio of 0 to 0.25 and  $L/B > 5$ ,  $I_p$  is taken as 0.5 when the last term approximates to  $\frac{0.5 W_b}{B E_b}$ . Values of  $E_b$  are obtained from plate loading tests at pile base level or from empirical relationships with the results of laboratory or in situ soil tests given in 5.2 and 5.3.

The pile shaft settlements given by the first term in equation 4.19 assume a transfer load to the soil uniformly down the pile shaft. It is clear from the discussion in 4.2.1 and 4.3.1 that the distribution is not uniform where a high proportion of the total load is carried in skin friction on the shaft of deeply penetrating piles. A simple method of simulating load transfer from pile to soil is the transfer function approach described by Vesic<sup>(4.23)</sup>. The pile is assumed to be divided into  $n$  elements which are considered as compressible short columns of length  $\Delta L = \frac{L}{n}$ . The soil is represented by a series of non-linear springs one to each element acting independently of one another. Another spring represents the compression of the soil beneath the pile base ([Figure 4.33](#)). Each column is subjected to an axial force,  $Q_i$ , skin friction,  $f_i$ . The latter can be calculated from:

$$f_i = \frac{Q_i - Q_{i-1}}{P \Delta L} = \frac{\Delta Q_i}{P \Delta L} \quad \dots(4.20)$$

where  $P$  is a perimeter of the column.

The elastic compression of the column is given by:

$$\Delta l_i = l_{i+1} - l_i = \frac{Q \Delta L}{E_p A_p} \quad \dots(4.21)$$

where  $A_p$  is the cross-sectional area of the column.

From the above a set of simultaneous equations can be derived which, providing the axial forces are known, and one boundary displacement is known or can be assumed, permit the displacements along the pile to be determined. The axial forces on the pile are calculated from the 'transfer function' which is an empirical or semi-empirical relationship of the form:

$$\Delta Q = Q_i - Q_{i-1} = \kappa(\Delta l_i) \quad \dots(4.22)$$

The transfer function,  $\kappa$ , is determined analytically or determined experimentally<sup>(4.42)</sup>.

The principal shortcoming of the transfer function method is that the 'springs' act independently of one another, that is, any soil layer represented by a spring is unaffected by movements in the layers above and below. The elastic continuum method avoids this problem by assuming that each element forms part of a rigid incompressible pile surrounded by compressible soil consisting of a homogeneous, elastic, isotropic continuum. The axial load at the pile head is resisted by frictional forces on each element. The displacements of each element are obtained from Mindlin's equation for the displacements due to a point

load in a semi-infinite mass. The resulting equations and the method of settlement analysis are described by Poulos([4.43](#)). Slip at the pile/soil interface can be allowed for by changing the magnitude of the friction force on the element. The equations used in the transfer function and elastic continuum methods can be written in matrix form and evaluated by digital computer. Both methods have the disadvantage that they over-simplify



working load settlements of small-diameter piles. However there is not the same mass of experience relating settlements to design loads obtained by Eurocode methods based on partial safety factors. Hence it is necessary to check that the design pile capacity does not endanger the serviceability limit state of the supported structure. Equation 4.19 can be used for this check. A material factor of unity should be adopted for the design value of  $E_d$ .

## 4.7 Piles bearing on rock

### 4.7.1 Driven piles

For maximum economy in the cross-sectional area of a pile it is desirable to drive the pile to virtual refusal on a strong rock stratum, thereby developing its maximum carrying capacity. Piles driven in this manner are regarded as wholly end bearing: skin friction on the shaft is not considered to contribute to the support of the pile. The depth of penetration required to reach virtual refusal depends on the thickness of any weak or heavily broken material overlying sound rock. If a pile can be driven to near refusal on to a strong *intact* rock the safe working load on the pile is governed by the permissible working stress on the material of the pile at the point of minimum cross-section; *i.e.*, the pile is regarded as a short column supported against buckling by the surrounding soil. Where piles are driven through water or through very soft clays and silts of fluid consistency, then buckling as a long strut must be considered (see [Section 7.5](#)).

When steel piles are adopted, working loads based on the permissible working stress on the steel may result in concentrations of very high loading on the rock beneath the toe of the pile. The ability of the rock to sustain this loading without yielding depends partly on the compressive strength of the rock and partly on the frequency and inclination of fissures and joints in the rock mass, and whether these discontinuities are tightly closed or are open or filled with weathered material. Very high toe loads can be sustained if the rock is strong, with closed joints either in a horizontal plane or inclined at only a shallow angle to the horizontal. If the horizontal or near-horizontal joints are wide there will be some yielding of the rock mass below the pile toe but the amount of movement will not necessarily be large since the zone of rock influenced by a pile of slender cross-section does not extend very deeply below toe level. However, the temptation to continue the hard driving of slender-section piles to ensure full refusal conditions must be avoided. This is because brittle rocks may be split by the toe of the pile, thus considerably reducing the base resistance. The splitting may continue as the pile is driven down, thus requiring very deep penetration to regain the original resistance.

Where bedding planes are steeply inclined with open transverse joints there is little resistance to the downward sliding of a block of rock beneath the toe and the movement will continue until the open joints have become closed, or until the rock mass becomes crushed and locked together. This movement and crushing will take place as the pile is driven down, as indicated by a progressive tightening-up in driving resistance. Thus there should be no appreciable additional settlement when the working load is applied. However, there may be some deterioration in the end-bearing value if the piles are driven in closely-spaced groups at varying toe levels. For this reason it is desirable to undertake re-driving tests whenever piles are driven to an end bearing into a heavily-jointed or steeply-dipping rock formation. If the re-driving tests indicate a deterioration in resistance, then loading tests must be made to ensure that the settlement under the working load is not excessive. Soil heave may also lift piles off their end bearing on a hard rock, particularly if there has been little penetration to anchor the pile into the rock stratum. Observations of the movement of the heads of piles driven in groups, together with re-driving tests indicate the occurrence of pile lifting due to soil heave. Methods of eliminating or minimizing the heave are described in 5.7.

Steel tubes driven with open ends, or H-section piles are helpful in achieving the penetration of layers of weak or broken rock to reach virtual refusal on a hard unweathered stratum. However, the penetration of such piles causes shattering and disruption of the weak layers to the extent that the skin friction may be seriously reduced or virtually eliminated. This causes a high concentration of load on the relatively small area of rock beneath the steel cross-section. While the concentration of load may be satisfactory for a strong intact rock it may be excessive for a strong but closely-jointed rock mass. The concentration of load can be reduced by welding stiffening rings or plates to the pile toe or, in the case of weak and heavily broken rocks, by adopting winged piles ([Figure 2.18](#)).

The methods given below for calculating the ultimate bearing capacity assume that this is the sum of the shaft and base resistance. Both of these components are based on correlations between pile loading tests and the results of field tests in rock formations or laboratory tests on core specimens. Hence when following Eurocode 7 recommendations the separate components of shaft and base resistance should be factored as described in [Section 4.2.1](#) for piles in clay.

Where the joints are spaced widely, that is at 600mm or more apart, or where the joints are tightly closed and remain closed after pile driving, the ultimate base resistance may be calculated from the equation:

$$q_b = 2 N_\phi q_{uc} \tag{4.23}$$

where the bearing capacity factor,

$$N_\phi = \tan^2 \left( 45^\circ + \frac{\phi}{2} \right)$$

For a coarse sandstone which typically has  $\phi$  values between 40° and 45°, the base resistance is stated by Pells and Turner (4.44) to be between 9 and 12 times  $q_{uc}$ . Wyllie(4.54) gives the following friction angles for intact rock which should be used only as a guide because of the wide variations which can occur due to site conditions.

Classification	Type	Friction angle (degrees)
Low friction	Schists (high mica content)	20 to 27
	Shale	
	Marl	
Medium friction	Sandstone	27 to 34
	Siltstone	
	Chalk	
	Gneiss	
	Slate	
High friction	Basalt	34 to 40
	Granite	

The values of  $N_\phi$  obtained from the friction angle of an intact rock can be reduced substantially if the rock mass has open or clay-filled joints, or if joints which are tightly closed in situ are subsequently opened by pile displacement and vibrations. In the case of open joints the ultimate base resistance may be no more than the unconfined compression strength,  $q_{uc}$ , of the intact rock.

It may be possible to measure, by laboratory testing, the parameters  $c$  and  $\phi$  of a jointed rock mass. Kulhawy and Goodman (4.45) state that the ultimate bearing capacity of the jointed rock beneath the pile toe can be obtained by the equation

$$q_{ub} = cN_c + \frac{\gamma B N_\gamma}{2} + \gamma D N_q \tag{4.23a}$$

- where  $c$  is the cohesion
- $B$  is the pile base width
- $D$  is the base depth below the rock surface
- $\gamma$  is the effective density of the rock mass
- $N_c, N_\gamma$  and  $N_q$  are bearing capacity factors related to  $\phi$  and shown in [Figure 4.35](#)

The above equation represents wedge failure conditions beneath a strip foundation and should not be confused with Terzaghi’s equation (4.15). It should also be noted that the bearing capacity factors are different from those in [Figure 4.27](#). Because equation 4.23a is for strip loading the value of  $cN_c$  should be multiplied by a factor of 1.25 for a square pile or 1.2 for a circular pile base. Also the term  $\gamma B N_\gamma / 2$  should be corrected by the factors 0.8 or 0.7 for square or circular bases respectively. The term  $\gamma B N_\gamma / 2$  is small compared with  $cN_c$  and is often neglected.

It can be difficult and expensive to obtain values of  $c$  and  $\phi$  from laboratory tests on large samples of jointed rock. Wyllie (4.54) has given some characteristic values obtained from back-analyses of failures in rock slopes. Kulhawy and Goodman

(4.45, 4.45a) have shown that these parameters can be related to the rock quality designation (RQD values) of the mass and they have suggested the following approximate relationships:

RQD (%)	Rock mass properties		
	$q_c$	$c$	$\phi$
0–70	$0.33q_{uc}^*$	$0.1q_{uc}$	$30^\circ$
70–100	0.33 to $0.8q_{uc}$	$0.1q_{uc}$	$30\text{--}60^\circ$

\*  $q_{uc}$  is the unconfined compression strength of the intact rock.

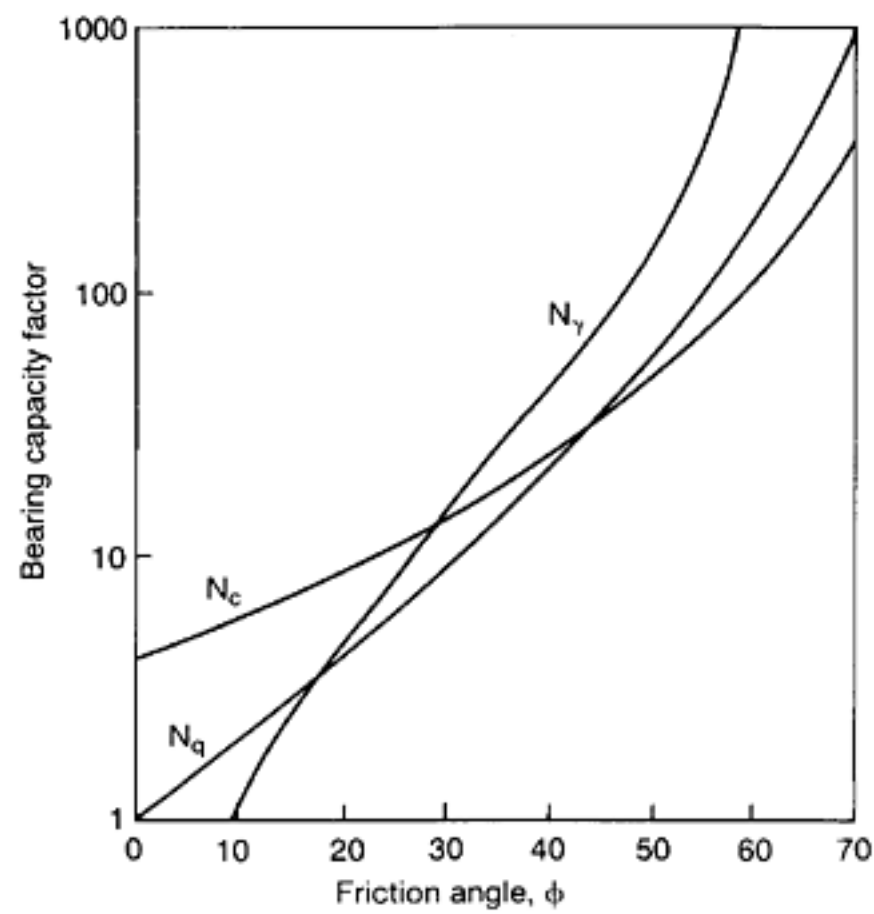


Fig. 4.35 Wedge bearing capacity factors for foundations on rock (Reprinted from Pells and Turner, 1980)

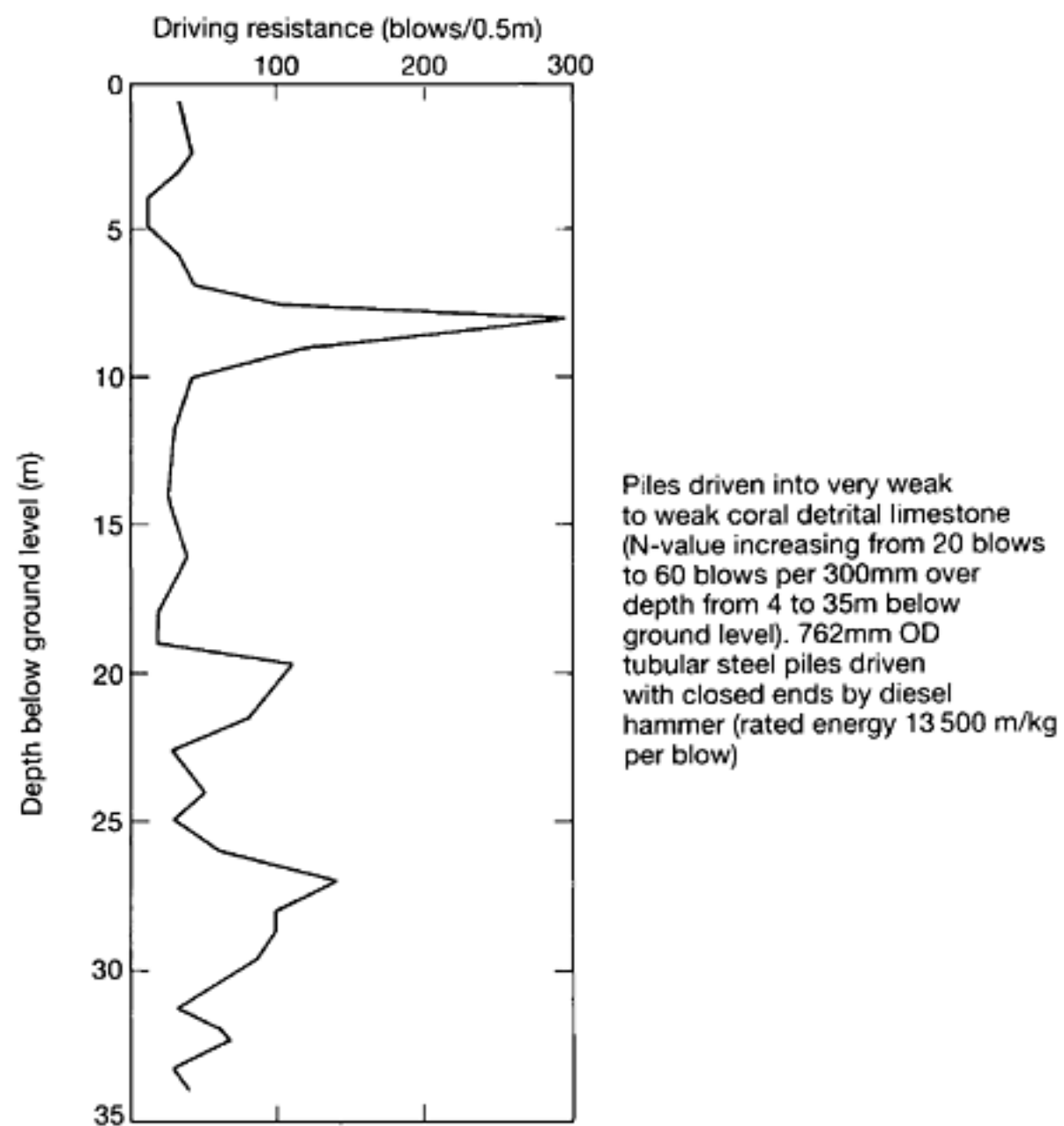


Fig. 4.36 Low resistance to driving of tubular steel piles provided by weak coral limestone

Table 4.4 Observed ultimate base resistance values from plate or pile loading tests on weak rocks

Description of rock	Pile type	Plate or pile diameter (mm)	Bearing pressure at failure (MN/m <sup>2</sup> )	Reference
Chalk (Grade I–II)	Plate	140	14.8–17.7	4.46
Chalk (Grade I–II)	Plate	140	6–8	4.46
Chalk (Grade III–IV)	Plate	450	4	4.46
Chalk (Grade III–IV)	Plate	150–760	2–5	4.46
Chalk (Grade III)	Plate	140	3.2–5.9	4.46
Chalk (Grade V)	Driven tube	364	3.2	4.46
Chalk (Grade IV–VI)	Jacket tube	170	6.6–16.4	4.47
Mudstone, weathered (Keuper Marl)	Bored	406	1.5	4.48
Mudstone, strong (Keuper Marl)	Driven	508	4.9	4.48
Mudstone (Keuper Marl) (Zone III–IV)	Bored	740	5.3	4.49
Mudstone/siltstone, moderately weak	Bored	900	$5.6 \left( \frac{q_b}{q_{uc}} = 0.5 \right)$	Unpubl.
Mudstone, highly to moderately weathered, weak	Plate	457	$9.2 \left( \frac{q_b}{q_{uc}} = 2.5 \right)$	Unpubl.
Cretaceous mudstone, weak, weathered, clayey	Bored	670	$6.8 \left( \frac{q_b}{q_{uc}} = 6 \right)$	4.50
Weak coral detrital limestone (carbonate siltstone/sandstone)	Driven	762	$5.1 \left( \frac{q_b}{q_{uc}} = 3.0 \right)$	Unpubl.
Calcareous sandstone, weak	Driven tube	1200	$3.0 \left( \frac{q_b}{q_{uc}} = 1.3 \right)$	Unpubl.
Sandstone, weak to moderately weak	Driven	275	$19 \left( \frac{q_b}{q_{uc}} = 3.5 \right)$	Unpubl.*

\* From dynamic pile tests.

It is important to note that to mobilize the maximum base resistance obtained from equation 4.23, the settlement at the pile toe is likely to be of the order of 20% of the base diameter. Therefore an ample safety factor, at least 2.5, should be adopted to ensure that settlements at the working load are within allowable limits (see 4.7.4).

An alternative method of obtaining the ultimate base resistance of piles on weak weathered rocks is to refer to published records obtained from pile or plate loading tests. Some published values are shown in Table 4.4. It will be noted that where the average unconfined compression strength,  $q_{uc}$ , was known, the ultimate base resistance was no more than six times this value.

Hobbs and Healy(4.46) have related the base resistance of piles in chalk to the standard penetration test N-values (blows/0.3m) by the expressions:

$$q_b = 0.24N \text{ MN/m}^2 \text{ for } N < 30 \quad \dots(4.24a)$$

$$q_b = 0.2N \text{ MN/m}^2 \text{ for } N > 40 \quad \dots(4.24b)$$

Full utilization of the maximum calculated base resistance or the permissible working stress on the material forming the pile may involve heavy driving, resulting in unseen damage to the pile shaft. Because of this some codes of practice place a limit on the maximum load which can be applied to a pile of a given type, irrespective of its cross-sectional area, as discussed in regard to the various types of pile in Chapter 2.

When in a completely or highly weathered state, rocks such as chalk, shales, siltstones and mudstones behave like soils of a clayey consistency and refusal is not reached until the piles have been driven to a stronger and relatively unweathered rock. The required depth of penetration can usually be obtained by an examination of rock cores taken in conjunction with an

assessment of the standard penetration-test values (see [11.1.4](#)) and the results of unconfined compression tests on rock specimens. Generally, it is preferable to drive open-ended steel tubular piles or H-piles deeply into weak weathered rocks to develop their resistance in combined end bearing and skin friction, rather than to employ solid plated ends with a shallow penetration. Heavily driven plated piles are liable to be lifted off their seating as a result of ground heave, or to rebound off the compressed strata beneath the toe. The penetration depth required to develop the full carrying capacity of piles in chalk is difficult to assess from an examination of borehole records and laboratory test data. This is because the entry of a pile pulverizes the chalk

and breaks down its cellular structure. The water entrapped in the cells is released and forms a slurry with the powdered rock, the surfaces of the pile shaft become lubricated, and the softened rock is extruded into open fissures. There is little increase in skin friction or end resistance, and in fissured chalk there is little or no displacement of the rock. Thus piles can be driven quite deeply into only moderately weathered chalk without any marked increase in the resistance to driving. It is believed, but reliable quantitative data are scanty, that the skin friction increases substantially with an increase in time after driving. For pile design either conservative assumptions as regards skin friction must be assumed, using for guidance the values given below, or test piles must be driven to, say, three different penetration depths and subjected to loading tests to failure. The loading tests must not be made until *at least two weeks* have elapsed from the date of driving.

The skin friction developed on piles driven into weak weathered rocks cannot always be calculated from the results of laboratory tests on rock cores. The skin friction depends on such factors as the formation of an enlarged hole around the pile, the slurring and degradation of rocks, the reduction in skin friction due to shattering of the rock by driving adjacent piles, and the presence of ground water. In the case of brittle coarse-grained rocks such as sandstones, igneous rocks and some limestones, it can be assumed that pile driving shatters the rock around the pile shaft to the texture of a loose to medium-dense sand. The ultimate skin friction can then be calculated from the second term in equation 4.8 using the appropriate values of  $K_s$  and  $\delta$ . Where rocks such as mudstones and siltstones weather to a clayey consistency making it possible to

obtain undisturbed samples from boreholes, the weathered rock can be treated as a clay and the skin friction calculated from the methods described in 4.2.1.

The effects of degradation of weakly cemented carbonate soils caused by pile driving have been discussed in [Section 4.3.3](#). Similar effects occur in carbonate rocks such as detrital coral limestones, resulting in very deep penetration of piles without any significant increase in driving resistance. An example of the low driving resistance provided by weak coral limestone to the penetration of closed-end tubular steel piles at a coastal site in Saudi Arabia is shown in [Figure 4.36](#).

As noted above, chalk is a special case. Hobbs and Healy<sup>(4.46)</sup> have listed the published values for piles in chalk in [Table 4.5](#). Hobbs and Healy stated from their analysis of the data in [Table 4.5](#) that the skin friction could be

Table 4.5 Observed ultimate skin friction on piles driven into chalk

Description of chalk	SPT N-value (blows/0.3m)	Type of pile	Ultimate skin friction (kN/m <sup>2</sup> )
Weathered, shattered	10–25	Raymond step-taper	20–100
Unweathered	25–40	Raymond step-taper	100–220
Weathered to unweathered	5–30	H-section	42
Weathered to unweathered	5–40	H-section	8–19 (av. 14)
Weathered	5–15	Steel, tubular	26
Weathered to unweathered	10–30	Steel, tubular	35–100
Weathered to unweathered	10–30	Steel, tubular	10–100
Soliflucted and weathered	5–25	Precast concrete shell	30–55
Soliflucted and weathered	5–15	Precast concrete shell	15–50
Weathered	15–25	Precast concrete shell	50–110

Table 4.6 Observed ultimate skin friction values for piles driven into weak and weathered rocks

Pile type	Rock description	Ultimate skin friction (kN/m <sup>2</sup> )	Reference
H-section	Moderately strong slightly weathered slaty mudstone	28*	4.52
H-section	Moderately strong slightly weathered slaty mudstone	158†	4.52
Steel tube	Very weak coral detrital limestone (carbonate sandstone/siltstone)	45	Unpubl.
Steel tube	Faintly to moderately weathered moderately strong to strong mudstone	127	Unpubl.
Steel tube	Weak calcareous sandstone	45	Unpubl.
Precast concrete	Very weak closely fissured argillaceous siltstone (Keuper Marl)	130	4.53

\* Penetration 1.25m † Penetration 2.2m

Table 4.7 Ultimate shaft resistance for various types of pile in weathered chalk

Type of pile	Ultimate shaft resistance (kN/m <sup>2</sup> )
A Small displacement (H-sections, open-end steel tubes)	8 to 87*
B Large displacement (Precast concrete, closed-end steel tubes and box-sections)	29 to 54 (Design average 35 to 40)
C Driven and cast-in-place (with zero slump concrete and expanded base)	98 to 192 (Design average 150)

\* Steel tube in strong chalk.

calculated in terms of effective stress. For the second term in equation 4.8 they give values of  $\tan \delta$  of 0.35 and 0.45 for steel and precast concrete piles respectively, and  $K_s$  may be expressed as  $K_s = 0.06N$ , where  $N$  is the standard penetration test N-

value. Other observed values of the skin friction on driven piles are given in [Table 4.6](#).

In the keynote address to the 1989 International Chalk Symposium, Lord([4.51](#)) expressed doubt on the validity of the effective stress method for driven piles in chalk. From the results of pile loading tests he gave a range of values shown in [Table 4.7](#) for different types of pile in weak and weathered chalk.

#### 4.7.2 Driven-and-cast-in-place piles

Driven-and-cast-in-place piles terminated on strong rock can be regarded as end-bearing. Their working load is governed by the permissible working stress on the pile shaft at the point of minimum cross-section, or by code of practice requirements (see [Table 2.10](#)). Where these piles are driven into weak or weathered rocks they should be regarded as partly friction and partly end-bearing piles. Published values of skin friction are limited to those for chalk given by Lord([4.51](#)) as listed above. It will be noted that they are higher than for driven piles. The driven-and-cast-in-place piles which incorporate an enlarged base are very economical for weak rock conditions because of the considerable increase in end-bearing resistance which can be obtained if a bulb of an appreciable size is capable of being hammered out in the soil immediately above rock level or within the weak rock.

#### 4.7.3 Bored-and-cast-in-place piles

Where these piles are installed by drilling through soft overburden onto a strong rock the piles can be regarded as end-bearing elements and their working load is determined by the safe working stress on the pile shaft at the point of minimum cross-section, or by code of practice requirements (see [Table 2.11](#)). Bored piles drilled down for some depth into weak or weathered rocks and terminated within these rocks act partly as friction and partly as end-bearing piles. Wyllie([4.54](#)) gives a detailed account of the factors governing the development of skin friction over the depth of the rock socket. The factors which govern the bearing capacity and settlement of the pile are summarized as:

1. The length to diameter ratio of the socket.
2. The strength and elastic modulus of the rock around and beneath the socket.
3. The condition of the side walls, *i.e.* roughness and the presence of drill cuttings or bentonite slurry.
4. Condition of the base of the drilled hole with respect to removal of drill cuttings and other loose debris.
5. Layering of the rock with seams of differing strength and moduli.
6. Settlement of the pile in relation to the elastic limit of the side wall strength.
7. Creep of the material at the rock/concrete interface resulting in increasing settlement with time.

The effect of the length/diameter ratio of the socket is shown in [Figure 4.37](#) for the condition of the rock having a higher elastic modulus than the concrete. It will be seen that if it is desired to utilize base resistance as well as socket friction the socket length should be less than four pile diameters. The high interface stress over the upper part of the socket will be noted. The condition of the side walls is an important factor. In a weak rock such as chalk, clayey shale, or clayey weathered marl, the action of the drilling tools is to cause softening and slurring of the walls of the borehole and, in the most adverse case, the skin friction corresponds to that typical of a smooth-bore hole in a soft clay. In stronger and fragmented rocks the slurring does not take place to the same

extent, and there is a tendency towards the enlargement of the drill hole, resulting in better keying of the concrete to the rock. If the pile borehole is drilled through soft clay this soil may be carried down by the drilling tools to fill the cavities and smear the sides of the rock socket. This behaviour can be avoided to some extent by inserting a casing and sealing it into the rock-head before continuing the drilling to form the rock socket, but the interior of the casing is likely to be heavily smeared with clay which will be carried down by the drilling tools into the rock socket. Wyllie<sup>(4.54)</sup> suggests that if bentonite is used as a drilling fluid the rock socket skin friction should be reduced to 25 % of that of a clean socket unless tests can be made to verify the actual friction which is developed.

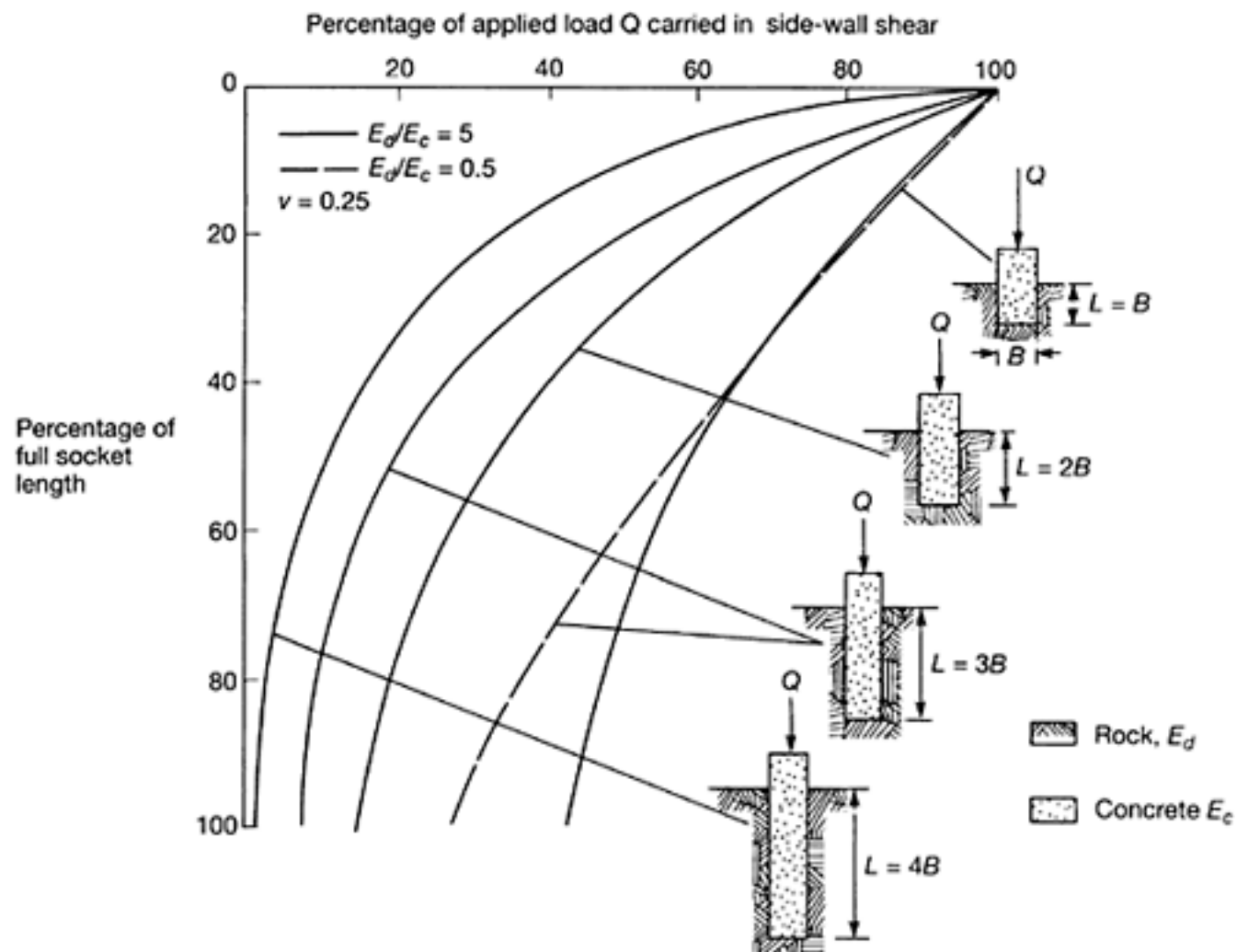


Fig. 4.37 Distribution of side wall shear stress in relation to socket length and modulus ratio (after Osterberg and Gill<sup>(4.55)</sup>)

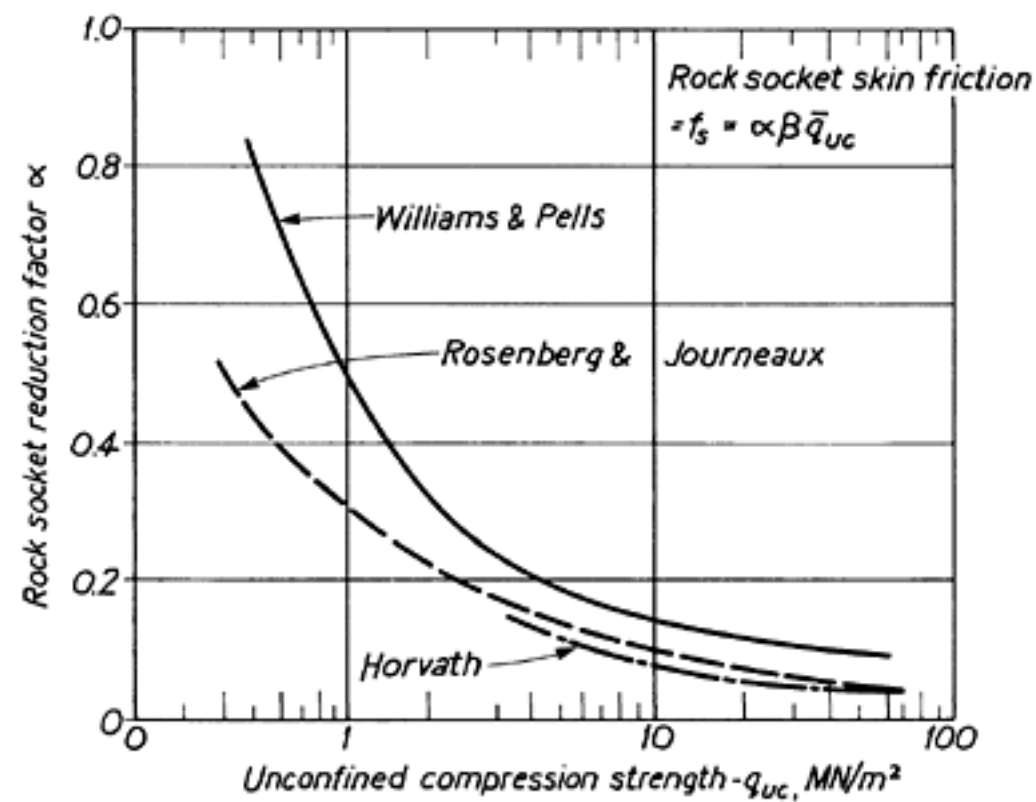


Fig. 4.38 Reduction factors for rock socket skin friction

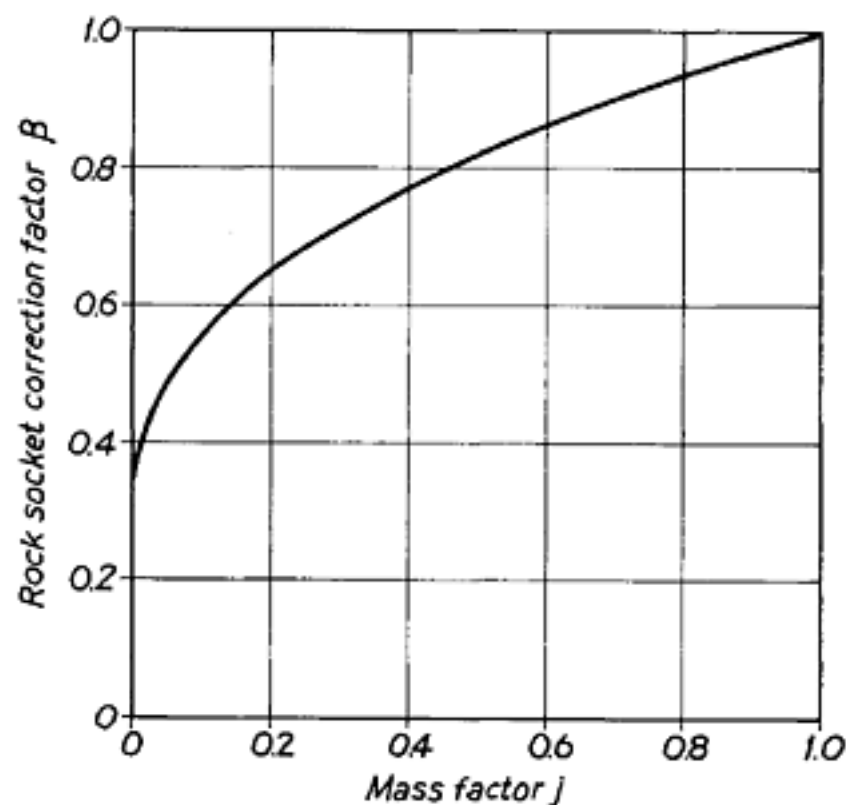


Fig. 4.39 Reduction factors for discontinuities in rock mass (after Williams and Pellis<sup>(4.58)</sup>)

It is evident that the keying of the shaft concrete to the rock and hence the strength of the concrete to rock bond is dependent on the strength of the rock. Correlations between the unconfined compression strength of the rock and rock socket bond stress have been established by Horvarth<sup>(4.56)</sup>, Rosenberg and Journeaux<sup>(4.57)</sup> and Williams and Pellis<sup>(4.58)</sup>. The ultimate bond stress,  $f_s$ , is related to the average unconfined compression strength,  $\bar{q}_{uc}$ , by the equation:

$$f_s = \alpha \beta q_{uc} \quad \dots(4.25)$$

where  $\alpha$  is a reduction factor relating to  $\bar{q}_{uc}$  as shown in [Figure 4.38](#)

$\beta$  is a correction factor related to the discontinuity spacing in the rock mass as shown in [Figure 4.39](#).

The curve of Williams and Pellis in [Figure 4.38](#) is higher than the other two, but the  $\beta$  factor is unity in all cases for the Horvarth and the Rosenberg and Journeaux curves. It should also be noted that the  $\alpha$  factors for all three curves do not allow for smearing of the rock socket caused by dragdown of clay overburden or degradation of the rock.

The  $\beta$  factor is related to the mass factor,  $j$ , which is the ratio of the elastic modulus of the rock mass to that of the intact rock as shown in [Figure 4.40](#). If the mass factor is not known from loading tests or seismic velocity measurements, it can be obtained approximately from the relationships with the rock quality designation (RQD) or the discontinuity spacing quoted by Hobbs<sup>(4.59)</sup> as follows:

RQD (%)	Fracture frequency per metre	Mass factor $j$
0–25	15	0.2
25–50	15–8	0.2
50–75	8–5	0.2–0.5
75–90	5–1	0.5–0.8
90–100	1	0.8–1

For the case of chalk, Hobbs and Healy<sup>(4.46)</sup> quote observed rock socket skin friction values between 36kN/m<sup>2</sup> and 100kN/m<sup>2</sup> for weathered becoming unweathered chalk with SPT N-values in the range of 7 to 26, and 100kN/m<sup>2</sup> to 470kN/m<sup>2</sup> for unweathered chalk with N-values between 20 and 38. However, Lord<sup>(4.51)</sup> is of the opinion that the standard penetration test is not appropriate for assessing skin friction resistance for either bored or driven piles. He recommends the use, for design purposes, of the range of values for bored piles shown in [Table 4.8](#). Generally the skin friction resistances are higher than those of driven piles reflecting the bond value given by keying of the cast-in-place concrete to the rough socket for bored piles compared with the slurring and degradation of chalk caused by the penetration of driven piles. Published and

unpublished values for rock socket friction in other rock types are given in [Table 4.8](#).

The allowable end-bearing resistance of bored-and-cast-in-situ piles in weak rocks again depends on drilling techniques. The use of percussive drilling tools can result in the formation of a very soft sludge at the bottom of the drill hole which can give a wrong impression of the true character of the rock.

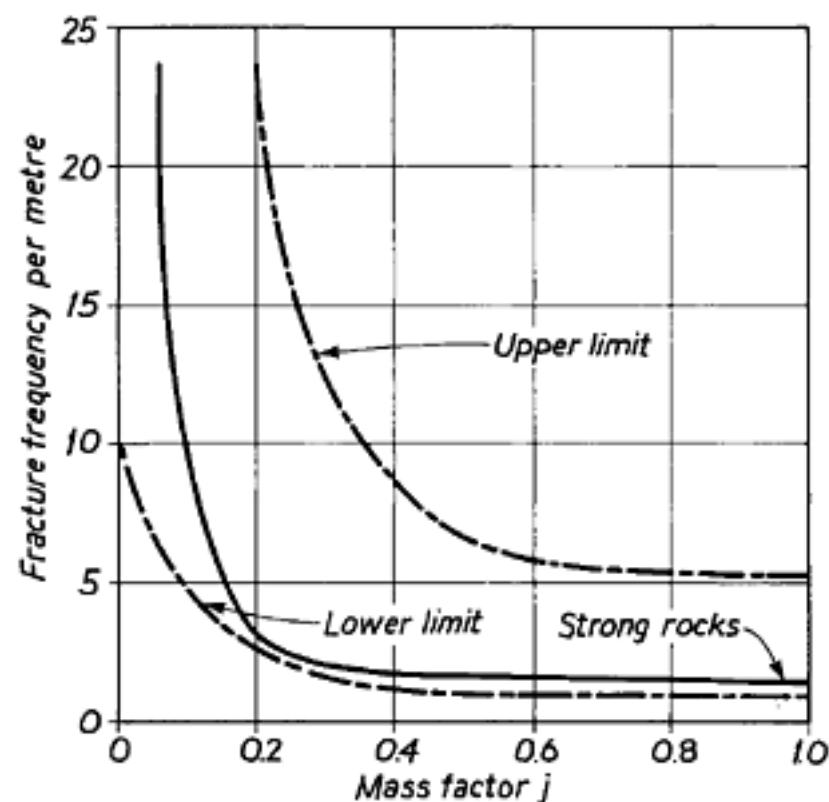


Fig. 4.40 Mass factor value (after Hobbs<sup>(4.59)</sup>)

Table 4.8 Observed ultimate skin friction values for bored piles in weak rocks

Description of rock	Ultimate skin friction (kN/m <sup>2</sup> )	Reference
Mudstone, weathered (Keuper Marl Zone II)	250–280	4.48
Mudstone, weathered (Keuper Marl Zone II)	210	4.49
Mudstone, weathered (Keuper Marl Zone III–IV)	150–180	4.48
Mudstone, weathered (Keuper Marl Zone III–IV)	119	4.49
Siltstone, weak, weathered, fractured ( $q_{uc}=2.6\text{MN/m}^2$ , $j=0.2$ )	550	Unpubl.
Mudstone/siltstone, moderately weak ( $q_{uc}=1\text{MN/m}^2$ , $j=0.2$ )	1030	Unpubl.
Shale, very weak ( $q_{uc}=0.45\text{MN/m}^2$ )	311	4.58
Shale, slaty, weathered	279*	Unpubl.
Cretaceous mudstone, weak, weathered, clayey ( $q_{uc}=1.1\text{MN/m}^2$ )	120–184	4.50
Diabase, highly weathered, weak, clayey ( $q_{uc}=0.3$ to $0.5\text{MN/m}^2$ )	122	4.60

\* From anchor pulling test.

The sludge should be baled out and, if necessary, flushed out with air and water. Standard penetration tests made at the bottom of the pile borehole give an indication of the quality of the rock, but conducting standard penetration tests at the base of each pile borehole can cause considerable delays to progress. Mechanical auger or grabbing rigs are not designed to handle sampling gear and rods in long lengths, and the operations of coupling-up lengths of guide tube and drill rod, lowering them to the base of the hole, and removing them on completion of the test can be very time-consuming. Therefore it is best to judge the required base level of the piles by examination and strength testing of large-diameter cores obtained from exploratory drilling at the site investigation stage (see 11.1.3), with later correlation by an examination of the drill cuttings from the pile boreholes. Assessment of pile base levels from rock cores is particularly necessary in thinly-bedded rock strata where weak rocks alternate with stronger strata. In these cases the allowable end-bearing pressure should be based on the characteristics of the weaker rocks, irrespective of the material in which the pile is terminated. It may be possible to assess pile base levels by correlations with the measured torque on the drill stem of the mechanical auger.

If the unconfined compression strength and angle of shearing resistance of the rock are known, the ultimate base resistance can then be calculated from equation 4.23 in the same manner as for driven piling. Again high calculated values should be adopted with caution because of the base settlement (of the order of 20% of the pile diameter) which is required to mobilize the ultimate resistance. Some authorities require the ultimate pile resistance to be determined either on the ultimate skin friction alone, or only on the ultimate base resistance. The substantial settlements required to mobilize a high proportion of the ultimate base resistance can cause breakdown of the rock socket bond. A reduction of 30% to 40% from the peak value

for shear displacements at the socket of little more than 15mm

have been observed(4.58). It can also be difficult to remove soft or loose debris from the whole area of the pile base at the time of final clean-out before concreting.

Where large diameter bored piles are constructed in weak compressible rocks the pile head settlement should be calculated by the methods described in 4.7.4.

For piles in chalk, Hobbs and Healy(4.46) state that equation 4.24 can be used for bored piles, but the calculated values should be reduced by about 50% if heavy chisels or other drop tools are used causing deterioration of the chalk at the pile base. This deterioration can cause serious reduction in the bearing capacity of an unweathered chalk. Some published values of base resistance obtained from plate or pile loading tests are given in [Table 4.4](#).

#### 4.7.4 The settlement of the single pile at the working load for piles in rocks

The effects of load transfer from shaft to base of piles on the pile head settlements have been discussed by Wyllie(4.54). Because of the relatively short penetration into rocks which is needed to mobilise the required total pile resistance, the simpler methods of determining pile head settlement described in 4.6 are suitable in most cases. For piles having base diameters up to 600mm the settlement at the working load should not exceed 10mm if a safety factor of 2.5 has been applied to the ultimate bearing capacity.

The settlement of large diameter piles can be calculated from equation 4.19. The modulus of deformation of the rock below the pile toe can be obtained from plate bearing or pressuremeter tests or from empirical relationships developed between the modulus and the unconfined compression strength of the rock given in 5.5.

These relationships are not applicable to high porosity chalk or weathered silty mudstone (Keuper Marl). Some published modulus values for these rocks applicable to piles are shown in [Table 4.9](#). It should also be noted that the relationships given in 5.5 assume fairly low stress levels. Therefore calculated values based on the unconfined compression strength of the rock should take into account the high bearing pressures beneath the base of piles.

Table 4.9 Values of deformation modulus applicable to piles and bearing on rock

Type of rock	Deformation modulus, $E_d$ (MN/m <sup>2</sup> )	Reference
Chalk (Grade I)	50–300	4.52
Chalk (Grade II)	30–50	4.52
Chalk (Grade III)	20–30	4.52
Chalk (Grade IV)	15–20	4.52
Chalk (Grade V)	12–15	4.52
Chalk (Grade VI)	<12	4.52
Chalk (Grade III)	52–72	Unpubl.
Mudstone (Keuper Marl, Zone I–II)	500–1200	4.53
Mudstone (Keuper Marl, Zone II)	45	4.42

Pells and Turner(4.61) have derived influence factors for calculating the settlement of a bored pile where the load is carried by rock socket skin friction only using the equation

$$\text{Settlement} = \rho = \frac{QI_p}{BE_d} \quad \dots(4.26)$$

where  $Q$  is the total load carried by the pile head

$I_p$  is an influence factor

$B$  is the diameter of the socket

$E_d$  is the deformation modulus of the rock mass surrounding the shaft.

The influences factors of Pells and Turner are shown in [Figure 4.41](#). Where the rock sockets are recessed below the ground surface or where a layer of soil or very weak rock overlies competent rock, a reduction factor is applied to equation 4.26. Values of the reduction factor are shown in [Figure 4.42](#).

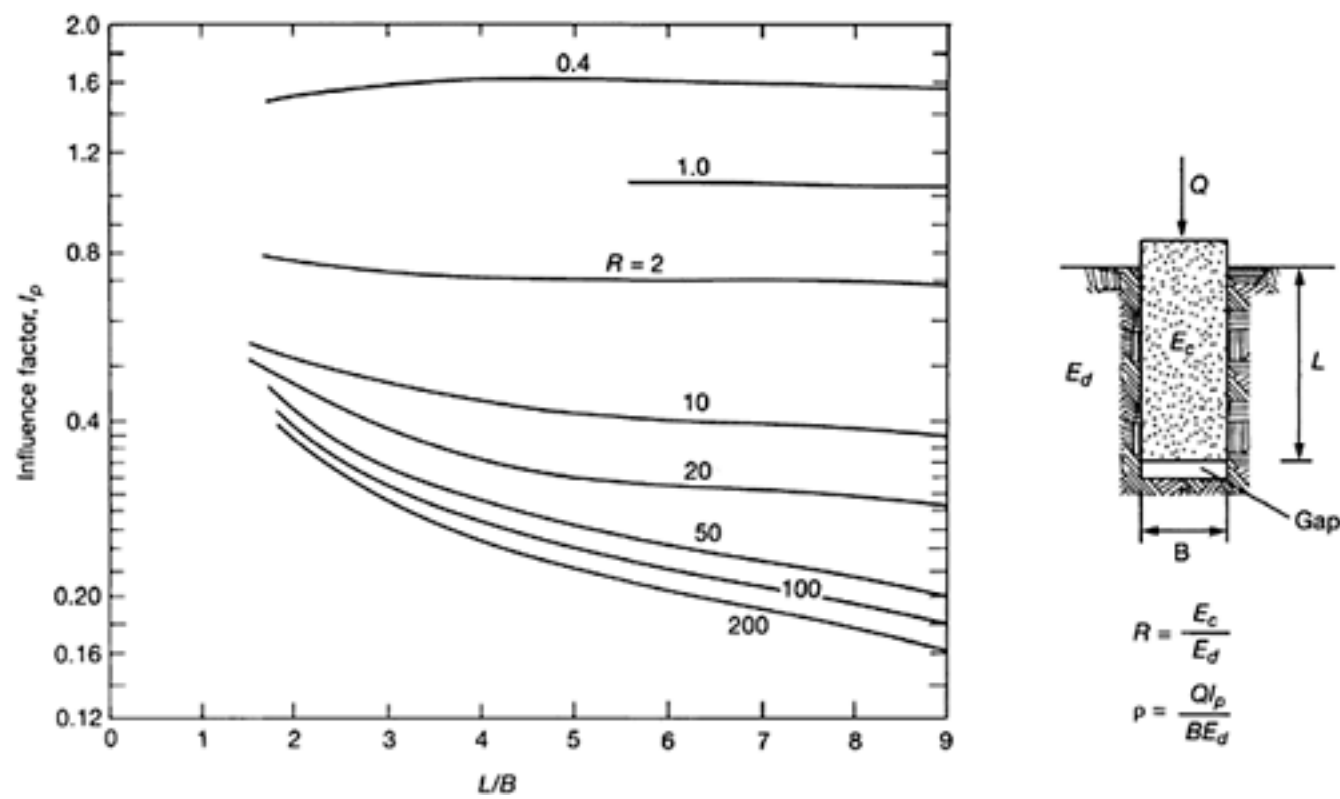


Fig. 4.41 Elastic settlement influence factors for rock-socket skin friction on piles (after Pells and Turner<sup>(4.6)</sup>, courtesy of Research Journals, National Research Council, Canada)

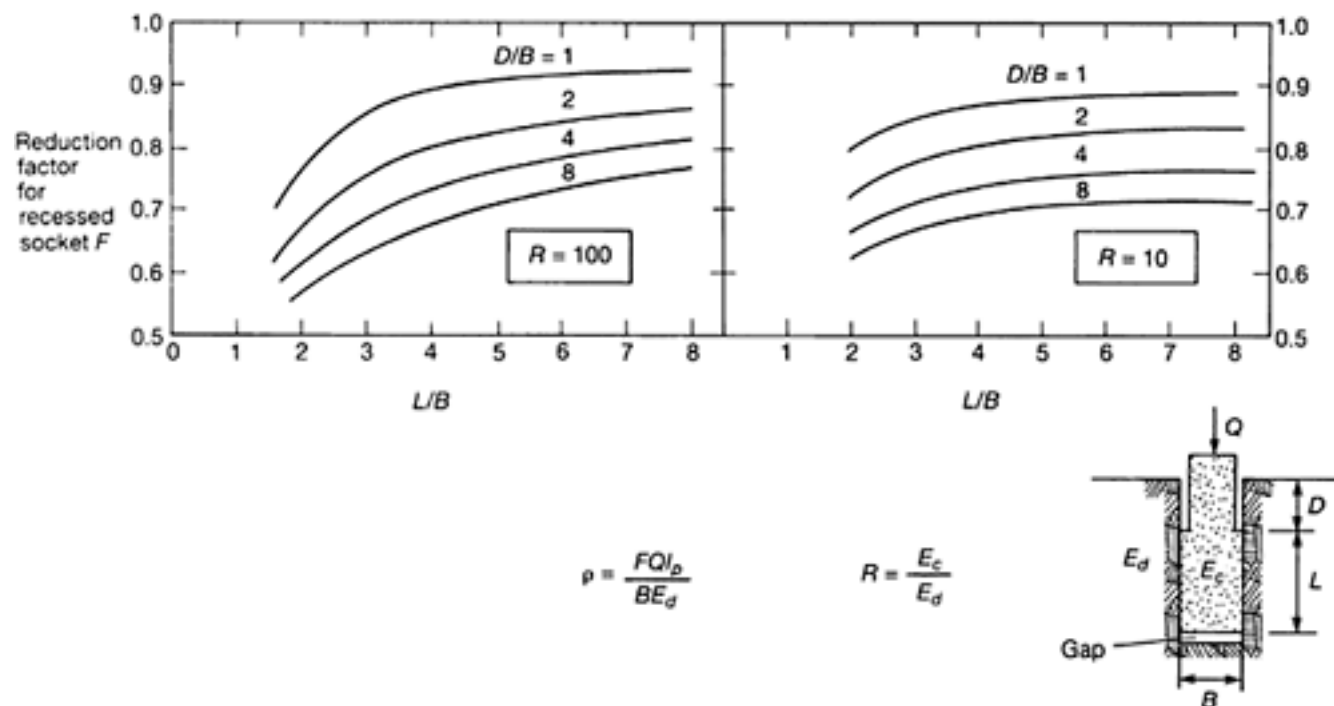


Fig. 4.42 Reduction factors for calculation of settlement of recessed sockets (after Pells and Turner<sup>(4.61)</sup>, courtesy of Research Journals, National Research Council, Canada)

## 4.8 Piles in fill—negative skin friction

### 4.8.1 Estimating negative skin friction

Piles are frequently required for supporting structures that are sited in areas of deep fill. The piles are taken through the fill to a suitable bearing stratum in the underlying natural soil or rock. No support for compressive loads from skin friction can be assumed over the length of the pile shaft through the fill. This is because of the downward movement of the fill as it compresses under its own weight or

under the weight of further soil or surcharge placed over the fill area. The downward movement results in drag-down forces, generally known as *negative skin friction*, on the pile shaft. Where fill is placed over a compressible natural soil the latter consolidates and moves downwards relative to the pile. Thus the negative skin friction occurs over the length of the shaft within the natural soil as well as within the fill.

Calculation of the magnitude of the negative skin friction is a complex problem which depends on the following factors.

1. The relative movement between the fill and the pile shaft.
2. The relative movement between any underlying compressible soil and the pile shaft.
3. The elastic compression of the pile under the working load.
4. The rate of consolidation of the compressible layers.

The simplest case is fill that is placed over a relatively incompressible rock with piles driven to refusal in the rock. The toe of the pile does not yield under the combined working load and drag-down forces. Thus the negative skin friction on the upper part of the pile shaft is equal to the fully mobilized value. Near the base of the fill its downward movement may be insufficiently large to mobilize the full skin friction, and immediately above rock-head the fill will not settle at all relative to the pile shaft. Thus negative skin friction cannot occur at this point. The distribution of negative skin friction on the shaft of the unloaded pile is shown in [Figure 4.43a](#). If a heavy working load is now applied to the pile shaft, the shaft compresses elastically and the head of the pile moves downwards relative to the fill. The upper part of the fill now acts in support of the pile although this contribution is neglected in calculating the pile resistance. The distribution of negative skin friction on the shaft of the loaded pile is shown in [Figure 4.43b](#). Where the fill has been placed at a relatively short period of time before installing the piles, continuing consolidation of the material will again cause it to slip downwards relative to the pile shaft, thus re-activating the down drag force.

The simplified profile of negative skin friction for a loaded pile on an incompressible stratum is shown in [Figure 4.43c](#). This diagram can be used to calculate the magnitude of the drag-down forces. The peak values for granular soils and fill material are calculated by the method described in 4.3.

In the case where negative skin friction is developed in clays, the rate of loading must be considered. It was noted in 4.2.4 that the capacity of a clay to support a pile in skin friction is substantially reduced if the load is applied to the pile at a very slow rate. The same consideration applies to negative skin friction, but in this case it works advantageously in reducing the magnitude of the drag-down force. In most cases of negative skin friction in clays the relative movement between the soil which causes drag-down and the pile takes place at a very slow rate. The movement is due to the consolidation of the clay under its own weight, or under imposed loading, and this process is very slow compared with the rate of application of the working load to the pile.

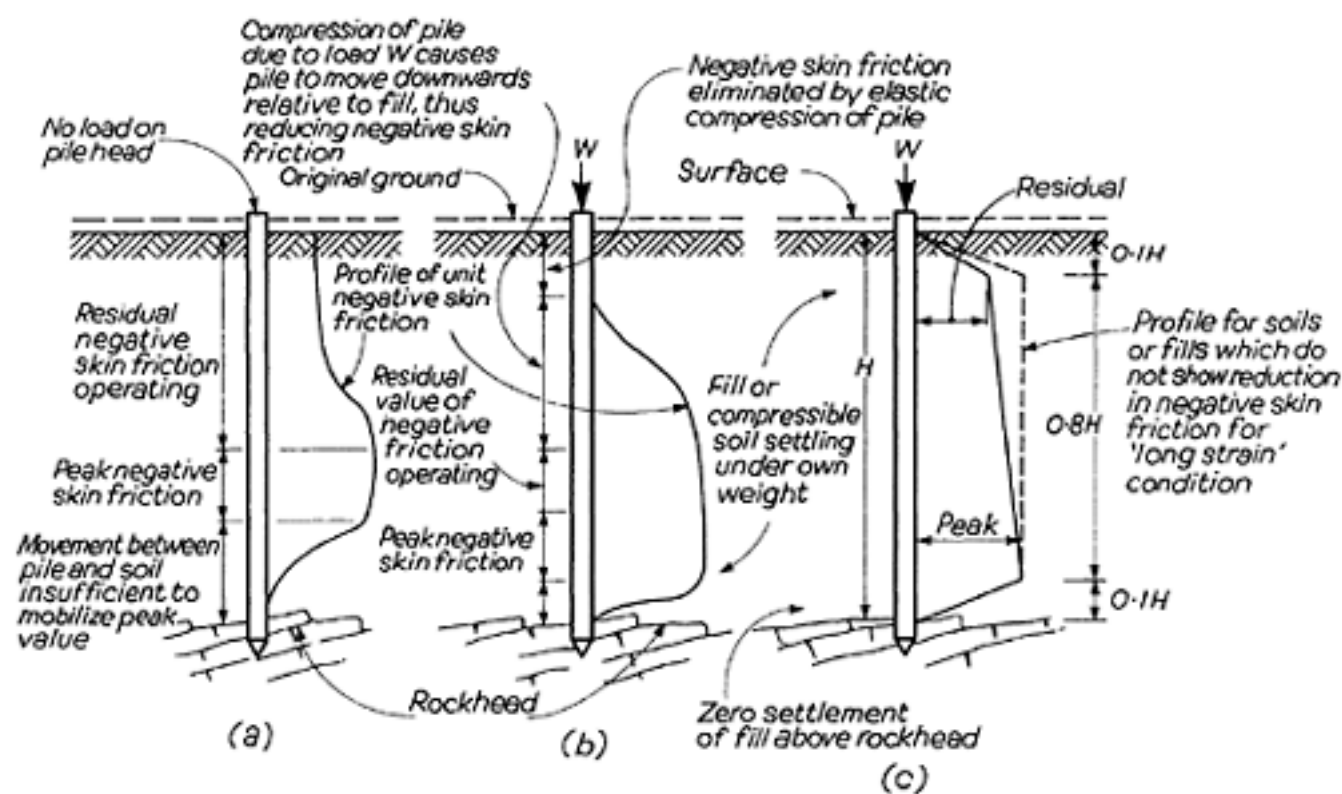


Fig. 4.43 Distribution of negative skin friction on piles terminated on relatively incompressible stratum

- (a) No load on pile head
- (b) Compressive load on pile head
- (c) Design curve for loaded pile

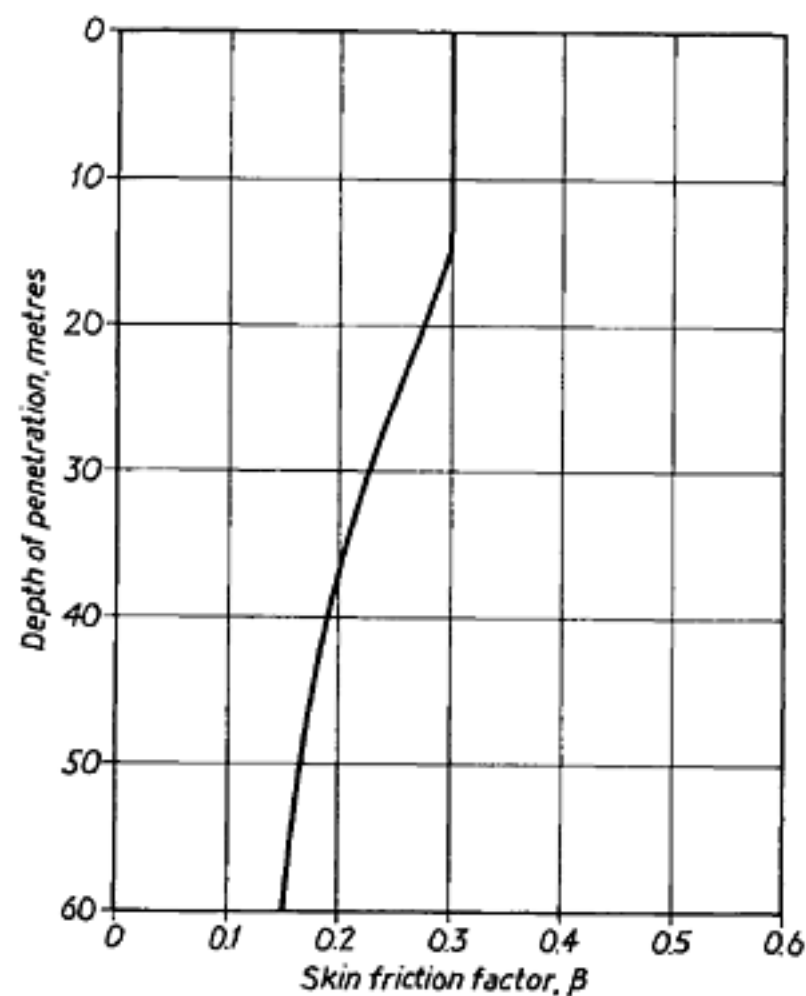


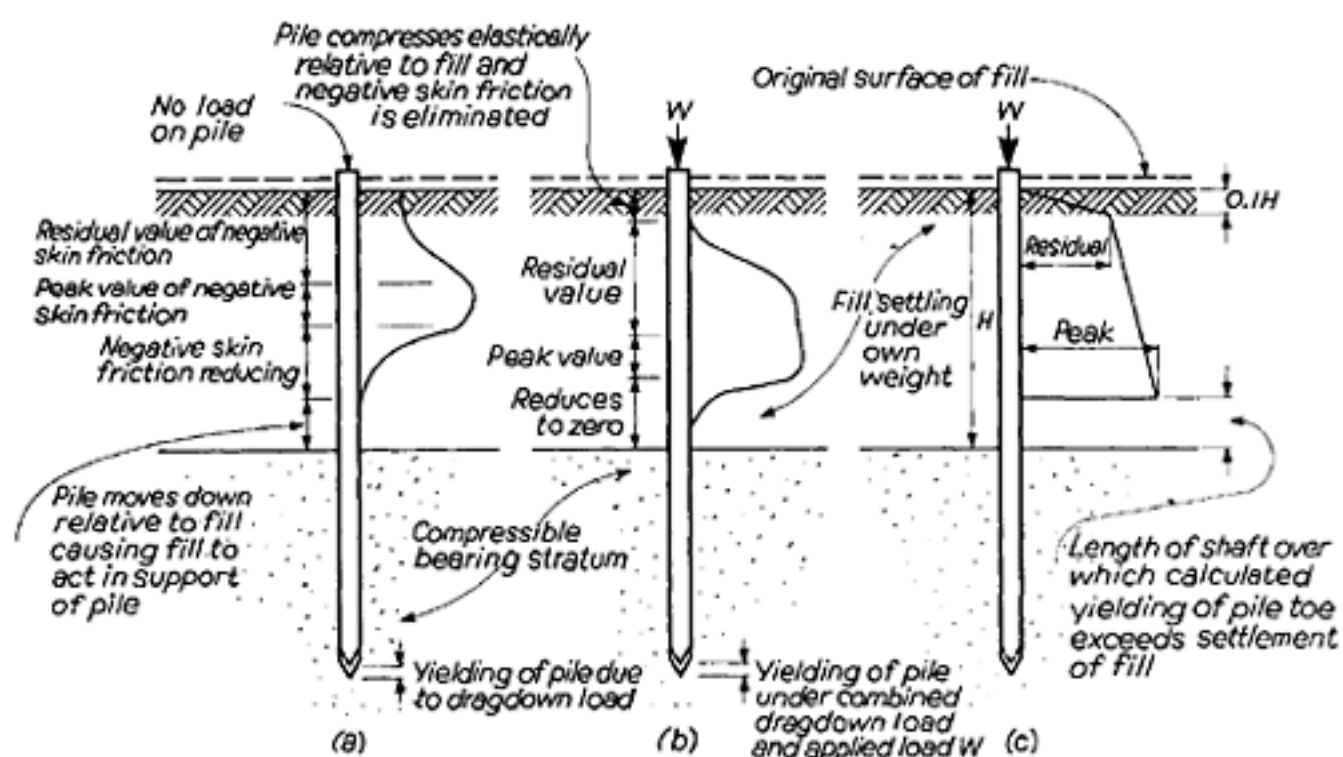
Fig. 4.44 Negative skin friction factors for piles driven into soft to firm clays (after Meyerhof<sup>(4.3)</sup>)

Meyerhof<sup>(4.3)</sup> advises that the negative skin friction on piles driven into soft to firm clays should be calculated in terms of effective stress from the equation:

$$\tau_{\text{neg}} = \beta \sigma'_{\text{vo}} \quad \dots(4.27)$$

Values of the skin friction factor allowing for reduction of the effective angle of friction with increasing depth to the residual value  $\delta_r$  are shown in [Figure 4.44](#).

Taking the case of a pile bearing on a compressible stratum, where yielding of the pile toe occurs under the drag-down forces and the subsequently applied working load, the downward movement of the pile relative to the lower part of the fill may then be quite large, and such that negative skin friction is not developed over quite an appreciable proportion of the length of the shaft within the fill. Over the upper part of the shaft the fill moves downwards relative to the pile shaft to an extent such that the negative skin friction operates, whereas in the middle portion of the pile shaft the small relative movement between the fill and the pile may be insufficient to mobilize the peak skin friction as a drag-down force. The distribution for the unloaded pile is shown in [Figure 4.45a](#).

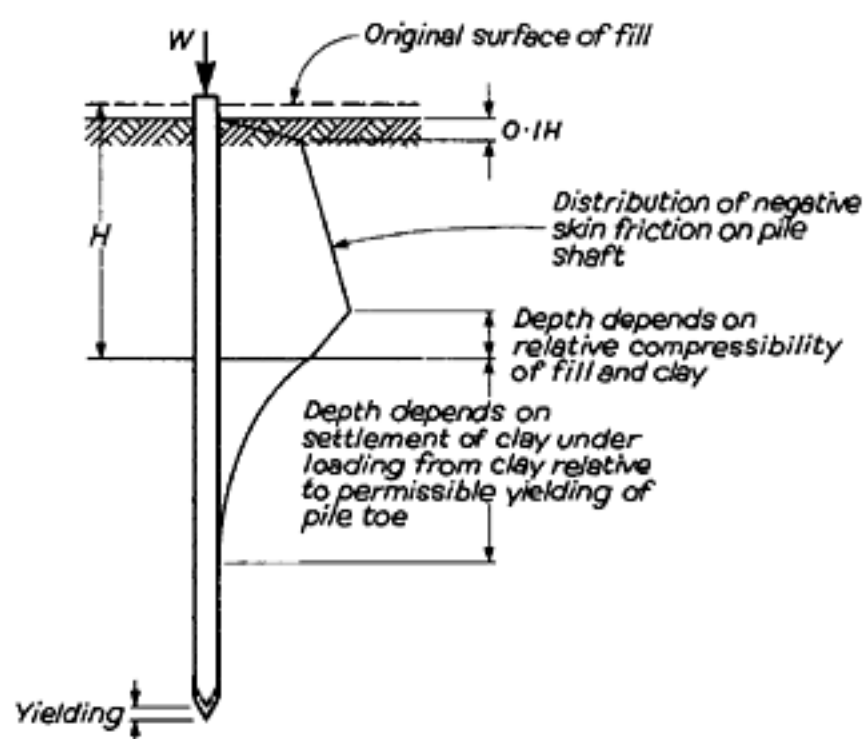


*Fig. 4.45 Distribution of negative skin friction on piles terminated in compressible stratum*

- (a) No load on pile head*
- (b) Compressed load on pile head*
- (c) Design curve for loaded pile*

When the working load is applied to the head of the pile, elastic shortening of the pile occurs, but since the load is limited by the bearing characteristics of the soil at the pile toe the movement may not be large enough to eliminate the drag-down force. The distribution of negative friction is then shown in [Figure 4.45b](#). The diagram in [Figure 4.45c](#) can be used for design purposes, with the peak value calculated as described in 4.3 for granular soils and fill and by using equation 4.27 and [Figure 4.44](#) for soft to firm clays.

It may be seen from [Figures 4.45a to 4.45c](#) that at no time does the maximum skin friction operate as a drag-down force over the full length of the pile shaft. It is not suggested that these simplified profiles of distribution of negative skin friction represent the actual conditions in all cases where it occurs, since so much depends on the stage reached in the consolidation of the fill, and the compression of the natural soil beneath the fill. The time interval between the installation of the pile and the application of the working load is also significant. In old fill which has become fully consolidated under its own weight and where it is not proposed to impose surcharge loading the negative skin friction may be neglected, but skin friction within the fill layer should not be allowed to help support the pile. In the case of recently-placed fill it may settle by a substantial amount over a long period of years. The fill may also be causing consolidation and settlement of the natural soil, within which the pile obtains its bearing. The case of recent fill placed over a compressible soil which becomes stiffer and less compressible with depth is shown in [Figure 4.46](#).



*Fig. 4.46 Distribution of negative skin friction on pile driven through recent fill into compressible clay stratum*

It is good practice to ignore the contribution to the support provided by skin friction over the length of a pile in soft clay, where the pile is driven through a soft layer to less compressible soil. This is because of the drag-down force on the pile shaft caused by heave and reconsolidation of the soft clay. The same effect occurs, of course, if a pile is driven into a stiff clay but the stiff clay continues to act in support of the pile if yielding at the toe is permitted.

Very large drag-down forces can occur on long piles. In some circumstances they may exceed the working load applied to the head of the pile. Fellenius<sup>(4.64)</sup> measured the progressive increase in negative skin friction on two precast concrete piles driven through 40m of soft compressible clay and 15m of less-compressible silt and sand. Reconsolidation of the soft clay disturbed by pile driving contributed 300kN to the drag-down load over a period of 5 months. Thereafter, regional settlement caused a slow increase in negative skin friction at a rate of 150kN per year. Seventeen months after pile driving a load of 440kN was added to each pile, followed by an additional load of 360kN a year later. Both these loads caused yielding of the pile at the toe to such an extent that all negative skin friction was eliminated, but when the settlement of the pile ceased under the applied load the continuing regional settlement caused negative skin friction to develop again on the pile shaft. Thus with a yielding pile toe the amount of negative skin friction which can be developed depends entirely on the downward movement of the pile toe relative to the settlement of the soil or fill causing the drag-down force. If the drag-down force is caused only by the reconsolidation of the heaved soil, and if the pile can be permitted to yield by an amount greater than the settlement of the ground surface due to this reconsolidation, then negative friction need not be provided for. If, however, the negative skin friction is due

to the consolidation of recent fill under its own weight or to the weight of additional fill, then the movement of the ground surface will be greater than the permissible yielding of the pile toe. Negative skin friction must then be taken into account, the distribution being as shown in [Figure 4.45c](#) or [Figure 4.46](#).

Much greater drag-down loads occur with piles driven onto a relatively unyielding stratum. Johanessen and Bjerrum<sup>(4.65)</sup> measured the development of negative skin friction on a steel pile driven through 53m of soft clay to rock. Sand fill was placed to a thickness of 10m on the sea bed around the pile. The resulting consolidation of the clay produced a settlement of 1.2m at the original sea-bed level and a drag-down force of about 1500kN at the pile toe. It was estimated that the stress in the steel near the toe could have been about 190N/mm<sup>2</sup>, which probably caused the pile to punch into the rock, so relieving some of the drag-down load. The average unit negative skin friction within the soft clay was equal to 100% of the undrained shearing strength of the clay.

#### 4.8.2 Safety factors for negative skin friction

Safety factors for piles subjected to negative skin friction require careful consideration. The concept of partial safety factors (see [4.6](#)) can be applied to the two components of permanent working load and negative skin friction. Thus if the negative skin friction  $P_n$  has been conservatively estimated before deciding on a value of  $Q_p$  to give a safety factor of 2.5 or more on

the combined loading, it is over-conservative to add this to the working load  $W$  on the pile in order to arrive at the total allowable pile load. It is more realistic to obtain the required *ultimate* pile load  $Q_p$  by multiplying the working load only by

the normal safety factor, and then to check that the safety factor given by the ultimate load divided by the working load plus the negative skin friction is still a reasonable value.

#### 4.8.3 Minimizing negative skin friction

The effects of drag-down can be minimized by employing slender piles (*e.g.* H-sections or precast concrete piles), but more positive measures may be desirable to reduce the magnitude of the drag-down forces. In the case of bored piles this can be done by placing in-situ concrete only in the lower part of the pile within the bearing stratum and using a precast concrete element surrounded by a bentonite slurry within the fill. Negative skin-frictional forces on precast concrete or steel tubular piles can be reduced by coating the portion of the shaft within the fill with soft bitumen.

Claessen and Horvat<sup>(4.66)</sup> describe the coating of 380 x 450mm precast concrete piles with a 10mm layer of bitumen having a penetration of 40 to 50mm at 25°C. The skin friction on the 24m piles was reduced to 750kN compared with 1600 to 1700kN for the uncoated piles.

Shell Composites Ltd. markets its Bitumen Compound SL for coating bearing piles. The material has the following characteristics.

Penetration at 25°C:	53 to 70mm
Softening Point (R and B):	57 to 63°C
Penetration Index:	Less than +2.

The bitumen is heated to 180°C (maximum) and sprayed or poured onto the pile to obtain a coating thickness of 10mm. Before coating, the pile surface should be cleaned and primed with Shell Composites Bitumen Solvent Primer applied by brush or spray at a rate of about 2kg/10m<sup>2</sup>. Alternatively, the SL Compound can be fluxed with 29% of white spirit to provide the primer. The bitumen slip layers should not be applied over the length of the shaft which receives *support* from skin friction, and Claessen and Horvat recommend that a length at the lower end of ten times the diameter or width of the pile should remain uncoated if the full *end-bearing resistance* is to be mobilized.

Negative skin friction is a most important consideration where piles are installed in groups. The overall settlement of pile groups in fill must be analysed as described in 5.5.

The above measures to minimize negative skin friction can be quite costly. In most cases it will be found more economical to increase the penetration of the pile into the bearing stratum thereby increasing its capacity to carry the combined loading.

Reddaway and Elson<sup>(4.67)</sup> observed that the cost of applying a bitumen slip coat to 285mm square precast concrete segmental piles was about 70% of the manufacturing cost of the pile segments on a metre for metre basis. The piles were used to support a bridge abutment at Newhaven where negative skin friction was induced by the surcharge from the approach embankment behind the abutment.

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## 4.10 Worked examples

### Example 4.1

A precast concrete pile for a jetty structure is required to carry a working load of 350kN in compression and 250kN in uplift. It is driven through 7m of very soft silty clay into stiff boulder clay with a fairly uniform undrained shearing strength of 110kN/m<sup>2</sup>. Determine the required penetration depth of a 350×350mm precast concrete pile.

The settlements of a jetty structure are not very critical to the structural design and a safety factor of 2 could be used to obtain the required ultimate carrying capacity. However, the effects of driving a pile through a soft clay to a limited penetration in a stiff clay are somewhat uncertain. It is therefore advisable to adopt a safety factor of 2.5 in this case. The skin friction in the soft clay will be ignored, partly because of the effects of heave and subsequent reconsolidation and partly because of the continuing disturbance of this layer due to the lateral forces acting on the jetty structure.

The safety factor of 2.5 requires an ultimate load in compression to be carried by stiff clay of  $2.5 \times 350 = 875\text{kN}$ .

From equation 4.2,

$$\text{ultimate end-bearing resistance} = 9 \times 110 \times 0.35^2 = 121 \text{ kN}.$$

Therefore the required skin-frictional resistance in compression =  $875 - 121 = 754 \text{ kN}$ .

For a penetration depth of 7.5m into stiff clay, from [Figure 4.7b](#) with  $L/B = 7.5/0.35 = 21$  and  $c_u = 110 \text{ kN/m}^2$ , the adhesion factor is 0.65. Therefore from equation 4.5,

$$\text{ultimate skin friction} = 0.65 \times 110 \times 4 \times 0.35 \times 7.5 = 751 \text{ kN}.$$

$$\text{Total ultimate resistance} = 121 + 751 = 872 \text{ kN}.$$

$$\text{Factor of safety on compressive load} = 872/350 = 2.49,$$

which is satisfactory.

$$\text{Factor of safety on uplift load} = 751/250 = 3,$$

which is also satisfactory.

The required penetration can be checked for compliance with Eurocode 7. Take average  $c_u$  of  $110 \text{ kN/m}^2$ . Because the empirical relationships between shaft friction and  $c_u$  shown in [Figure 4.7](#) are based on correlation of static load tests  $Q_{sk}$  should be divided by a factor of 1.4 (not 1.5 because the adhesion factors in [Figure 4.7](#) are conservative). To allow for the possibility of  $c_u$  at the base being lower than the average, a characteristic value of 0.75 times the average is taken and divided by the material factor of 1.8 (code Table 2b). Therefore:

$$\text{Characteristic base resistance} = Q_{bk} = \frac{9 \times 0.75 \times 110 \times 0.35^2}{1.8 \times 1.4} = 36 \text{ kN}$$

$$\text{Characteristic shaft resistance} = Q_{sk} = \frac{0.65 \times 110 \times 4 \times 0.35 \times 7.5}{1.4} = 536 \text{ kN}$$

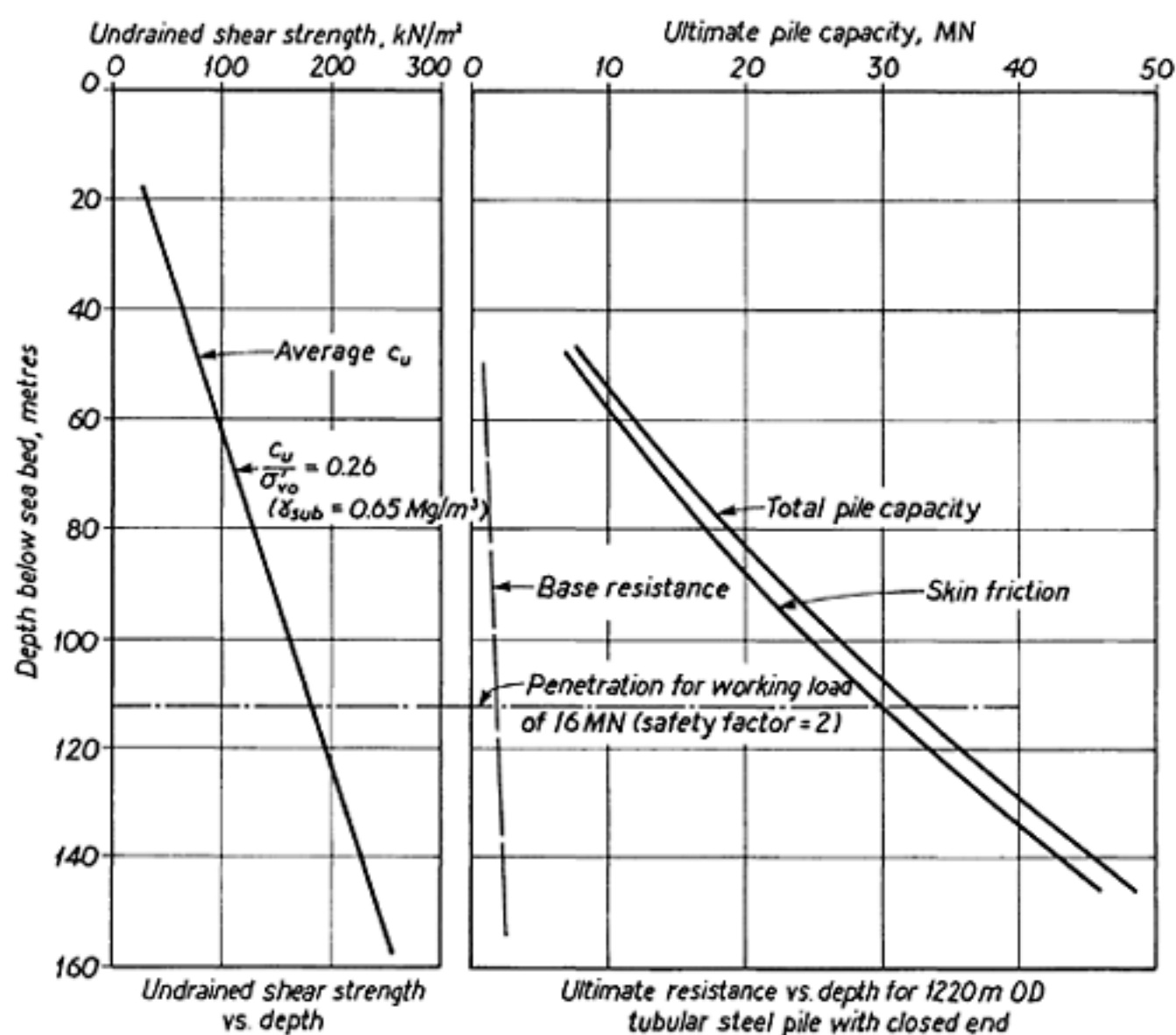
$$\text{Design bearing capacity} = 36/1.3 + 536/1.3 = 440 \text{ kN}.$$

Therefore a lesser penetration would be required for design to the Eurocode on the given load of  $350 \text{ kN}$  but the partial load  $\gamma_F$  for actions (Table 2a of the code) would be increased to allow for the effect of dynamic loads from berthing ships,

whereas with the 'permissible stress' method the safety factor of 2.5 would be deemed to cover this type of loading.

### Example 4.2

A steel tubular pile 1.220m in outside diameter forming part of a berthing structure is required to carry a working load in compression of  $16 \text{ MN}$  and a uplift of  $8 \text{ MN}$ . The pile is driven with a closed end into a deep deposit of normally-consolidated marine clay. The undrained shearing strength-depth profile of the clay is shown in [Figure 4.47](#). Determine the depth to which the pile must be driven to carry the working load with a safety factor of 2.0.





In dealing with problems of this kind it is a good practice to plot the calculated values of ultimate skin friction, end bearing and total resistance for various depths of penetration. The required pile length can then be read off from the graph. This is convenient procedure for a marine structure where the piles may have to carry quite a wide range of loading.

$$\text{Outside perimeter of pile} = \pi \times 1.220 = 3.83\text{m.}$$

$$\text{Overall base area of pile} = \frac{1}{4} \times \pi \times 1.220^2 = 1.169\text{m}^2.$$

From [Figure 4.47](#), at 160m,

$$\frac{c_u}{\sigma'_{vo}} = \frac{260}{0.65 \times 9.81 \times 160} = 0.25.$$

From [Figure 4.9a](#), the adhesion factor,  $\alpha_p$  is 1.0 over the full depth.

At 50m below the sea bed:

$$\text{Average shearing strength along shaft} = \frac{1}{2} \times 80 = 40\text{kN/m}^2.$$

From [Figure 4.9b](#), adhesion factor for  $L/B$  value of  $50/1.22 = 41$  is 1.0.

$$\begin{aligned} \text{From equation 4.6, total skin friction on outside of shaft} &= \frac{1.0 \times 1.0 \times 40 \times 3.83 \times 50}{1000} = 7.66\text{MN} \\ &= \frac{9 \times 80 \times 1.169}{1000} = 0.84\text{MN} \end{aligned}$$

From equation 4.2, end-bearing resistance = 8.50MN.

Thus total pile resistance

At 75m below the sea bed:

$$\text{Average shearing strength along shaft} = \frac{1}{2} \times 120 = 60\text{kN/m}^2.$$

Length factor for  $L/B$  value of 61 is 0.9.

$$\begin{aligned} \text{Thus total skin friction on outside of shaft} &= \frac{1 \times 0.9 \times 60 \times 3.83 \times 75}{1000} = 15.51\text{MN} \\ &= \frac{9 \times 120 \times 1.169}{1000} = 1.26\text{MN} \end{aligned}$$

End-bearing resistance

Thus total pile resistance = 16.77MN.

Similarly the total pile resistances at depths of 100m, 125m and 150m below the sea bed are 26.19, 38.01 and 50.78MN respectively.

The calculated values of pile resistance are plotted in [Figure 4.47](#), from which it may be seen that a penetration depth of 113m is required to develop an ultimate resistance of 32MN, which is the value required to support a compressive load of 16MN with a safety factor of 2. At a depth of 113m the skin friction is 30MN. Therefore the safety factor on uplift is  $30/8$ , i.e. nearly 4.

Checking the working stress on the steel,

$$= \frac{16 \times 10^6}{\frac{1}{4}\pi(1270^2 - 1234^2)} = 170\text{N/mm}^2.$$

For a wall thickness of 25 mm max. compressive stress

This is 50% of the yield stress for high-tensile steel which is satisfactory for the easy driving conditions ([Table 2.9](#)). The thickness can be decreased to 16mm in the lower 50m of the pile.

Checking the design bearing capacity at 75m penetration by the Eurocode recommendations:

$$Q_{bk} = \frac{9 \times 0.75 \times 120 \times 1.169}{1.8 \times 1.5 \times 1000} = 0.35\text{MN}$$

$$Q_{d1} = 15.51/1.5 = 10.34\text{MN}$$

$$\text{Design bearing capacity} = \frac{0.35}{1.3} + \frac{10.34}{1.3} = 8.2\text{MN}$$

This is practically the same as the allowable pile load at 75m which would have been obtained by dividing the calculated ultimate pile resistance of 16.77MN by the specified safety factor of 2.

### **Example 4.3**

A building column carrying a working load of 1400kN is to be supported by a single bored pile installed in firm to stiff fissured London clay ([Figure 4.48](#)). Select suitable dimensions and penetration depth

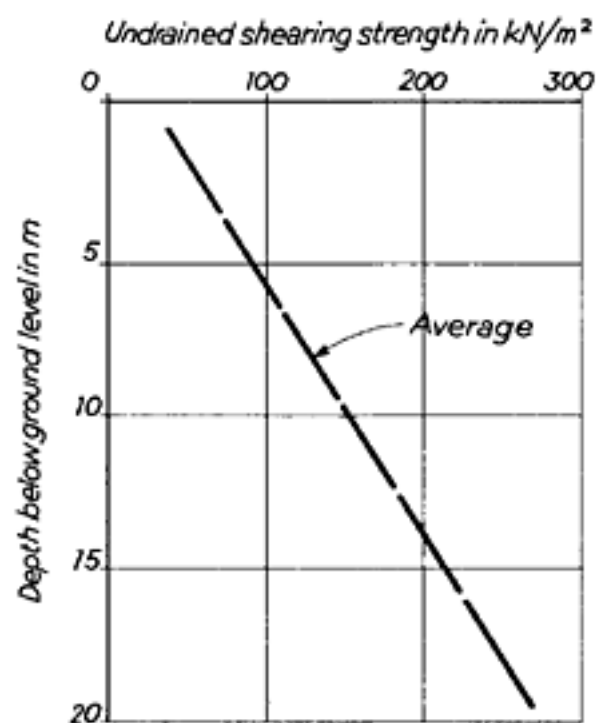


Fig. 4.48

to obtain a safety factor of 2 in total pile resistance, or safety factors of 3 in end bearing and unity in shaft friction. Calculate the immediate settlement at the working load.

A pile of 1m diameter is suitable. A penetration depth of 13m (12m below cut-off level) will be tried.

Average shearing strength along pile shaft (from [Figure 4.48](#))

$$= \frac{1}{2}(35 + 182) = 108.5 \text{ kN/m}^2.$$

Take an adhesion factor of 0.45. Then from equation 4.5,

$$\text{total skin friction on pile shaft} = 0.45 \times 108.5 \times \pi \times 1 \times 12 = 1841 \text{ kN}.$$

Because the clay is fissured it is desirable to reduce the average shearing strength at pile base level to obtain the end-bearing resistance. Thus

$$\text{end-bearing resistance} = 9 \times 0.75 \times 182 \times \frac{1}{4} \pi \times 1^2 = 965 \text{ kN}.$$

For a safety factor of 2 on the total pile resistance,

$$\text{allowable working load} = \frac{1}{2}(1841 + 965) = 1403 \text{ kN}.$$

For a safety factor of 3 on the base resistance and 1 on the skin friction,

$$\text{allowable working load} = \left(\frac{1}{3} \times 965\right) + 1841 = 2163 \text{ kN}.$$

Checking the allowable load by the Eurocode recommendations, the average  $c_u$  at base level is regarded as a characteristic strength and is divided by a material factor of 1.8. Hence

$$Q_{bk} = \frac{965}{1.8 \times 1.5} = 357 \text{ kN} \quad \text{and} \quad Q_{sk} = \frac{1841}{1.5} = 1227 \text{ kN}.$$

$$\text{design bearing capacity} = \frac{357}{1.6} + \frac{1227}{1.3} = 1167 \text{ kN}.$$

This is rather more conservative than the value of 1403kN obtained by the foregoing 'permissible stress' method.

The alternative of a bored pile with an enlarged base is now considered. Take the same shaft diameter and a base diameter of 2m, a base level for the pile being at 10m below ground level. The top of the base enlargement will be at 9m and the base resistance must be ignored for 2m above this level ([Figure 4.12](#)).

Average shearing strength from cut-off level at 1m to 7m =  $\frac{1}{2}(35 + 111) = 73 \text{ kN/m}^2$ .

An adhesion factor of 0.3 should be used to allow for delays and softening effects while drilling the enlarged base. Thus

$$\text{total skin friction on pile shaft} = 0.3 \times 73 \times \pi \times 1 \times 6 = 413 \text{ kN}.$$

As before a reduction factor of 0.75 should be used on the shearing strength of  $145 \text{ kN/m}^2$  at a depth of 10m. Thus from equation 4.2,

$$\text{end-bearing resistance} = 9 \times 0.75 \times 145 \times \frac{1}{4}\pi \times 2^2 = 3075 \text{ kN}.$$

For a safety factor of 2 on the total pile resistance,

$$\text{allowable working load} = \frac{1}{2}(413 + 3075) = 1744 \text{ kN}.$$

For a safety factor of 3 on base resistance and 1 on skin friction,

$$\text{allowable working load} = (\frac{1}{3} \times 3075) + 413 = 1438 \text{ kN}.$$

Therefore a 1m diameter bored pile with an enlarged base of 2m diameter at a depth of 10m below ground level is satisfactory for a working load of 1400kN.

When using the Eurocode recommendations to design the enlarged base pile a more sophisticated method than a simple factor of  $0.75 \times \bar{c}_u$  would be needed to determine the characteristic shear strength of the soil beneath the pile toe. It would be necessary to look at the individual test results, perhaps to make a statistical analysis of the data, and also to assess the effects of the installation procedure.

Comparing the two types of pile it would clearly be more economical to adopt the deeper straight-sided pile because of the faster drilling rate and the avoidance of difficulties often experienced when installing enlarged bases.

Considering the settlement of the straight-sided pile, the ultimate shaft friction of 1841kN exceeds the working load, and therefore the settlement of the pile will be no more than that required to mobilize the ultimate resistance; *i.e.*, a settlement of about 10mm may be expected.

The immediate settlement of the enlarged base pile can be calculated from equation 4.18. For the full mobilization of shaft resistance at the working load,

$$\text{load on base} = 1400 - 413 = 987 \text{ kN},$$

$$\text{Unit pressure on base} = \frac{987}{\frac{1}{4}\pi \times 2^2} = 314 \text{ kN/m}^2,$$

$$\text{Unit ultimate base resistance} = 9 \times 0.75 \times 145 = 979 \text{ kN/m}^2.$$

In the absence of plate bearing test data, take  $K$  in equation 4.17 as 0.02. Thus

$$\rho_i = 2 \times 0.02 \times \frac{314}{979} \times 1000 = 13 \text{ mm}.$$

This is an acceptable value.

Checking from equation 4.19, taking the drained deformation modulus of the clay at pile base level as  $140c_u$  (for long-term loading), we get:

$$\begin{aligned} \rho &= \frac{(413 + 2 \times 987) \times 8000}{2 \times 0.7854 \times 30 \times 10^6} + \frac{0.5 \times 987 \times 1000}{2 \times 140 \times 145} \\ &= 0.4 + 12.2 \\ &= 13 \text{ mm} \end{aligned}$$

#### Example 4.4

A precast concrete pile 450mm square forming part of a jetty structure is driven into a medium dense over-consolidated sand.

Standard penetration tests made in the sand gave an average value of  $N$  of 15 blows/300mm. The pile is required to carry a compressive load of 250kN and an uplift load of 180kN. Determine the required penetration depth of the pile for a safety factor of at least 2.5 on both compressive and uplift loads.

The unit skin friction developed on a pile in sand is rather low, and thus the penetration depth in this case is likely to be governed by the requirements for uplift resistance.

Take a trial penetration depth of 7m below sea bed. From [Figure 4.13](#) for  $N = 15$ ,  $\phi = 31^\circ$ . The submerged density of the sand may be taken as  $1.2\text{Mg/m}^3$ . For an over-consolidated sand we can take

$K_o = 1$ . Table 4.1 gives  $K_s/K_o = \text{say, } 1.5$ , giving  $K_s = 1.5$ . From Table 4.2, take  $\delta = 0.8\phi = 0.8 \times 31^\circ = 24.8^\circ$

$$\begin{aligned}\text{Total skin friction} &= \frac{1}{2} \times 1.5 \times 1.2 \times 9.81 \times 7 \times \tan 24.8^\circ \times 7 \times 4 \times 0.45 \\ &= 360 \text{ kN}\end{aligned}$$

$$\text{Factor of safety on uplift load} = \frac{360}{180} = 2 \text{ which is satisfactory.}$$

Checking by the Eurocode recommendations the  $\phi$  value of  $31^\circ$  was derived from the average SPT resistance. Therefore the shaft friction derived from it must be divided by a factor of 1.5. Hence design uplift capacity

$$= Q_{bk}/\gamma_s = 360/1.5 \times 1.3 = 185 \text{ kN}$$

Calculating the base resistance by using equation 4.9 and ignoring the first term. Assuming loose sand conditions, and effective overburden pressure

$$= \frac{1.2 \times 7}{10.7} = 0.78 \text{ tons}$$

From equation 4.11,  $I_r = \frac{30}{\sqrt{0.78 \times \tan 28^\circ}} = 64$

Figure 4.15b gives  $N_q^* = 40$  for  $\phi = 31^\circ$

$$= 40 \times 1.2 \times 9.81 \times 7$$

Therefore, unit base resistance  $= 3296 \text{ kN/m}^2$

Total base resistance  $= 3296 \times 0.45^2 = 667 \text{ kN}$

Total pile resistance  $= 360 + 667 = 1027 \text{ kN}$

Safety factor on compressive load  $= \frac{1027}{250} = 4$  which is satisfactory.

Using the Eurocode recommendations, design bearing capacity  $= 360/1.5 \times 1.3 + 667/1.5 \times 1.3 = 527 \text{ kN}$

Checking the base resistance using Berezantsev's value of  $N_q$  in equation 4.8, from Figure 4.17 with

$\phi = 31^\circ$ ,  $N_q = 25$  ( $D/B = 15$ ). Thus

$$Q_b = 25 \times 1.2 \times 9.81 \times 7 \times 0.45^2 = 417 \text{ kN.}$$

$$\text{Total pile resistance} = 417 + 360 = 777 \text{ kN.}$$

$$\text{Safety factor on compressive load} = \frac{777}{250} = 3.1,$$

which is satisfactory.

### Example 4.5

Isolated piles are required to carry a working load of 900 kN on a site where borings and static cone penetration tests recorded the soil profile shown in Figure 4.49. Select suitable types of pile and determine their required penetration depth to carry the working load with a safety factor of 2.5. Previous tests in the area have shown that the ultimate base resistance of piles driven into the dense sand stratum is equal to the static cone resistance.

The piles will attain their bearing within the sand stratum. Any type of bored-and-cast-in-place pile will be uneconomical compared with the driven type. A driven-and-cast-in-place pile is suitable and this can be compared with a closed-end steel tubular pile.

*Driven-and-cast-in-place pile*

For 25-grade concrete and a working stress of 25% of the works cube strength (BS 8004), working stress on concrete  $= 0.25 \times 25 = 6.25 \text{ N/mm}^2$ . Therefore

$$\text{required diameter of pile shaft} = \sqrt{\frac{900 \times 1000 \times 4}{\pi \times 6.25}} = 428\text{mm.}$$

Say nominal shaft diameter of 450mm.

A saving in the length of the pile can be obtained by forming an enlarged base. The size attainable in a fairly dense sand is problematical but it should be possible to form a base with an effective diameter of 600mm.

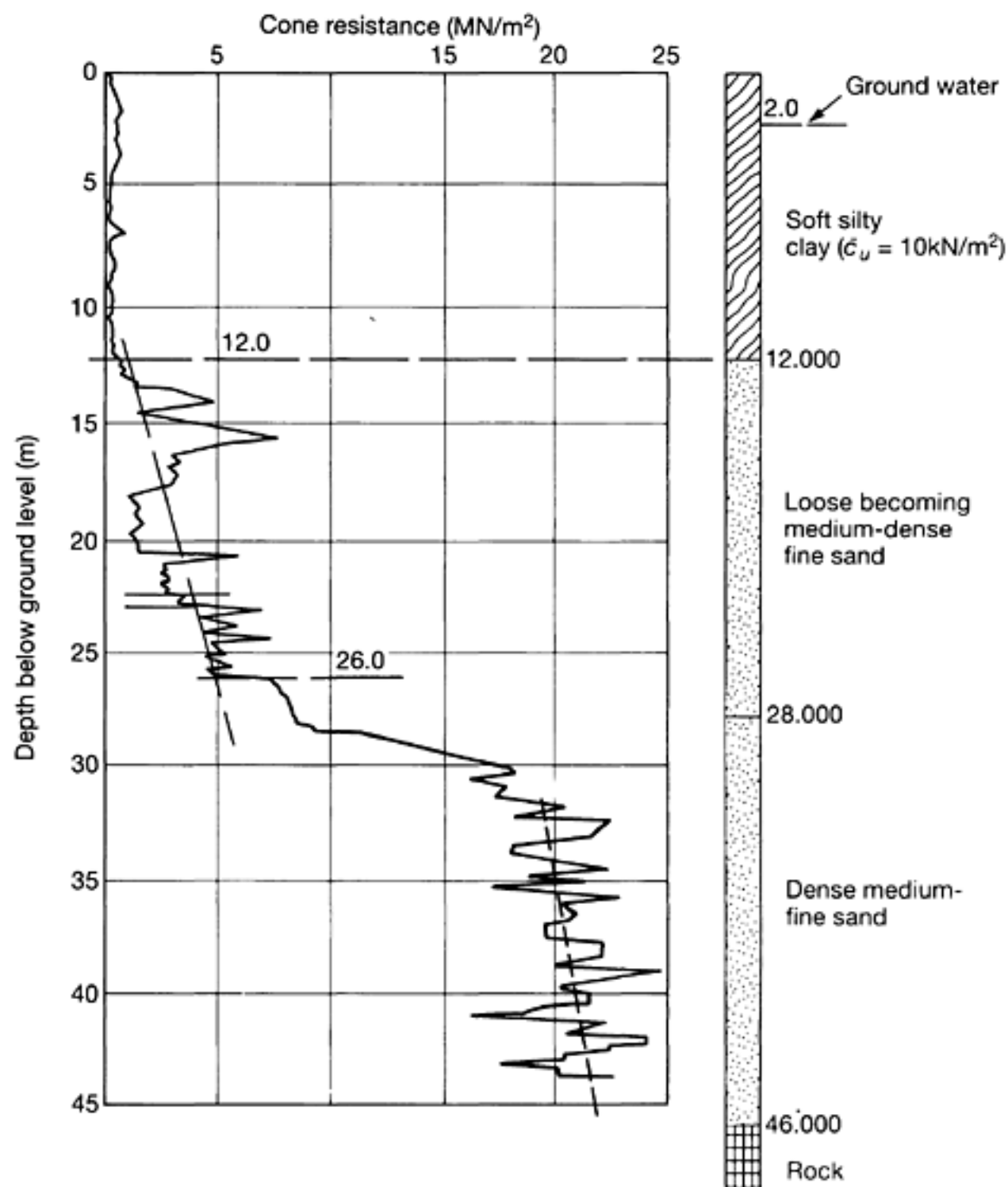


Fig. 4.49

For a wholly end-bearing pile,

$$\text{base pressure at ultimate load} = \frac{900 \times 2.5}{\frac{1}{4}\pi \times 0.6^2 \times 10^3} = 7.9\text{MN/m}^2$$

For a unit base pressure of  $7.9\text{MN/m}^2$  a penetration depth of about 28m is required. Taking into account the available skin-frictional resistance of the pile shaft the likely penetration depth would be 25m. Ignoring the skin friction within the soft clay, the skin friction within the sand stratum can be calculated by reference to [Table 4.3](#). A coefficient for a driven and cast-in-place pile is not shown in this table but it should not be less than 0.012.

$$\text{Ultimate skin friction at 12m} = 0.012 \times 1 \times 10^3 = 12\text{kN/m}^2$$

$$\text{Ultimate skin friction at 25m} = 0.012 \times 5 \times 10^3 = 60\text{kN/m}^2$$

$$\text{Total skin friction} = \frac{1}{2}(12 + 60) \times \pi \times 0.45 \times 13 = 662\text{kN}$$

$$\text{Required ultimate unit resistance beneath enlarged base} = \frac{900 \times 2.5 - 662}{0.7854 \times 0.6^2} = 5616\text{kN/m}^2$$

From inspection of [Figure 4.49](#) it will not be possible to obtain this resistance as an average over a distance of eight pile diameters above and four diameters below a pile toe level at 25m. For a toe level at 26m the cone resistance increases continuously over this level.

Therefore take  $\bar{q}_{c-1}$  over a distance  $0.7 \times 0.45 = 0.3\text{m}$  giving an average of  $6\text{MN/m}^2$ . Over a height of

$8 \times 0.45 = 3.6\text{m}$  the  $\bar{q}_{c-2}$  value is given by

$$\bar{q}_{c-2} = \frac{4.5 \times 2.8 + 3 \times 0.8}{3.6} = 4.2\text{MN/m}^2$$

From equation 4.14,

$$\bar{q}_c = \frac{6 + 4.2}{2} = 5.1 \text{ MN/m}^2$$

$$\begin{aligned} \text{Ultimate base resistance} &= 5.1 \times 0.7854 \times 0.6^2 \times 10^3 \\ &= 1442 \text{ kN} \end{aligned}$$

Approx. total pile resistance =  $662 + 1442 = 2104 \text{ kN}$

$$= \frac{2104}{900} = 2.3 \quad (\text{satisfactory})$$

Factor of safety on working load

The size of the enlarged base was conjectured and therefore it is necessary to undertake test loading to ensure that the working load can be applied without excessive settlement.

#### *Steel tubular pile*

For mild steel to BS 4360 Grade 43A having a yield stress of  $250 \text{ N/mm}^2$  and a working stress of 30% of the yield value (BS 8004),

$$\text{required area of steel} = \frac{900 \times 1000}{0.3 \times 250} = 12000 \text{ mm}^2.$$

A steel tubular pile having an outside diameter of 508mm and a wall thickness of 7.94mm has a steel area of  $12470 \text{ mm}^2$ . If the pile is wholly end bearing,

$$\text{unit pressure at base at ultimate load} = \frac{900 \times 2.5}{\frac{1}{4}\pi \times 0.508^2} = 11100 \text{ kN/m}^2.$$

From [Figure 4.49](#) an average cone resistance of  $11.1 \text{ MN/m}^2$  is attainable at rather less than 30m penetration. Allowing for a reduction in the end-bearing load due to shaft friction, take a trial penetration depth of 26m below ground level.

From [Table 4.3](#),

$$\text{Ultimate unit skin friction at 12m} = 0.012 \times 1 \times 10^3 = 12 \text{ kN/m}^2$$

$$\text{Ultimate unit skin friction at 26m} = 0.012 \times 5 \times 10^3 = 60 \text{ kN/m}^2$$

$$\text{Total skin friction} = \frac{1}{2}(12 + 60) \times \pi \times 0.508 \times 14 = 804 \text{ kN}$$

$$\text{Required ultimate unit base resistance} = \frac{900 \times 2.5 - 804}{0.7854 \times 0.508^2} = 7134 \text{ kN/m}^2$$

The average  $q_c$  value at 26m has been shown to be only  $5 \text{ MN/m}^2$ . However inspection of [Figure 4.49](#) shows that the required value is obtainable at a depth of 28m.

Therefore the working load of 900kN can be carried either by a driven-and-cast-in-place pile with the 450mm diameter shaft enlarged at the base or by a 508mm outside-diameter steel tubular pile with a wall thickness of 7.94mm and the end closed by a 15mm steel plate. The required penetration depths for the two types of pile are 26 and 28m respectively.

### **Example 4.6**

Calculate the ultimate bearing capacity of a 914mm OD×19mm wall thickness tubular steel pile driven with an open end to a depth of 17m below ground level in the soil conditions shown in [Figure 4.23a](#).

Pile characteristics are: External perimeter 2.87m

Internal perimeter 2.75m

For 38mm shoe, net cross-sectional area at toe= $0.1046 \text{ m}^2$

Gross base area= $0.656 \text{ m}^2$

The skin friction on the pile shaft is calculated from the average curve shown in [Figure 4.23a](#). Ignore any skin friction in the soft clay. The coefficient from [Table 4.3](#) is 0.008

$$\text{At } 6.5\text{m} \quad \text{ultimate unit skin friction} = 0.008 \times 4 = 0.032\text{MN/m}^2$$

$$\text{At } 14.5\text{m} \quad \text{ultimate unit skin friction} = 0.008 \times 9 = 0.072\text{MN/m}^2$$

$$\text{At } 17.0\text{m} \quad \text{ultimate unit skin friction} = 0.008 \times 9.9 = 0.079\text{MN/m}^2$$

Total external skin friction 6.5 to 17.0m

$$= \frac{1}{2}[(0.032 + 0.072) \times 8 + (0.072 + 0.079) \times 2.5] \times 2.87$$

$$= 1.74\text{MN}$$

$$\text{Total internal skin friction 6.5 to 17.0m} = 1.74 \times \frac{2.75}{2.87} = 1.67\text{MN}$$

From average curve in Figure 4.23a, net base resistance =  $9.9 \times 0.1046 = 1.03\text{MN}$

$$\text{Gross base resistance} = 9.9 \times 0.656 = 6.49\text{MN}$$

Figure 4.24 shows that the limiting value for a fine to coarse sand is not exceeded. The value of the gross base resistance assumes a solid end to the pile. Figure 4.21 shows that it would be unwise to assume a unit base resistance higher than  $3.5\text{MN/m}^2$ . Therefore the base resistance of the open-end pile plugged with sand reduces to  $2.30\text{MN}$ . This is higher than the calculated internal skin friction. Hence a plug of sand will not be carried down and the total pile resistance will be the sum of the external and internal skin friction plus the net base resistance.

$$\text{Total ultimate pile resistance} = 1.74 + 1.67 + 1.03 = 4.44\text{MN}$$

$$\text{Adopting a factor of safety of 2.5, allowable load} = \frac{4.44}{2.5} = 1.78\text{MN}$$

Checking the calculated base resistance from the lower bound curve in Figure 4.23a

$$\text{Total ultimate pile resistance} = 1.74 + 1.67 + 8 \times 0.1046 = 4.25\text{MN}$$

$$\text{Safety factor on working load} = \frac{4.25}{1.78} = 2.4 \quad (\text{satisfactory})$$

The net base resistance can be checked by converting the cone resistance to the equivalent angle of shearing resistance of the sand. Assume ground water level is at ground level.

$$\text{Effective overburden pressure at 17m} = 8 \times 6.5 + 10 \times 10.5 = 157\text{kN/m}^2$$

From Figure 4.14, for  $\bar{q}_c = 9.9\text{MN/m}^2$ ,  $\phi = 37^\circ$ . From Figure 4.17,  $N_q = 70$ , Therefore

$$\text{ultimate unit end-bearing resistance} = 70 \times 157 \times 10^{-3} = 11\text{MN/m}^2,$$

which is close to the value taken directly from Figure 4.23.

### Example 4.7

A bored and cast-in-place pile is required to carry a working load of  $9000\text{kN}$  at a site where  $4\text{m}$  of loose sand overlies a weak jointed cemented mudstone. Core drilling into the mudstone showed partly open joints and RQD values increased from an average of  $15\%$  at rockhead to  $35\%$  at a depth of  $10\text{m}$ . Tests on rock cores gave an average unconfined compression strength of  $4.5\text{MN/m}^2$ . Determine the required depth of the pile below rockhead and calculate the settlement of the pile at the working load.

$$\text{The stress on the shaft of a 1.5m pile is: } \frac{9}{(\pi/4) \times 1.5^2} = 5.1\text{MN/m}^2$$

This is satisfactory for concrete with a cube crushing strength of  $25\text{MN/m}^2$  (allowable stress =  $6\text{MN/m}^2$ ). Load carried in skin friction in the loose sand will be negligible.

From Figure 4.38 for  $q_{uc} = 4.5\text{MN/m}^2$ ,  $\alpha = 0.2$ . The mass factor,  $j$ , for RQD from  $15\%$  to  $35\%$  is  $0.2$ . Therefore,  $\beta$ , from Figure 4.39 is  $0.65$ .

$$= 0.2 \times 0.65 \times 4\,500$$

From equation 4.25, rock socket skin friction  $= 585\text{kN/m}^2$

Taking a 7m socket length, total ultimate skin friction  $= 585 \times \pi \times 1.5 \times 7 = 19\,297\text{kN}$

$$= \frac{19\,297}{9\,000} = 2.1$$

Therefore, factor of safety on shaft friction

Because of the open joints in the rock it will be advisable to assume that the base resistance does not exceed the unconfined compression strength of the rock.

$$\text{Total base resistance} = \frac{\pi}{4} \times 1.5^2 \times 4500 = 7952\text{kN}$$

$$\text{Total pile resistance} = 19297 + 7952 = 27249\text{kN}$$

$$\text{Factor of safety} = \frac{27249}{9000} = 3.0 \text{ which is satisfactory}$$

If the rock socket skin friction were to be only half the calculated value, no load would be transferred to the pile base. Therefore, the pile head settlement will be caused by compression in the rock socket only.

From [Section 5.5](#) the modulus ratio of a cemented mudstone is 150, and for a mass factor of 0.2 the deformation modulus of the rock mass is  $0.2 \times 150 \times 4.5 = 135\text{MN/m}^2$ . In [Figure 4.41](#) the modulus ratio  $E_c/E_d$  is  $20 \times 10^3/135 = 148$  and for  $L/B = 7/1.5 = 4.7$  the influence factor  $I$  is 0.24. The ratio  $D/B$  for a recessed socket is  $4/1.5 = 2.7$ . Therefore the reduction factor from [Figure 4.42](#) is about 0.8. Hence from equation 4.26:

$$\text{Pile head settlement} = \frac{0.8 \times 9 \times 10^3 \times 0.25}{1.5 \times 135} = 9\text{mm}$$

As noted above the whole of the working load is carried by shaft friction. Eurocode 7, [Chapter 7](#), gives the pile design bearing capacity as  $19297/1.5 \times 1.3 = 9896\text{kN}$ . Adoption of this working load would increase the settlement from 9 to only 10mm.

### Example 4.8

A tubular steel pile with an outside diameter of 1067mm is driven with a closed end to near refusal in a moderately strong sandstone (average)  $q_{uc} = 20\text{MN/m}^2$  overlain by 15m of soft clay. Core drilling in the rock showed a fracture frequency of 5 joints per metre. Calculate the maximum working load which can be applied to the pile and the settlement at this load. Only a small penetration below rockhead will be possible with sandstone of this quality, and the rock will be shattered by the impact. Hence skin friction support both in the soft clay and the rock will be negligible compared with the base resistance. Pile driving impact is likely to open joints in the rock hence the base resistance should not exceed the unconfined compression strength of the intact rock.

$$\begin{aligned} \text{Total ultimate pile resistance} &= \text{base resistance} \\ &= \frac{\pi}{4} \times 1.067^2 \times 20 = 17.9\text{MN} \end{aligned}$$

$$\text{For a safety factor of 2.5 allowable load} = \frac{17.9}{2.5} = 7.2\text{MN}$$

For a wall thickness of 19mm in mild steel ( $f_y = 240\text{MN/m}^2$ )

$$\text{Allowable load} = 0.5 \times 240 \times 0.0626 = 7.5\text{MN}$$

Therefore take a maximum working load of 7.2MN

Pile driving impact may increase the fracture frequency from 5 to 10, say, fractures per metre giving a mass factor of 0.2.

From [section 5.4](#) the modulus ratio of sandstone is 300.

$$\text{Deformation modulus} = 0.2 \times 300 \times 20 = 1200\text{MN/m}^2$$

From equation 4.19:

$$\begin{aligned} &= \frac{7.2 \times 15 \times 1000}{0.0626 \times 2 \times 10^5} + \frac{0.5 \times 7.2 \times 1000}{1.067 \times 1200} \\ &= 8.6 + 2.8 \end{aligned}$$

$$\text{Settlement of pile head} = 11.4\text{mm (say 10 to 15mm)}$$

### Example 4.9

A 5m layer of hydraulic fill consisting of sand is pumped into place over the ground shown in [Figure 4.49](#). The calculated time-settlement curve for the surface of the hydraulic fill is shown in [Figure 4.50](#). Two years after the completion of filling a closed-end steel cased pile with an outside diameter of 517mm is driven to a penetration of 27m to carry a working load of 900kN (as Example 4.5). Calculate the negative friction which is developed on the pile shaft and assess whether or not any deeper penetration is required to carry the combined working load and negative skin friction.

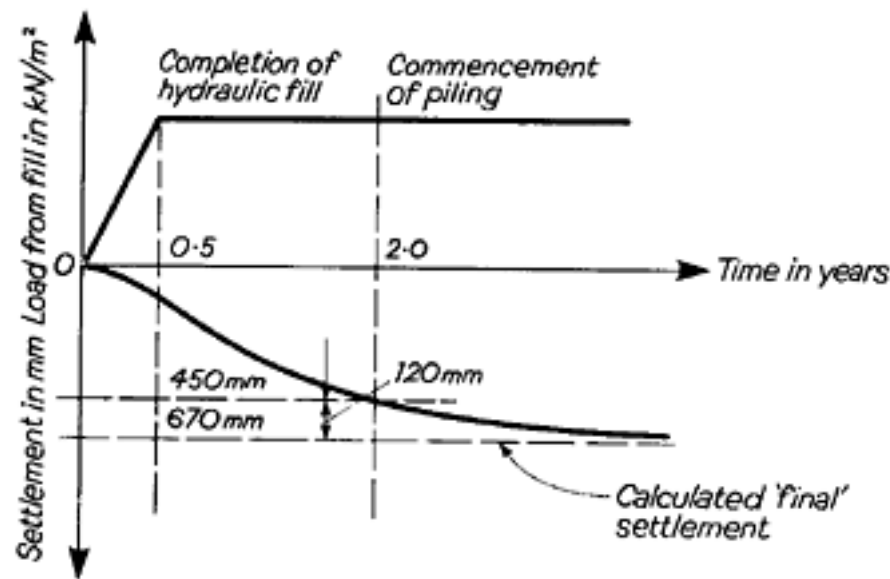


Fig. 4.50

It can be seen from the time-settlement curve that about 120mm of settlement will take place from the time of driving the pile until the clay beneath the fill layer is fully consolidated. This movement is considerably larger than the compression of the pile head under the working load (about 10mm of settlement would be expected under the working load of 900kN).

Therefore negative skin friction will be developed over the whole depth of the pile within the hydraulic fill. Considering now the negative skin friction within the soft clay, if it is assumed that drag-down will not occur if the clay settles relatively to the pile by less than 5mm, then adding the settlement of the pile toe (10mm at the working load) negative skin friction will not be developed below the point where the clay settles by less than 15mm relative to site datum. After pile driving, the full thickness of the clay settles by 120mm at the surface of the layer. By simple proportion, a settlement of 5mm occurs at a point  $12 \times 15 / 120 = 1.5\text{m}$  above the base of the layer. This assumes uniform compressibility in the clay, but there is decreasing compressibility with increasing depth such that the settlement decreases to less than 15mm at a point not less than 2m above the base of the layer. A closer estimate could be obtained by calculating the settlement in each of a number of horizontal layers and drawing a profile of the movement. However, the above approximate assessment will be adequate for the present case.

Adopting Meyerhof's values from [Figure 4.44](#) for the negative skin friction Unit negative skin friction 2m above the base of clay layer

$$= 0.3\sigma'_{vo} = 0.3 \times 9.81 [(5 \times 2) + (2 \times 1.9) + (8 \times 0.9)] \\ = 62\text{kN/m}^2$$

Unit negative skin friction at top of clay stratum

$$= 0.3 \times 9.81 \times 5 \times 2 = 29\text{kN/m}^2$$

Unit negative skin friction 2m below top of clay stratum (at groundwater level)

$$= 0.3 \times 9.81 [(5 \times 2) + (2 \times 1.9)] = 41\text{kN/m}^2$$

Total negative skin friction in clay

$$= \pi \times 0.517 [\frac{1}{2}(29 + 41) \times 2 + \frac{1}{2}(41 + 62) \times 8] = 783\text{kN}$$

Drainage of the fill will produce a medium-dense state of compaction for which  $K_o$  is 0.45 and  $K_s$  in equation 4.8 is

$1.5 \times 0.45 = 0.67$  ([Table 4.1](#)) and  $\delta = 0.7 \times 30^\circ = 21^\circ$ . Therefore

Negative skin friction

$$\begin{aligned}
 &= \frac{1}{2} \times 0.67 \times 9.81 \times 2 \times 5 \times \tan 21^\circ \times \pi \times 0.517 \times 5 \\
 &= 102 \text{ kN}
 \end{aligned}$$

Total negative skin friction on pile =  $102 + 783 = 885 \text{ kN}$

Total load on pile =  $885 + 900 = 1785 \text{ kN}$

From example 4.5, the average cone resistance at a penetration of 27m is about 7MN/m<sup>2</sup> and the skin friction on the shaft is about 1000kN.

Ultimate pile resistance

$$= (\frac{1}{4}\pi \times 0.517^2 \times 7000) + 1000 = 2470\text{kN}$$

Safety factor on combined working load and negative skin friction  $= \frac{2470}{1785} = 1.4$

A safety factor of at least 1.5 should be provided on the combined loading. This could be achieved by driving the pile to 27.5 or 28m.

The working stress on the pile shaft must be checked under the combined loading. For a wall thickness of 4.47mm the steel area is 7193mm<sup>2</sup>.

Permissible load on steel at 30% yield stress

$$= \frac{0.3 \times 255 \times 7193}{1000} = 550\text{kN}$$

If the pile is filled with concrete with a working stress of 25% of the cube strength:

$$= \frac{1}{4}\pi \times 0.508^2 \times 0.25 \times 25 \times 1000$$

Allowable load on concrete = 1267kN

Total allowable load = 1267 + 550 = 1817kN

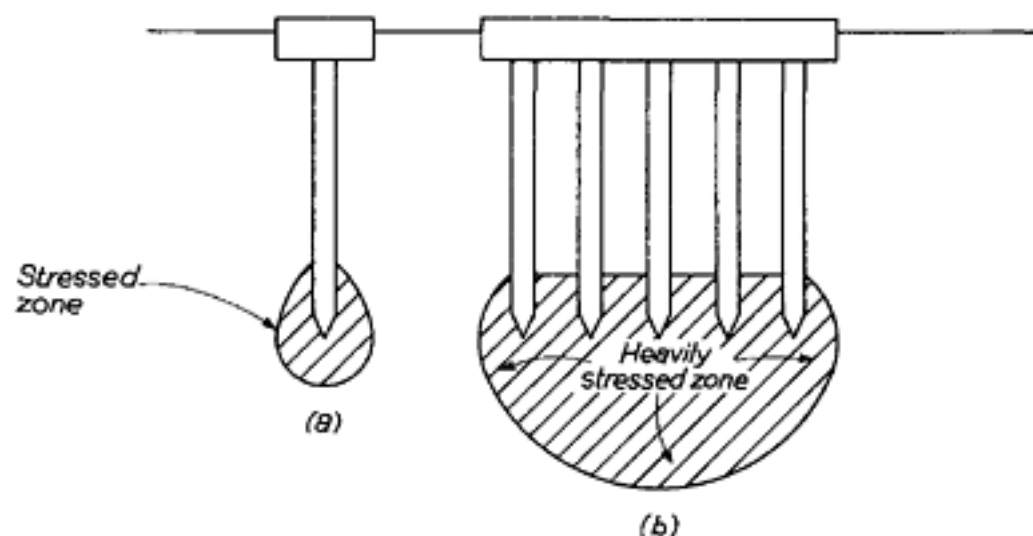
If there is concern about long-term corrosion of the steel section in the hydraulic fill, the cube strength of the concrete filling could be increased so that the whole of the load is carried by the concrete.

## CHAPTER 5

# Pile groups under compressive loading

### 5.1 Group action in piled foundations

The supporting capacity of a group of vertically loaded piles can, in many situations, be considerably less than the sum of the capacities of the individual piles comprising the group. In all cases the elastic and consolidation settlements of the group are greater than those of a single pile carrying the same working load as that on each pile within the group. This is because the zone of soil or rock which is stressed by the entire group extends to a much greater width and depth than the zone beneath the single pile ([Figure 5.1](#)). Even when a pile group is bearing on rock the elastic deformation of the body of rock within the stressed zone can be quite appreciable if the piles are loaded to their maximum safe capacity.



*Fig. 5.1 Comparison of stressed zones beneath single pile and pile group*

*(a) Single pile (b) Pile group*

Group action in piled foundations has resulted in many recorded cases of failure or excessive settlement, even though loading tests made on a single pile have indicated satisfactory performance. A typical case of foundation failure is the single pile driven to a satisfactory set in a compact or stiff soil layer underlain by soft compressible clay. The latter formation is not stressed to any significant extent when the single pile is loaded ([Figure 5.2a](#)) but when the load from the superstructure is applied to the whole group, the stressed zone extends down into the soft clay. Excessive settlement or complete general shear failure of the group can then occur ([Figure 5.2b](#)). A classic example of this action is the foundation failure of the Charity Hospital at New Orleans described by Terzaghi and Peck ([5.1](#)). The building was constructed on about 20000 timber piles 7.9m long driven through a soft compressible soil and terminating in a 1.8m layer of dense sand underlain by 50m or more of soft compressible clay. A test pile was loaded to about 300kN and the settlement under the test load was only 6.5mm. A working load of 150kN was adopted, but during and after erection of the building the group settled. A movement of more than 300mm was observed over a period of 2 years after construction, and severe damage was caused to the steel frame and stone cladding.

The allowable loading on pile groups is sometimes determined by the so-called 'efficiency formulae', in which the efficiency of the group is defined as the ratio of the average load per pile when failure of the complete group occurs, to the load at failure of a single comparable pile. The various efficiency ratios are based simply on experience without any relationship to soil mechanics principles. For this reason the author does not consider this to be a desirable or logical approach to the problem and prefers

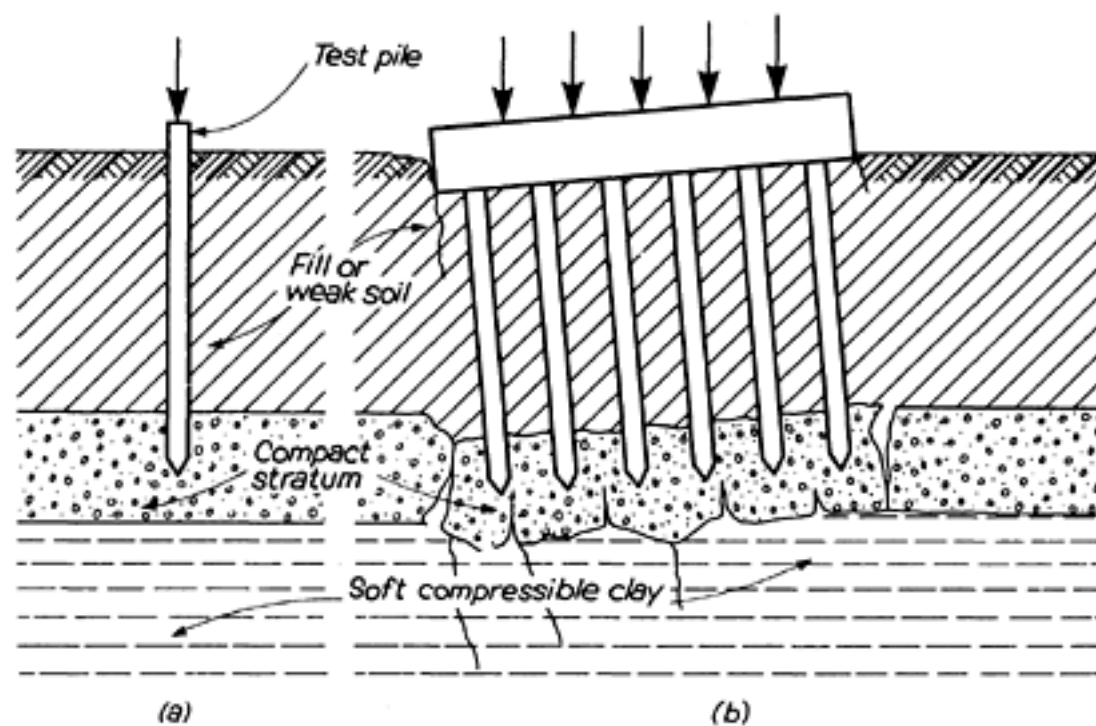


Fig. 5.2 Shear failure of pile group

(a) Test load on single isolated pile when soft clay is not stressed significantly (b) Load applied to group of piles when soft clay is not stressed significantly is stressed heavily

to base design methods on the assumption that the pile group behaves as a block foundation with a degree of flexibility which depends on the rigidity of the capping system and the superimposed structure. By treating the foundation in this manner, normal soil mechanics practice can be followed in the calculations to determine the ultimate bearing capacity and settlement. Load transfer in skin friction from the pile shaft to the surrounding soil is allowed for by assuming that the load is spread from the shafts of friction piles at an angle of 1 in 4 from the vertical. Three cases of load transfer are shown in Figure 5.3a to 5.3c.

In German practice (DIN 1054) the periphery of an equivalent block foundation is taken as that bounded by a line passing at three times the pile shaft diameter outside the axes of the peripheral piles. The depth of the block is defined as 'the depth which is significant for absorbing the compressive forces'.

An important point to note in the application of soil mechanics methods to the design of pile groups is that, whereas in the case of the single pile the installation method has a very significant effect on the selection of design parameters for skin friction and end bearing, the installation procedure is of lesser importance when considering group behaviour. This is because the zone of disturbance of the soil occurs only within a radius of a few pile diameters around and beneath the individual pile, whereas the soil is significantly stressed to a depth to or greater than the width of the group (Figure 5.1). The greater part of this zone is well below the ground which has been disturbed by the pile installation.

Computer programs have been established to model pile/soil interaction behaviour from which the settlement of pile groups can be determined. Poulos<sup>(4.43)</sup> has listed four programs based on continuum analysis methods.

These are:

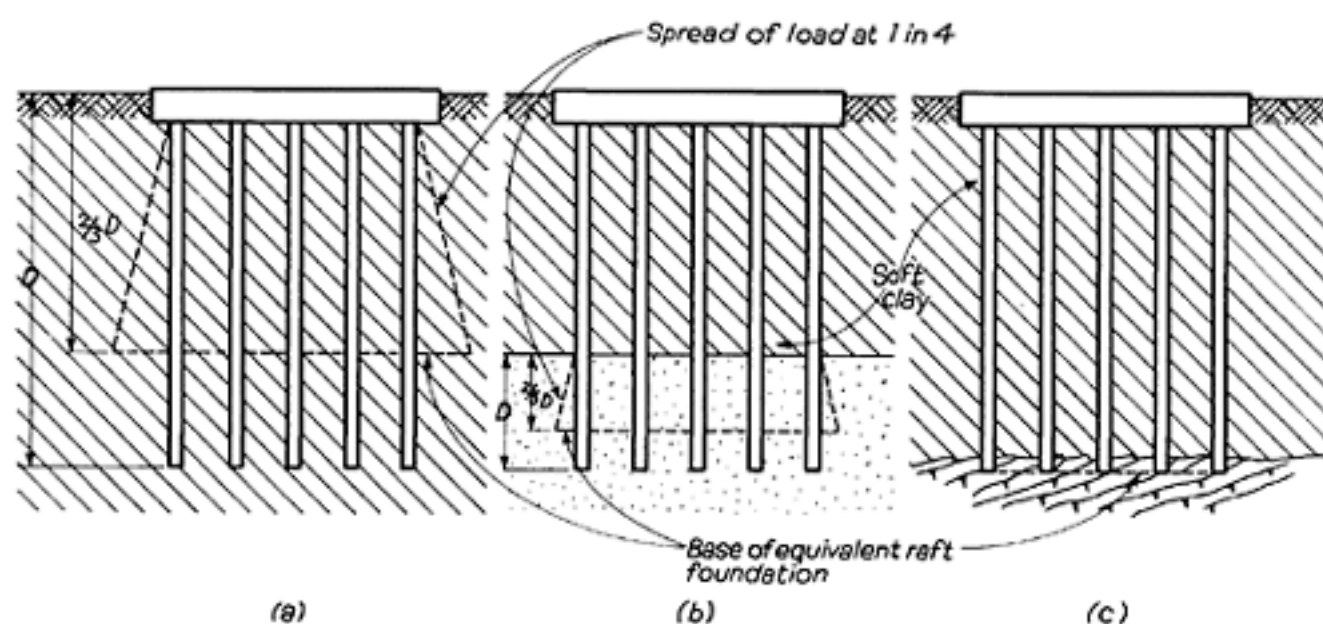


Fig. 5.3 Load transfer to soil from pile group

(a) Group of piles supported predominantly by skin friction  
(b) Group of piles driven through soft clay to combined skin friction and end bearing in stratum of dense

*granular soil*

*(c) Group of piles supported in end bearing on of dense granular soil hard rock stratum*

DEFPIG	Non-linear continuum analysis using interaction factors
GAPFIX	Non-linear continuum analysis, complete solution
M-PILE	Simplified continuum analysis using interaction factors
PGROUP	Complete linear continuum analysis

In the above programs soil behaviour is modelled on the basis of the theory of elasticity. Poulos<sup>(4.43)</sup> states ‘Despite the gross simplification which this model involves when applied to real soil, it provides a useful basis for the prediction of pile behaviour provided that appropriate elastic parameters are selected for the soil. A significant advantage of using an elastic model for soil is that it provides a rational means of analysis of pile groups and evaluation of immediate and final movement of a pile. In determining immediate movements, the undrained elastic parameters of the soil are used in the theory, whereas for final movements the drained parameters are used.’ A useful comparison of the M-PILE and PGROUP programs is given in the UK Department of Transport Publication BD 25/88.

In view of the above reservations and the difficulties of obtaining representative values of the undrained and drained deformation parameters (particularly the latter) from field or laboratory testing of soils and rock, the author believes that the equivalent raft method is sufficiently reliable for most day-to-day settlement predictions. Nevertheless it could be convenient and time-saving to use an available computer program particularly when making studies to determine the effect of varying parameters such as pile diameter, length and spacing.

In most practical problems piles are taken down to a stratum of relatively low compressibility and the resulting total and differential settlements are quite small such that an error of plus or minus 50% due to deficiencies in theory or unrepresentative deformation parameters need not necessarily be detrimental to the structure carried by the pile group (see also [Section 11.1.4](#)).

An example of the small differences between pile group settlements calculated by the equivalent raft method and those determined from charts based on the computer program listed above is given in [Section 5.4](#).

The principal problems concerned with pile groups are constructional effects such as ground heave, the interference of closely-spaced piles which have deviated from line during driving, and the possibilities of damage to adjacent structures and services. It is, of course, necessary to calculate the total and differential settlements of pile groups and overall piled areas to ensure that these are within limits acceptable to the design of the superstructure. The criteria of relative deflections, angular distortion, and horizontal strain which can be tolerated by structures of various types have been reviewed by Burland and Wroth<sup>(5.2)</sup>.

## 5.2 Pile groups in cohesive soils

### 5.2.1 Ultimate bearing capacity

If piles in groups are driven through soft clays, loose sand, or fill, to terminate in a stiff clay, there is no risk of general shear failure of the group provided that there is an adequate safety factor against failure of the single pile. However, the settlement of the group must be calculated as described in the following section of this chapter.

If it is necessary to terminate a group of piles entirely within a soft clay (this is not a desirable practice) then the safety factor against ‘block failure’ of the group must be calculated. The ultimate bearing capacity of the block of soil encompassed by the group shown in [Figure 5.4](#) is calculated from the following equation.

$$Q = 2D(B + L)\bar{c} + 1.3c_b s N_c BL \quad \dots(5.1)$$

where  $D$  is the depth of the piles below ground level,  $B$  is the overall width of the group,  $L$  is the overall length of the group,  $\bar{c}$  is the average cohesion of the clay over the pile embedment depth,  $c_b$  is the cohesion of the clay at the pile base level or

within the zone of soil below the base affected by the loading,  $s$  is a shape factor, and  $N_c$  is the bearing capacity factor.

If the pile group is required to carry the full working load within a few days or weeks of the piles being installed the remoulded shearing strength should be taken for  $\bar{c}$ . In most cases the undisturbed cohesion can be used for  $c_b$ , since the

greater part of the zone in which general shear failure would take place remains in an undisturbed state. The values of  $N_c$

depend on the depth to width ratio of the group ([Figure 5.5](#)). The shape factor depends on the length to width ratio, and suitable values are shown in [Figure 5.6](#).

Where piles are installed in relatively small numbers there is a possibility of excessive base settlement if two or more piles

deviate from line and come into near or close contact at the toe. Thus in [Figure 5.7](#) the toe loads are concentrated over a small area and, while failure would not occur if the safety

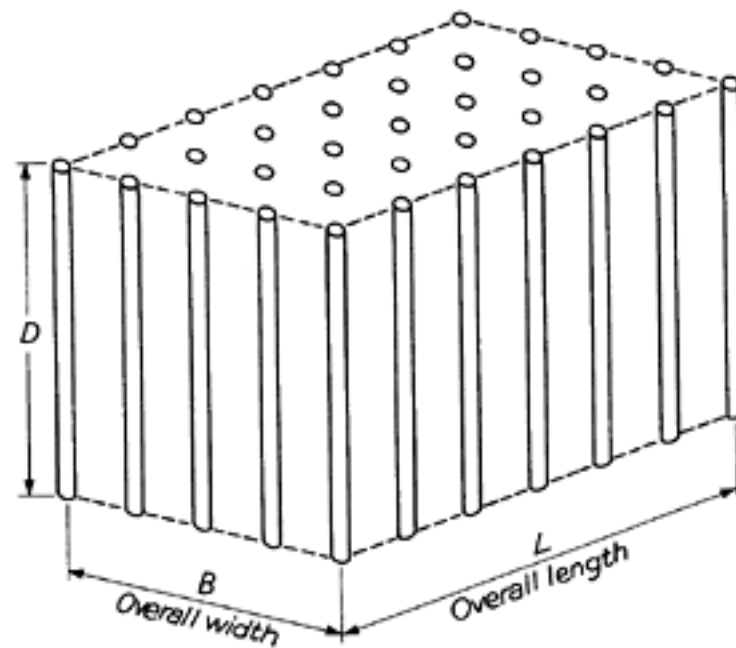


Fig. 5.4 Pile group acting as block foundation

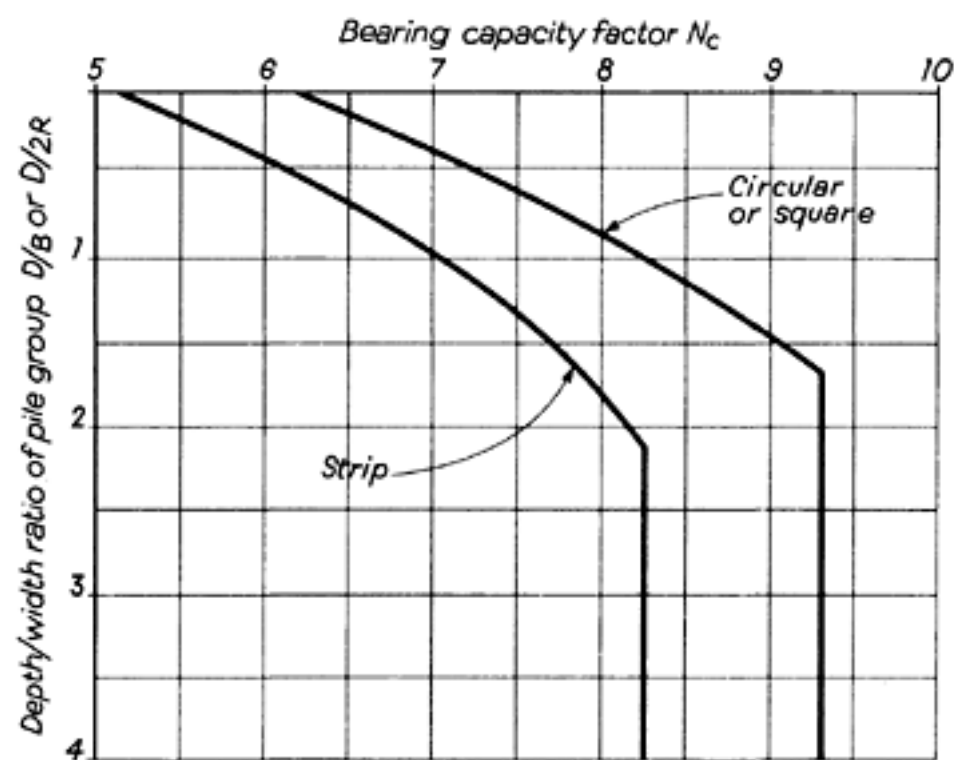


Fig. 5.5 Bearing capacity factor  $N_c$  (after Meyerhof<sup>(4.1)</sup>)

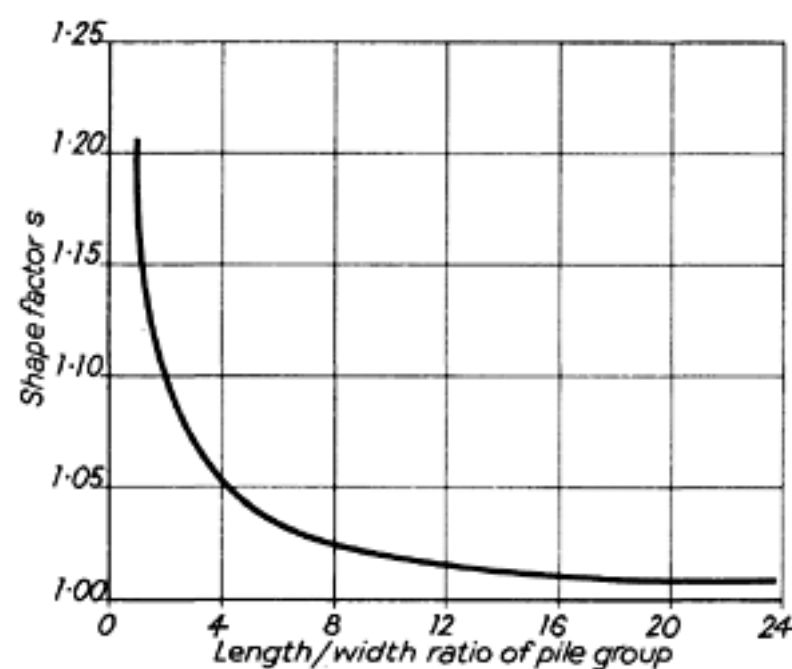


Fig. 5.6 Shape factor for rectangular pile groups (Meyerhof-Skempton)

factor in end bearing was adequate, the settlement would be higher than that which would occur when the piles were at their design spacing. This would lead to differential settlement between the piles in the group. A safeguard against this occurrence is the adoption of a centre-to-centre spacing of piles in clay of at least three pile diameters, with a minimum of 1m. BS 8004 recommends a centre-to-centre

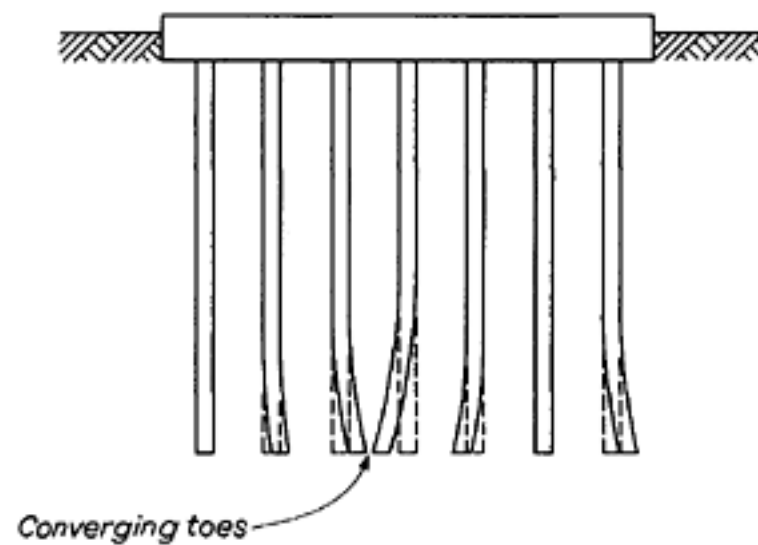


Fig. 5.7 Effect of deviation of piles from correct alignment in group

spacing for friction piles of not less than the perimeter of the pile or for circular piles three times the diameter. Closer spacing can be adopted for piles carrying their load mainly in end bearing but the space between adjacent piles must not be less than their least width. Special consideration must be given to the spacing of piles with enlarged bases, including a study of interaction of stresses and the effect of construction tolerances. The requirements of the German codes DIN 4026 for driven piles and DIN 4014 for bored piles are shown in Figures 5.8 and 5.9 respectively.

The Swedish piling code gives the following minimum centre-to-centre spacing.

Predicted pile length	End-bearing and friction piles		'Cohesion' piles	
	Circular	Square	Circular	Square
Less than 10m	3×diameter	3.4×width	4×diameter	4.5×width
10 to 25m	4×diameter	4.5×width	5×diameter	4.5×width
More than 25m	5×diameter	5.6×width	6×diameter	6.8×width

In all cases the centre-to-centre spacing should not be less than 0.8m.

### 5.2.2 Settlement

The settlement of a pile group in clay is the sum of the *immediate* or elastic settlement  $\rho_i$  and the long-term *consolidation* settlement  $\rho_c$ . The general equation for calculating  $\rho_i$ , for a flexible foundation at the ground surface level is

$$\rho_i = q_n \times 2B \times \left( \frac{1 - \nu^2}{E_u} \right) \times I_p \quad \dots(5.2)$$

where  $\rho_i$  is the settlement at the *centre* of the flexible loaded area,  $q_n$  is the net foundation pressure,  $B$  is the width of an equivalent foundation flexible raft,  $\nu$  is the undrained Poisson's ratio for clay (generally taken as equal to 0.5),  $I_p$  is an influence factor, and  $E_u$  is the deformation modulus for the undrained loading conditions.

The values of  $I_p$  depend on the ratio  $H/B$  of the depth of the compressible soil layer to the pile group width and on the length to width ratio  $L/B$  of the group. Curves (Figure 5.10) established by Steinbrenner were published by Terzaghi(5.3) which enable the values of  $I_p$  and hence the immediate settlement of a surface foundation, to be obtained. It is then necessary to use

Fox's correction curves(5.4) to obtain the immediate settlement of a raft foundation equivalent to the pile group. Generally, where the deformation modulus is reasonably constant with depth it will be found more convenient to use the influence factors of Christian and Carrier(5.5) to obtain the *average* immediate settlement of a foundation at depth  $D$  below the surface where

$$\text{average settlement } \rho_i = \frac{\mu_i \mu_o q_n B}{E_u} \quad \dots(5.3)$$

In the above equation Poisson's ratio is assumed to be equal to 0.5. The factors  $\mu_i$  and  $\mu_o$ , which are related to the depth of the equivalent raft, the thickness of the compressible soil layer and the length/width ratio of the equivalent raft foundation, are shown in [Figure 5.11](#).

The value of the deformation modulus  $E_u$  is obtained from the stress-strain curve resulting from compressive loading on the soil under undrained conditions. Such a curve typically shows purely elastic behaviour only over low stress levels (line AB in [Figure 5.12](#)) and the adoption of a modulus of elasticity

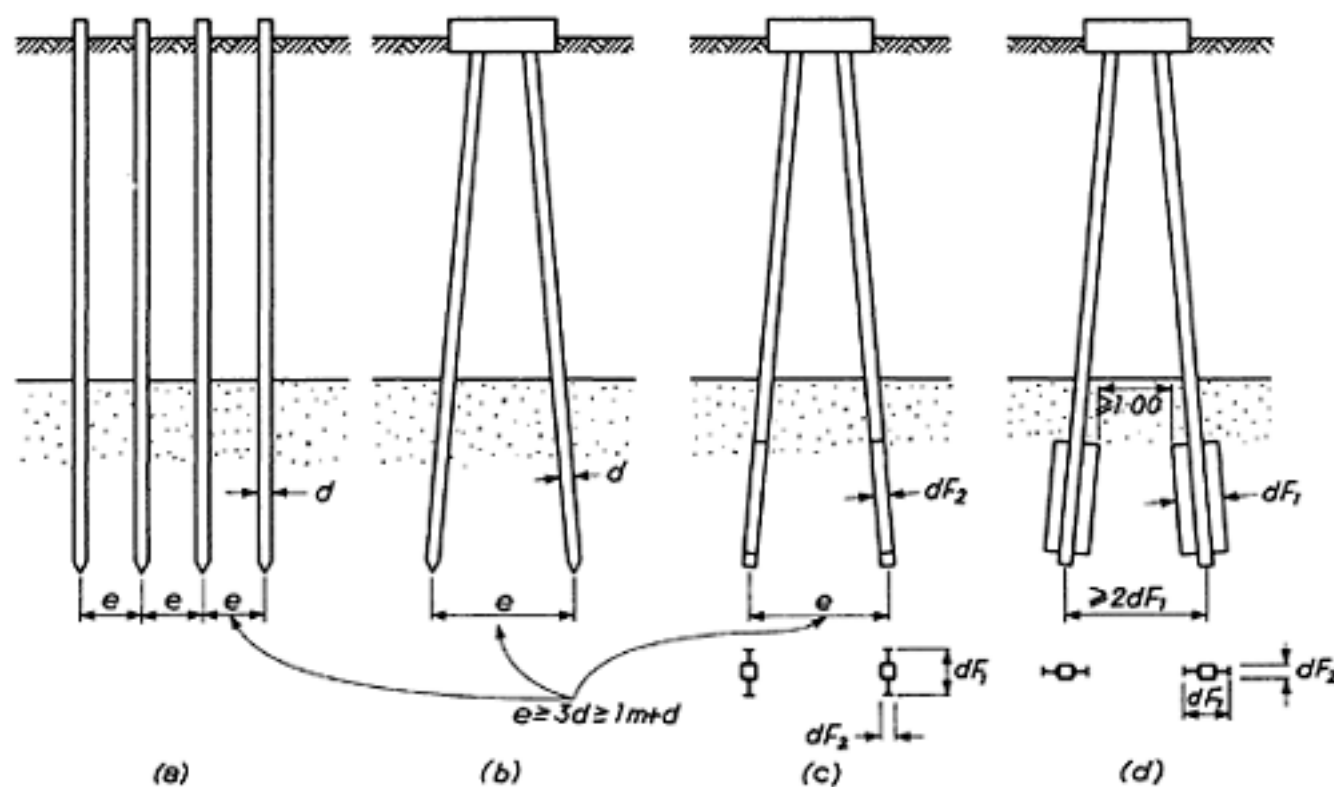


Fig. 5.8 Requirements of German Code DIN 4026 for spacing of driven piles in groups

- (a) Vertical piles
- (b) Raking piles
- (c) and
- (d) Raking winged piles

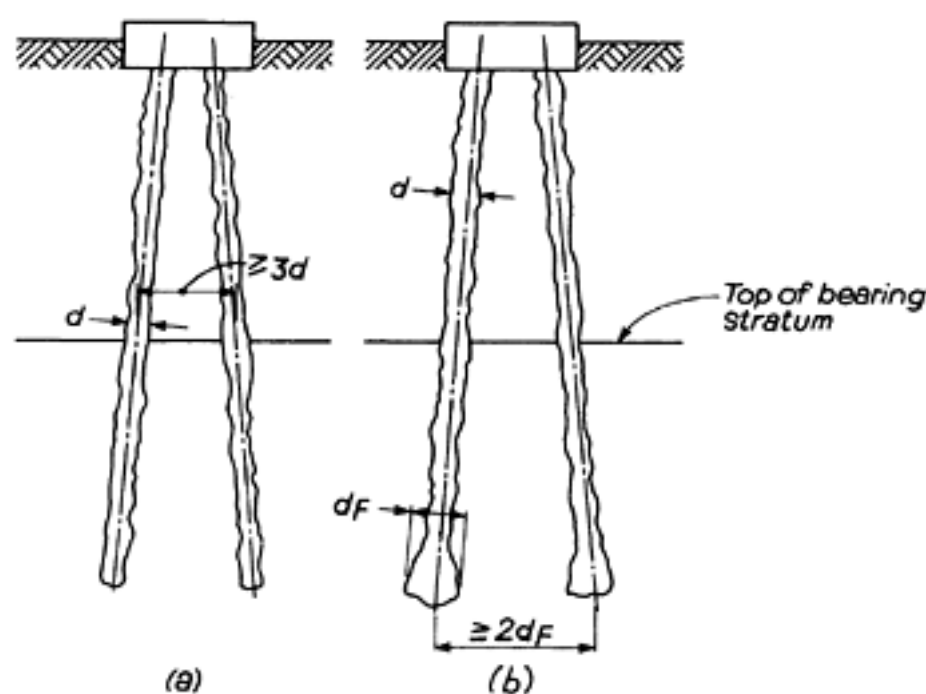


Fig. 5.9 Requirements of German Code DIN 4014 for spacing of bored-and-cast-in-situ piles in groups

- (a) Straight-sided piles
- (b) Piles with enlarged bases

(Young's modulus) corresponding to the straight-line portion might result in the immediate settlement being underestimated. The usual procedure is to draw a secant AC to the stress-strain curve corresponding to a compressive stress that is equal to the net foundation pressure at the base of the equivalent block foundation. More conservatively the secant AD can be drawn at a compressive stress of 1.5 or some other suitable multiple of the foundation pressure. The deformation modulus  $E_u$  is then

obtained as shown in [Figure 5.12](#).

Due to sample disturbance, unrealistically-low modulus values are obtained from stress-strain curves produced from conventional unconfined or triaxial compression tests in the laboratory. It is more reliable to obtain the modulus values from plate bearing tests made in boreholes or trial pits or from field testing using the pressuremeter or Camkometer ([Chapter 11](#)). Alternatively,  $E_u$  can be related to the undrained shearing strength  $c_u$  of the clay. In a general review of this relationship for

London clay, Butler<sup>(5.6)</sup> stated that the assumption of  $E_u = 400c_u$  is a reasonable compromise between divergent data

representing, on one hand the relationship established from laboratory testing, and on the other the observations of the settlement of full-scale structures. Equipment has been developed recently for obtaining piston-driven tube samples of stiff clays and there have been improvements in techniques for coring weak rocks. Thus samples which have suffered only small disturbance can be provided for

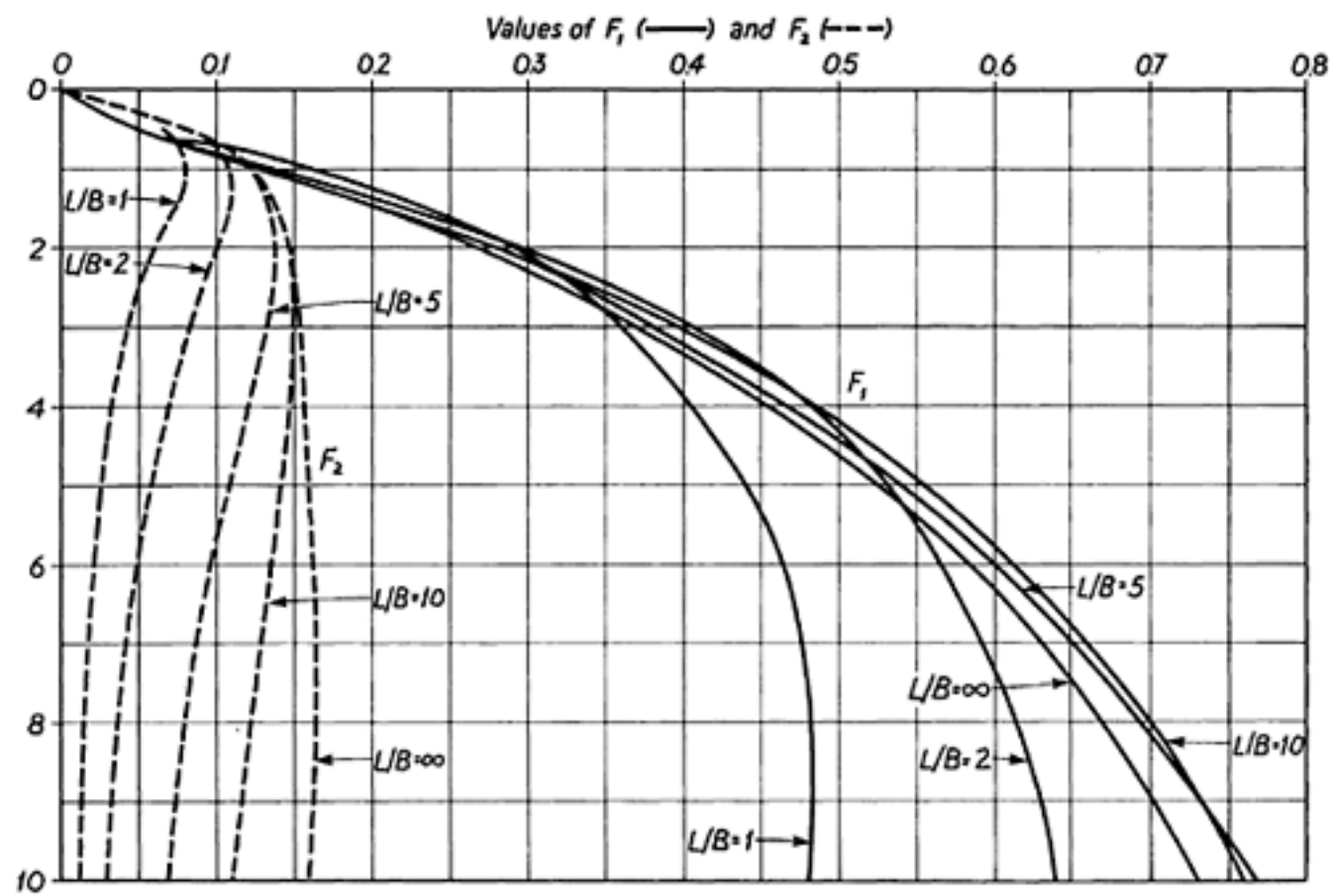


Fig. 5.10 Values of Steinbrenner's influence factor  $I_p$  (for  $\nu$  of 0.5,  $I_p = F_1$ , for  $\nu = 0$ ,  $I_p = F_1 + F_2$ )

Note: When using this diagram to calculate  $p_i$  at the centre of a rectangular area take  $B$  as half the foundation width to obtain  $H/B$  and  $L/B$

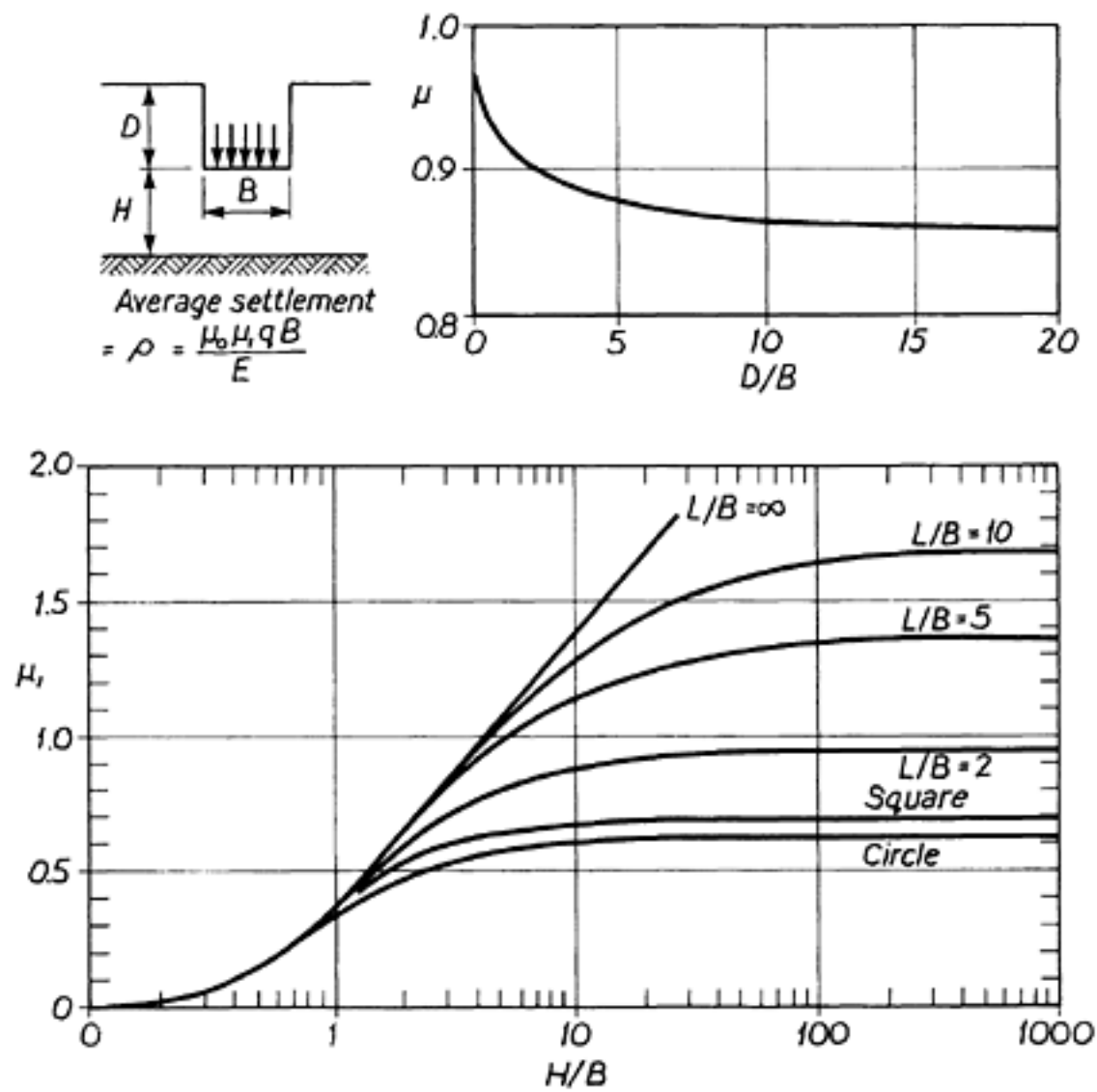


Fig. 5.11 Influence factors for calculating immediate settlements of flexible foundations of width  $B$  at depth  $D$  below ground surface (after Christian and Carrier<sup>(5.5)</sup>).

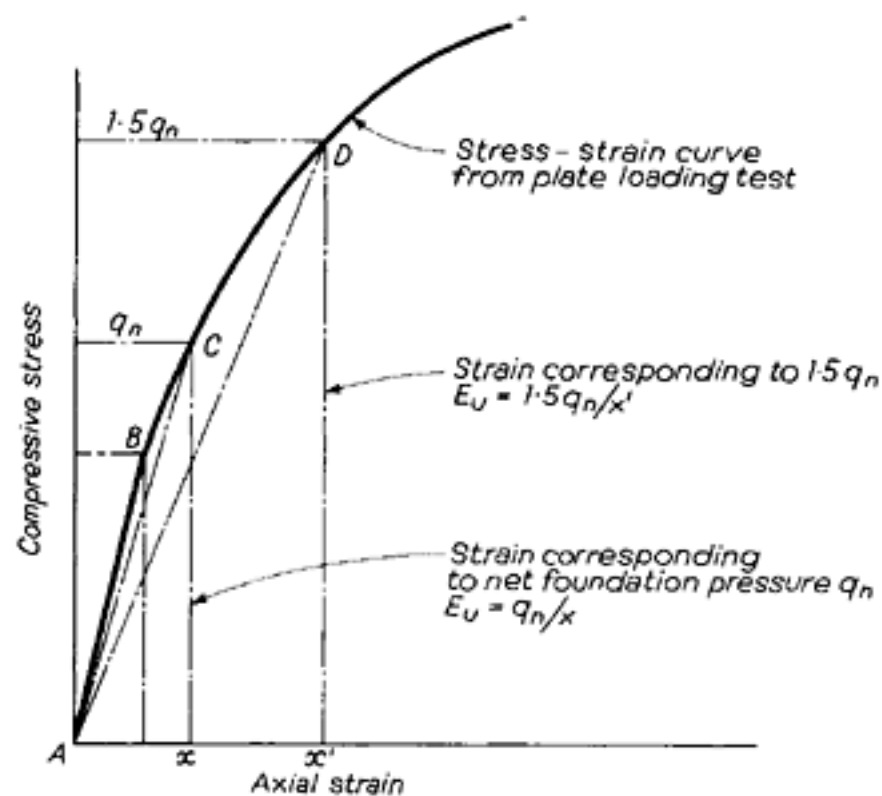


Fig. 5.12 Determining deformation modulus  $E_u$  from stress-strain curve

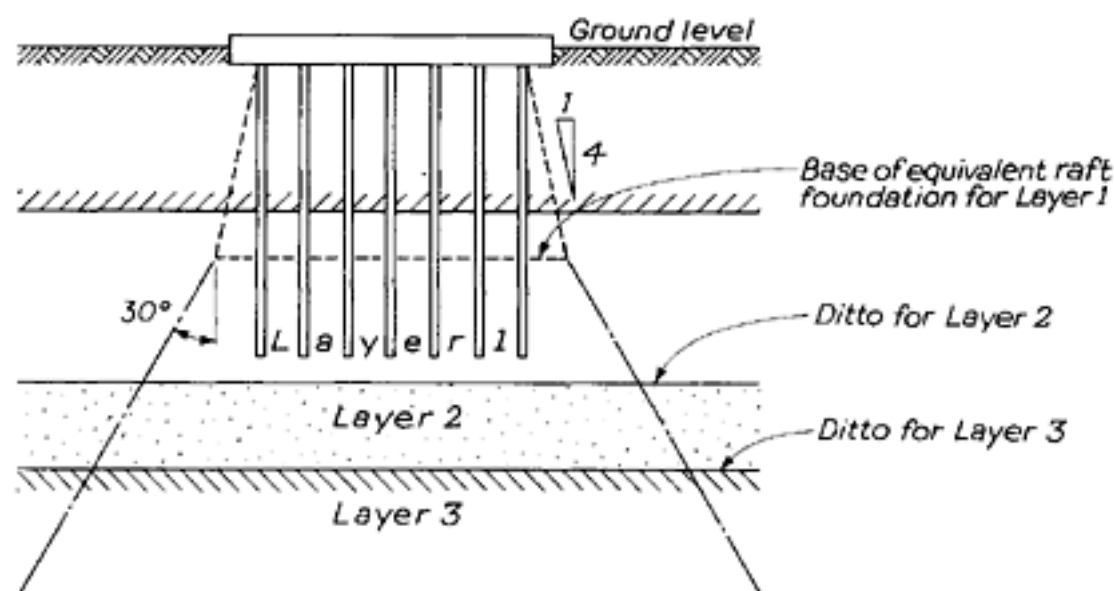


Fig. 5.13 Load distribution beneath pile group in layered soil formation

laboratory testing to obtain modulus values. Equipment is available for measuring very small strains in uniaxial or triaxial compression tests to obtain small strain modulus values.

In layered soils with different values of the deformation modulus  $E_u$  in each layer or in soils which show a progressively

increasing modulus with increases in depth, the strata below the base of the equivalent raft are divided into a number of representative horizontal layers and an average value of  $E_u$  is assigned to each layer. The dimensions  $L$  and  $B$  in [Figure 5.11](#)

are determined on the assumption that the load is spread to the *surface* of each layer at an angle of  $30^\circ$  from the edges of the equivalent raft ([Figure 5.13](#)). The total settlement of the piled foundation is then the sum of the average settlements calculated for each soil layer from equation 5.3.

The influence values in [Figures 5.10](#) are based on the assumption that the deformation modulus is constant with depth.

However, in most natural soil and rock formations the modulus increases with depth such that calculations based on a constant modulus give exaggerated estimates of settlement. Butler<sup>(5.6)</sup> developed a method for settlement calculations for the conditions of a deformation modulus increasing linearly with depth within a layer of finite thickness. The value of the modulus at a depth  $z$  below foundation level is given by the equation:

$$E_u = E_f(1 + kz/B) \quad \dots(5.4a)$$

and

$$\text{and} \quad \rho_i = \frac{q_n B I'_p}{E_f} \quad \dots(5.4b)$$

where  $E_f$  is the modulus of deformation at foundation level (the base of the equivalent raft) and  $\rho_i$  is the settlement at the *corner* of the loaded area

The value of  $k$  is obtained by plotting measured values of  $E_u$  with depth and drawing a straight line through the plotted points to obtain values for substitution in equation 5.4a. Where a plot of undrained

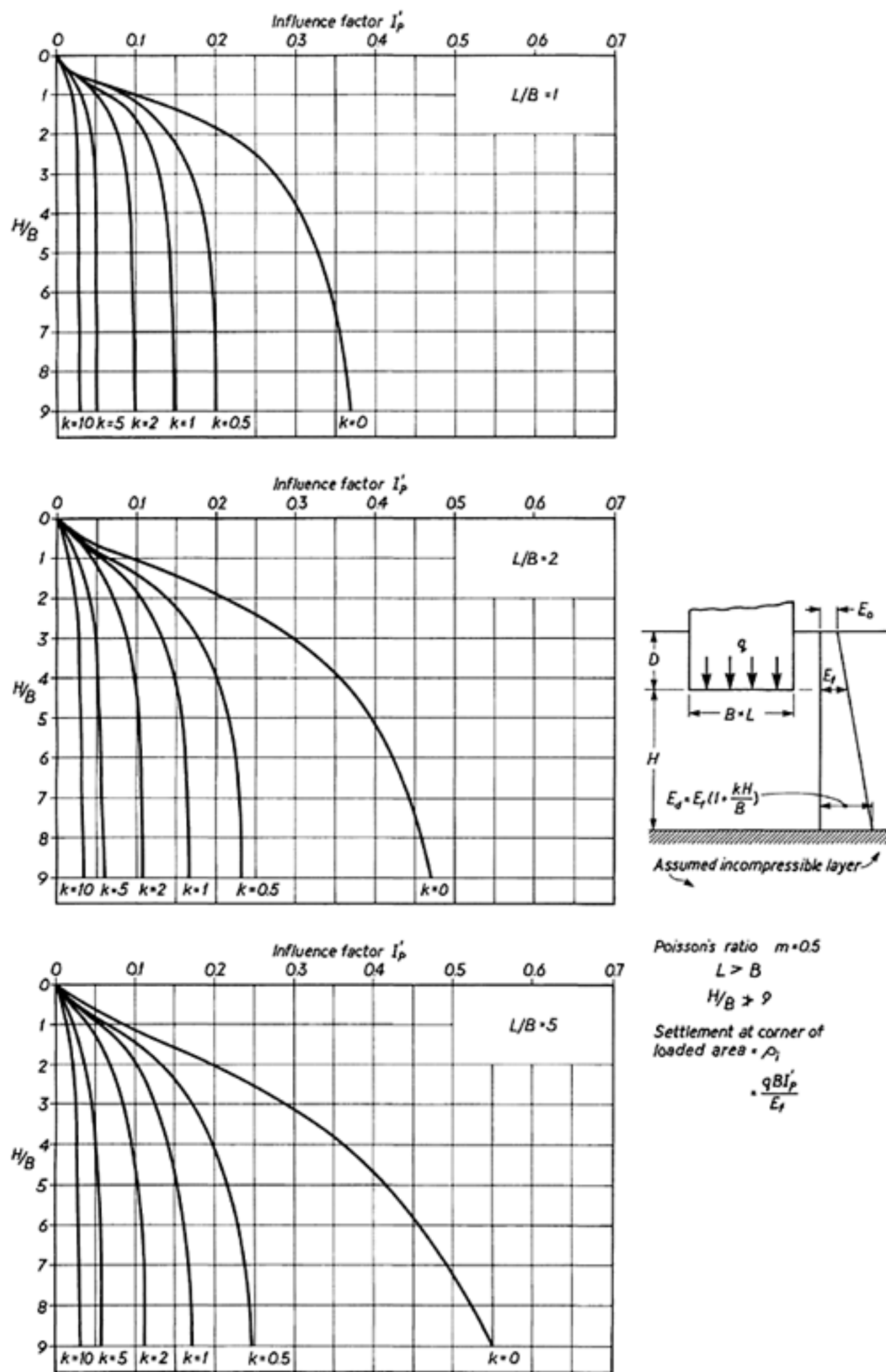


Fig. 5.14 Values of the influence factor  $I'_p$  for deformation modules increasing linearly with depth and modular ratio of 0.5 (after Butler<sup>(5.6)</sup>)

shear strength with depth has been obtained the  $E_u$  vs depth line can be derived from the empirical relationships given above.

Having obtained  $k$ , the appropriate factor for  $I'p$  is obtained from Butler's curves shown in [Figure 5.14](#). These are for different ratios of  $L/B$  at the level of the equivalent raft, and are applicable for a compressible layer thickness not more than  $9 \times B$ . The curves are based on the assumption of a Poisson's ratio of 0.5 for undrained conditions, this is for immediate application of the load.

Where the pile group covers a large area and is thus relatively flexible, it may be necessary to calculate the settlements at the corners as well as the centre of the area. The method of calculation is given in 5.5.

The *consolidation* settlement  $\rho_c$  is calculated from the results of oedometer tests made on clay samples in the laboratory. The curves for the pressure/voids ratio obtained from these tests are used to establish the coefficient of volume compressibility  $m_v$ .

In hard glacial tills or weak highly weathered rock it may be difficult to obtain satisfactory undisturbed samples for oedometer tests. If the results of standard penetration tests are available, values of  $m_v$  (and also  $c_u$ ) can be obtained from

empirical relationships established by Stroud<sup>(5.7)</sup> shown in [Figure 5.15](#).

Having obtained a representative value of  $m_v$  for each soil layer stressed by the pile group, the *oedometer settlement*  $\rho_{oed}$  for this layer at the centre of the loaded area is calculated from the equation

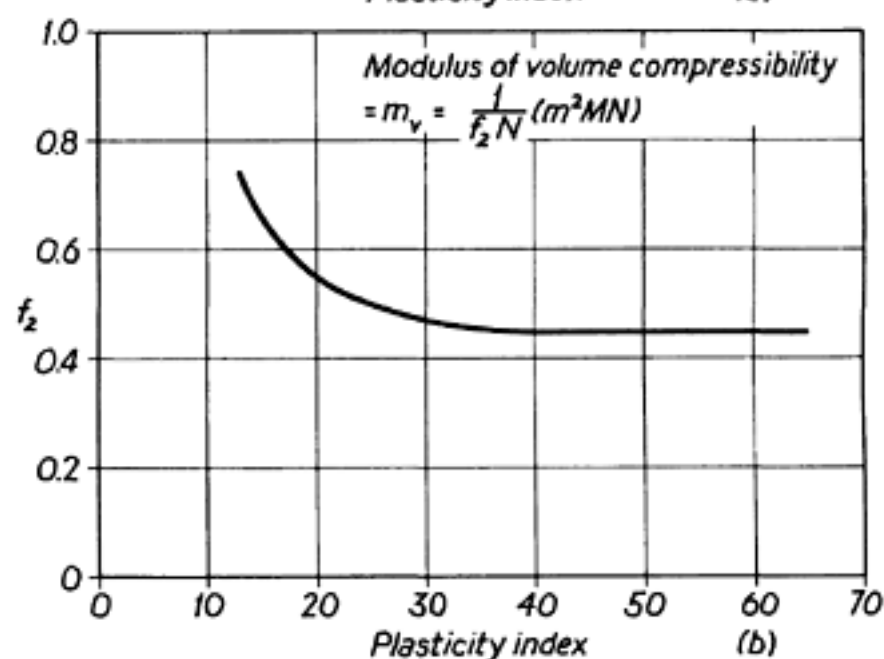
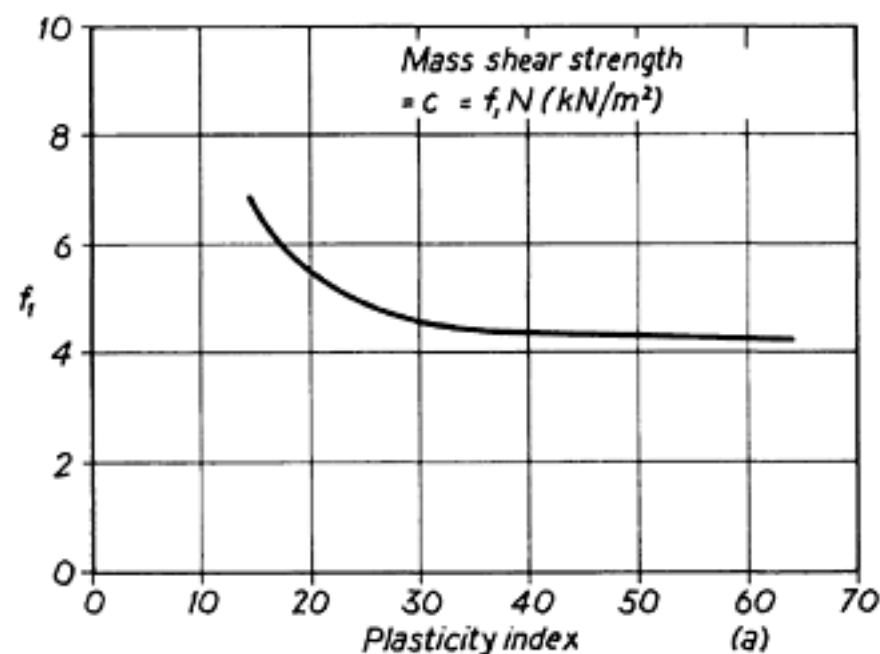
$$\rho_{oed} = \mu_d m_v \times \sigma_z \times H \quad \dots(5.5)$$

where  $\mu_d$  is a depth factor,  $\sigma_z$  is the average effective vertical stress imposed on the soil layer due to the net foundation

pressure  $q_n$  at the base of the equivalent raft foundation and  $H$  is the thickness of the soil layer. The depth factor is obtained

from Fox's correction curves<sup>(5.4)</sup> shown in [Figure 5.16](#). To obtain the average vertical stress  $\sigma_z$  at the centre of each soil layer

the coefficients in [Figure 5.17](#) should be used. The oedometer settlement must now be corrected to obtain the field value of the consolidation settlement. The correction is made by applying a 'geological factor'  $\mu_g$  to the oedometer settlement, where



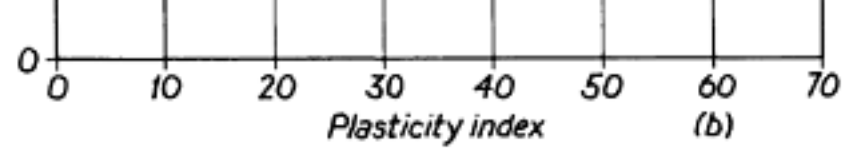


Fig. 5.15 Relationship between mass shear strength, modulus of volume compressibility, plasticity index and standard penetration test *N*-values (after Stroud<sup>(5.7)</sup>) (a) *N*-value vs. undrained shear strength (b) *N*-value vs. modulus of volume compressibility

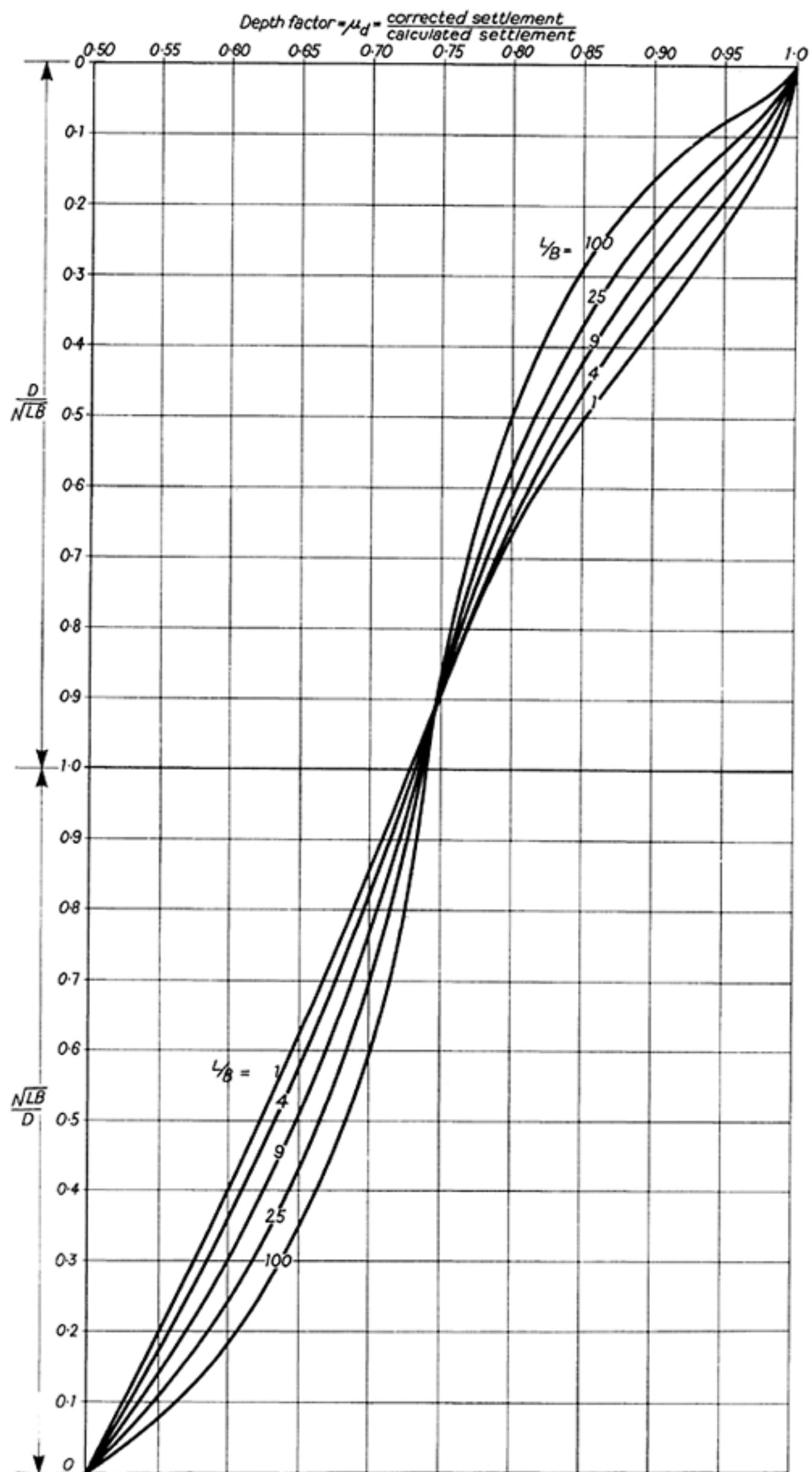


Fig. 5.16 Depth factor  $\mu_d$  for calculating oedometer settlements (after Fox<sup>(5.5)</sup>)

$$\rho_c = \mu_g \times \rho_{oed}$$

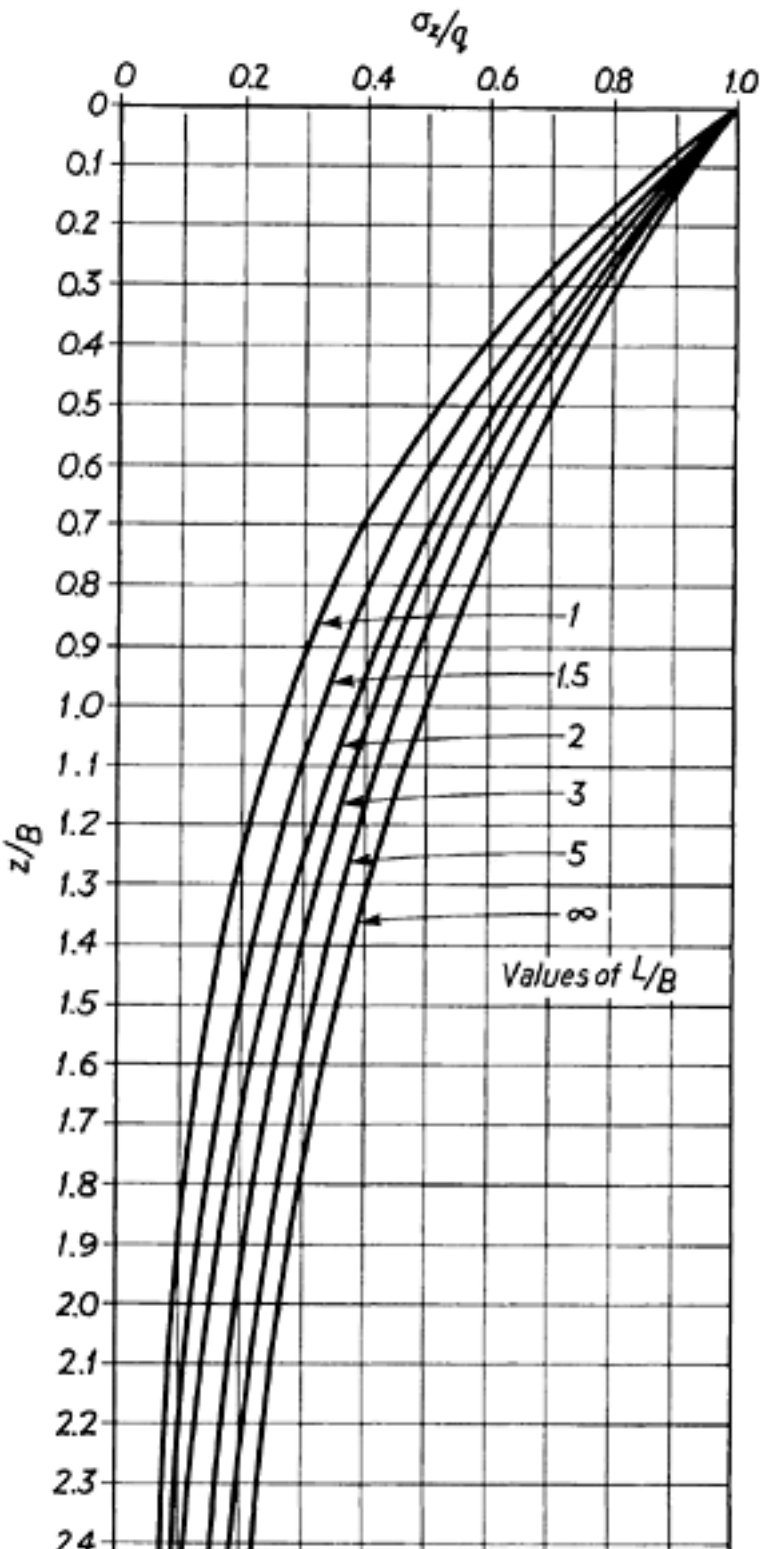
...(5.6)

Published values of  $\mu_g$  have been based on comparisons of the settlement of actual structures with computations made from laboratory oedometer tests. Values established by Skempton and Bjerrum<sup>5.8</sup> are shown in [Table 5.1](#).

Table 5.1 Value of geological factor  $\mu_g$

Type of clay	$\mu_g$ value
Very sensitive clays (soft alluvial, estuarine and marine clays)	1.0–1.2
Normally-consolidated clays	0.7–1.0
Over-consolidated clays (London clay, Weald, Kimmeridge, Oxford and Lias clays)	0.5–0.7
Heavily over-consolidated clays (unweathered glacial till, Keuper Marl)	0.2–0.5

The total settlement of the pile group is then the sum of the immediate and consolidation settlements calculated for each separate layer. A typical case is a gradual decrease in compressibility with depth. In such a case the stressed zone beneath the pile group is divided into a number of separate horizontal layers, the value of  $m_v$  for each layer being obtained by plotting  $m_v$  against the depth as determined from the laboratory oedometer tests. The base of the lowermost layer is taken as the level at which the vertical stress has decreased to one-tenth of  $q_n$ . The depth factor  $\mu_d$  is applied to the sum of the *consolidation* settlements calculated for each layer. It is not applied to the immediate settlement if the latter has been calculated from the factors in [Figure 5.11](#).



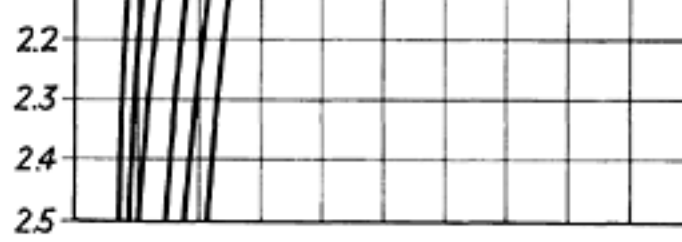


Fig. 5.17 Calculation of mean vertical stress ( $\sigma_z$ ) at depth  $z$  beneath rectangular area  $a \times b$  on surface loaded at uniform pressure  $q$

Where a pile group is surmounted by a deep rigid cap, or if the group supports a rigid superstructure, the pile group can be regarded as being equivalent to a rigid block foundation having a uniform settlement. The value of the latter is obtained by applying a 'rigidity factor' to the consolidation settlement obtained from the equivalent flexible raft foundation. The immediate settlement obtained from equation 5.3 is the average settlement corresponding to that given by a rigid foundation. A commonly accepted value for the rigidity factor is 0.8. Thus

$$\frac{\text{settlement of rigid pile group}}{\text{settlement of flexible pile group}} = 0.8.$$

A pile group consisting of a number of small clusters or of individual piles connected by ground beams or by a flexible ground floor slab can be regarded as being equivalent to a flexible raft foundation at depth for this condition. The consolidation settlements at the corners of the piled area are roughly one-half of the settlement at the centre of the group, as calculated in the manner described above.

Another method of estimating the *total* settlement of a structure on an *over-consolidated* clay is to use equation 5.2, making the substitution of a deformation modulus obtained for loading under drained conditions. This modulus is designated by the term  $E_v'$ , which is substituted for  $E_u$  in the bottom of equation 5.2. It is approximately equal to  $1/m_v$ . The equation implies a

homogeneous and elastic material and thus it is not strictly valid when used to calculate consolidation settlements. However, when applied to over-consolidated clays for which the settlements are relatively small, the method has been found by experience to give reasonably reliable predictions. Success in using the method depends on the collection of sufficient data correlating the observed settlements of structures with the determinations of from plate loading tests and laboratory tests on good undisturbed samples of clay. Butler<sup>(5.6)</sup> in his review of the settlement of structures on over-consolidated clays has related  $E_v'$  to the undrained cohesion  $c_u$  and arrived at the relationship  $E_v' = 130c_u$  for London clay.

Various correlations between the soil modulus and the undrained shear strength of clays for piles with a length to diameter ratio equal to or greater than 15 are shown in [Figure 5.18](#). In commenting on these data Poulos<sup>(4.43)</sup> stated that they should be taken as representing values of the undrained modulus. He commented on the wide spread of the data suggesting that this could be due to differences in the method of measuring  $c_u$  and the soil modulus, differences in the level of loading at which

the modulus was measured, and differences between the type and over-consolidation ratio of the various clays. Where the undrained shear strength increases linearly with depth, equation 5.4 can be used to obtain  $E_v'$  and hence the total settlements from [Figure 5.14](#). From an extensive review of published and unpublished data, Burland and Kalra<sup>(5.9)</sup> established the relationship for London clay: (MN/m<sup>2</sup>) where  $z$  is the depth below ground level.  $E_v' = 7.5 + 3.9z$

Generally, the author prefers the more logical method of considering immediate and consolidation settlements separately. This properly takes into account time effects and the geological history of the site. Provided that a sufficient number of good undisturbed samples have been obtained at the site investigation stage, the prediction of consolidation settlements from oedometer tests made in the laboratory has been found to lead to reasonably accurate results. The adoption of the method based on the total settlement deformation modulus depends on the collection of adequate observational data, first regarding the relationship between the undrained shearing strength and the deformation modulus, and secondly regarding the actual settlement of structures from which the relationships can be checked. Any attempt to obtain a deformation modulus from triaxial compression tests in the laboratory is likely to result in serious error. The modulus is best obtained from the  $E_u/c_u$  and

$E_u/E_v'$  relationships, which must be established from well-conducted plate bearing tests and field observations of settlement. For a general discussion of the settlement of foundations on clays, the reader is referred to a CIRIA report by Padfield and Sharrock<sup>(5.10)</sup>. They state that for stiff over-consolidated clays the immediate settlement  $\rho_i$  is approximately equal to 0.5 to

0.6 times the oedometer settlement,  $\rho_{oed}$  and the consolidation settlement,  $\rho_c$ , is approximately equal to 0.4 to 0.5 times  $\rho_{oed}$ .

For normally consolidated soft clays  $\rho_i$  is approximately equal to 0.1  $\rho_{oed}$  and  $\rho_c$  is approximately equal to  $\rho_{oed}$ .

The steps in making a settlement analysis of a pile group in, or transmitting stress to, a cohesive soil can be summarized as follows.

1. For the required length of pile, and form of pile bearing (*i.e.* friction pile or end-bearing pile), draw the equivalent flexible raft foundation represented by the group (see [Figure 5.3](#)).
2. From the results of field or laboratory tests assign values to  $E_u$  and  $m_v$  for each soil layer significantly stressed by the equivalent raft.
3. Calculate the immediate settlement of  $\rho_i$  of each soil layer using equation 5.3, and assuming a spread of load of 30°

from the vertical to obtain  $q_n$  at the surface of each layer ([Figure 5.13](#)). Alternatively calculate on the assumption of a linearly increasing modulus.

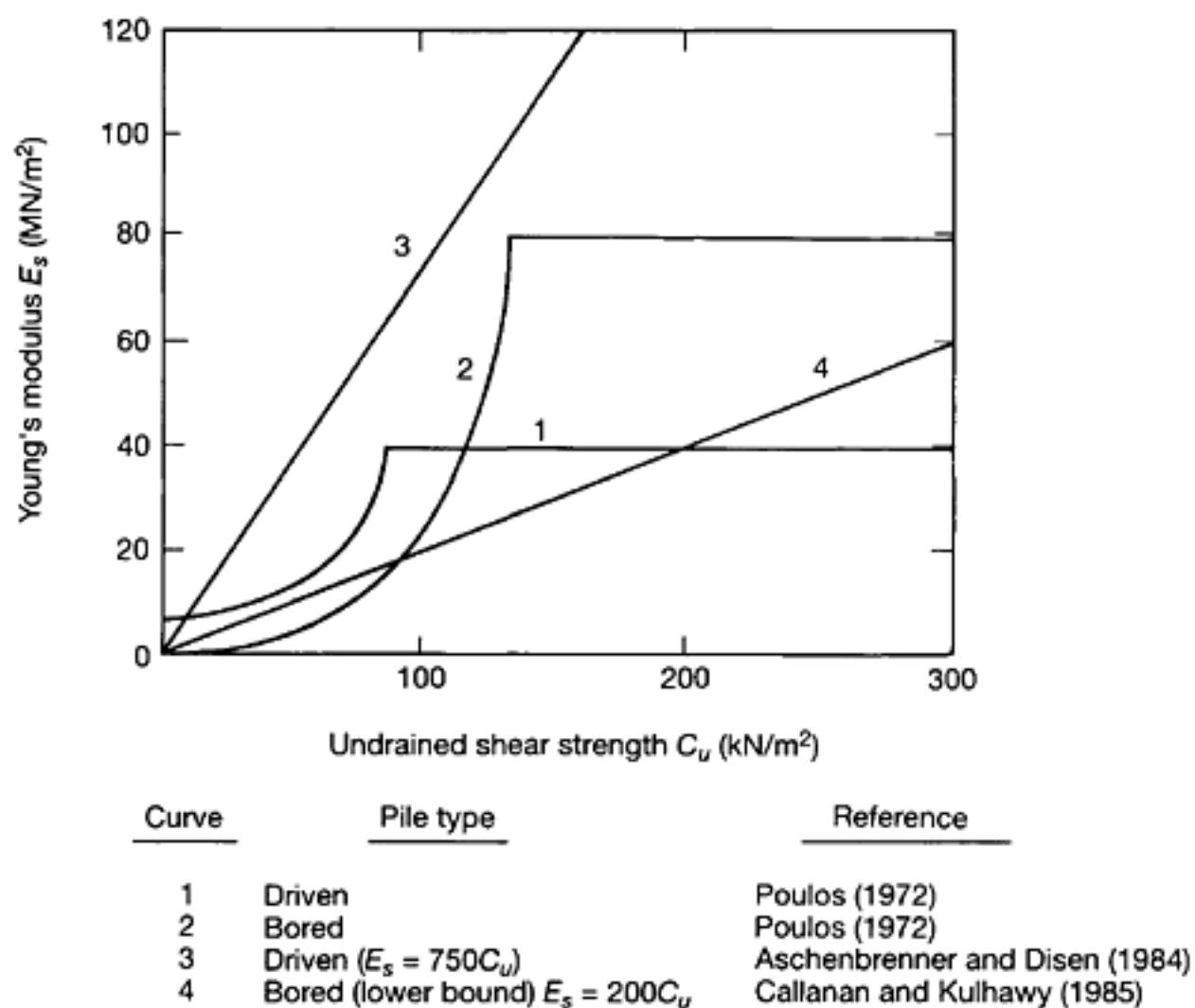


Fig. 5.18 Correlations for soil modulus for piles in clay (after Callanan and Kulhawy, for references see Poulos(4.43))

4. Calculate the consolidation settlement  $\rho_c$  for each soil layer from equations 5.5 and 5.6, using Figure 5.17 to obtain the vertical stress at the centre of each layer.
5. Apply a rigidity factor to obtain the average settlement for a rigid pile group.

The consolidation settlement calculated as described above is the final settlement after a period of some months or years after the completion of loading. It is rarely necessary to calculate the movement at intermediate times, *i.e.* to establish the time-settlement curve, since in most cases the movement is virtually complete after a period of a very few years and it is only the final settlement which is of interest to the structural engineer. If time effects are of significance, however, the procedure for obtaining the time-settlement curve can be obtained from standard works of reference on soil mechanics.

### 5.3 Pile groups in cohesionless soils

#### 5.3.1 Estimating settlements from standard penetration tests

Provided that the individual pile has an adequate safety factor against failure under compressive loading there can be no risk of the block failure of a pile group terminated in and applying stress to a cohesionless soil. For economy, the piles must be designed to have high-end bearing loads. As in the case of piles terminated in a clay, there is a risk of differential settlement between adjacent piles if the toe loads of a small group become concentrated in a small area when the piles deviate from their intended line (Figure 5.7). The best safeguard against this occurrence is to adopt a reasonably wide spacing between the piles. Methods of checking the deviation of piles caused by the installation method are described in Chapter 11. The immediate settlement of the pile group due to 'elastic' deformation of the cohesionless soil beneath the equivalent flexible raft foundation must be calculated. Equation 5.3 is applicable to this case and the deformation modulus  $E_d$  can be obtained

from plate loading tests in trial pits, or from standard penetration, pressuremeter, or Camkometer tests, made in boreholes. Schultze and Sherif(5.11) used case histories to establish a method for predicting foundation settlements from the results of standard penetration tests using the equation

$$\rho = \frac{s.p}{N^{0.87}(1 + 0.4D/B)} \quad \dots(5.7)$$

where  $s$  is a settlement coefficient

$p$  is the applied stress at foundation level

$N$  is the average SPT N-value over a depth of  $2B$  below foundation level, or  $d_s$  if the depth of cohesionless soil is less than  $2B$

$D$  and  $B$  are the foundation depth and width respectively

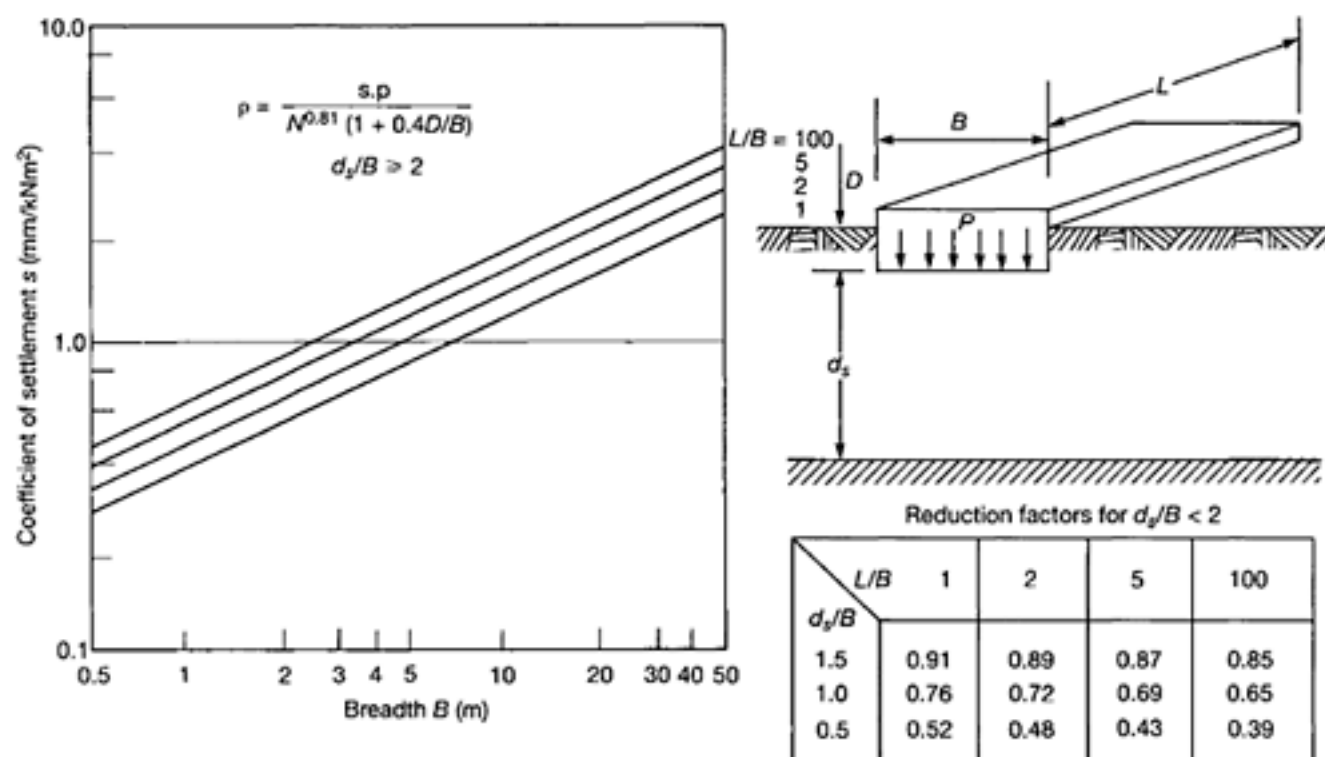


Fig. 5.19 Determining foundation settlements from results of standard penetration tests (after Schultze and Sherif<sup>(5.11)</sup>)

Values of the coefficient  $s$  and  $d_s$  are obtained from Figure 5.19. Burland and Burbidge<sup>(5.12)</sup> have developed an empirical relationship between standard penetration test N-values and a term they have called the foundation subgrade compressibility,  $a_f$ . This term is used in the equations:

$$I_c = \frac{a_f}{B^{0.7}} \quad \dots(5.8)$$

and

$$a_f = \frac{\Delta \rho_i}{\Delta q} \text{ (in mm per kN/m}^2\text{)} \quad \dots(5.9)$$

Where  $I_c$  is a compressibility index

$B$  is the foundation width

$\Delta \rho_i$  is the immediate settlement in mm

$\Delta q$  is the increment of foundation pressure in kN/m<sup>2</sup>

$I_c$  and  $a_f$  are related to the standard penetration test results shown in Figure 5.20 for normally consolidated granular soils. In very fine and silty sands below the water table where  $N$  is greater than 15 the Terzaghi and Peck correction factor should be applied, giving:

$$N(\text{corrected}) = 15 + 0.5(N - 15) \quad \dots(5.10)$$

Where the material is gravel or sandy gravel, Burland and Burbidge recommend a correction:

$$N(\text{corrected}) = 1.25 \times N \quad \dots(5.11)$$

It should be noted that the  $I_c$  values in Figure 5.20 are based on the average  $N$ -values over the depth of influence,  $z_f$  of the foundation pressure. The depth of influence is related to the width of the loaded area  $B$  in Figure 5.21 for cases where  $N$

increases or is constant with depth. Where  $N$  shows consistent decrease with depth,  $z_f$  is taken as equal to  $2B$  or the base of the compressive layer, whichever is the lesser. The average  $N$  in [Figure 5.20](#) is the arithmetic mean of the  $N$ -values over the depth of influence.

In a normally consolidated sand the immediate average settlement,  $\rho_i$ , corresponding to the average net applied pressure,  $q'$ , is given by:

$$\rho_i = q' \times B^{0.7} \times I_c \text{ (in mm)} \quad \dots(5.12)$$

In an over-consolidated sand or for loading at the base of an excavation for which the maximum previous overburden pressure was  $\sigma_{vo}$  and where  $q'$  is greater than  $\sigma_{vo}$ , the immediate settlement is given by:

$$\rho_i = (q' - \frac{2}{3} \sigma_{vo}) B^{0.7} \times I_c \text{ (in mm)} \quad \dots(5.13a)$$

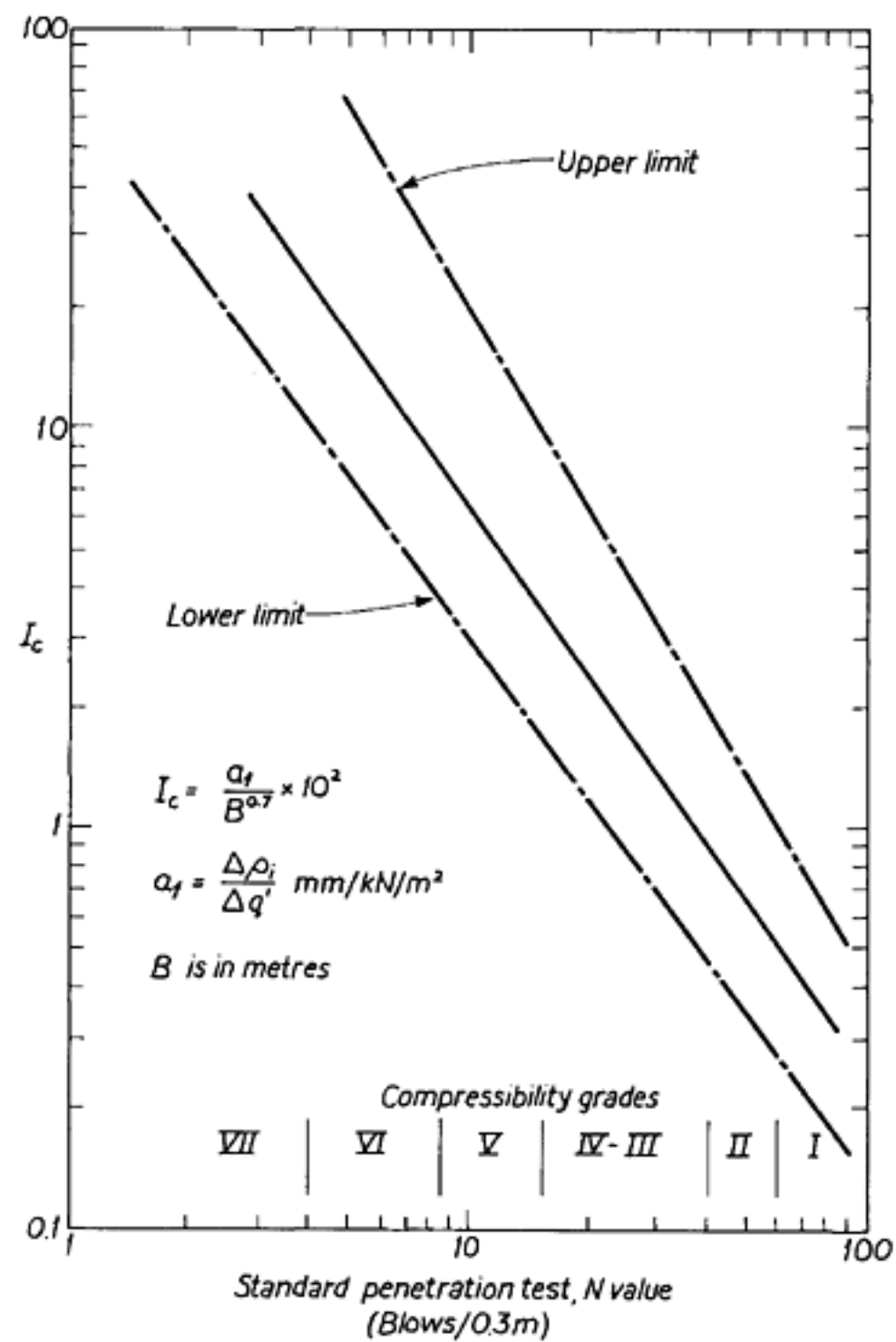


Fig. 5.20 Relationship between compressibility index and average  $N$ -value over depth of influence (after Burland and Burbidge<sup>(5.12)</sup>)

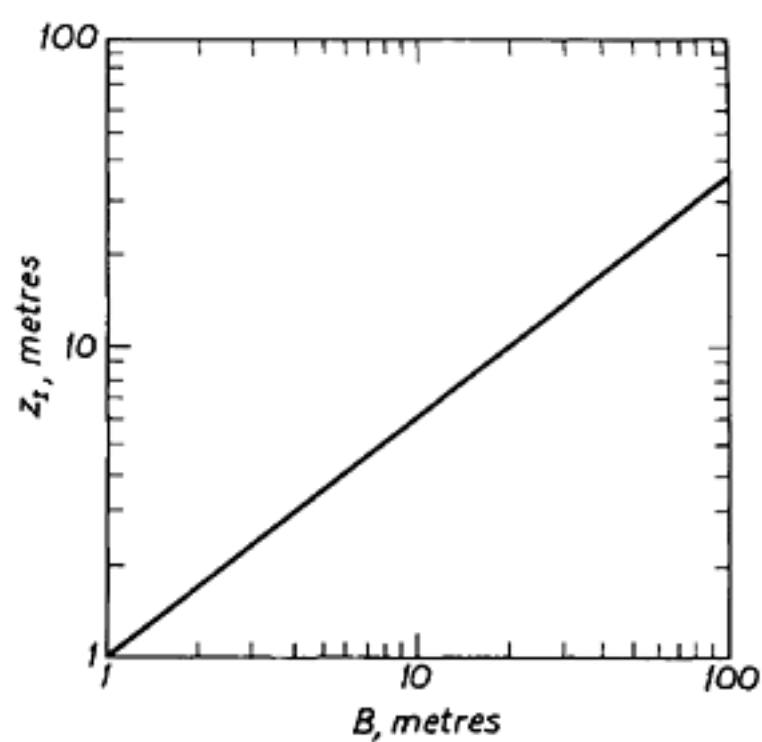


Fig. 5.21 Relationship between breadth of loaded area and depth of influence  $z_1$  (after Burland and Burbidge<sup>(5.12)</sup>)

where  $q'$  is less than  $\sigma_{vo}$  equation 5.13a becomes

$$\rho_i = q' \times B^{0.7} \times \frac{I_c}{3} (\text{in mm}) \quad \dots(5.13b)$$

In the case of pile groups the width  $B$  is the width at the base of the equivalent raft as shown in [Figure 5.3](#). The Burland and Burbridge method was developed essentially for shallow foundations and correla-

tions with published settlement records given in their paper were mainly confined to foundations where their depth was not greater than their width. They state that the depth to width ratio did not influence the settlements to any significant degree and hence a depth factor of the type shown in [Figure 5.9](#) should not be applied. However, a correction should be applied to allow for the foundation shape and for the thickness of the compressible layer beneath the foundation where this is less than the depth of influence,  $z_f$ .

correction factors are:

$$\text{Shape factor} = f_s = \left( \frac{1.25 L/B}{L/B + 0.25} \right)^2 \quad \dots(5.14a)$$

$$\text{Thickness factor} = f_t = \frac{H_s}{z_f} \left( 2 - \frac{H_s}{z_f} \right) \quad \dots(5.14b)$$

where  $L$  is the length of the loaded area ( $L > B$ )  
 $B$  is the width of the loaded area  
 $H_s$  is the thickness of the compressible layer ( $H_s < z_f$ )

Burland and Burbidge state that most settlements on granular soils are time-dependent, that is, they show a long-term creep settlement and a further time correction factor is applied using the equation:

$$f_t = \frac{\rho_t}{\rho_i} = \left[ 1 + R_3 + R \log \frac{t}{3} \right] \quad \dots(5.15)$$

where  $t$  is equal to or greater than 3 years  
 $R_3$  is the proportion of the immediate settlement which takes place in the loaded area  
 $R$  is the creep ratio expressed as the proportion of the immediate settlement that takes place per log cycle of time

Burland and Burbidge give conservative values of  $R$  and  $R_3$  as 0.2 and 0.3 respectively for static loading and 0.8 and 0.7 respectively for fluctuating loads.

Summarizing all the above corrections, the *average consolidation settlement* is given by:

$$\rho_c = f_s f_t f_i [(q' - \frac{1}{3} \sigma'_{vo}) \times B^{0.7} \times I_c] \text{ (in mm)} \quad \dots(5.16)$$

The wide range of  $I_c$  values between the upper and lower limit shown by [Figure 5.20](#) can cause difficulty in obtaining a reasonably close estimate of pile group settlements, particularly where the group is underlain by medium-dense sands. For example the average  $I_c$  value for a sand with an SPT of 10 is 6 compared with upper and lower limit values of 20 and 3 respectively, giving an upper limit of settlement of three times that calculated from the average curve. However, in most cases piles are taken down to dense sands to obtain the maximum end-bearing resistance, where the settlement calculated from the upper limit curve is likely to be relatively small.

### 5.3.2 Estimating settlements from static cone penetration tests

Where total and differential settlements are shown to be large and critical to the superstructure design it is desirable to make static cone penetration tests ([Section 11.1.4](#)) from which the soil modulus ( $E_d$ ) values can be derived and then to use the Steinbrenner ([Figure 5.10](#)) or Christian and Carrier ([Figure 5.11](#)) charts to obtain the group settlement. Relationships between the cone-resistance ( $q_c$ ) values and the drained Young's modulus for normally-consolidated quartz sands are shown in [Figure 5.22](#). The  $E_{25}$  and  $E_{50}$  values represent the drained modulus at a stress level of 25% and 50% respectively of the 'failure

stress. In a general review of the application of cone penetration testing to foundation design, Meigh<sup>(5.15)</sup> stated that the  $E_{25}$  values are appropriate for most foundation problems but the  $E_{50}$  values may be more relevant to calculating settlements of the single pile.

The  $E$  values in [Figure 5.22](#) greatly overestimate settlements in over-consolidated sands. Lunne and Christoffersen<sup>(5.16)</sup> established a relationship between initial tangent constrained modulus (the reciprocal of the modulus of volume compressibility  $m_v$ ) and  $q_c$  for normally and over-consolidated sands as shown in [Figure 5.23](#).

Another method of estimating the settlements of pile groups in granular soils based on static cone penetration test values has been developed by Schmertmann<sup>(5.17)</sup> and Schmertmann *et al.*<sup>(5.18)</sup>. Their basic equation for the settlement of a loaded area is

$$\rho = C_1 C_2 \Delta_p \sum_0^{2B} \frac{I_z}{E_d} \Delta_z \quad \dots(5.17)$$

where  $C_1$  is a depth correction factor (see below),  $C_2$  is a creep factor (see below),  $\Delta_p$  is the net increase of load on the soil at the base of the foundation due to the applied loading,  $B$  is the width of the loaded area,  $I_z$  is the vertical-strain influence factor (see [Figure 5.24](#)),  $E_d$  is the deformation modulus, and  $\Delta_z$  is the thickness of the soil layer.

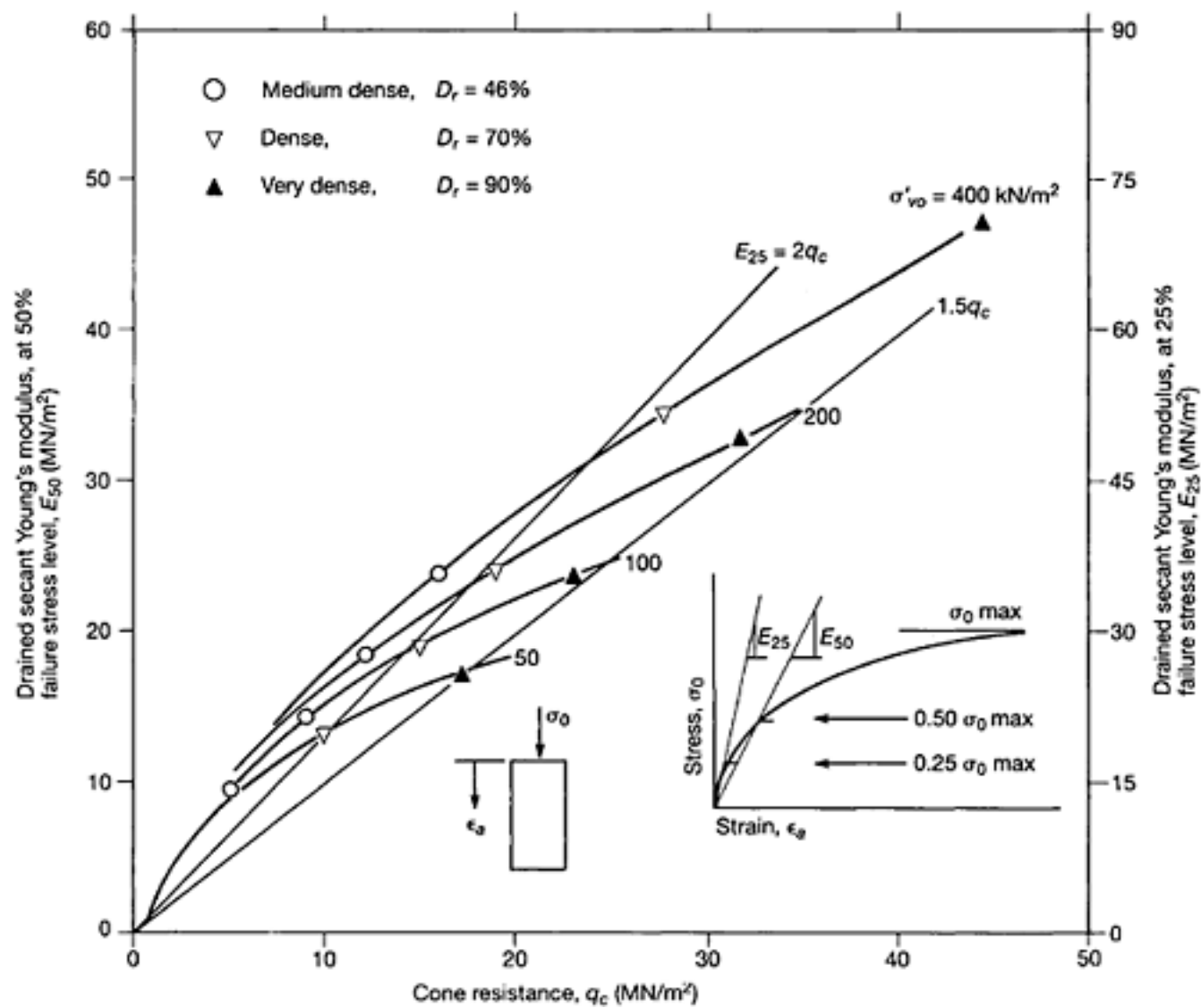


Fig. 5.22 Drained deformation modulus values ( $E_d$ ) for uncemented normally-consolidated quartz sands in relation to cone resistance (after Meigh<sup>(5.15)</sup>, Robertson et al.<sup>(5.13)</sup>, Baldi et al.<sup>(5.14)</sup>)

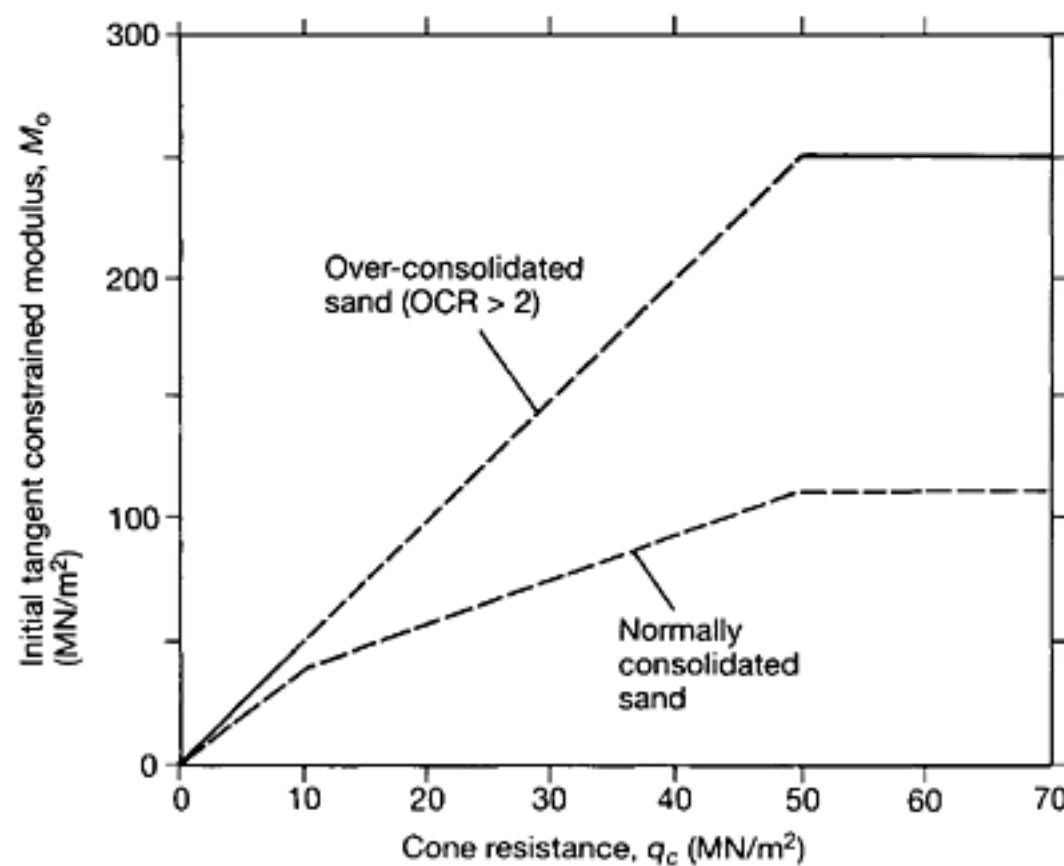


Fig. 5.23 Initial tangent constrained modulus for normally-consolidated and over-consolidated sand related to cone resistance (after Lunne and Christoffersen<sup>(5.16)</sup>)

The value of the depth correction factor is given by

$$C_1 = 1 - 0.5 \left( \frac{\sigma'_{v0}}{\Delta_p} \right) \quad \dots(5.18)$$

where  $\sigma'_{v0}$  is the effective overburden pressure at foundation level (*i.e.* at the base of the equivalent raft).

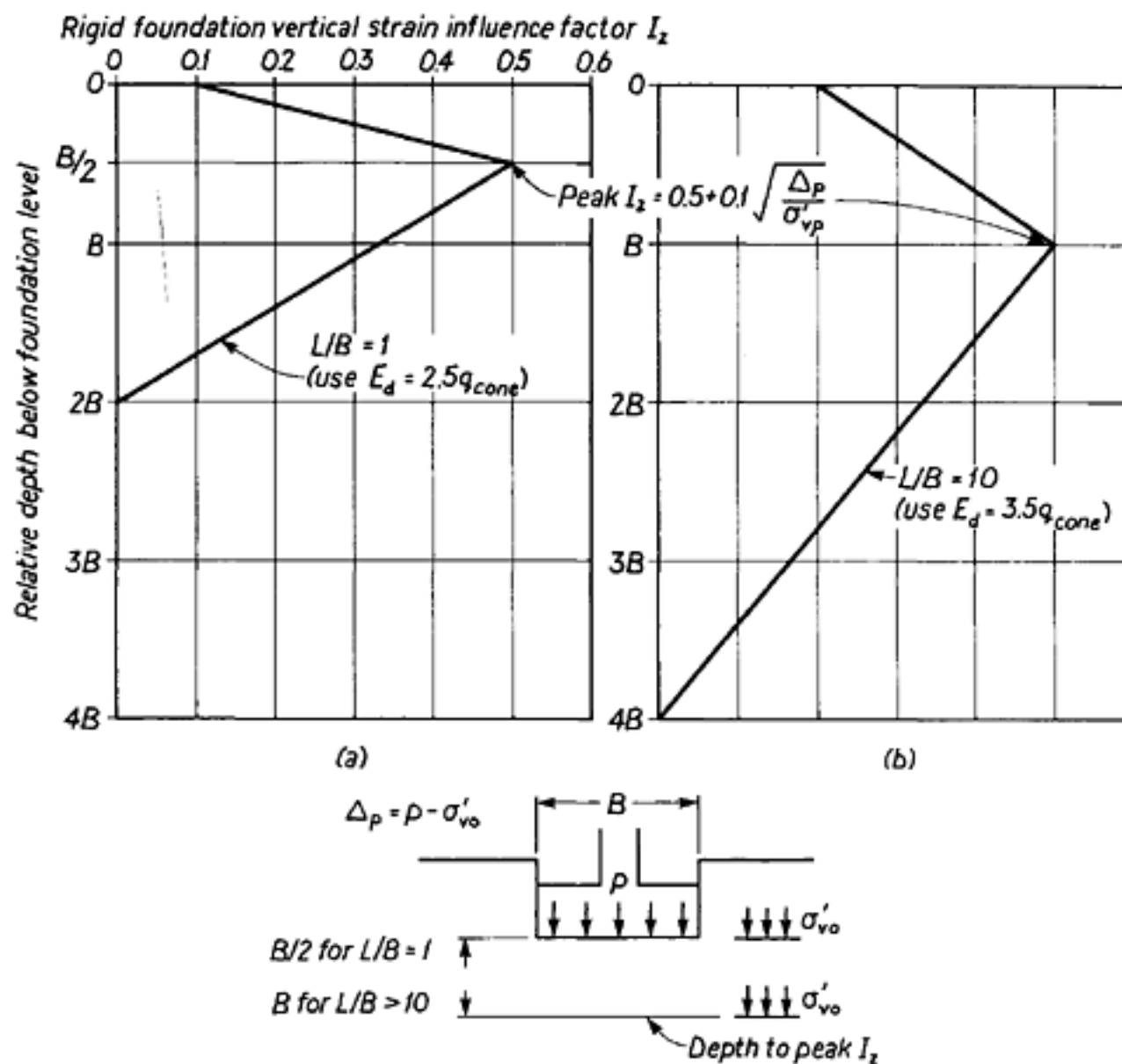


Fig. 5.24 Schmertmann's influence factors for calculating immediate settlements of foundations on sands (after Schmertmann *et al.* (5.18))

Schmertmann<sup>(5.17)</sup> states that while the settlement of foundations on granular soils is usually regarded as immediate, *i.e.* the settlement is complete within a short time after the completion of the application of load, observations have frequently shown long-continuing secondary settlement or creep. He gives the value of the green factor as

$$C_2 = 1 + 0.2 \log_{10} \left( \frac{\text{time in years}}{0.1} \right) \quad \dots(5.19)$$

Schmertmann *et al.* (5.18) have established an improved curve for obtaining the vertical-strain influence factor based on elastic half-space theory where the factor  $I_z$  is related to the foundation width, as shown in Figure 5.24.

The vertical strain influence factor is obtained from one of the two curves shown in Figure 5.24. For square pile groups (axisymmetric loading) the curve in Figure 5.24a should be used. For long pile groups (the plane strain case) where the length is more than ten times the breadth, use the curve in Figure 5.24b. Values for rectangular foundations for  $L/B$  of less than 10 can be obtained by interpolation.

The deformation modulus for square and long pile groups in normally-consolidated sands is obtained by multiplying the static cone resistance,  $q_c$ , by a factor of 2.5 and 3.5 respectively. The deformation modulus applicable for a stress increase of

$\Delta_p$  above the effective overburden pressure,  $\sigma'_{vo}$ , is given by the equation:

$$E_d = E \sqrt{\frac{\sigma'_{vo} + (\Delta_p/2)}{\sigma'_{vo}}} \quad \dots(5.20)$$

Where standard penetration tests only are available the static cone resistance (in MN/m<sup>2</sup>) can be obtained by multiplying the SPT  $N$ -values (in blows/300mm) by an empirical factor  $N$  for which Schmertmann suggests the following values

Silts, sandy silts, and slightly cohesive silty sands:

$N=0.2$

Clean fine to medium sands, slightly silty sands:

$N=0.35$

Coarse sands and sands with a little gravel:

$N=0.5$

Sandy gravel and gravels:

$N=0.6$

Where static cone resistance data are available the relationships in [Figures 5.22](#) or 5.23 can be used to obtain values of  $E_d$  for substitution in equation 5.17.

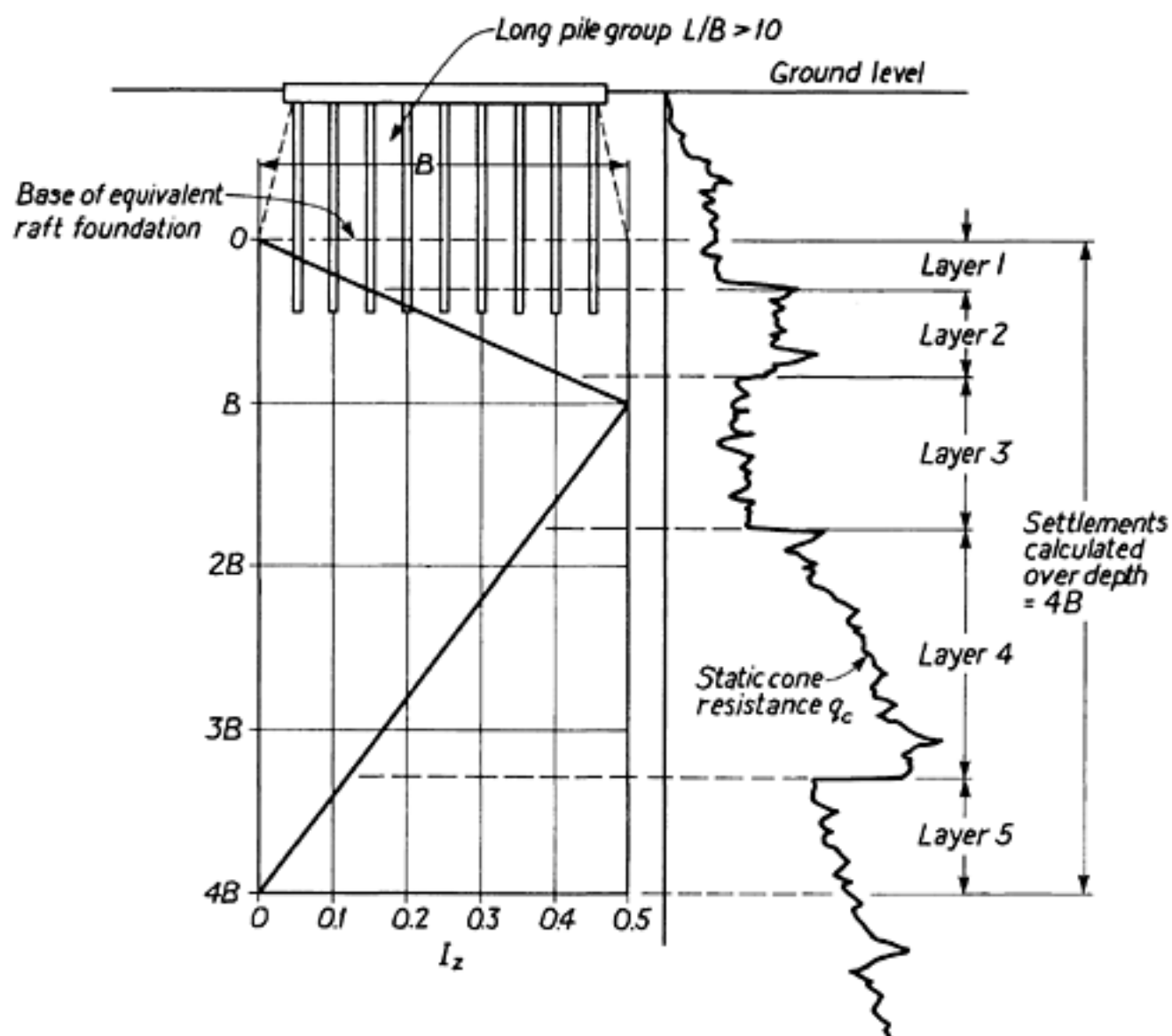


Fig. 5.25 Establishing the vertical strain from static cone penetration tests

The procedure for estimating settlements by the Schmertmann method is first to divide the static cone resistance diagram into layers of approximately equal or representative values of  $q_c$  in a manner shown in [Figure 5.25](#). The base of the equivalent raft

representing the pile group is then drawn to scale on this diagram and the influence curve is superimposed beneath the base of the raft. The settlements in each layer resulting from the loading  $\Delta_p$  at the base of the equivalent raft are then calculated

using the values of  $E_d$  and  $I_z$  appropriate to each of the representative layers. The sum of these settlements is corrected for

depth and creep from equations 5.18 to 5.20. The various steps in the calculation are made in tabular form as illustrated in Example 5.3.

Where piles are terminated in a cohesionless soil stratum underlain by compressible clay, the settlements within the zone of clay stressed by the pile group are calculated by the methods described in [Section 5.2.2](#). The form of load distribution to be used in this analysis to obtain the dimensions of the equivalent raft on the surface of the clay layer is shown in [Figure 5.13](#).

## 5.4 Deep pile groups in cohesive and cohesionless soils

The equivalent raft method of determining the settlement of pile groups described in 5.2 and 5.3 is a simple one and applicable to ordinary foundation work where the overall loading at the level of the equivalent raft is not high and the penetration of the piles into the bearing stratum is relatively short. The method is particularly suitable for layered soils. Where deep penetration of the piles is required to transfer heavy loading to a compressible stratum it may be necessary to analyse the interaction of the piles forming the group. Thus when an individual pile forming the group is loaded the soil surrounding the pile is compressed and, depending on the spacing of the piles, the compressed zone extends to the neighbouring piles which also compress the soil around them. The extent to which the settlement of the individual pile increases the settlement of its neighbours is calculated by means of interaction factors. These depend on the geometry, stiffness and spacing of the piles and the elastic modulus of the soil between and beneath them.

The analysis is a complex one and involves the splitting of the piles into a number of discrete elements as described for the case of the single pile in 4.6. Various simplifying assumptions are made and iterative methods are required for the solution of the problem.

A description of the various analytical methods based on boundary element and finite element techniques and requiring a digital computer for their solution is beyond the scope of this book. Reference should be made to the paper by Poulos<sup>(4.43)</sup>. It

was noted at the beginning of this chapter that

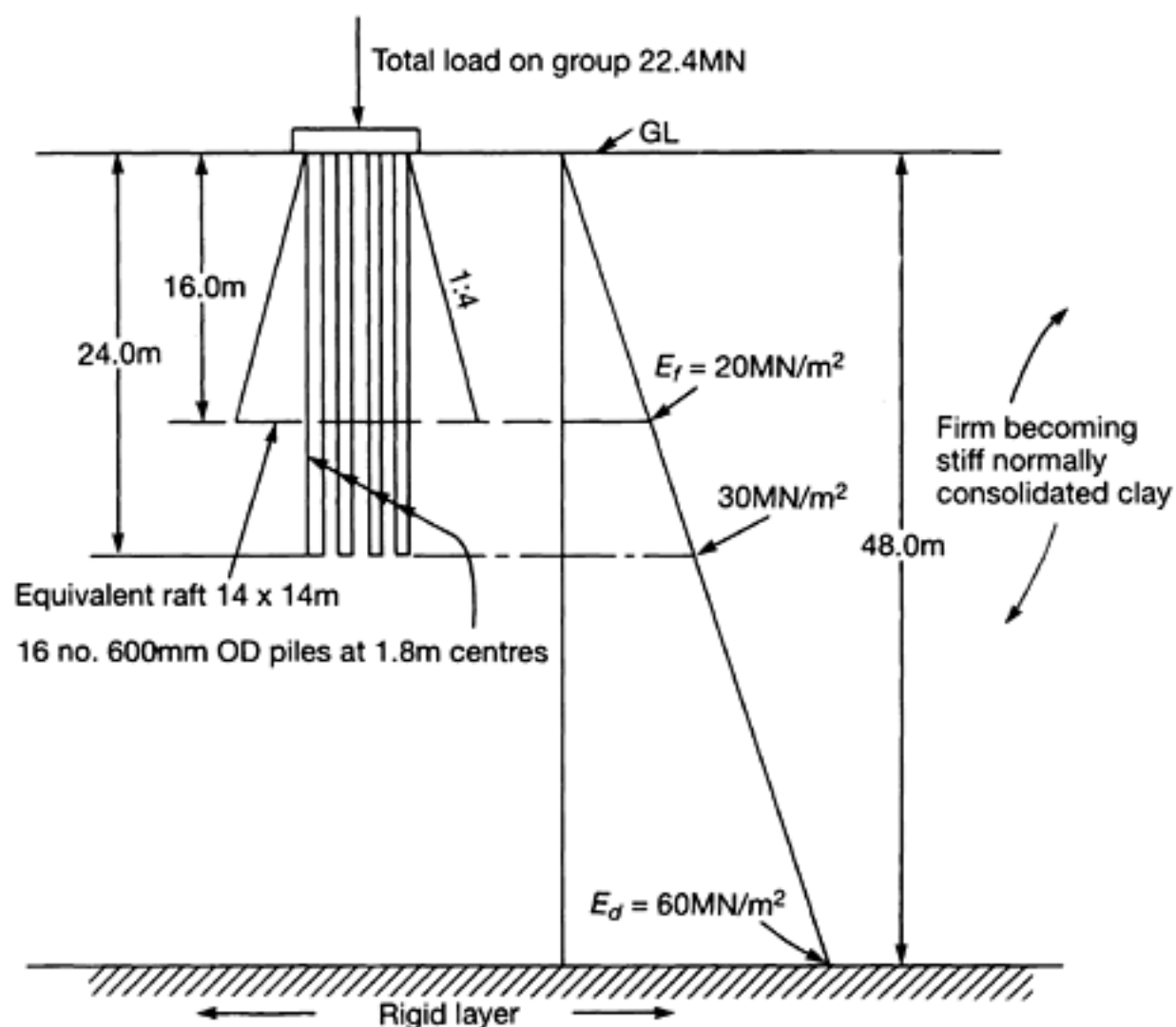


Fig. 5.26 Pile group settlement by equivalent raft method

pile group settlements obtained from computer-based analyses were not necessarily more accurate than those calculated by the equivalent raft method (Figure 5.3). As an example of the relative accuracy of the methods Figure 5.26 shows a 4×4 pile group where the piles spaced at 3 diameters centre to centre are taken down to a depth of 24m into a firm becoming stiff normally consolidated clay where the undrained shear strength and compressibility vary linearly with depth. The group settlements calculated by the equivalent raft method used the influence factors of Butler (Figure 5.14).

The comparative group settlements were:

DEFPIG	—42mm
PGROUP	—31mm
Equivalent raft	—30mm

## 5.5 Pile groups terminating in rock

The stability of a pile group bearing on a rock formation is governed by that of the individual pile. For example, one or more of the piles might yield due to the presence of a pocket of weathered rock beneath the toe. There is no risk of block failure unless the piles are terminated on a sloping rock formation, when sliding on a weak clay-filled bedding plane might occur if the bedding is unfavourably inclined to the direction of loading (Figure 5.27). The possibility of such occurrences must be studied in the light of the information available on the geology of the site.

The settlement of a pile group may be of significance if the piles are heavily loaded. Immediate settlements can be calculated as described in Section 5.2.2, and equations 5.2 and 5.3 are applicable, where  $E_d$  is reasonably constant with depth.

The values of  $E_d$  for use in equation 5.3 are not the same as those shown in Table 4.2. The latter values are those appropriate

to the fairly high stress levels beneath the base of an individual pile. When the spread of load beneath a group of piles is considered, the stress level beneath the equivalent raft foundation is considerably less and the deformation modulus has a correspondingly higher value (see Figure 5.12). Some typical values applicable to pile groups where the anticipated settlement is of the order of 0.01% to 0.05% of the group width are listed in Table 5.2.

It is also possible to obtain a rough estimate of the deformation modulus of a jointed rock mass from empirical relationships with the unconfined compression strength of the intact rock. BS8004 gives a relationship as  $E_d = j \times M_r \times q_c$  where  $j$  is the mass factor (see 4.7.3 for values) and  $M_r$  is the ratio of the elastic modulus of the intact rock to its unconfined compression

strength. BS 8004 gives the following values for  $M_r$ :

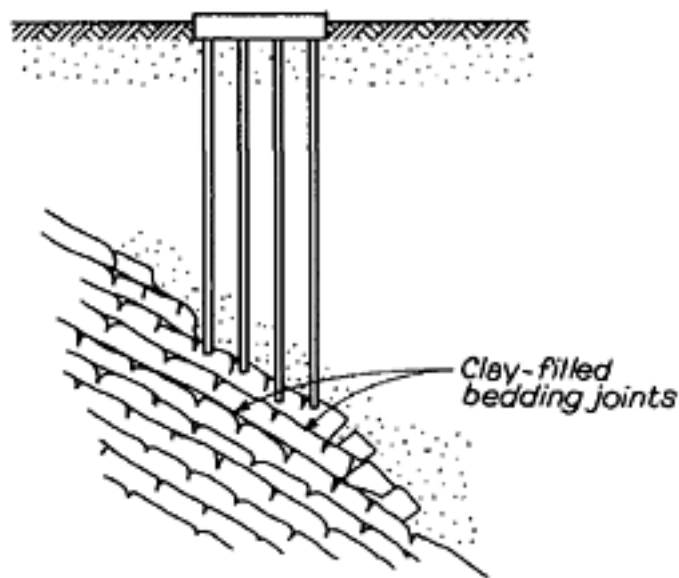


Fig. 5.27 Instability of pile group bearing on sloping rock surface

		Values for $M_r$
Group 1	Pure limestones and dolomites Carbonate sandstones of low porosity	600
Group 2	Igneous Oolitic and marly limestones Well-cemented sandstones Indurated carbonate mudstones Metamorphic rocks including slates and schists (flat cleavage/foliation)	300
Group 3	Very marly limestones Poorly cemented sandstones Cemented mudstones and shales Slates and schists (steep cleavage/foliation)	150
Group 4	Uncemented mudstones and shales	75

Chalk and Keuper Marl are excluded from the above groups. Some observed values of  $E_d$  for these are given in [Table 5.2](#).

Table 5.2 Values of deformation modulus of rock for low stress levels

Type of rock	Deformation modulus, $E_d$ MN/m <sup>2</sup>	Reference
Chalk (Grade I)	3900	5.19
Chalk (Grade II)	1900–3900	5.19
Chalk (Grade III)	950–1900	5.19
Chalk (Grade IV)	450–950	5.19
Chalk (Grade V)	450	5.19
Chalk (Grade II)	1880	Unpubl.
Chalk (Grade I–IV)	113–164	5.20
Chalk (Grade V–VI)	75–130	4.59
Keuper Marl (Zone I)	26–250	4.48
Keuper Marl (Zone II)	9–70	4.48
Keuper Marl (Zone III)	2–48	4.48
Keuper Marl (Zone IV)	2–13	4.48
Siltstone, weathered, RQD.0–20%	1–6	5.21

It is likely that weathered rocks will show an increase in  $E_d$  with depth as the state of weathering decreases from complete at rockhead to the unweathered condition. If it is possible to draw a straight line through the increasing values the influence

factors in [Figure 5.14](#) can be used in conjunction with equation 5.4 to obtain the settlement at the centre of the loaded area. These curves were established by Butler([5.6](#)) for a Poisson's ratio of 0.5, but most rock formations have lower ratios. Meigh ([5.22](#)) stated that the Poisson's ratio of Triassic rocks is about 0.1 to 0.3. Burland and Lord([5.23](#)) established a value of 0.2 for the Middle Chalk.

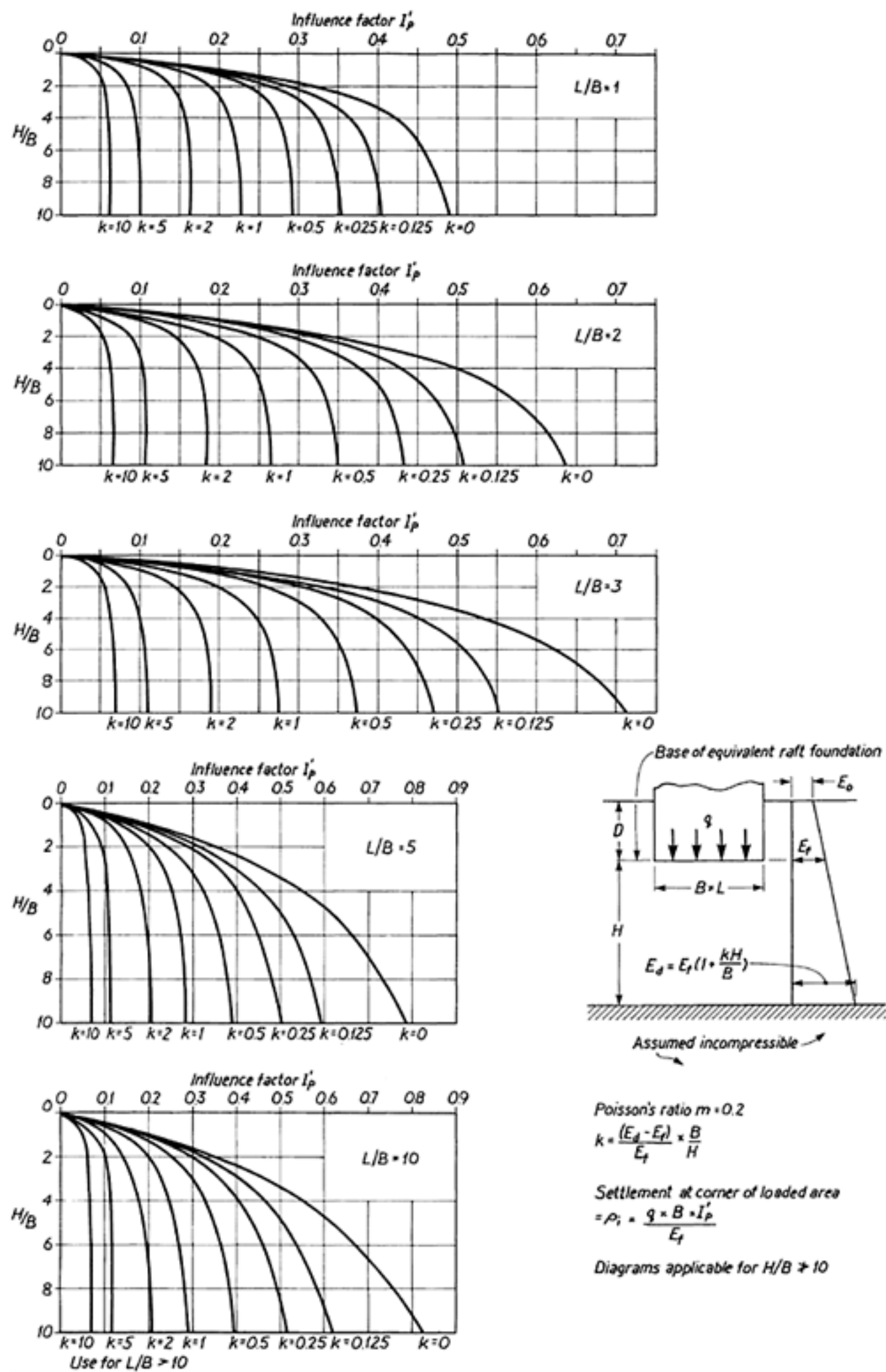


Fig. 5.28 Values of influence factor for deformation modulus increasing linearly with depth and modular ratio of 0.2 (after Meigh<sup>(5.22)</sup>)

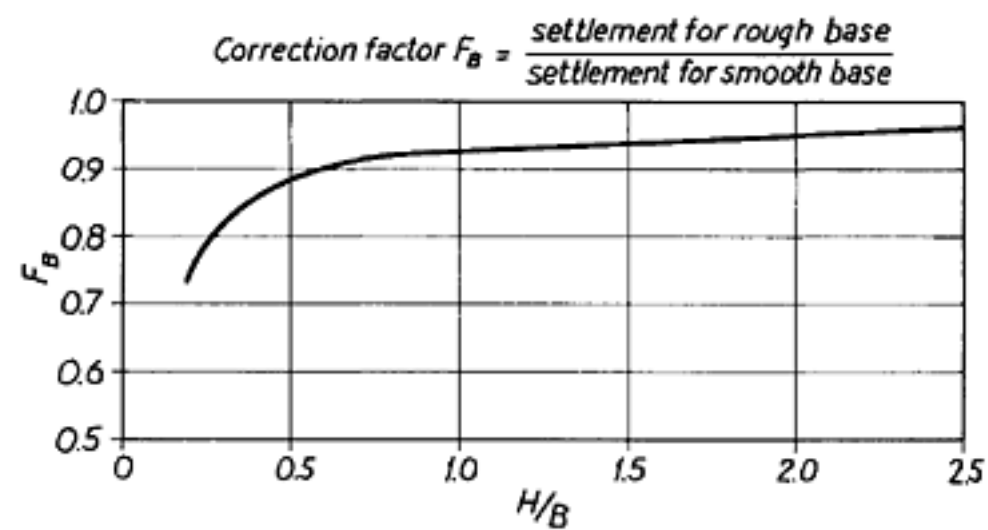


Fig. 5.29 Correction factors for roughness of base of foundation (after Meigh<sup>(5.22)</sup>)

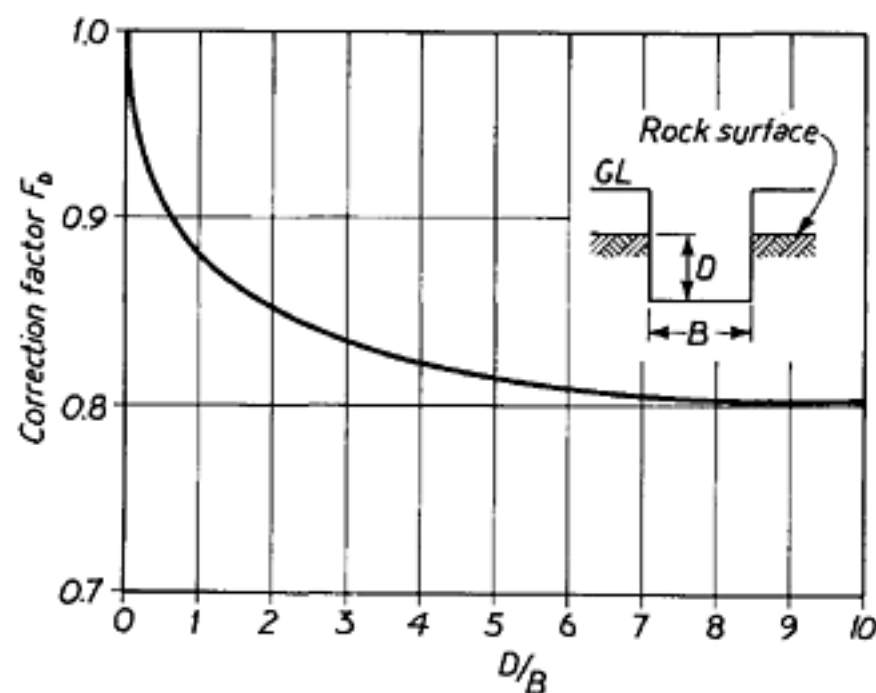


Fig. 5.30 Correction factors for depth of embedment of foundation below surface of rock (after Meigh<sup>(5.22)</sup>)

Meigh<sup>(5.22)</sup> derived curves for the influence factors shown in [Figure 5.28](#) for various values of the constant  $k$  in equation 5.21 where:

$$k = \frac{(E_d - E_f) B/H}{E_f} \quad \dots(5.21)$$

and for a Poisson's ratio of 0.2. He applied further corrections to the calculation of the settlement at the *corner* of the foundation where:

$$\text{Settlement at corner} = \rho_i = \frac{q_n B I'_p}{E_f} \quad \dots(5.22)$$

The corrected settlement is given by:

$$\rho_c (\text{corrected}) = \frac{q_n B I'_p}{E_f} \times F_B \times F_D \quad \dots(5.23)$$

where  $F_B$  is the correction factor for roughness of base ([Figure 5.29](#))

$F_D$  is the correction factor for depth of embedment ([Figure 5.30](#))

The equivalent raft is assumed to have a rough base and is divided into four equal rectangles and the settlement computed for the corner of each rectangle from equation 5.22. The settlement at the centre of the pile group is then four times the corner settlement.

## **5.6 Pile groups in filled ground**

The problem of negative skin friction or drag-down on the shafts of isolated piles embedded in fill was discussed in 4.8. This drag-down is caused by the consolidation of the fill under its own weight, or under the weight of additional imposed fill. If the fill is underlain by a compressible clay the consolidation of the clay under the weight of the fill also causes negative skin friction in the portion of the shaft

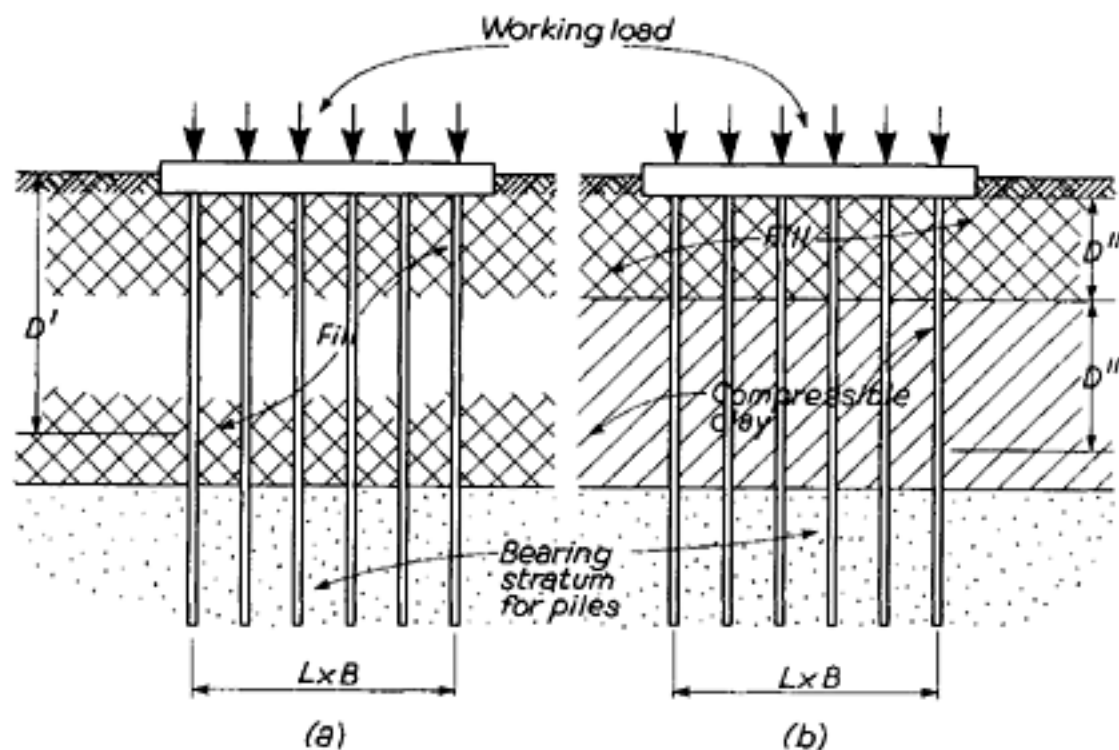


Fig. 5.31 Negative skin friction on pile groups in filled ground

(a) Fill overlying relatively incompressible bearing stratum

(b) Fill placed on compressible clay layer

within this clay. Negative skin friction also occurs on piles installed in groups but the addition to the working load on each of the piles in the group is not necessarily more severe than that calculated for the isolated pile. The basis for calculating the negative skin friction as described in 4.8.1, is that the ultimate skin friction on the pile shaft is assumed to act on that length of pile over which the fill and any underlying compressible clay move downwards relative to the shaft. The magnitude of this skin friction cannot increase as a result of grouping the piles at close centres, and the total negative skin friction acting on the group cannot exceed the total weight of fill enclosed by the piles. Thus in [Figure 5.31a](#),

$$\text{total load on pile group} = \text{working load} + (B \times L \times \gamma' D') \quad \dots(5.24)$$

where  $\gamma'$  is the unit weight of fill, and  $D'$  is the depth over which the fill is moving downwards relative to the piles. Where the fill is underlain by a compressible clay, as in [Figure 5.31b](#),

$$\text{total load on pile group} = \text{working load} + B \times L(\gamma' D' + \gamma'' D'') \quad \dots(5.25)$$

where  $D''$  is the total thickness of fill,  $\gamma''$  is the unit weight of compressible clay, and  $D'''$  is the thickness of compressible clay moving downwards relative to the piles.

It should also be noted that the negative skin friction acting on the piles in the group does not increase the *settlement* of the group caused by the working load on the piles. If the filling has been in place for a long period of years any underlying compressible soil will have been fully consolidated and the only additional load on the compressible soil causing settlement of the group is that from the working load on the piles. However, if the fill is to be placed only a short time before driving the piles, then any compressible soil below the fill will consolidate. The amount of this consolidation can be calculated separately and added to the settlement caused by the working load on the piles. The negative skin friction on the piles is not included in the working load for the latter analysis.

## 5.7 Effects on pile groups of installation methods

When piles are driven in groups into clay the mass of soil within the ground heaves and also expands laterally, the volume of this expansive movement being approximately equal to the volume occupied by the piles. High pore pressures are developed in the soil mass, but in the course of a few days or weeks these pore pressures dissipate and the heaving directly caused by pore pressure subsides. In soft clays the subsidence of the heaved soil can cause negative skin friction to develop. It is not usual to add this negative skin friction to the working load since it is of relatively short duration, but its effect can be allowed for by ignoring any *support* provided in skin friction to the portion of the pile shaft within the soft clay. Methods of calculating the surface heave within a pile group have been discussed by Hagerty and Peck([5.24](#)). Chow and Teh([5.25](#)) have

established a theoretical model relating the pile

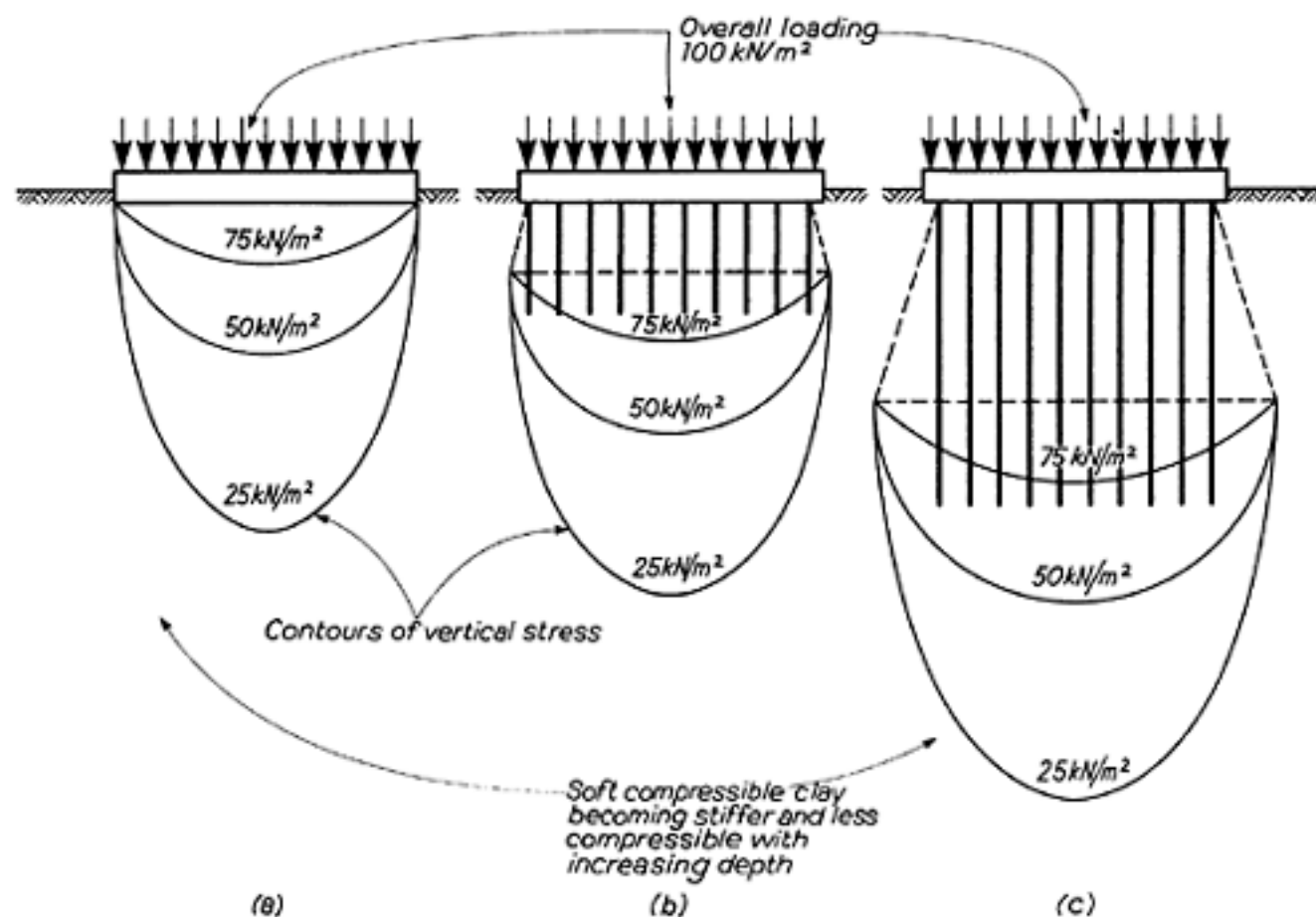


Fig. 5.32 Comparison of stress distribution beneath shallow raft foundation and beneath pile groups

- (a) Shallow raft
- (b) Short friction piles
- (c) Long friction piles

head heave/diameter ratio to the pile spacing/diameter ratio for a range of length/diameter ratios in soft, firm, and stiff clays. It is not good practice to terminate pile groups within a soft clay since the reconsolidation of the heaved and remoulded soil can result in the substantial settlement of a pile group, and neighbouring structures can be affected. It may be seen from [Figure 5.32](#) that there is little difference between the extent of the stressed zone around and beneath a surface raft and a group of short friction piles. The soil beneath the raft is not disturbed during construction and hence the settlement of the raft may be much less than that of a pile group carrying the same overall loading. This was illustrated by Bjerrum<sup>(5.26)</sup>, who compared the settlement of buildings erected on the two types of foundation construction on the deep soft and sensitive clays of Drammen near Oslo.

A building where the gross loading of  $65 \text{ kN/m}^2$  was reduced by excavation for a basement to a net loading of  $25 \text{ kN/m}^2$  was supported on 300 timber friction piles 23m long. In 10 years the building had settled by 110mm and the surrounding ground surface had settled by 80mm. A nearby building with a gross loading of  $55 \text{ kN/m}^2$  had a fully compensated unpiled foundation, *i.e.* the weight of the soil removed in excavating for the basement balanced the superstructure and substructure giving a net intensity of loading of zero on the soil. Nearly 30mm of heave occurred in the base of excavation and thus the settlement of the building was limited to the re-consolidation of the heaved soil. The net settlement 9 years after completing the building was only 5mm.

Lateral movement of a clay soil and the development of high pore pressures can damage structures or buried services close to a pile group. Adams and Hanna<sup>(5.27)</sup> measured the pore pressures developed within the centre of a large group of driven piles at Pickering Nuclear Power Station, Ontario. The horizontal ground strains were also measured at various radial distances from the centre. The group consisted of 750 piles driven within a circle about 46m in diameter. Steel H-section piles were selected to give a minimum of displacement of the 15m of firm to very stiff and dense glacial till, through which the piles were driven to reach bedrock. From measurements of the change in the distance between adjacent surface markers it was calculated that the horizontal earth pressure at a point 1.5m from the edge of the group was  $84 \text{ kN/m}^2$  while at 18.8m from the edge the calculated pressure was only  $1 \text{ kN/m}^2$ . Earth pressure cells mounted behind a retaining wall 9m from the group showed no increase in earth pressure due to the pile driving. Very high pore pressures were developed at the centre of the piled area, the increase being  $138 \text{ kN/m}^2$  at a depth of 6m, dissipating to  $41 \text{ kN/m}^2$ , 80 days after completing driving of the instrumented pile, when all pile driving in the group had been completed.

The average ground heave of 114mm measured over the piled area represented a volume of soil

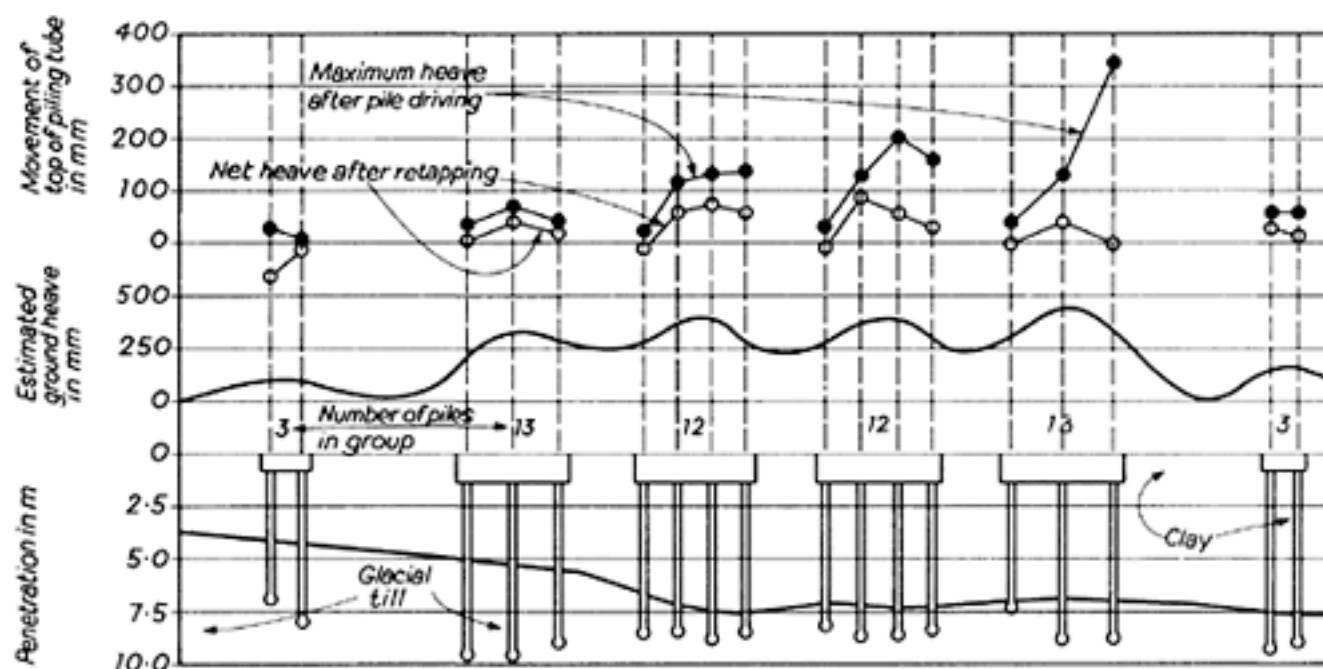


Fig. 5.33 Observations of heave due to pile driving in clay (after Brzezinski *et al.* (5.28))

displacement greater than the volume of steel piles which had been driven into the soil, for which the theoretical ground heave was 108mm.

Substantial heave accompanied by the lifting of piles already driven can occur with large displacement piles. Brzezinski *et al.* (5.28) made measurements of the heave of 270 driven-and-cast-in-place piles in a group supporting a 14-storey building in Quebec. The piles had a shaft diameter of 406mm and the bases were expanded by driving. The piles were driven through 6.7m to 11m of stiff clay to a very dense glacial till. Precautions against uplift were taken by providing a permanent casing to the piles and the concrete was not placed in the shafts until the pile bases had been re-driven by tapping with a drop hammer to the extent necessary to overcome the effects of uplift. The measured heave of a cross-section of the piled area is shown in Figure 5.33. It was found that the soil heave caused the permanent casing to become detached from the bases, as much as 300mm of separation being observed. Heave effects were not observed if the piles were driven at a spacing wider than 12 diameters. This agrees with the curves established by Chow and Teh (5.25) which show a pile head heave of only about 1mm for a spacing of 12 diameters.

Similar effects were observed by Cole (5.29). At three sites the heave was negligible at pile spacings wider than 8 to 10 diameters. Cole observed that uplift was more a function of the pile diameter and spacing than of the soil type or pile length. Where piles carry their load mainly in end bearing the effect of uplift is most damaging to their performance and on all sites where soil displacement is liable to cause uplift, precautions must be taken as described in 5.8. Heave is not necessarily detrimental where piles are carried by skin friction in firm to stiff clays in which there will be no appreciable subsidence of the heaved soil to cause negative skin friction to develop on the pile shaft. On a site where a 12-storey block of flats was supported by driven-and-cast-in-situ piles installed in 5m of firm London clay to terminate at the base of a 4m layer of stiff London clay, about 0.5m of heave was observed in the ground surface after seventy piles had been driven within the 24×20m area of the block. A pile was tested in an area where 220mm of heave had occurred. The settlement at 1300kN (*i.e.* twice the working load) was 23mm, while the settlement at the working load was only 2.5mm.

Heaving and the development of high pore pressures do not occur when bored-and-cast-in-place piles are installed in groups. However, general subsidence around the piled area can be caused by the 'draw' or relaxation of the ground during boring. In soft sensitive clays the bottom of a pile borehole can heave up due to 'piping', with a considerable loss of ground. These effects can be minimized by keeping the pile borehole full of water or bentonite slurry during drilling and by placing the concrete within a casing which is only withdrawn after all concrete placing is completed.

Detrimental effects from heave are not usually experienced when driving piles in groups in cohesionless soils. A loose soil is densified, thus requiring imported filling to make up the subsidised ground surface within and around the group. Adjacent structures may be damaged if they are within the area of subsidence. A problem can arise when the first piles to be installed drive easily through a loose sand but, as more piles are driven, the sand becomes denser thus preventing the full penetration of all the remaining piles. This problem can be avoided by paying attention to the order of driving, as described in 5.8.

Subsidence due to the loss of ground within and around a group in a cohesionless soil can be quite severe when bored-and-cast-in-place piles are installed, particularly when 'shelling' is used as the boring

method, (see [3.3.5](#)). The subsidence can be very much reduced, if not entirely eliminated, by the use of rotary drilling with the assistance of a bentonite slurry (see [3.3.6](#)).

## 5.8 Precautions against heave effects in pile groups

It will have been noted from [Section 5.7](#) that the principal problems with soil heave and the uplift of piles occur when large displacement piles are driven into clay. In cohesionless soils the problems can be overcome to a great extent by using small displacement piles such as H-sections or open-ended steel tubes. To adopt a spacing between piles of 10 or more diameters is not usually practical if pile group dimensions are to be kept within economical limits. Pre-boring the pile shaft is not always effective unless the pre-bored hole is taken down to the pile base, in which case the skin friction will be substantially reduced if not entirely eliminated. Jetting piles is only effective in a cohesionless soil and the problems associated with this method are described in 3.1.9. The most effective method is to re-drive any risen piles, after driving all the piles in a cluster that are separated from adjacent piles by at least 12 diameters has been completed.

In the case of driven-and-cast-in-place piles, a permanent casing should be used and the re-driving of the risen casing and pile base should be effected by tapping the permanent casing with a 3-tonne hammer, as described by Brzezinski *et al.* ([5.28](#)). Alternatively, the 'Multitube' method described by Cole ([5.29](#)) can be used. This consists of providing sufficient lengths of withdrawable casing to enable all the piling tubes to be driven to their full depth and all the pile bases to be formed before the pile shafts in any given cluster are concreted. An individual cluster dealt with in this way must be separated from a neighbouring cluster by a sufficient distance to prevent the uplift of neighbouring piles or to reduce this to an acceptable amount. On the three sites described by Cole it was found possible to drive piles to within 6.5 diameters of adjacent clusters without causing an uplift of more than 3mm to the latter. This movement was not regarded as detrimental to the load/settlement behaviour. Cole stated that, although the 'Multitube' system required eight driving tubes to each piling rig, the cost did not exceed that of an additional 2m on each pile.

Curtis ([3.20](#)) states that it is possible to re-drive risen driven-and-cast-in-situ piles using a 3- to 4-tonne hammer with a drop not exceeding 1.5m. The head of the pile should be protected by casting on a 0.6m capping cube in rapid-hardening cement concrete.

Colclough ([5.29](#)) stated that the order of driving piles did not affect the incidence of risen piles but it did change the degree of uplift on any given pile in a group. Generally the aim should be to work progressively outwards or across a group, and in the case of an elongated group from end to end or from the middle outwards in both directions. This procedure is particularly important when driving piles in cohesionless soils. If piles are driven from the perimeter towards the centre of a group, a cohesionless soil will 'tighten-up' so much due to ground vibrations that it will be found impossible to drive the interior piles.

It is desirable to adopt systematic monitoring of the behaviour of all piles installed in groups by taking check levels on the pile heads, by carrying out re-driving tests, and by making loading tests on working piles selected at random from within the groups. Loading tests undertaken on isolated piles before the main pile driving commences give no indication of the possible detrimental effects of heave. Lateral movements should also be monitored as necessary.

## 5.9 Pile groups beneath basements

Basements may be required beneath a building for their functional purpose, *e.g.* as an underground car park or for storage. The provision of a basement can be advantageous in reducing the loading which is applied to the soil by the building. For example, if a basement is constructed in an excavation 7m deep the soil at foundation level is relieved of a pressure equivalent to 7m of overburden, and the gross loading imposed by the building is reduced by this amount of pressure relief. It is thus possible to relieve completely the net loading on the soil. An approximate guide to the required depth of excavation is the fact that a multistorey dwelling block in reinforced concrete with brick and concrete external walls, lightweight concrete partition walls, and plastered finishes weighs about 12.5kN/m<sup>2</sup> per storey. This loading is inclusive of 100% of the dead load and 60% of the design imposed load. Thus a 20-storey building would weigh 250kN/m<sup>2</sup> at ground level, requiring a basement to be excavated to a depth of about 20m to balance the loading (assuming the ground-water level to be 3m below ground level and taking the submerged density of the soil below water level).

Deep basement excavations in soft compressible soils can cause considerable constructional problems due to heave, instability, and the settlement of the surrounding ground surface. Because of this it may

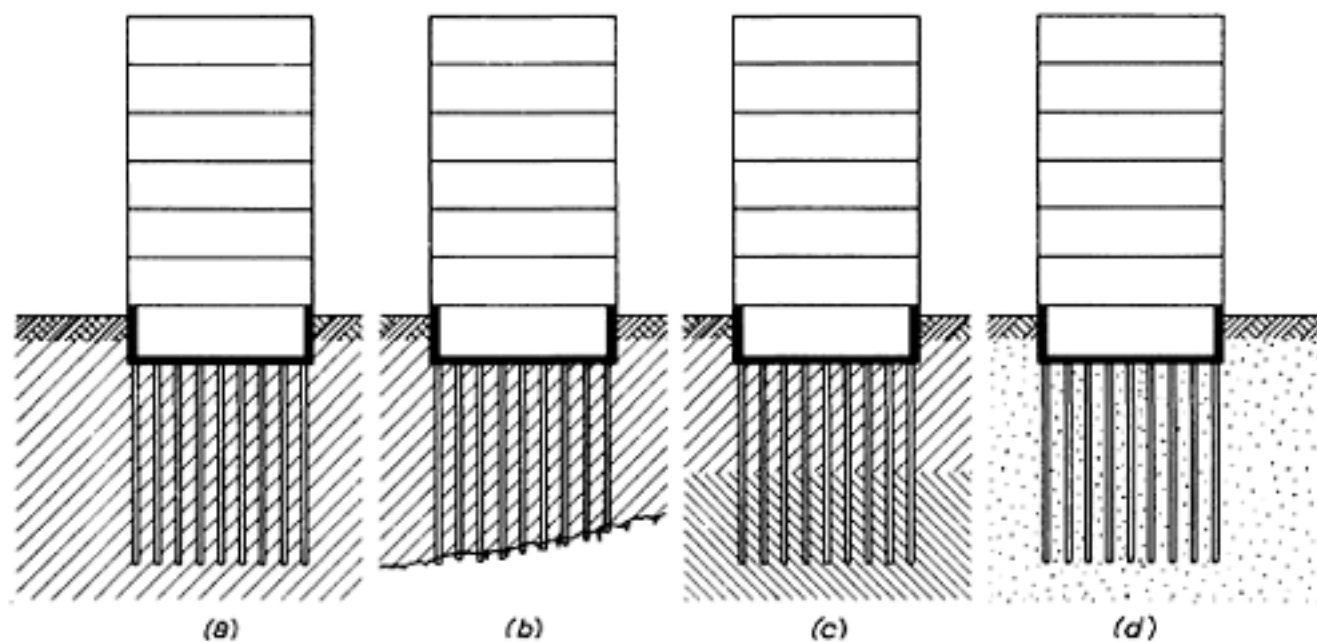


Fig. 5.34 Piled basements in various ground conditions

- (a) Wholly in compressible clay
- (b) Compressible clay over bedrock
- (c) Soft clay over stiff clay
- (d) Loose sand becoming denser with depth

be desirable to adopt only a partial relief of loading by excavating a basement to a moderate depth and then carrying the net loading on piles taken down to soil having a lesser compressibility.

In all cases where piles are installed to support structures it is necessary to consider the effects of soil swelling and heave on the transfer of load from the basement floor slab to the piles. Four cases can be considered as described below and shown in [Figure 5.34](#).

#### **Piles wholly in compressible clay ([Figure 5.34a](#))**

In this case the soil initially heaves due to swelling consequent on excavating the foundation, and further heave results from pile driving. The heaved soil is then trimmed off to the correct level and the basement slab concreted. If the concreting is undertaken within a few days or a week after the pile driving there is a tendency for the heaved soil to slump down, particularly in a soft clay which developed high pore pressures. A space may tend to open between the underside of the concrete and the soil surface. When the superstructure is erected the piles will carry their working load and if correctly designed they will settle to an acceptable degree. This will in turn cause the basement slab to settle but pressure will not develop on its underside because the soil within and beneath the settling piles will move down with them. Thus the maximum pressure on the underside of the basement slab is due to the soil swelling at an early stage before partial slumping of the heaved soil takes place and before the piles carry any of their designed loading. The uplift pressure on the basement slab will be greater if bored piles are used since no heaving of the soil is caused by installing the piles, and if the basement slab is completed and attached to the piles soon after completing the excavation the swelling pressures on the underside of the slab will cause tension to be developed in the piles. This is particularly liable to happen where bored piles are installed from the ground surface before the excavation for the basement commences. Concreting of the pile shaft is terminated at the level of the underside of the basement slab and the construction of the basement slab usually takes place immediately after the completion of excavation and before any heave of the excavation can take place to relieve the swelling pressure. Generally, in any piled basement where *bored piles* are installed wholly in compressible clay the basement slab should be designed to withstand an uplift pressure equal at least to one-half of the dead and sustained imposed load of the superstructure.

Alternatively, a void can be provided beneath the basement slab by means of collapsible cardboard or plastics formers. The piles can be designed to be anchored against uplift or they can be sleeved over the zone of swelling. Anchoring the piles against uplift by increasing the shaft length to increase skin friction below the swelling zone is often the most economical solution to the problem. Where void formers made of cardboard or plastics are used to eliminate swelling pressure beneath the basement slab there is a risk of bio-degradation of the organic materials causing an accumulation of methane gas in the void. Venting the underside of the slab can be difficult and costly.

Providing an increased shaft length can be made more economical than sleeving the pile shaft within the swelling zone.

Fleming and Powderham<sup>(5.30)</sup> recommended that where piles are reinforced to restrain uplift the friction forces should not be underestimated and they suggest that if the forces are estimated conservatively it would be appropriate to reduce the load factors on the steel, perhaps to about 1.1.

Hydrostatic pressure will, of course, act on the basement slab in water-bearing soil. The piles must

be designed to carry the net full weight of the structure (*i.e.* the total weight less the weight of soil and soil water excavated from the basement).

When installing piles for 'top-down' construction the empty borehole from the pile head below the lowest basement floor slab up to the ground surface is supported by temporary casing. Steel columns are set on the pile heads and are surrounded with sand before the casing is extracted. Particular care is necessary in establishing the position of the pile borehole and maintaining vertically in drilling. If this is not done there could be considerable error in the position of the pile head and difficulties in locating the column in the design position.

The tolerance in pile position permitted by the Institution of Civil Engineers' Specification for Piling<sup>(2.1)</sup> is 75mm and the permitted deviation from the vertical of a bored pile is 1 in 75. Taking as an example a 3-storey basement with an overall depth from ground surface to pile head level (beneath the lowest floor slab) of 15m the centre of the pile could be 275mm from the design position. Consequently either the column set in its intended position in the superstructure would be off-plumb or the column would apply an eccentric load to the pile head. The permitted tolerances should be kept in mind when considering the pile diameter and the design of reinforcement to provide for eccentric loading.

#### **Piles driven through compressible clay to bedrock (Figure 5.34b)**

In this case soil swelling takes place at the base of the excavation followed by heave if driven piles are employed. As before, the heaved soil tends to slump away from the underside of the basement slab if the latter is concreted soon after pile driving. Any gap which might form will be permanent since the piles will not settle except due to a very small elastic shortening of the shaft. If bored piles are adopted, with a long delay between concreting the base slab and applying the superstructure loading to the piles, the pressure of the underside of the slab due to long-term soil swelling might be sufficient to cause the piles to lift from their seating on the rock. The remedy then is to provide a void beneath the slab, and to anchor the piles to rock or to sleeve them through the swelling zone.

#### **Piles driven through soft clay into stiff clay (Figure 5.34c)**

This case is intermediate between the first two. There is a continuing tendency for the heaved soft clay to settle away from the underside of the basement slab, because the settlement of the piles taking their bearing in the stiff clay is less than that caused by the reconsolidation of the heaved and disturbed soft clay. Uplift pressure occurs on the underside of the base slab if bored piles are used, and a design value equal at least to one-half of the combined dead and imposed load of the superstructure should be considered. Alternatively the effects of heave should be eliminated as described above.

#### **Piles driven into loose sand (Figure 5.34d)**

In this case it is presumed that the piles are driven through loose sand to an end bearing in deeper and denser sand. The slight heave of the soil caused by excavating the basement is an instantaneous elastic movement. No heave occurs because either pile driving causes some settlement of the ground surface due to densification, or a loss of ground results due to pile boring. When the superstructure load is applied to the piles they compress, but the soil follows the pile movement, and any soil pressures developed on the underside of the basement slab are relatively small. Hydrostatic pressure occurs in a water-bearing soil.

In all cases when designing piled basements the full design working load should be considered as acting on the piles and, in the case of piles bearing on rock or granular soils of low compressibility, the load on the underside of the basement slab can be limited to that caused by the soil pressure (*i.e.*, the overburden pressure measured from the ground surface around the basement) and hydrostatic pressure. Sometimes a tall building is constructed close to a low-rise podium (Figure 5.35) and both structures are provided with a piled basement. Piling beneath the podium is required to reduce differential movement between the heavily-loaded tower block and the podium. Uplift of the latter may occur if the weight of the superstructure is less than that of the soil removed in excavating for the basement. In such a case the piles must be anchored below the zone of soil swelling and designed to take or eliminate tension. The pressure on the underside of the podium basement slab will be equal to the swelling pressure exerted by the soil unless a void former is used to eliminate the pressure. A vertical movement joint passing completely through the basement and superstructure should be provided between the tower and podium to allow freedom of movement.

Measurements of the relative loads carried by the piles and the underside of the slab of a piled basement raft were described by Hooper<sup>(5.31)</sup>. The measurements were made during and subsequently to the construe-

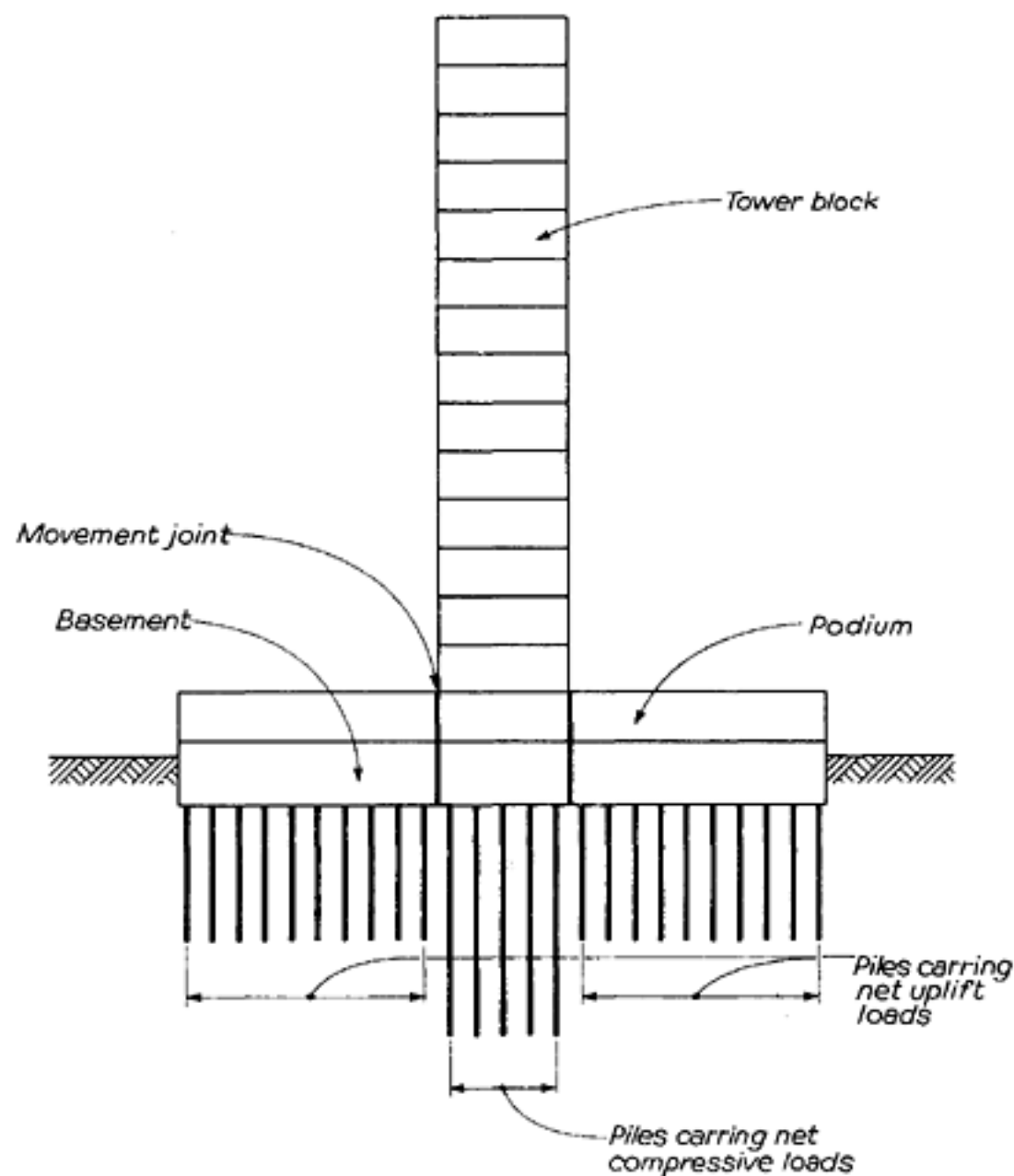


Fig. 5.35 Tower block and podium supported by piled basement

tion of the 31-storey building of the Hyde Park Cavalry Barracks in London. The 90m high building was constructed on the piled raft 8.8m below ground level. The 51 bored-and-cast-in-place piles supporting the raft had a shaft diameter of 910mm and an enlarged base 2400mm in diameter (Figure 5.36a). The piles were installed by drilling from ground level and concreting the shaft up to raft level before commencing the bulk excavation.

The weight of the building (including imposed load but excluding wind load) was calculated to be 228MN. The weight of soil removed when excavating through gravel on to the stiff London clay at raft level was 107MN, giving a net load to be transferred by the raft and piles to the London clay of 121MN, or a net bearing pressure at raft level of  $196\text{kN/m}^2$ . Load cells were installed in three of the piles to measure the load transferred from the raft to the pile shaft, and three earth pressure cells were placed between the raft and the soil to measure the contact pressures developed at this interface. Settlements of the raft at various points were also measured by means of levelling points installed at ground level.

The observations of pile loadings and contact pressures were used to estimate the proportion of the total load carried by the piles and the basement raft from the initial stages of construction up to 3 years after completing the building. The results of these calculations are shown in Figure 5.36b and are compared with the calculated total weight of the building at the various stages of construction. Hooper<sup>(5.31)</sup> estimated that at the end of construction 60% of the building load was carried by the piles and 40% by the underside of the raft. In the post-construction period there was a continuing trend towards the slow transfer of more load to the piles, about 6% of the total downward structural load being transferred to the piles in the three-year period.

### 5.10 The optimization of pile groups to reduce differential settlements in clay

Cooke *et al.*<sup>(5.32)</sup> measured the proportion of load shared between the piles and raft and also the distribution of load to selected piles in different parts of a 43.3m by 19.2m piled raft supporting a 16-storey building in London clay at Stonebridge Park. There were 351 piles in the group with a diameter of 0.45m and



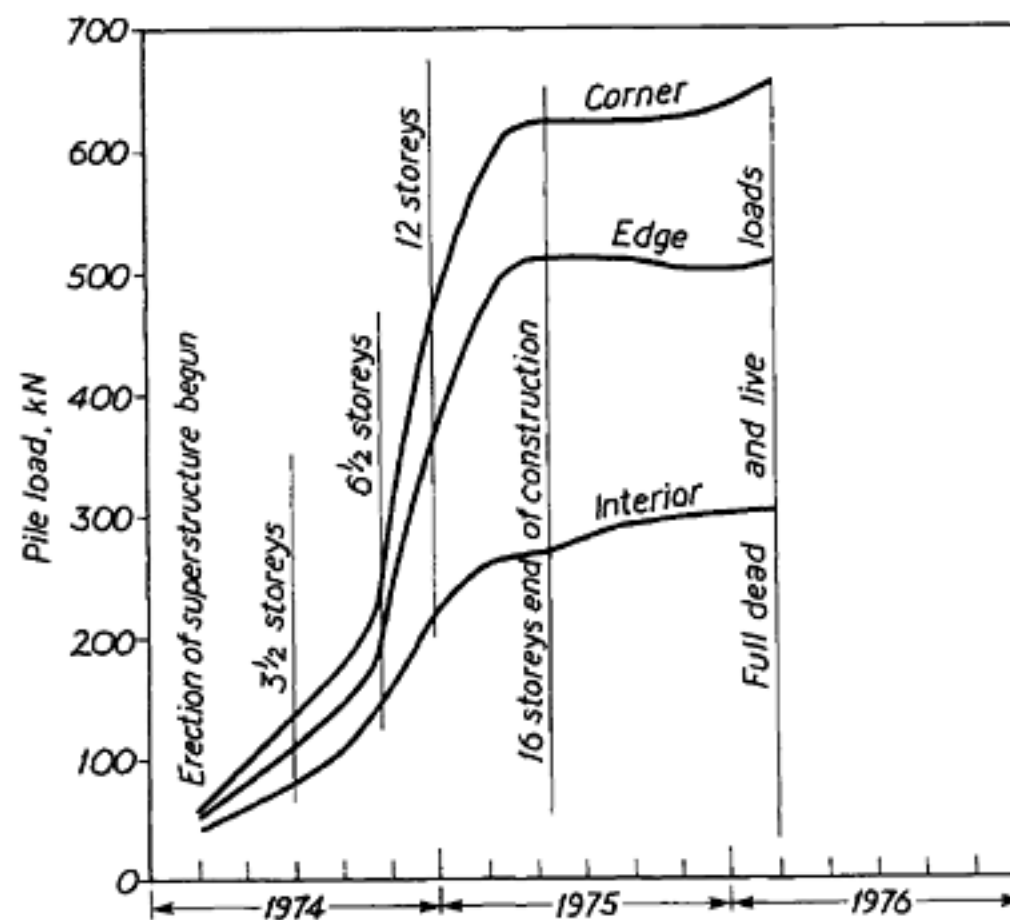


Fig. 5.37 Load distribution on piled raft in London clay (after Cooke et al.[\(5.32\)](#))

Load distribution between the piles is achieved through the continuous pile cap which must be designed to be stiff enough to achieve this. With perfect optimization differential settlement can be reduced to zero. The analysis to achieve optimization is complex and involves interaction factors discussed in 5.4. A computer is required to perform the necessary calculations. It is also necessary to check that the stress is not excessive on the shafts of the central piles which are designed to carry a high proportion of the load.

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## 5.12 Worked examples

### Example 5.1

Piles driven to a depth of 13.90m below ground level into a firm to stiff clay are arranged in a group consisting of ten rows of seven piles, each pile carrying a working load of 360kN. The profiles of deformation modulus  $E_u$  and coefficient of compressibility  $m_v$  are shown in [Figure 5.38](#). Calculate the settlement of the pile group.

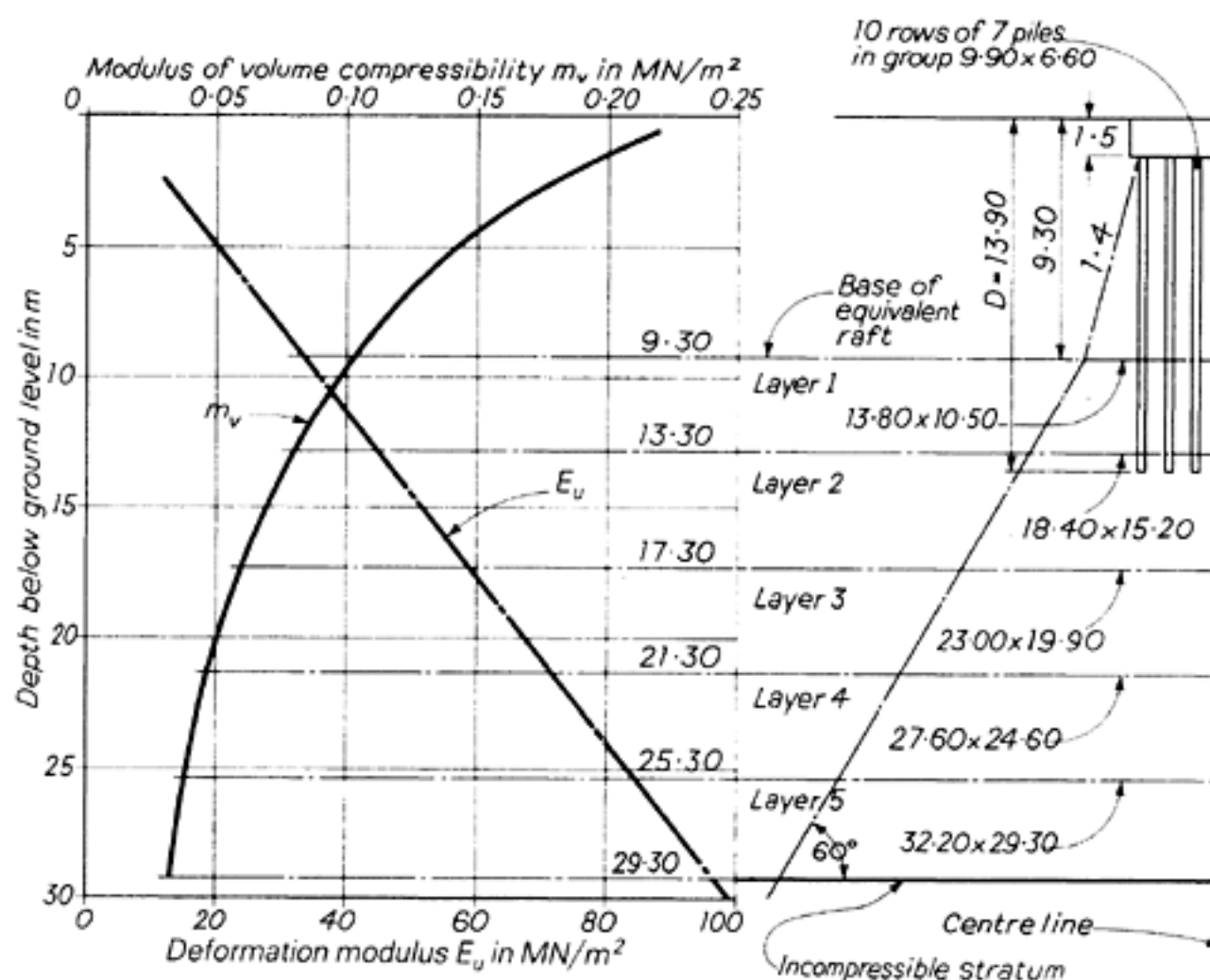


Fig. 5.38

For the arrangement of the piles shown in [Figure 5.38](#), the overall dimensions of the group are  $9 \times 1.1 = 9.9\text{m}$ , by  $6 \times 1.1 = 6.6\text{m}$ . The spread of the load shown in [Figure 5.3a](#) applies.

$$\text{Depth to centre of equivalent raft} = \frac{2}{3} \times 13.90 = 9.3\text{m}.$$

$$\begin{aligned} \text{Dimensions of equivalent raft} &= 6.60 + (\frac{1}{4} \times 7.80 \times 2) = 10.5\text{m} \\ \text{and } &6.60 + (\frac{1}{4} \times 7.80 \times 2) = 13.8\text{m} \end{aligned}$$

$$\text{Pressure at level of equivalent raft } q_n = \frac{70 \times 360}{10.5 \times 13.8} = 174\text{kN/m}^2$$

The settlements are calculated over the zone of soil down to the level of the incompressible stratum, that is at a depth of 20m below the base of the equivalent raft. It is convenient to divide the soil into five 4m layers commencing at 9.30m and extending to 29.30m. The immediate and consolidation settlements are then calculated for each layer.

*Immediate settlement in Layer 1*

From Figure 5.38, average  $E_u = 39\text{MN/m}^2$ . From Figure 5.11, for  $H/B = 4/10.5 = 0.38$ , and  $L/B = 13.8/10.5 = 1.3$ ,  $\mu_1 = 0.15$ , and for  $D/B = 9.3/10.5 = 0.9$ , and  $L/B = 1.3$ ,  $\mu_0 = 0.93$ . Therefore from equation 5.3.,

$$\text{immediate settlement} = \rho_i = \frac{0.15 \times 0.93 \times 174 \times 10.5 \times 1000}{39 \times 1000} = 6.5\text{mm}.$$

The settlements in the underlying four layers are calculated in a similar manner, the calculations for all five layers being tabulated thus.

Layer	B(m)	L(m)	$q_n$ (kN/m <sup>2</sup> )	$\mu_1$	$\mu_0$	$E_u$ (MN/m <sup>2</sup> )	$\rho_i$ (mm)
1	10.5	13.8	174	0.15	0.93	39	6.5
2	15.1	18.4	90	0.06	0.93	52	1.5
3	19.7	23.0	55	0.03	0.92	64	0.5
4	24.3	27.6	37	0.02	0.92	76	0.2
5	28.9	32.2	27	0.01	0.93	88	0.1
Total immediate settlement=8.8mm							

The immediate settlement can be checked from equation 5.4b because the deformation modulus increases linearly with depth. At level of equivalent raft  $E_u$  is 32MN/m<sup>2</sup> and at 20m below this level it is 97MN/m<sup>2</sup>. Therefore from equation 5.4a:

$$97 = 32 (1 + 20k/10.5)$$

$$k = 1.1$$

Dividing equivalent raft into four rectangles, each 6.9×5.25m. From Figure 5.14 for  $L/B = 1.3$ ,  $H/B = 20/5.25 = 3.8$  and  $k = 1.1$ ,  $I'_p$  is 0.13. From equation 5.4b:  $L/B = 6.9/5.25$

$$\text{Settlement at corner of rectangle} = \frac{174 \times 5.25 \times 0.13 \times 1000}{32 \times 1000} = 3.7\text{mm}$$

$$\text{Settlement at centre of equivalent raft} = 4 \times 3.7 = 14.8\text{mm}$$

*Oedometer settlement for Layer 1*

$$\text{Depth to centre of layer} = 9.3 + 2.0 = 11.3\text{m}.$$

From Figure 5.17 with  $L/B = 13.8/10.5 = 1.3$  and  $z/B = 2/10.5 = 0.19$ , stress at the centre of layer  $= 0.8 \times 174\text{kN/m}^2$ . From Figure 5.35 average  $m_v$  at centre of layer  $= 0.09\text{MN/m}^2$ . Therefore

$$\text{oedometer settlement from equation 5.5} = \rho_{oed} = \frac{0.09 \times 0.80 \times 174 \times 4 \times 1000}{1000} = 50.1\text{mm}.$$

The oedometer settlements for all five layers are calculated in a similar manner and are tabulated thus.

Layer	Depth to centre of layer (m)	$z$ (m)	$z/B$	$\sigma_z$	$m_v$ (MN/m <sup>2</sup> )	$\rho_{oed}$
1	11.3	2	0.19	0.80×174	0.09	50.1
2	15.3	6	0.57	0.51×174	0.07	24.8
3	19.3	10	0.95	0.33×174	0.05	11.5
4	23.3	14	0.33	0.22×174	0.04	6.1
5	27.3	18	0.71	0.15×174	0.04	4.2
Total oedometer settlement=96.7mm						

From [Figure 5.16](#) the depth factor for  $D/\sqrt{LB} = 9.30/\sqrt{13.8 \times 10.5} = 0.77$  , is 0.78 and for London clay the geological factor  $\mu_g$  is about 0.5. Therefore

$$\text{corrected consolidation settlement} = \rho_c = 0.5 \times 0.78 \times 96.7 = 37.7\text{mm.}$$

$$\text{Total settlement of pile group} = \rho_i + \rho_c = 8.8 + 37.7 = 46.5\text{mm.}$$

In practice a settlement between 30mm and 60mm would be expected.

### Example 5.2

Part of the jetty structure referred to in Example 4.4 carries bulk handling equipment with a total dead and imposed load of 6MN. Design a suitable pile group to carry this equipment and calculate the settlement under the dead and imposed loading. It has been calculated in Example 4.4 that a 450×450mm precast concrete pile driven to 7m below the sea bed could carry a working load of 250kN in compression with a safety factor on the ultimate resistance of a single pile of 3.1. For uniformity in design and construction it is desirable to adopt a pile of the same dimensions to carry the bulk handling plant. A group of forty-two piles arranged in seven rows of six piles should be satisfactory.

Spacing the piles at centres equal to three times the width the dimensions of the group are  $6 \times 1.35 = 8.10\text{m}$  by  $5 \times 1.35 = 6.75\text{m}$ . A suitable pile cap in the form of a thick slab would be  $10.5 \times 9.0 \times 1.25\text{m}$  deep. Take a depth of water of 12m and a height of 4m from water level to the underside of the pile cap.

Weight of pile group above sea bed level

$$= 9.81 \left( [10.5 \times 9.0 \times 1.25 \times 2.5] + \{42 \times 0.45^2 [(12 \times 1.5) + (4 \times 2.5)] \} \right) = 5\,233\text{kN.}$$

$$\text{Working load on each pile} = \frac{(6 + 5.233)}{42} \times 1\,000 = 267\text{kN.}$$

Safety factor on ultimate load of 777kN (Example 4.4)  $= 777/267 = 2.9$ , which is satisfactory.

Because the piles are driven into a uniform sand carrying their load partly in skin friction and partly in end bearing the distribution of load shown in [Figure 5.3a](#) applies.

$$\text{Depth below sea bed to equivalent raft} = \frac{2}{3} \times 7 = 4.67\text{m.}$$

Thus the dimensions of the equivalent raft are

$$L = 8.1 + (\frac{1}{4} \times 2 \times 4.67) = 10.4\text{m}$$

$$B = 6.75 + (\frac{1}{4} \times 2 \times 4.67) = 9.1\text{m.}$$

In calculating settlements it is only necessary to consider the dead and imposed loading from the bulk handling plant. The piles and pile cap settle immediately as they are constructed and the pile cap is finished to a level surface.

$$\text{Pressure on sand below raft due to weight of plant} = \frac{6 \times 1\,100}{10.4 \times 9.1} = 63\text{kN/m}^2.$$

$$\text{At level of raft, effective overburden pressure} = 1.2 \times 9.81 \times 4.67 = 55\text{kN/m}^2.$$

From [Figure 5.20](#) for a standard penetration test  $N$ -value of 15 blows/300mm,  $I_c$  is  $4 \times 10^{-2}$ .

Assume for the purposes of illustration that the previous overburden pressure was 75kN/m<sup>2</sup>. Then from equation 5.13b the immediate settlement for an effective pressure increase of 63kN/m<sup>2</sup> is:

$$\rho_i = 63 \times 9.1^{0.7} \times \frac{4 \times 10^{-2}}{3} = 3.9\text{mm}$$

From [Figure 5.21](#) the depth of influence  $z_f$  for  $B$  of 9.1m is 5m. This is less than the thickness of the compressible layer.

Hence the thickness factor,  $f_s$ , is unity. From equation 5.14a:

$$\text{shape factor} = \left( \frac{1.25 \times 10.4/9.1}{10.4/0.1 + 0.25} \right)^2 = 1.05$$

The time factor for settlement at 30 years and static loading condition is:

$$f_t = 1 + 0.3 + 0.2 \frac{\log 30}{3} = 2.5$$

Therefore, consolidation settlement

$$\begin{aligned} &= 1.05 \times 1 \times 2.5 \times 3.9 \\ &= 10.2\text{mm} \end{aligned}$$

The imposed loading would be intermittent in operation.

Checking from equation 5.7, for  $d_s$  greater than  $2B$  and  $L/B=1.1$ , [Figure 5.19](#) gives  $s=1.1$ .

$$\text{Immediate settlement} = \frac{1.1 \times 63}{15^{0.87} \left( 1 + 0.4 \times \frac{4.67}{9.1} \right)} = 5\text{mm}$$

Therefore the pile group would be expected to settle between 5mm and 10mm under the dead and imposed loading from the bulk handling equipment.

### Example 5.3

The driven-and-cast-in-place piles in Example 4.6 each carry a working load of 900kN and are arranged in a group of twenty rows of fifteen piles spaced at 1.60m centres in both directions. Calculate the settlement of the pile group using the static cone resistance diagram in [Figure 4.49](#). Length of pile group  $= 19 \times 1.6 = 30.4\text{m}$ . Width of pile group  $= 14 \times 1.6 = 22.4\text{m}$ . The transfer of load from the piles to the soft clay in skin friction is relatively small, and therefore the distribution of load shown in [Figure 5.3b](#) applies.

Depth to equivalent raft foundation  $= 3 \times 15 = 10\text{m}$  below the surface of the sand stratum or 22m below ground level, as shown in [Figure 5.39](#).

$$\text{Length of equivalent raft } L = 30.4 + (2 \times 10 \times \frac{1}{4}) = 35.4\text{m}$$

$$\text{Width of equivalent raft } B = 22.4 + (2 \times 10 \times \frac{1}{4}) = 37.4\text{m}$$

Pressure on soil beneath  
raft

$$= \frac{270 \times 1000}{35.4 \times 27.4} = 278\text{kN/m}^2.$$

The settlement can be calculated by the Schmertmann method. It is convenient to divide the cone resistance diagram shown in [Figure 4.49](#) into three layers between the base of the equivalent raft and

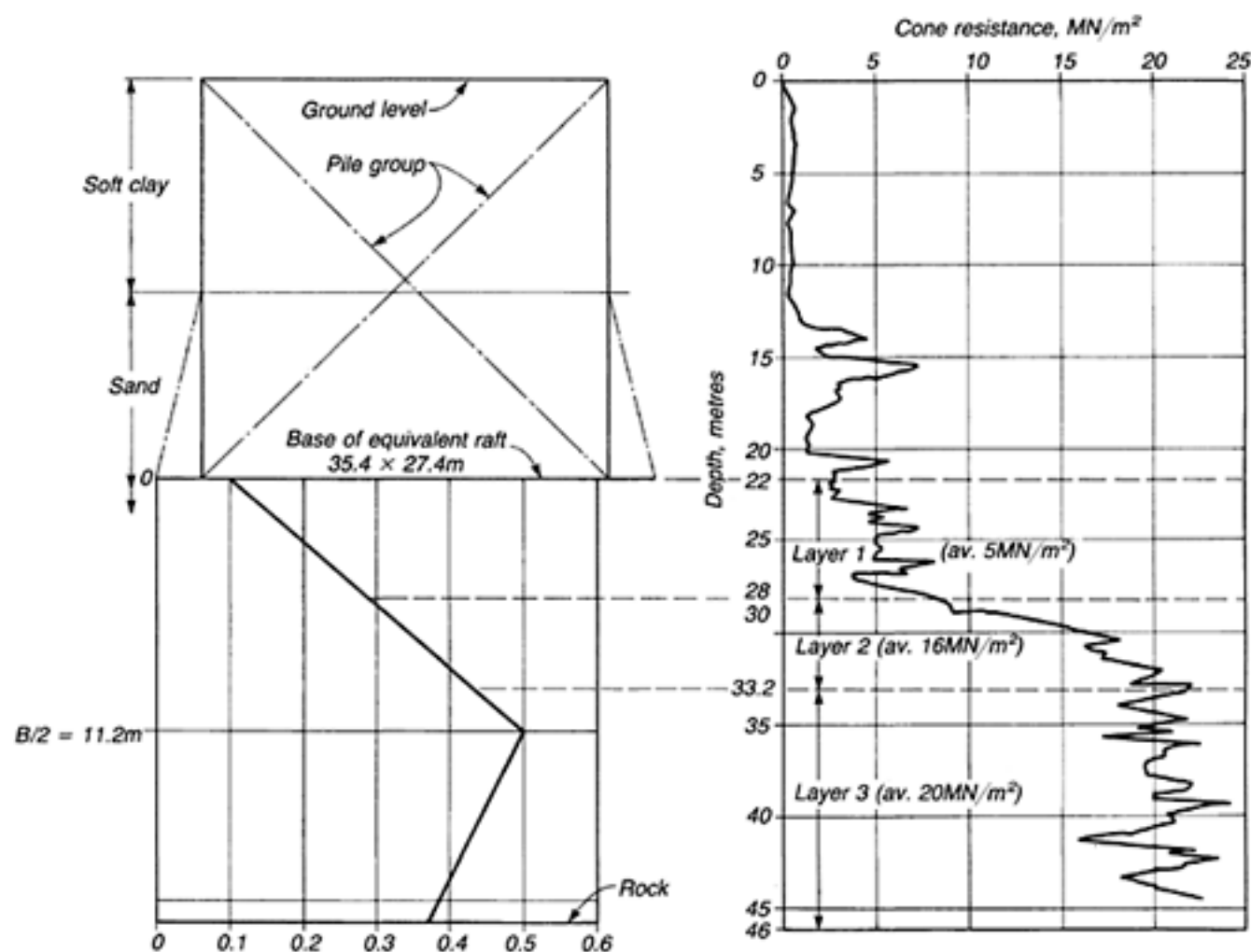


Fig. 5.39

rock head. The sub-division of these layers and the superimposition of the Schmertmann curves beneath the base of the raft are shown in [Figure 5.39](#). The settlement is calculated over a period of 25 years.  
From [Figure 5.39](#),

Layer	For $L/B=1$	$I_z$	For $L/B=10$
1	0.20	}	0.24
2	0.36		0.30
3a	0.46		0.39
3b	0.40		
Layer	For $L/B=1$	$E_d$	For $L/B=10$
1	$5 \times 2.5 = 12.5\text{MN/m}^2$		$5 \times 3.5 = 17.5\text{MN/m}^2$
2	$16 \times 2.5 = 40\text{MN/m}^2$		$16 \times 3.5 = 56\text{MN/m}^2$
3	$20 \times 2.5 = 50\text{MN/m}^2$		$20 \times 3.5 = 70\text{MN/m}^2$

For axisymmetric loading ( $L/B=1$ ) from equation 5.17:  
Uncorrected settlement is given by:

Layer 1 = $\frac{278 \times 0.20 \times 6 \times 1000}{12.5 \times 1000}$	=27mm
Layer 2 = $\frac{278 \times 0.36 \times 5.2 \times 1000}{40 \times 1000}$	=13mm
Layer 3a = $\frac{278 \times 0.46 \times 2.5 \times 1000}{50 \times 1000}$	=6mm
Layer 3b = $\frac{278 \times 0.4 \times 10.3 \times 1000}{50 \times 1000}$	=23mm
Total	<hr/> =69mm <hr/>

Similarly, for  $L/B>10$  the uncorrected settlements are:

Layer 1	=19mm
Layer 2	=9mm
Layer 3	=21mm
Total	<hr/> =49mm <hr/>

By interpolation the settlement for  $L/B = 1.3$  is 66mm  
Effective overburden pressure at base of raft  $= 9.81[(2 \times 1.9) + (10 \times 0.9) + (10 \times 0.9)] = 214\text{kN/m}^2$   
From equation 5.18,  $C_1 = 1 - 0.5 \times \frac{214}{278} = 0.62$ .  
From equation 5.19,  $C_2 = 1 + 0.2 \lg \frac{25}{0.1} = 1.48$ .  
Corrected settlement at 25 years  $= 0.62 \times 1.48 \times 66 = 61\text{mm}$

### Example 5.4

Nuclear reactors, their containment structures and ancillary units weighing 900MN are to be constructed on a base 70m×32m sited on 8m loose to medium-dense sand overlying a moderately strong sandstone. Rotary cored boreholes showed that below a thin zone of weak weathered rock the RQD value of the sandstone was 85% and the average unconfined compression strength was 14MN/m<sup>2</sup>. Design a suitable piled foundation and calculate the settlement. say between 50mm and 75mm

$$\text{Overall loading on base} = \frac{900}{70 \times 32} = 0.402 \text{ MN/m}^2.$$

Under this loading the settlement of the sand will be excessive. A piled foundation is required and relatively few large-diameter piles will be more economical than a large number of lightly loaded piles.

Adopt piles 1.5m in diameter taken 2m below weak weathered rock on to the moderately strong sandstone. It will be possible to permit loading of the concrete forming the bored piles to the maximum working stress on the concrete of the pile shaft. For 25-grade concrete, max. working stress  $= 0.25 \times 25 = 6.25 \text{ MN/m}^2$  (factor of safety on)  $q_{uc} = 2.2$  and since max.

working load on pile  $= \frac{1}{4} \times \pi \times 1.5^2 \times 6.25 = 11 \text{ MN}$ , required number of piles  $= 900/11 = 82$

A suitable arrangement of piles consists of fourteen rows of six piles placed at 5m centres in both directions.

Length of pile group  $L = 13 \times 5 = 65 \text{ m}$ .

Width of pile group  $B = 5 \times 5 = 25 \text{ m}$ .

The transfer of load in skin friction to the sand is relatively small and the piles can be regarded as end bearing on the rock.

The base of the equivalent raft will be shown in [Figure 5.3c](#).

$$\text{Overall loading at base of raft} = \frac{900}{65 \times 25} = 0.55 \text{ MN/m}^2.$$

From 4.7.3 for RQD of 85%, mass factor  $= 0.7$  and from 5.5 the modulus ratio of a well-cemented sandstone is 300,

deformation modulus of sandstone  $= E_d = 0.7 \times 300 \times 14 = 2940 \text{ MN/m}^2$ , say  $3000 \text{ MN/m}^2$ .

From [Figure 5.10](#) with  $H/B = \infty$ , and  $L/B = 65/25 = 2.6$ ,  $\mu_1 = 1.1$ , and with and  $L/B = 2.6$ ,  $\mu_0 = 0.95$ . From equation 5.3,  $D/B = (8 + 2)/25 = 0.4$

$$\text{settlement of foundation} = \frac{1.1 \times 0.95 \times 0.55 \times 25 \times 1000}{3000} = 5 \text{ mm}.$$

### Example 5.5

A site where the ground conditions consist of 5.5m of soft organic silty clay overlying 35m of stiff to very stiff over-consolidated clay followed by rock, is reclaimed by placing and compacting 4m of sand fill covering the entire site area. Six months after completing the reclamation a 12-storey building imposing an overall dead load and sustained imposed load of  $160 \text{ kN/m}^2$  on a ground floor area of  $48 \text{ m} \times 21 \text{ m}$  is to be constructed on the site. The average undrained shearing strength of the stiff clay stratum is  $90 \text{ kN/m}^2$  at the surface of the stratum, increasing to  $430 \text{ kN/m}^2$  at rockhead. Measurements of the deformation modulus and modulus of volume compressibility show a linear variation, with average values at the top and bottom of the stiff clay stratum as follows.

At top:  $E_u = 40 \text{ MN/m}^2$ ,  $m_v = 0.08 \text{ m}^2/\text{MN}$ .

At bottom:  $E_u = 120 \text{ MN/m}^2$ ,  $m_v = 0.04 \text{ m}^2/\text{MN}$ .

Design suitable piled foundations and estimate the settlement of the completed building.

Because of the heavy loading it is economical to provide large-diameter bored-and-cast-in-situ piled foundations. A suitable arrangement consists of fourteen rows of six piles ([Figure 5.40](#)). Trial-and-adjustment calculations show that a pile diameter of 1200mm is suitable. The pile spacing must be a minimum of 3 diameters, giving a spacing of at least 3.6m. Adopt a spacing of say 3.75m in both directions. Thus the dimensions of the pile group are  $5 \times 3.75 = 18.75 \text{ m}$  and  $13 \times 3.75 = 48.75 \text{ m}$ .

$$\text{Average load carried by piles} = \frac{48 \times 21 \times 160}{14 \times 6} = 1920 \text{ kN per pile}.$$

The central two rows of piles carry higher loads than the outer two rows on each side. A likely loading for the centre rows is  $2200 \text{ kN}$  per pile. The required penetration of the piles will be calculated on this loading. The exterior piles will be taken to the same depth but adopting a reduced diameter as required by the lesser loading.

The piles carry negative skin friction due to the consolidation of the soft clay under the imposed loading of the sand fill. At 6 months, settlement of the soft clay will be continuing at a very slow rate and it is appropriate to use [Figure 4.44](#) to calculate the negative skin friction in this layer.

Unit negative skin friction at top of layer

$$= 0.30 \sigma'_{v0} = 0.30 \times 9.81 \times 2.1 \times 4 = 4.7 \text{ kN/m}^2.$$

Unit negative skin friction at ground-water level (see [Figure 5.40](#))

$$= 0.30 \times 9.81 [(2.1 \times 4) + (1.6 \times 3)] = 8.8 \text{ kN/m}^2.$$

Unit negative skin friction at bottom of layer

$$= 0.30 \times 9.81[(2.1 \times 4) + (1.6 \times 3) + (0.6 \times 2.5)] = 43.3 \text{ kN/m}^2.$$

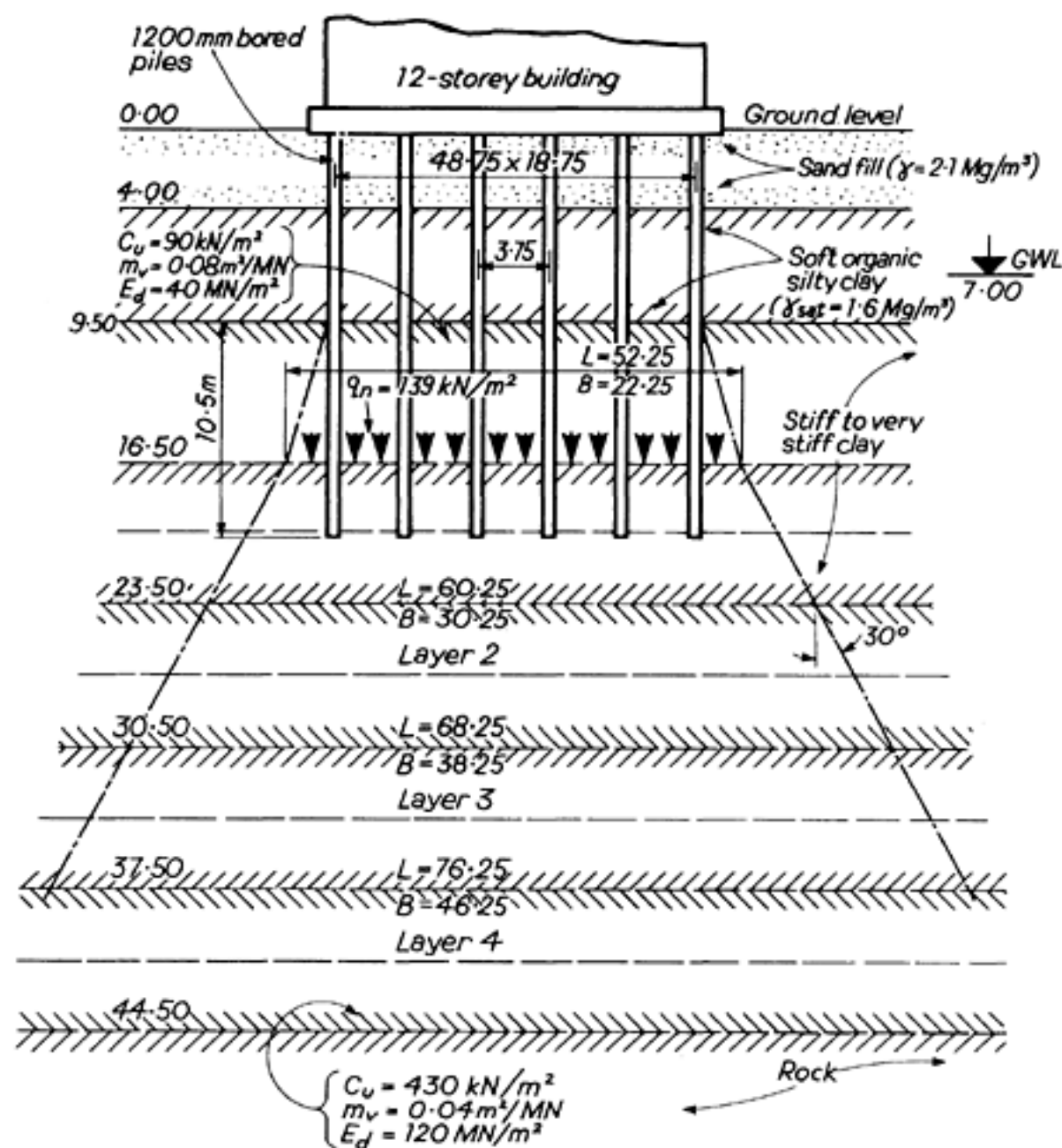


Fig. 5.40

Therefore total negative skin friction in soft clay

$$= \pi \times 1.2 \times \left[ \frac{1}{2}(4.7 + 8.8) \times 3 + \frac{1}{2}(8.8 + 43.3) \times 2.5 \right] = 746 \text{ kN.}$$

Because the pile will settle due to yielding of the stiff clay when the full working load is applied, the pile will move downwards relative to the lower part of the soft clay. Thus negative skin friction will be developed only over about 80% of the length within the soft clay. Thus

approx. total negative skin friction in soft clay =  $0.8 \times 746 = 597 \text{ kN}$ .

The negative skin friction in the sand can be calculated using Kulhawy's coefficients. Although the compacted sand fill is dense it will be loosened by pile boring to give a coefficient  $K_s$  of 1 and a value of  $\phi$  of  $30^\circ$ . From equation 4.13,

negative skin friction on pile in sand fill

$$= \frac{1}{2} \times 1 \times 9.81 \times 2.1 \times 4 \times \tan 30^\circ \times \pi \times 1.2 \times 4 = 359 \text{ kN.}$$

$$\text{Total negative skin friction on pile} = 359 + 597 = 956 \text{ kN.}$$

$$\text{Total applied load on piles in centre rows} = 956 + 2200 = 3156 \text{ kN.}$$

The required pile penetration depth is calculated on the basis of the building loading, with a check being made to ensure that the safety factor on the combined building load and negative skin friction is adequate.

Required ultimate pile resistance for overall safety factor of 2 (Section 4.6) =  $2 \times 2200 = 4400 \text{ kN}$

Take a trial penetration depth of 10m into the stiff clay stratum. At the pile base level and the average value of  $c_u$  on the shaft

is  $140 \text{ kN/m}^2$ . Thus  $c_b = 190 \text{ kN/m}^2$

$$\text{Ultimate base resistance} = \frac{1}{4} \times \pi \times 1.2^2 \times 9 \times 190 = 1\,935 \text{ kN.}$$

$$\text{Load to be carried in skin friction} = 4\,400 - 1\,935 = 2\,465 \text{ kN.}$$

The adhesion factor for a straight-sided pile can be taken as 0.45. Therefore from equation 4.5,

$$\text{total load} = 2\,465 = 0.45 \times 140 \times \pi \times 1.2 \times l,$$

from which  $l = 10.4\text{m}$  (say 10.5m) and the trial depth is satisfactory.

Checking the criterion of a safety factor of 3 in end bearing and unity in skin friction, allowable load

$= (\frac{1}{3} \times 1935) + 2465 = 3110\text{kN}$ , which roughly equals the building load plus the negative skin friction. Checking the overall safety factor on the combined loading,

$$\text{safety factor} = (1935 + 2465)/3156 = 1.4$$

This is satisfactory since the negative skin friction on the piles will not contribute to the settlement of the pile group. The transfer of load from the pile group to the soil will be as shown in [Figure 5.3b](#).

$$\begin{aligned} \text{Depth below ground level to base of equivalent raft} \\ = 4 + 5.5 + (\frac{2}{3} \times 10.5) = 16.5\text{m}. \end{aligned}$$

The dimensions of the equivalent raft are:

$$L = 48.75 + (\frac{2}{3} \times 10.5 \times 2 \times \frac{1}{4}) = 52.25\text{m}.$$

$$B = 18.75 + (\frac{2}{3} \times 10.5 \times 2 \times \frac{1}{4}) = 22.25\text{m}.$$

$$\text{Pressure on base of equivalent raft due to building load} = \frac{48 \times 21 \times 160}{52.25 \times 22.25} = 139\text{kN/m}^2.$$

*Calculating the immediate settlement*

At a level of equivalent raft,  $E_u = E_f = 65\text{MN/m}^2$

At rockhead,  $E_u = 120\text{MN/m}^2$

$$120 = 65(1 + 28k/22.25)$$

From equation 5.4a,  $k = 0.7$

Divide equivalent raft into four rectangles, each  $26.1 \times 11.1\text{m}$

From [Figure 5.14](#) for  $L/B = 26.1/11.1 = 2.3$ ,  $H/B = 28/11.1 = 2.5$  and  $k = 0.7$ ,  $I'_p$  is 0.14. From equation 5.4b

$$\text{Settlement at corner of rectangle} = \frac{139 \times 11.1 \times 0.14 \times 1000}{65 \times 1000} = 3.3\text{mm}$$

$$\text{Settlement at centre of equivalent raft} = 4 \times 3.3 = 13.2\text{mm}$$

*Calculating the consolidation settlement*

To calculate the settlement of the pile group due to the building loads only, the 28m layer of clay between the equivalent raft and rockhead is divided into four 7m layers.

*Oedometer settlement in Layer 1.*

From [Figure 5.17](#) with  $z/B = 3.5/22.2 = 0.16$  and  $L/B = 52.2/22.2 = 2.3$ , stress at centre of rectangle  $= 0.83 \times 139 = 118\text{kN/m}^2$ . Modulus of volume compressibility  $= 0.07\text{m}^2/\text{MN}$ . Then from equation 5.5,

$$\text{oedometer settlement uncorrected for depth factor} = \frac{0.07 \times 118 \times 7 \times 1000}{1000} = 57.8\text{mm}.$$

The settlements in the remaining layers are calculated similarly, the results for the four layers are tabulated as follows.

Layer	Depth to centre of layer (m)	$z$ (m)	$z/B$	$\sigma_z$ (kN/m <sup>2</sup> )	$m_v$ (MN/m <sup>2</sup> )	$\rho_{oed}$ (mm)
1	20.00	3.5	0.16	118	0.07	57.8
2	27.00	10.5	0.47	88	0.06	37.0
3	34.00	17.5	0.79	64	0.05	22.4
4	41.00	24.5	1.10	50	0.04	14.0

The above summation must be corrected by a depth factor which is given by [Figure 5.16](#), with  $D/\sqrt{LB}$   
 $= 16.5/\sqrt{52.25 \times 22.25} = 0.48$  and  $L/B = 2.35$ , as  $\mu_d = 0.85$ .

To obtain the consolidation settlement  $\rho_c$  the summation is also multiplied by the geological factor  $\mu_g$ , which is 0.5 for an overconsolidated clay. Therefore

$$\text{total consolidation settlement} = 0.85 \times 0.5 \times 131.2 = 55.8\text{mm}.$$

$$\text{Total settlement of pile group due to building load only} = \rho_t + \rho_c = 13.2 + 55.8 = 69.0\text{mm}.$$

To this figure must be added the consolidation settlement of the stiff clay due to the sand filling. The immediate settlement is not taken into account since this will have taken place before commencing the construction of the building.

Oedometer settle due to 4m of sand fill for an average  $m_v$  of 0.06m<sup>2</sup>/MN in clay layer

$$= \frac{0.06 \times 9.81 \times 2.1 \times 4 \times 1000}{1000} = 4.9\text{mm}.$$

Correcting for the geological factor,

$$\rho_c = 0.5 \times 4.9 = 2.4\text{mm}.$$

A time-settlement calculation would show that about one-third of this settlement would be complete before completing the pile installation. Thus

settlement of 12-storey building due to combined loading from building and sand layer

$$= 69.0 + (\frac{1}{3} \times 2.4) = 70.6\text{mm}.$$

Therefore it would be reasonable to assume that the final settlement of the building would be between 50 and 100mm.

It will be noted that the negative skin friction on the piles was not added to the loading on the equivalent raft when calculating the settlement of the building. However, it is necessary to check that the individual piles will not settle excessively under the combined building load and negative skin friction.

Maximum load on pile=3156kN. If shaft friction on pile is fully developed, the end-bearing load is 3156–2465=691kN, and thus

$$\text{end-bearing pressure} = \frac{691}{\frac{1}{4}\pi \times 1.2^2} = 611\text{kN/m}^2.$$

$$\text{Ultimate unit base resistance} = 9 \times 190 = 1710\text{kN/m}^2.$$

From equation 4.17, with  $K = 0.01$

$$\rho_i = 0.01 \times \frac{611}{1710} \times 1200 = 4\text{mm}$$

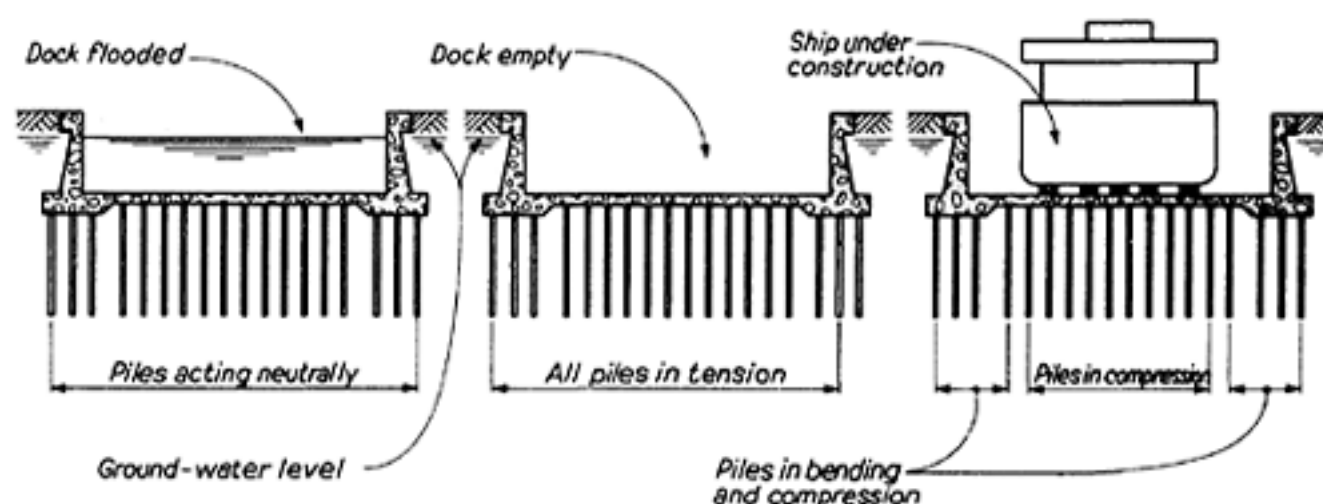
Therefore individual piles will not settle excessively and the critical factor is the overall settlement of the complete pile group, for which a movement of 50 to 100mm over a long period of years is by no means excessive.

## CHAPTER 6

# The design of piled foundations to resist uplift and lateral loading

### 6.1 The occurrence of uplift and lateral loading

When vertical piles are installed beneath buoyant structures such as dry docks, basements, and pumping stations they are required to resist uplift loads. Where the hydrostatic pressure always exceeds the downward loading, as in the case of some underground tanks and pumping stations, the anchorages are permanently under tension and cable anchors may be preferred to piles. However, in the case of the shipbuilding dock floor in [Figure 6.1](#), for example, the anchorages may be under tension only when the dock is unwatered before the commencement of shipbuilding. As the loading on the floor from ship construction increases to the stage at which the uplift pressure is exceeded, the anchor piles are required to carry compressive loads. Cable anchors might not then be suitable if the dock floor was underlain by soft or loose soil.



*Fig. 6.1 Tension/compression piles beneath floor of shipbuilding dock*

Vertical piles are used to restrain buildings against uplift caused by the swelling of clay soils. Swelling can occur, for example, when mature trees are removed from a building site. The desiccated soil in the root zone of the trees gradually absorbs water from the surrounding clay, and the consequent swelling of the clay may amount to an uplift of 50 to 100mm of the ground surface, causing severe damage to buildings sited over the root zone. In sub-tropical countries where there is a wide difference in seasonal climatic conditions, *i.e.* a hot dry summer and a cool wet winter, the soil zone affected by seasonal moisture changes can extend to a depth of several metres below the ground surface. In clay soils these changes cause the ground surface to alternately rise and fall with a differential movement of 50mm or more. The depth to which these swelling (or alternate swelling and shrinkage) movements can occur, usually makes the use of piled foundations taken below the zone of soil movements more economical and technically more suitable than deep strip or pad foundations.

Vertical piles must have a sufficient depth of penetration to resist uplift forces by the development of skin friction in the soil beneath the zone of soil movements ([Figure 6.2](#)). Uplift on bored piles can be reduced by casting the concrete in the upper part of the pile within a smooth polyvinylchloride (pvc) sleeve, or by coating a precast concrete or steel tubular pile with soft bitumen (see [4.8.3](#)). Uplift can be further reduced by supporting the superstructure clear of the ground surface, or by providing a compres-

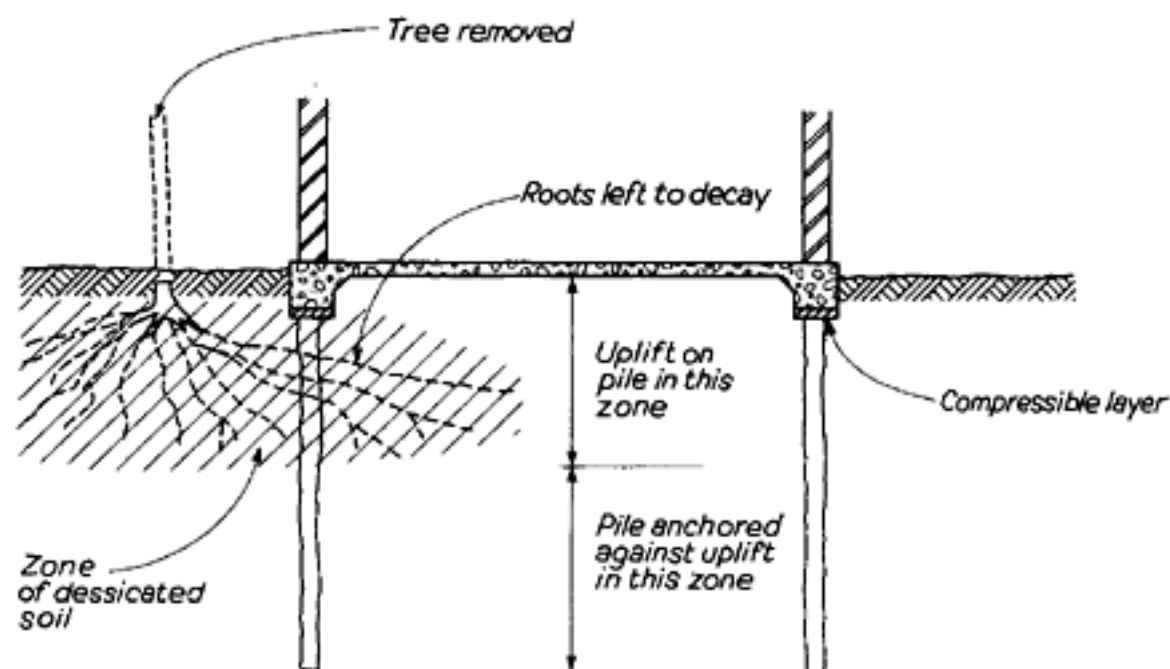


Fig. 6.2 Uplift on pile due to swelling of soil after removal of mature tree

sible layer beneath pile caps and ground beams (see [Figure 7.14](#)). Piles in large groups may also be lifted due to ground heave, as described in [Section 5.7](#).

In countries where frost penetrates deeply below the ground surface, frost expansion of the soil can cause uplift on piles, resulting in severe effects in 'permafrost' regions, as described in [Section 9.4](#). Floating ice on lakes and rivers can jam between piles in groups causing them to lift when water levels rise or when the ice sheet buckles.

The most frequent situation necessitating design against lateral and uplift forces occurs when the piles are required to restrain forces causing the sliding or overturning of structures. Lateral forces may be imposed by earth pressure ([Figure 6.3a](#)), by the wind ([Figure 6.3b](#)), by earthquakes, or by the traction of braking vehicles ([Figure 6.3c](#)). In marine structures lateral forces are caused by the impact of berthing ships ([Figure 6.4](#)), by the pull from mooring ropes, and by the pressure of winds, currents, waves and floating ice. A vertical pile has a very low resistance to lateral loads and, for economy, substantial loadings are designed to be resisted by groups of inclined or raking piles (sometimes referred to as 'batter' piles). Thus in [Figure 6.5](#) the horizontal force can be resolved into two components, producing an axial compressive force in pile A and a tensile force in pile B. It is usual to ignore the restraint offered by the pile cap; thus the magnitude of the each component is obtained from a simple triangle of forces as shown. Where lateral forces are transient in character, e.g. for wind loadings, they may be permitted to be carried wholly or partly by the pile cap where this is bearing on the ground (see [7.8](#)). If raking piles are installed in fill or compressible soil which is settling under its own weight or under a surcharge pressure, considerable bending stresses can be induced in the piles, requiring a high moment of resistance to withstand the combined axial and bending stresses ([Figure 6.55](#)).

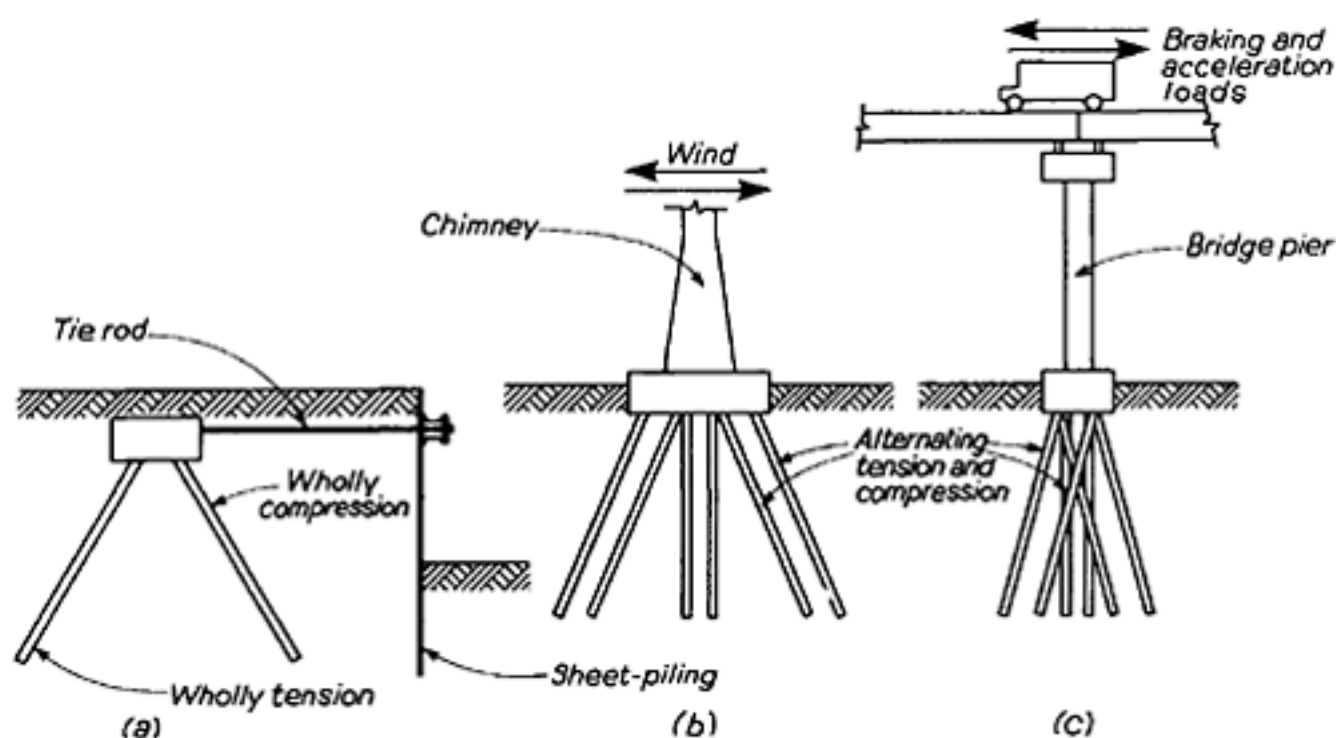


Fig. 6.3 Raking piles to resist overturning forces

- (a) Piled anchorage to tie rods restraining sheet-piled retaining wall
- (b) Raking piles to withstand wind forces on chimney
- (c) Raking piles to withstand traction forces from vehicles on bridge

## 6.2 Uplift resistance of piles

### 6.2.1 General

The simplest method of restraining piles against uplift is to employ a pile shaft that is sufficiently long to take the whole of the uplift load in skin friction. However, where there is rock beneath a shallow soil overburden it may not be possible to drive the piles deeply enough to mobilize the required skin-frictional resistance. In such cases the shaft resistance must be augmented by adding dead weight to the pile to overcome the uplift load, or by anchoring the pile to the rock. Adding dead weight to counteract uplift loading is not usually feasible or economical. The piles may be required to carry alternating uplift and compressive loads, in which case the added dead weight

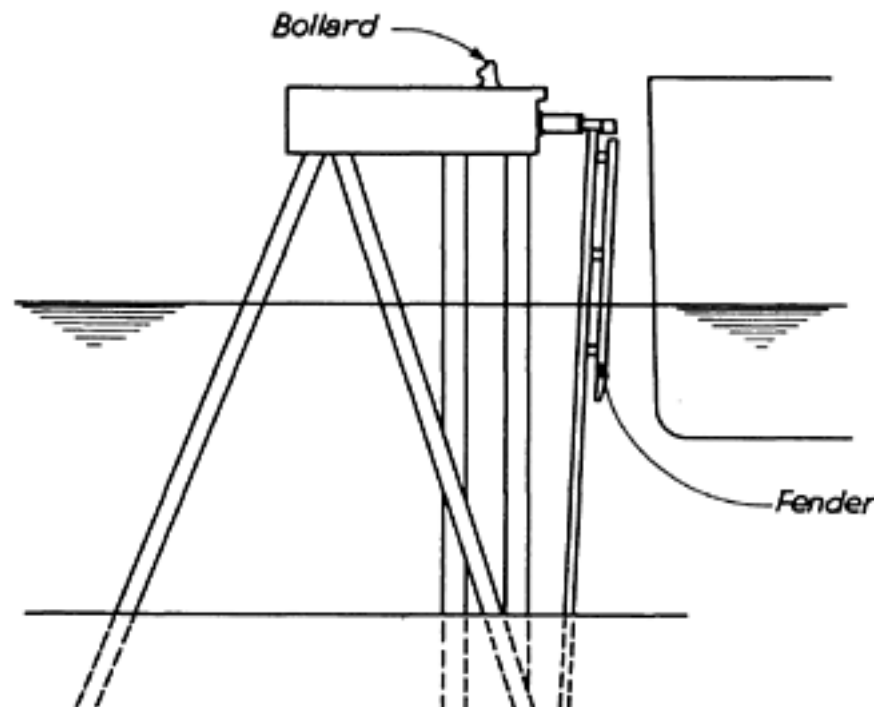


Fig. 6.4 Raking and vertical piles in breasting dolphin

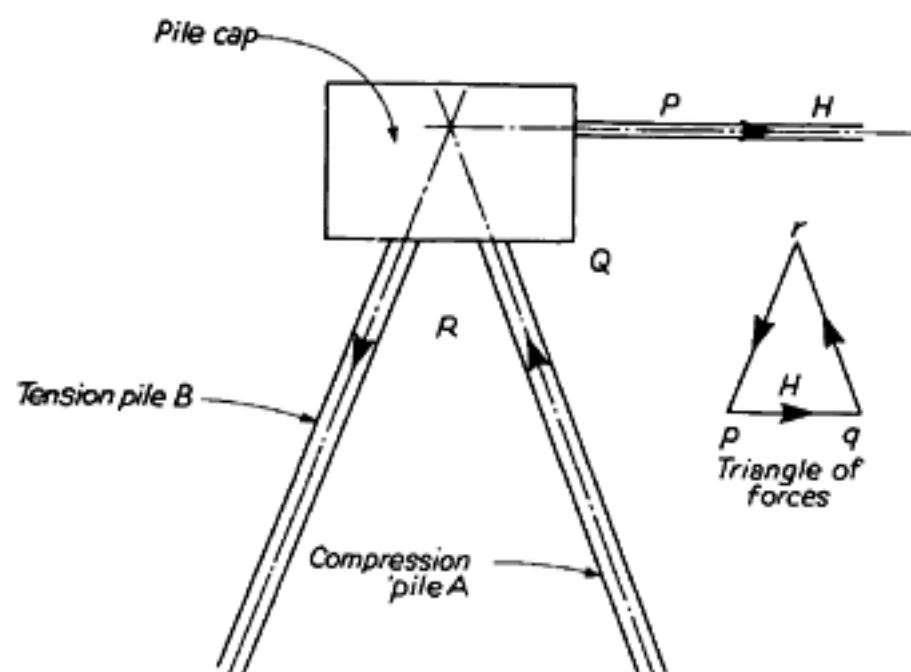


Fig. 6.5 Restraint of horizontal force by raking piles

would result in a large increase in the compressive loading. In the case of shipbuilding dock floors ([Figure 6.1](#)), dead weight in the form of a thick floor would add considerably to the construction costs, and in piled dolphins ([Figure 6.4](#)) the provision of a massive pile cap could make a substantial addition to the load on the compression rakers. Experience has shown that anchors in the form of grouted-in bars, tubes, or cables are the most economical means of providing the required uplift resistance for piles taken down to a shallow rock layer.

### 6.2.2 The uplift resistance of friction piles

The resistance of straight-sided piles in skin friction to statically applied uplift loads is calculated in exactly the same way as the skin friction on compression piles, and the calculation methods given in

Sections 4.2 to 4.5 can be used. However, for cyclic loading the skin friction resistance is influenced by the rate of application of the load and the degree of degradation of the soil particles at the interface with the pile wall. Some published test results show that the uplift resistance in skin friction on a pile shaft is less than its resistance in compression, possibly up to 50% less in a granular soil. In the short term, the uplift resistance of a bored pile in clay is likely to be equal to its skin-frictional resistance in compression; however, Radharkrishna and Adams<sup>(6.1)</sup> noted a 50% reduction in the uplift resistance of cylindrical augered footings and a 30 to 50% reduction in belled footings in clay when sustained loads were carried over a period of 3 to 4 months. It was considered that the reduction in uplift was due to a loss of suction beneath the pile base and the dissipation of negative pore pressures set up at the initial loading stage. These authors pointed out that such reductions are unlikely for piles where the depth/width ratio is greater than 5.

St John *et al.*<sup>(6.2)</sup> calculated that the first pull on a previously unloaded pile in clay would give an uplift resistance equal to the ultimate resistance in skin friction. However under conditions of cyclic loading, or creep caused by sustained loading, the uplift skin friction could fall from the peak to the residual value, particularly in the case of long piles. In the latter case it was stated that irreversible uplift movements will not occur until the peak cyclic shear stress is 80% of the ultimate static capacity. For short piles under sustained uplift loading, consideration of creep may reduce the uplift resistance to 50% of the resistance in static compression.

Differences between skin friction capacity in compression and uplift on piles in cohesionless soils are important because of the relatively low values of unit resistance compared with values obtainable in stiff clays. St John *et al.*<sup>(6.2)</sup> attributed the loss in uplift capacity under cyclic tension loading to changes in the packing of the soil grains and the attrition of the grains. The mineralogy of the soil is an important factor. Poulos<sup>(6.3)</sup> observed that the skin friction resistance in compression of calcareous sands is reduced by cyclic loading to only 50% of the value for static resistance, compared with a 25% reduction for silica sands. This order of degradation is given by cyclic shear displacements greater than the amount required to develop the ultimate static capacity. The reduction in skin friction is a function of the magnitude of displacement of the pile relative to the soil. Peak skin friction resistance in a granular soil is reached at a displacement of about 1% of the pile diameter.

Cyclic degradation effects can be quantified as discussed by Poulos<sup>(4.43)</sup>.

Tests by Peuch<sup>(6.4)</sup> in loose sandy silts and sands showed that the ultimate skin friction in cyclic uplift loading was 30% to 40% less than the static sustained loading. The various researches described above have been in connection with driven piles supporting offshore structures. In the absence of other published information the same reductions for uplift and cyclic loading should be allowed for cast-in-place piles.

Generally a safety factor of 3 on the ultimate resistance as calculated for compression is usually adequate to determine the uplift capacity of piles in cohesive and cohesionless soils, bearing in mind that an upward movement of only 0.5 to 1.0% of the pile width is required to mobilize the peak resistance. Because this movement is very small the tapered pile is unlikely to show any significant difference between the skin-frictional resistance in compression and tension, although it is clear that the latter falls off very quickly as the tapered pile is lifted out of the ground. Skin friction to resist uplift load in granular soils can be increased greatly by skin grouting as described in Section 4.3.4.

Where vertical piles are arranged in closely-spaced groups the uplift resistance of the complete group may not be equal to the sum of the resistances of the individual piles. This is because, at ultimate-load conditions, the block of soil enclosed by the pile group is lifted. The manner in which the load is transferred from the pile to the soil is complex and depends on the elasticity of the pile, the layering of the soil, and the disturbance to the ground caused by installing the pile. A spread of load of 1 in 4 from the pile to the soil provides a simplified and conservative estimate of the volume of a cohesionless soil available to be lifted by the pile group, as shown in Figure 6.6. For simplicity in calculation, the weight of the pile embedded in the ground is assumed to be equal to that of the volume of soil it displaces. If the weight of the block of soil is calculated by using a diagram of the type shown in Figure 6.6, then the safety factor against uplift can be taken as unity, since skin friction around the periphery of the group is ignored in the calculation. The submerged weight of the soil should be taken below ground-water level.

In the case of cohesive soils the uplift resistance of the block of soil in undrained shear enclosed by the pile group in Figure 6.7 is given by the equation

$$Q_u = (2LH + 2BH) \bar{c}_u + W \quad \dots(6.1)$$

where  $Q_u$  is the total uplift resistance of the pile group,  $L$  and  $B$ , are the overall length and width of the group, respectively,  $H$  is the depth of the block of soil below pile cap level,  $\bar{c}_u$  is the value of average undisturbed undrained cohesion of the soil around the sides of the group, and  $W$  is the combined weight of the block of soil enclosed by pile group plus the weight of the piles and pile cap.

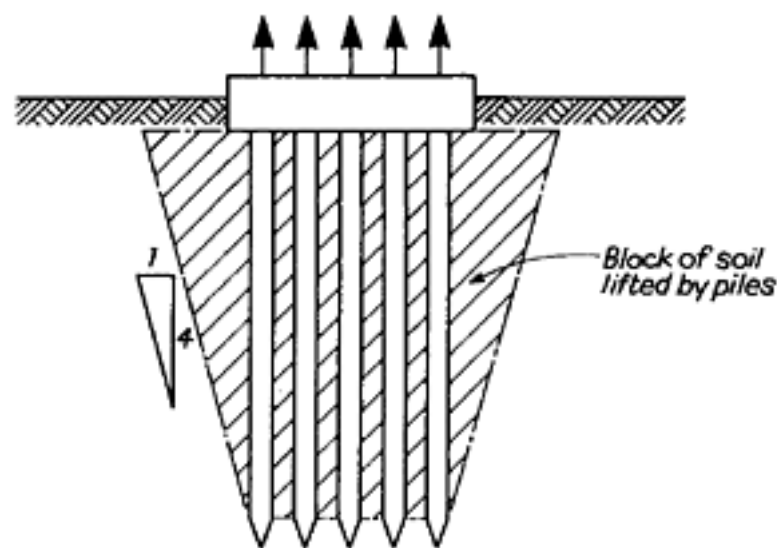


Fig. 6.6 Uplift of group of closely-spaced piles in cohesionless soils

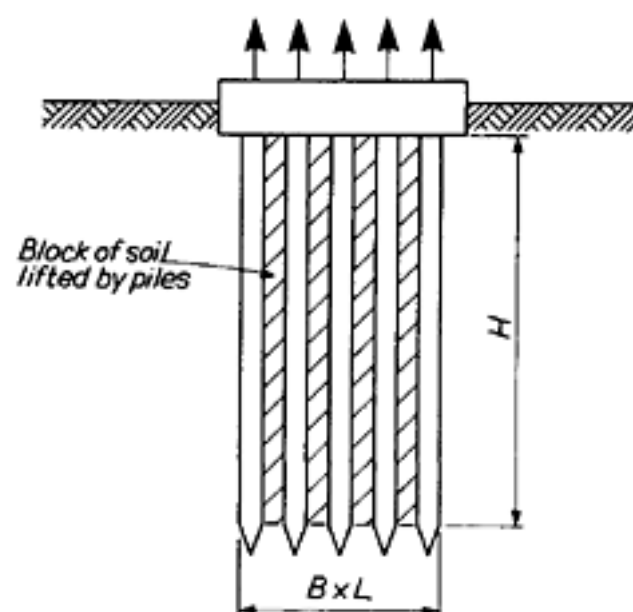


Fig. 6.7 Uplift of group of piles in cohesive soils

A safety factor of 2 should be used with equation 6.1 to allow for the possible weakening of the soil around the pile group caused by the method of installation. For long-term sustained loading a safety factor of 2.5 to 3 would be appropriate. If either of the above two methods is used to calculate the combined uplift resistance of a pile group, the allowable resistance must not be greater than that provided by the sum of the skin-frictional resistance of the individual piles in the group divided by the appropriate safety factor.

When calculating the uplift of a single pile or group of piles using the Eurocode 7 recommendations it should be noted that where the uplift load is derived from live loading, such as wind, the live load should be increased by a partial factor of  $\gamma_F=1.3$  (or higher and as much as 1.5 for uplift on masts or towers). Where the dead load of the structure is favourable in resisting uplift and critical it is reduced by a partial factor  $\gamma_F=0.95$ .

The uplift capacity of a single pile is calculated from the results of field tests or laboratory tests on soils and rocks using the same procedure as described for compression loading in [Chapter 4](#). Particular emphasis is placed in the Eurocode [Chapter 7](#) on possible weakening of skin friction resistance to uplift where this is in the form of cyclic loading or reversals of load. The prescribed dimensions of the block of soil resisting uplift on a pile group as shown in [Figure 7.7a](#) of the code should be noted.

### 6.2.3 Piles with base enlargements

When bored piles are constructed in clay soils, base enlargements can be formed to anchor the piles against uplift. The enlargements are made by the belling tools described in [Section 3.3.1](#). Enlargements cannot be formed in cohesionless soils unless the borehole is drilled with the support of a bentonite slurry. The size and stability of an enlargement made in this way is problematical. Full-scale loading tests are essential to prove the reliability of the bentonite method for any particular site. Reliable predictions cannot be made of the size and shape of base enlargements formed by hammering out a bulb of concrete at the bottom of a driven-and-cast-in-situ pile as described in 2.3.2. End enlargements formed on precast concrete or steel piles, although providing a substantial increase in compressive

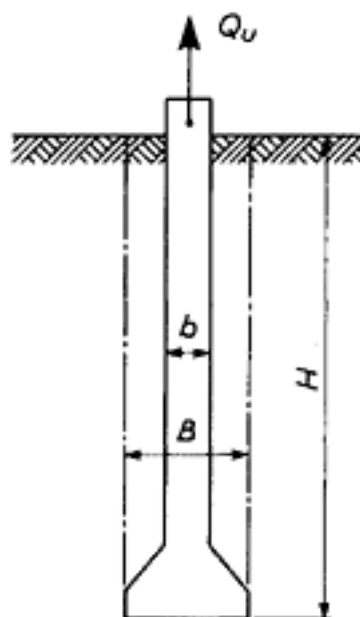


Fig. 6.8. Uplift of single pile with base enlargement in cohesive soil ( $\phi = 0$ )

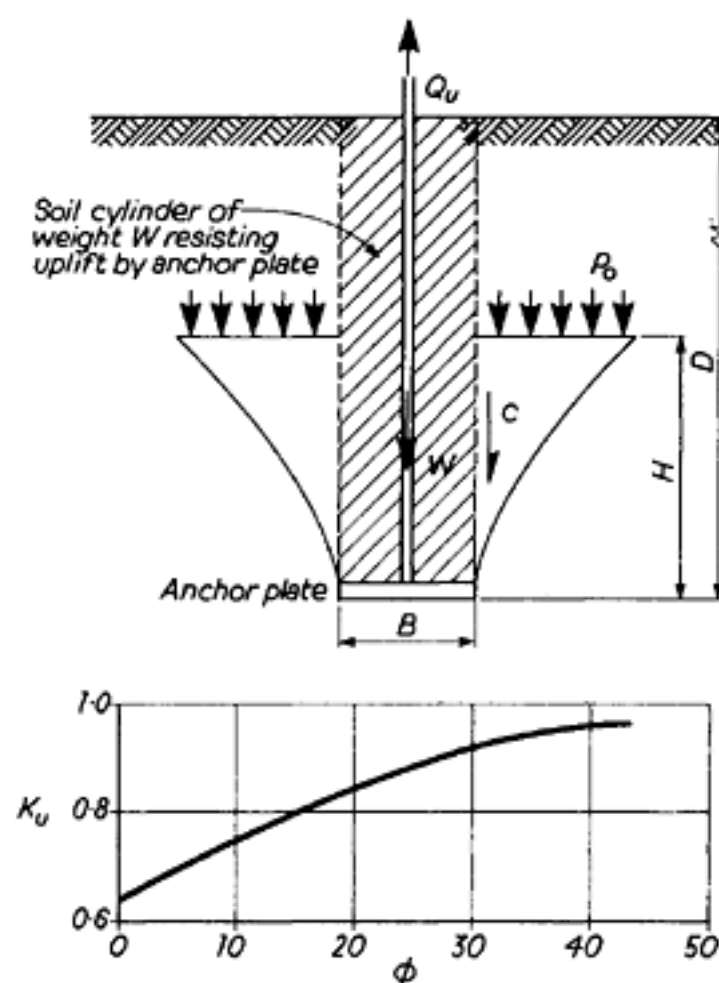


Fig. 6.9 Uplift of circular plate in partly-cohesive ( $c - \phi$ ) or cohesionless ( $c=0$ ) soil (after Meyerhof and Adams<sup>(6.5)</sup>)

resistance when driven to a dense or hard stratum, do not offer much uplift resistance since a gap of loosened soil is formed around the shaft as the pile is driven down.

Meyerhof and Adams<sup>(6.5)</sup> investigated the uplift resistance of a circular plate embedded in a partly cohesive ( $c - \phi$ ) soil and established the formula

$$Q_u = \pi c B H + s \times \frac{1}{2} \pi \times \gamma \times B (2D - H) H K_u \tan \phi + W \quad \dots(6.2)$$

where  $Q_u$  is the uplift resistance of the plate,  $B$  is the diameter of the plate,  $H$  is the height of the block of soil lifted by the

pile (Figure 6.9),  $c$  is the cohesive strength of the soil,  $s$  is a shape factor (see below),  $\gamma$  is the density of the soil (the submerged density being taken below ground-water level),  $D$  is the depth of the plate,  $K_u$  is a coefficient obtained from

Figure 6.9,  $\phi$  is the angle of shearing resistance of the soil, and  $W$  is the weight of the soil resisting uplift by the plate.

If equation 6.2 is adapted to a pile with an enlarged base the weight of the pile is taken in conjunction with the weight of the soil when calculating  $W$ .

It will be noted that for deeply-embedded plates or pile enlargements,  $H$  does not extend up to ground level and its value can

be obtained from tests made by Meyerhof which gave the following results.

$\phi$	20°	25°	30°	35°	40°	45°	48°
$H/B$	2.5	3	4	5	7	9	11

The shape factor  $s$  for deep foundations (including piles) is equal to  $1+mH/B$ , where  $m$  depends on the angle of shearing resistance  $\phi$  of the soil. Meyerhof's values of  $m$  and the maximum permissible values of the shape factor are as follows.

$\phi$	20°	25°	30°	35°	40°	45°	48°
$m$	0.05	0.1	0.15	0.25	0.35	0.50	0.60
max. $s$	1.12	1.30	1.60	2.25	3.45	5.50	7.60

The value of  $Q_u$  calculated from equation 6.2 must not exceed the combined resistance of the enlarged base (considered as a buried deep foundation) and the pile skin friction. Thus for a partly-cohesive soil the base resistance can be calculated using Terzaghi and Peck's coefficients  $N_c$ ,  $N_q$ , and  $N_\gamma$  (Figure 4.27).

$$Q_{ub} = \frac{1}{4}\pi(B^2 - b^2)[1.3c_b N_c + \gamma d(N_q - 1) + 0.4\gamma B N_\gamma] \quad \dots (6.3a)$$

For a fully-cohesive soil, (i.e.  $\phi = 0$ ; Figure 6.8),

$$Q_{ub} = \frac{1}{4}\pi(B^2 - b^2) \times 9 \times c_b \quad \dots (6.3b)$$

where  $B$  is the diameter of the enlarged base,  $b$  is the diameter of the shaft,  $c_b$  is the cohesion of the soil at the base of the pile,  $\gamma$  is the density of the soil (the submerged density being taken below ground-water level), and  $d$  is the depth to the base of the pile.

The skin friction on the pile shaft is calculated as described in Sections 4.2.3, 4.3.6, and 4.7.3. The shaft length is taken as the overall depth of the pile, from which the depth of the enlargement and any allowance made for the shrinkage of the soil away from the pile at the ground surface are deducted. Where piles in clay have to carry long-term sustained uplift loading, and the ratio of the depth of these piles to the width of the enlarged base is less than 5, the uplift resistance, as calculated by equation 6.2 or 6.3, should be reduced by one-half.

Where piles with base enlargements are installed in groups the uplift resistance of the group can be calculated as described in Section 5.2.1.

## 6.2.4 Anchoring piles to rock

Rock anchors are provided for tension piles when the depth of soil overburden is insufficient to develop the required uplift resistance on the pile shaft in skin friction. In weak rocks such as chalk or marl it is possible to drive piles into the rock, or to drill holes for bored piles so that the skin friction can be obtained on the pile shaft at its contact surface with the rock.

However, driving piles into a strong rock achieves only a small penetration and so shatters the rock that no worthwhile skin friction can be obtained. The cost of drilling into a strong rock to form a bored pile is not usually economical compared with that of drilling smaller and deeper holes for anchors as described below, although drilling-in large-diameter piles to carry ship berthing forces in marine structures is sometimes practised (see Section 8.2).

Anchorage in rock are formed after driving an open-ended hollow section pile to seat the toe of the pile into the rock surface. The pile must not be driven too hard at this stage as otherwise the toe will buckle, thus preventing the entry of the cleaning-out tools and the anchor drilling assembly. The soil plug within the pile is removed by baling, washing, or 'airlifting'. If a bored pile is to be anchored, the borehole casing is drilled below rock level to seal off the overburden. All the soil within the piling tube is cleaned out, and drilling pipes with centralizers are lowered down to the rock level. The anchor hole is then drilled to the required depth. The cuttings washed out of the hole are removed by reverse circulation up the drilling pipe or through a conductor tube up to the surface. The anchor, which can consist of a high-tensile steel bar or a stranded cable, is fed down the hole. A small-bore nylon tube is taped to the anchor and used to inject the grout at the bottom of the drilling hole (Figure 6.10). At this level the bar or cable is provided with a compression fitting to ensure that the full bonded length of the anchor acts in resisting uplift. Stranded cables are parted (after removing the sheath) and the strands are degreased over the lower part which is bonded to the grout.

Grout is injected through the nylon tube to fill the annulus completely, and it is also allowed to fill the piling tube to the required level. Where the anchors are stressed, the bar or cable is carried up to the top of the pile or pile cap to which the stress from the anchor is transferred by a stressing head and jack. The anchor is greased and sheathed in a plastics tube supplemented by wrapping with waterproof tape to protect the unbonded length from corrosion. The space surrounding the sheathed

length can be filled with grout or concrete, or left as a void. The latter is usually required in the case of piles in marine structures to allow them to flex under lateral loading.

Unstressed or 'dead' anchors can consist of steel tubes installed by drilling them down into rock. On reaching the required depth, grout is pumped down the drilling pipe where it emerges at the drilling bit and fills the annulus between the anchor tube and the rock. A sealing plate prevents the grout from entering the space between the anchor tube and the drilling pipe, as shown in [Figure 6.11](#). The grout is allowed to fill the pile to the height necessary to cover the top of the anchor tube, so as to protect it from corrosion and to serve as the medium transferring the uplift load from the pile on to the anchor. Where large uplift loads are carried, the transfer of load is effected by welding a mild steel strip on to the interior surface of the pile and the exterior of the anchor tube to act as a shear key, as described in the following section. The drilling bit is left in place at the bottom of the tube where it acts as a compression fitting, but the drilling rods are disconnected at a special coupling.

### 6.2.5 The uplift resistance of drilled-in rock anchors

The resistance to pull-out of anchors drilled and grouted into rock depends on five factors, each of which must be separately evaluated. They are as follows.

1. The safe working stress on the steel forming the anchor.
2. The allowable bond stress between the anchor and the grout.
3. The allowable bond stress between the grout and the rock.
4. The dead weight of the mass of rock and any overlying soil which is lifted by the anchor, if prior failure does not occur due to the preceding three factors.
5. The dead weight of the mass of rock and any soil overburden which is lifted by a group of closely-spaced anchors.

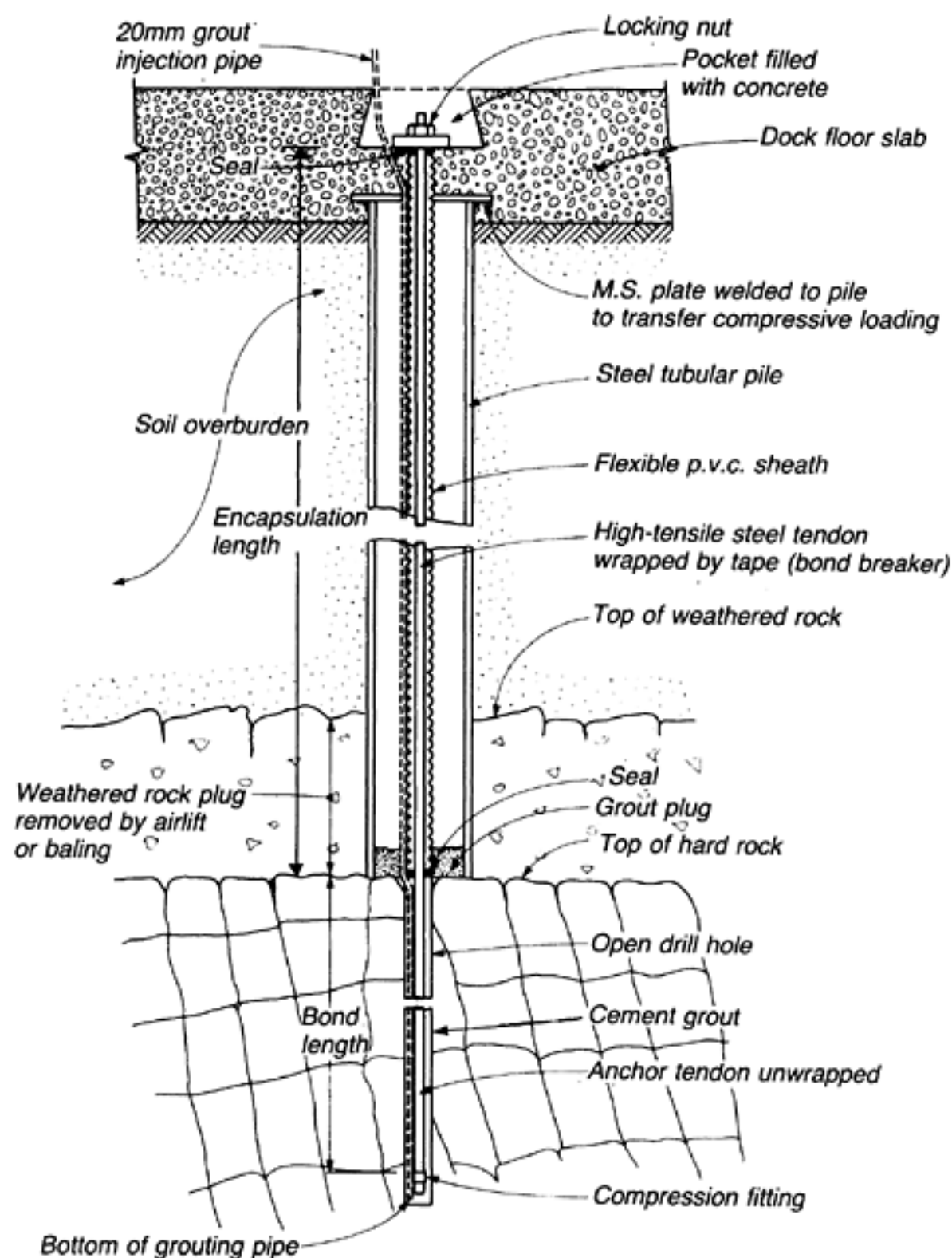


Fig. 6.10 Stressed bar tendon in steel tubular pile supporting dock floor

Bar anchors are manufactured in low-alloy steel to BS 4486 and cable anchors in non-alloy steel wire to BS 5896. The breaking stress depends on the diameter of the bar or wire or made-up strand. In the latter case it also depends on the manufacturer's treatment of the wire after uncoiling and reforming it into strand. Manufacturers' handbooks should be consulted to obtain the breaking load of a bar or strand of a particular diameter.

Steel tubes for 'dead' anchors not requiring high strength can be manufactured from smooth or deformed bar to BS 4449, 4461 or 6744. It is usual to work to 50 to 62.5% of the ultimate tensile strength to obtain the allowable working load on the anchor. An allowable stress of 50% of the ultimate permits stressing to 75% of the ultimate strength if the individual anchors are stressed to 1.5 times the working load.

The allowable bond stress between the anchor and the grout depends on the compressive strength of the grout, the amount of keying or roughening given to the steel surface, the diameter of the anchor and the influence of the bottom compression fitting in short anchors. The anchor diameter is of significance since with large-diameter high-capacity anchors there is an appreciable diminution of diameter caused by the inward radial strain that occurs under the tensile load. This creates a tendency to weaken the bond between the steel and the grout.

Recommendations for the bond strength at the grout to tendon interface are given in British Standard 8081, Code of Practice for Ground Anchors. This code provides a wealth of useful and practical

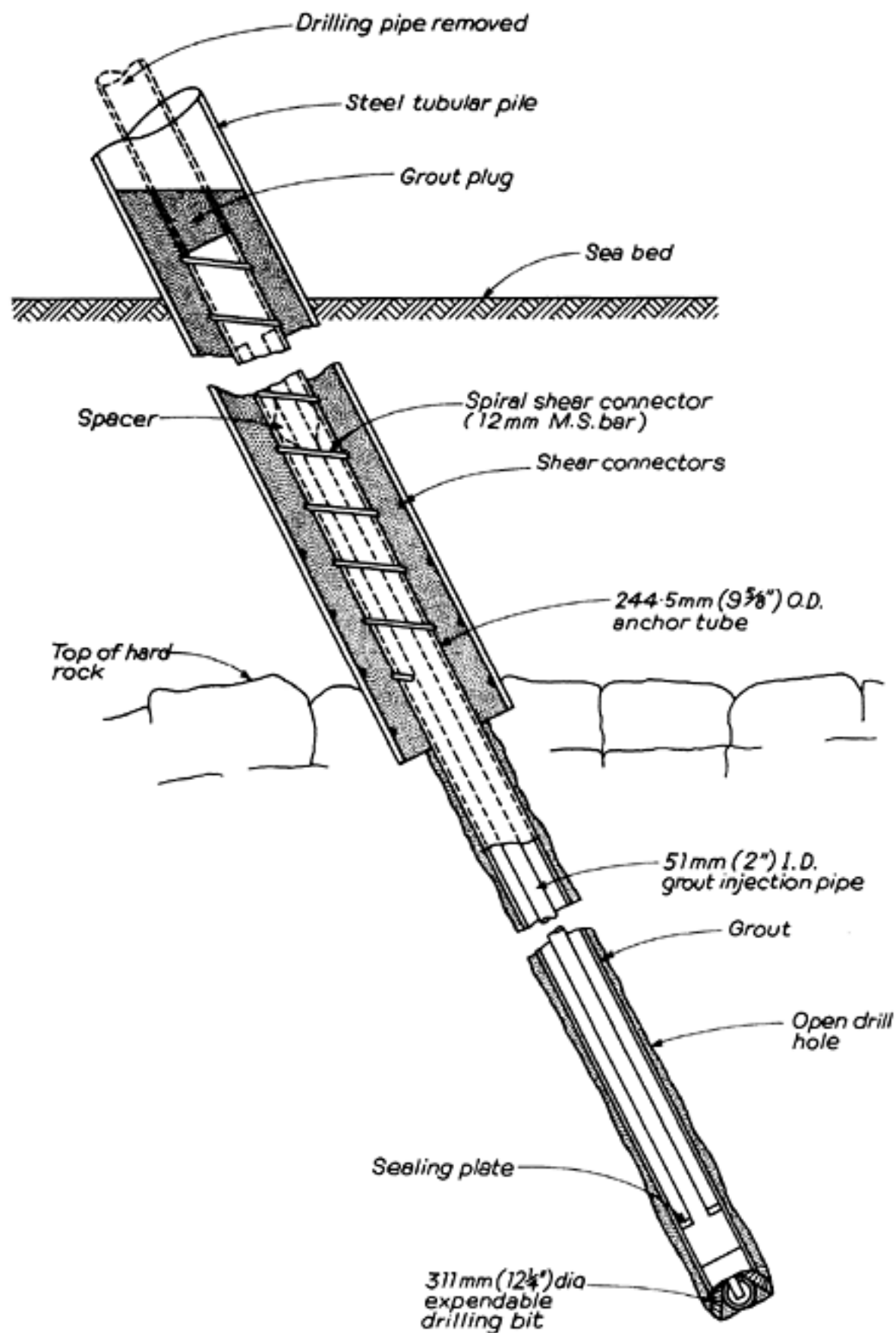


Fig. 6.11 'Dead' anchor in raking steel tubular pile for mooring dolphin

information on the design, installation and testing of anchors in soils and rock. The code recommendations for ultimate bond strength are:

Plain bar	—	not greater than 1N/mm <sup>2</sup>
Clean strand or deformed bar	—	not greater than 2N/mm <sup>2</sup>
Locally noded strand	—	not greater than 3N/mm <sup>2</sup>

Mixes of pumpable normal Portland cement grout have compressive strengths in the range of 14 to 21N/mm<sup>2</sup> at 3 days. Special grouts are formulated for injection into the annulus between an anchor and a tubular pile or between a pile and the surrounding sleeve of a 'jacket' structure (see 8.2.1). The grouts incorporate plasticizers, expanding agents and fibrous bonding materials. By adopting a water-cement ratio of about 0.5 compressive strengths of the order of 24N/mm<sup>2</sup> are attainable at 3 days. Alternatively for marine work a mix consisting of 100 parts of API Oilwell B cement to 34 parts of seawater will develop a characteristic cube strength of about 22N/mm<sup>2</sup> at 3 days. However, when such special grouts are used to transfer the load between large-diameter piles and the surrounding jacket sleeve, correspondingly high bond stresses cannot be achieved. This is because of the shrinkage of the grout in the relatively wide annulus and the diminution in the diameter of the inner member due to the inward radial strain when under tensile load.

The transfer of load from a pile to the sleeve can be effected wholly through shear keys formed on the inner surface of the sleeve, and outer surface of the pile, and these should be in the form of beads of weld metal or welded-on steel strips.

The ultimate grout to steel bond strength on the surface of tubular piles on pile sleeves either with or without mechanical shear connectors can be calculated by an equation recommended by the UK Department of Energy<sup>(6.6)</sup> as follows:

$$f_{buc} = K.C_L(9C_s + 1100h/s) \times (f_{cu})^{0.5} \quad \dots (6.4)$$

where $f_{buc}$	is the <i>characteristic</i> bond strength in N/mm <sup>2</sup>
$f_{cu}$	is the <i>characteristic</i> grout compressive strength in N/mm <sup>2</sup>
$K$	is the stiffness factor (see below)
$C_L$	is the coefficient for grout length to pile diameter ratio
$C_s$	is the surface condition factor
$h$	is the minimum shear connector outstand in mm
$s$	is the nominal shear connector spacing in mm

The stiffness factor is given by:

$$K = \frac{1}{m} \left( \frac{D}{t} \right)_g^{-1} + \left[ \left( \frac{D}{t} \right)_p + \left( \frac{D}{t} \right)_s \right]^{-1} \quad \dots (6.5)$$

where  $m$  is the modular ratio of steel to grout (take conservatively as 18 in absence of other data for longterm, i.e. 28 days or more),  $D$  is the outside diameter,  $t$  is the wall thickness. The suffixes  $g$ ,  $p$  and  $s$  refer to the grout, pile and sleeve respectively.

The basis of equation 6.4 was a research programme described in references 6.7 (a) to (c). This covered a limited range of geometry of the pile connections. For this reason the Department of Energy have applied limits to the values of  $C_L$  as follows:

$L/D_p$	$C_L$
2	1.0
4	0.9
8	0.8
$\geq 12$	0.7

Where  $L$  is the nominal grouted connection length. Obtain intermediate values for  $L/D_p < 12$  by linear interpolation.

The surface condition factor  $C_s$  should be taken according to the following:

- (i) If shear connectors are present and satisfy requirement  $h/s > 0.005$  take  $C_s$  as 1.0
- (ii) For plain pipe connections and for connections with shear connectors but with  $h/s < 0.005$ , in the absence of test data take  $C_s = 0.6$

The values in (i) and (ii) refer to shotblasted or lightly rusted surface conditions. Other conditions (e.g. painted surfaces) should receive special consideration.

Equation 6.4 should be applied only to connections which satisfy the geometrical limits given below, and where mechanical shear connectors are used they are present both on pile and sleeve surfaces in contact with grout; the connectors are at uniform spacing along the length of the connection; outstand and spacing of shear connectors on pile and sleeve are the same; shear connectors on driven piles should cover sufficient length to ensure contact with grout after driving; shear connector cross-section and welds on each grout/steel interface should be designed to transmit total load applied to the grouted connection. The geometrical limits are:

Sleeve geometry  $50 \leq (D/t)_s \leq 140$  Pile geometry  $24 \leq (D/t)_p \leq 40$

Grout annulus geometry  $10 \leq (D/t)_g \leq 45$   $L/D_p \geq 2$

Shear connector height ratio  $0^* \leq h/D_p \leq 0.006$

Shear connector spacing ratio  $0^* \leq D_p/s \leq 8$

Shear connector ratio  $0^* \leq h/s \leq 0.04$

Shear connector shape factor  $1.5 \leq w/h \leq 3$  (consistent with welded square bar or hemispherical weldbeads)

where  $w$  is the nominal width of the shear connector including welds.

The American Petroleum Institute recommendations<sup>(6.8)</sup> are much simpler. The equations are:

For operating and environmental conditions combined with dead loads and maximum or minimum live loads appropriate to normal operations of the platform:

$$f_{ba} = 0.138 + 0.5f_{cu} \frac{h}{s} \text{ (MN/m}^2\text{)} \quad \dots(6.6)$$

For design environmental conditions with dead loads and maximum or minimum live loads appropriate for combining with extreme conditions:

$$f_{ba} = 0.184 + 0.67f_{cu} \frac{h}{s} \text{ (MN/m}^2\text{)} \quad \dots(6.7)$$

where  $f_{ba}$  is the *allowable* axial load transfer strength (steel to grout bond strength) and other terms are as previously defined.

It is evident from equations 6.6 and 6.7 that the first term represents the bond strength value for the plain pipe. The equations are again subject to limitations as follows:

Grout compression strength to be greater than 17.25N/mm<sup>2</sup> but less than 110N/mm<sup>2</sup>

Ratio  $D_p/t_p$  not more than 40

Ratio  $D_s/t_s$  not more than 80

Ratio  $D_g/t_g$  greater than 7, not more than 45

Ratio  $Dp/s$  greater than 2.5, not more than 8

Ratio  $h/s$  not greater than 0.10

Ratio  $w/h$  greater than 1.5, not less than 3

Product  $f_{cu} \times h/s$  equal to or not less than 5.5MN/m<sup>2</sup>

The allowable bond stress between grout and rock depends on the compressive strength of the intact rock, the size and spacing of joints and fissures in the rock, the keying of the rock effected by the drilling bit, and the cleanliness of the rock surface obtained by the flushing water. The size of the drill hole and the size of the annular space between the anchor and the wall of the hole are also important. Usually the diameter of the drill hole is taken as 1.7 to 2.5 times the diameter of the anchor. The lower end of the range is used in a strong massive rock and the higher end in a weak fractured rock. A large-diameter hole or a thick annulus gives rise to problems due to the shrinkage of the grout and the consequent weakening of the grout-to-rock bond. These difficulties can be overcome to some extent by using a special bonding grout as described above. Also, by using a compression fitting at the bottom of the anchor, part of the grout column is put in compression. The smaller the annulus and the shorter the bonded length, the higher is the compressive stress on the grout and hence its ability to lock into the surrounding rock. The value of the bond between grout and rock will be small if the rock softens to a slurry under the action of drilling and flushing. This occurs with chalk, weathered marl and weathered clayey shales. Some observed values of bond stress at failure for drill holes of up to 75mm in diameter are given in [Table 6.1](#).

If the bond stress between the grout and the rock is a critical factor in designing the anchors, the allowable value should be obtained by increasing the length of the anchor rather than by increasing the diameter of the drill hole, for the reasons already stated. However, in certain conditions it is possible that the bond stress will not be reduced in direct proportion to the increase in bond length. This is

Table 6.1

Type of rock	Bond stress between grout and rock at failure N/mm <sup>2</sup>	Reference
Chalk (Grade I)	0.21	Littlejohn <sup>6.9</sup>
Chalk (Grade III)	0.80	Littlejohn <sup>6.9</sup>
Keuper Marl (Zones I and II)	0.17–0.25	Littlejohn <sup>6.9</sup>
Chalk	1.0	Hutchinson <sup>6.10</sup>
Weathered shaley slate	0.27	Unpublished <sup>6.11</sup>
Hard shaley slate	1.0–1.7	Unpublished <sup>6.11</sup>
Billings shale (Ottawa)	3.0	Freeman <i>et al.</i> <sup>6.12</sup>
Sandstone	>0.6	Unpublished <sup>6.11</sup>

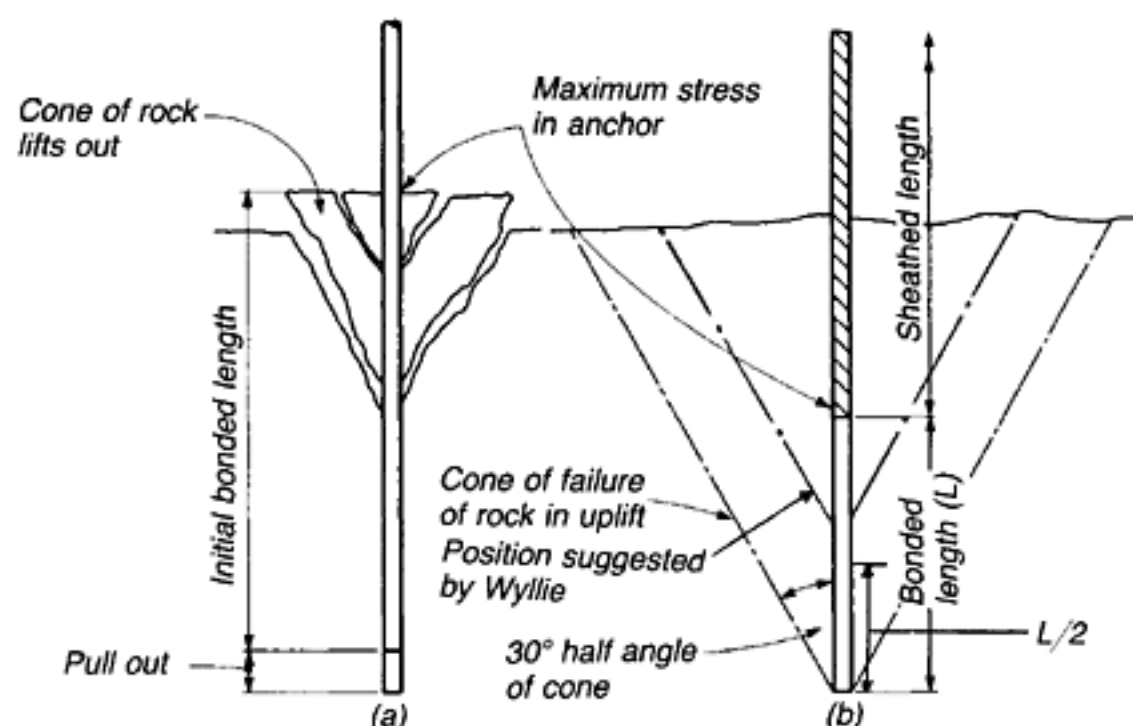


Fig. 6.12 Pull-out of cone of rock

(a) Fully bonded anchor

(b) Upper part sheathed, lower part bonded

because of the possibility of progressive failure in a hard rock. The maximum stretch in the anchor occurs at the top of the bonded length, and this may cause local bond failure with the rock or the pulling out of a small cone of rock (Figure 6.12a). Progressive failure then extends down to the bottom of the anchor. By limiting the bond length and sheathing the upper part of the anchor within the rock (Figure 6.12b), the pulling-out of a cone of rock is prevented and the whole column of grout is compressed and acts in bond resistance with the rock.

The pull-out resistance of the mass of rock (as shown in Figure 6.12b) is the final criterion of the performance of an individual anchor. The actual shape of the mass of rock lifted (if failure does not occur due to the failure of the steel-to-grout or rock-to-grout bond) depends on the degree of jointing and fissuring of the rock and the inclination of the bedding planes. Various forms of failure are sketched in Figure 6.13. A cone with a half angle of 30° gives a conservative value for the pull-out resistance and represents conditions for a heavily jointed or shattered rock. Wyllie<sup>(4.54)</sup> suggests that the base of the cone should be taken at the mid-point of the bonded length (Figure 6.12b), but this arrangement would not apply for the case of a compression fitting at the bottom of the anchor. Because shear at the interface between the surface of the cone and the surrounding rock is neglected, a safety factor of unity can be taken on the weight of the rock cone, where the rock is bedded horizontally or at moderate angles from the vertical (Figure 6.13a). Where the bedding planes or other joint systems are steeply inclined, as shown in Figures 6.13b to 6.13d, either an increased factor should be allowed, or an attempt should be made to calculate the uplift resistance of the rock mass by rock mechanics' principles. The submerged weight should be taken for rock below ground-water level or below the sea. The uplift resistance of the cylinder or cone of soil overburden above the rock cone can be calculated as described in Section 6.2.3. The dimensions  $B$  and  $H$  in equations 6.2 and 6.3 are as shown in Figure 6.14. Skin friction on the pile does not operate to resist uplift for this mode of failure. The mode of failure of a group of anchors, assuming no failure occurs in the bond between grout and steel or grout and rock, is shown in Figure 6.15. The anchors can be splayed out as shown in Figure 6.16 to increase the volume of rock bounded by the group.

The calculation of the volume of rock  $V_c$  in a single cone with a half-angle of  $30^\circ$  at various angles

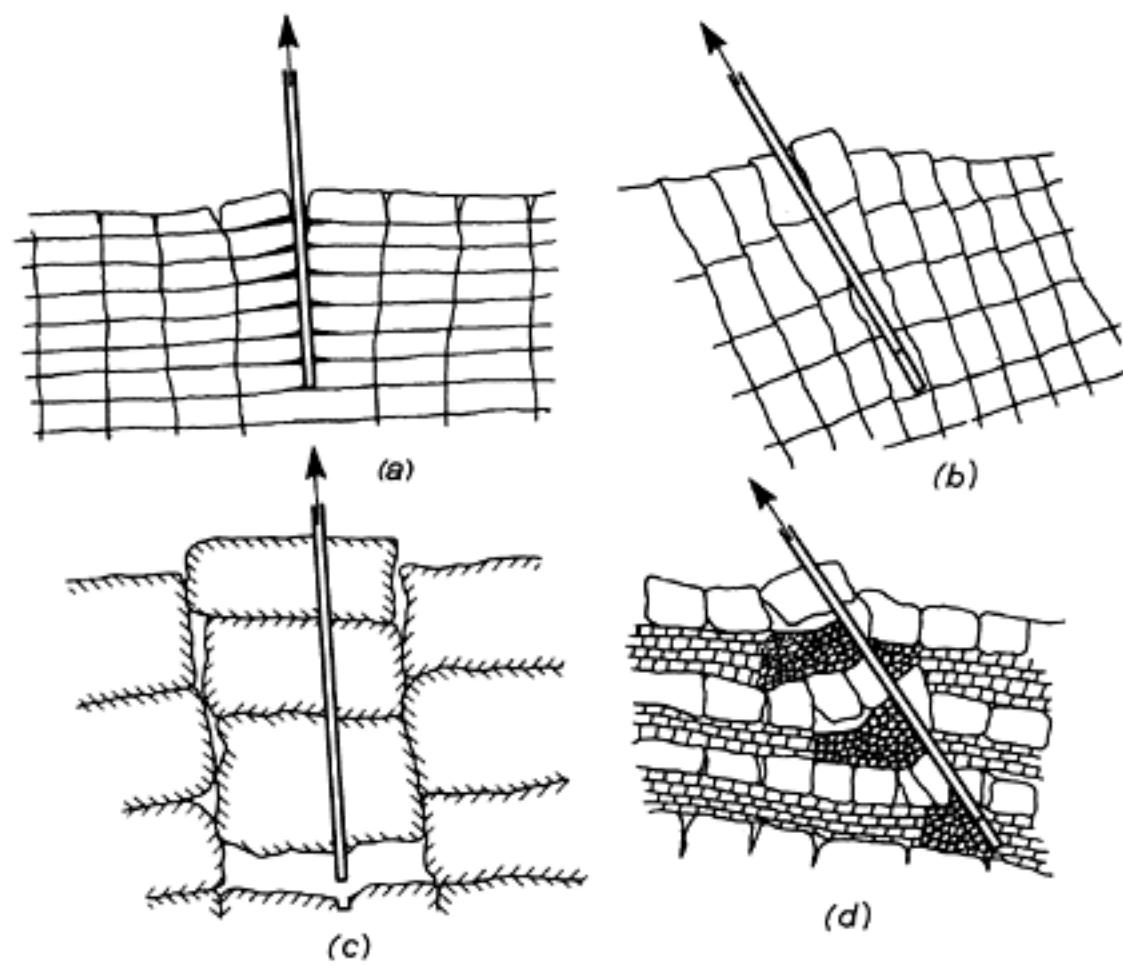


Fig. 6.13 Pull-out failure in rock anchors

- (a) Horizontally bedded rock (thinly bedded)
- (b) Steeply inclined bedding planes with anchor raked in direction of bedding joints
- (c) Horizontally bedded rock
- (d) Alternating thinly and thickly bedded rocks

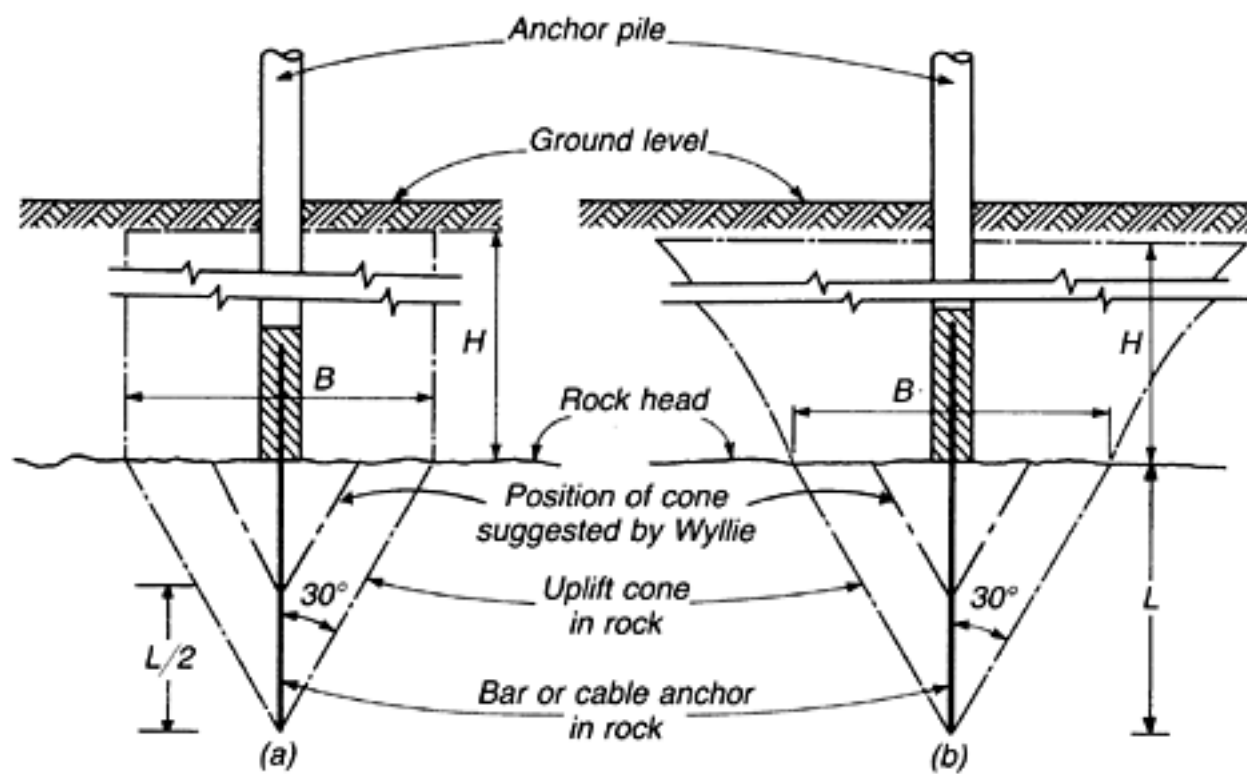
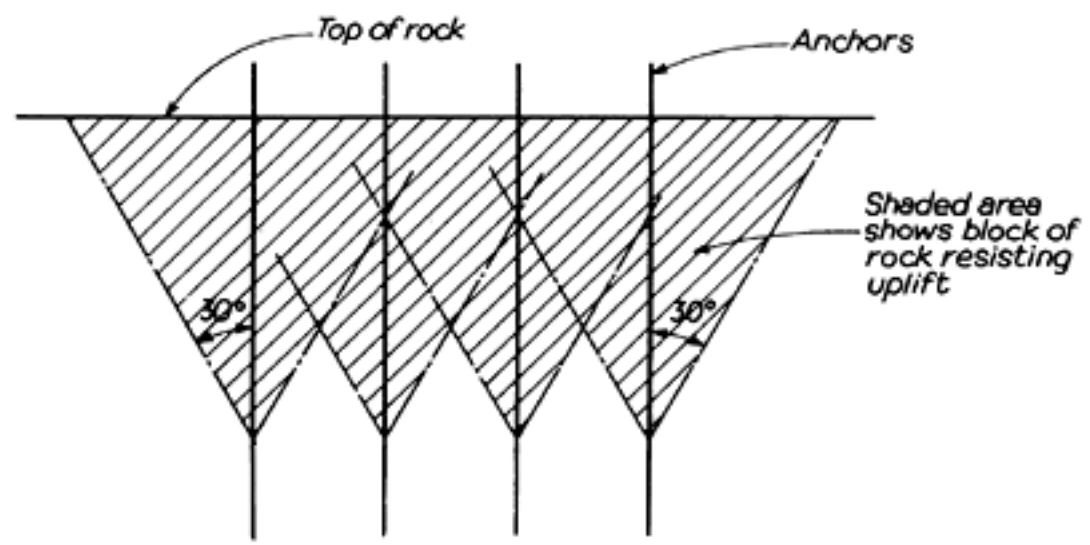


Fig. 6.14 Approximate method of calculating ultimate uplift resistance of rock anchors with soil overburden

- (a) Clay overburden
- (b) Granular soil overburden



*Fig. 6.15 Failure condition at group of anchors in rock*

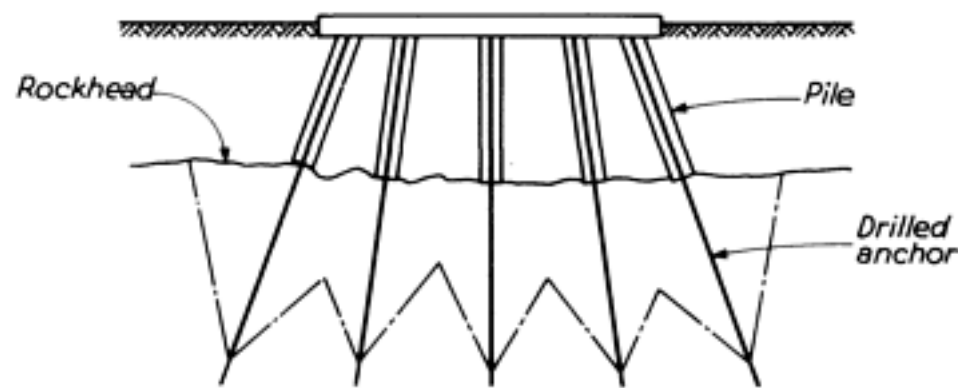


Fig. 6.16 Splaying anchors in group to increase uplift resistance

of inclination  $\theta$  to a horizontal rock surface can be performed with the aid of the curve for  $V_c/L^3$  in [Figure 6.17a](#). The effect of overlapping cones of rock in groups of vertical or raking anchors can be calculated by reference to [Figures 6.17a and 6.17b](#). These charts enable the overlapping volumes  $\Delta V_m$  and  $\Delta V_n$  to be calculated for a group of anchors arranged on a rectangular grid. They are not applicable to a diagonal (*i.e.* ‘staggered’) pattern. All the anchors in the group are assumed to be arranged at the same angle of inclination to the horizontal and the charts are based on a cone with a  $30^\circ$  half-angle. The charts are not valid if the sum of  $(P/n)^2$  and  $(S/m)^2$  is less than 4 when composite overlapping occurs. In such a case the total volume acting against uplift should be estimated from the geometry of the system. Because of the various uncertainties in the design of rock anchors as described above, it is evident that it is desirable to adopt stressed anchors with every anchor individually stressed and hence checked for pull-out resistance at a proof load of 1.5 times the working load. However, it should be noted that the technique of stressing anchors by jacking against the reaction provided by the pile does not check the pull-out resistance of the cone of rock: this is clear from [Figure 6.10](#). The resistance offered by the mass or rock can be tested only by providing a reaction beam with bearers sited beyond the influence of the conjectural rock cone. Tests of this description are very expensive to perform and it is usual to avoid them by adopting conservative assumptions for the dimensions of the cone, and applying a safety factor to the calculated weight if required.

### 6.3 Single vertical piles subjected to lateral loads

The ultimate resistance of a vertical pile to a lateral load and the deflection of the pile as the load builds up to its ultimate value are complex matters involving the interaction between a semi-rigid structural element and the soil, which deforms partly elastically and partly plastically. Taking the case of a vertical pile unrestrained at the head, the lateral loading on the pile head is initially carried by the soil close to the ground surface. At a low loading the soil compresses elastically but the movement is sufficient to transfer some pressure from the pile to the soil at a greater depth. At a further stage of loading the soil yields plastically and transfers its load to greater depths. A short rigid pile unrestrained at the top and having a length to width ratio of less than 10 to 12 ([Figure 6.18a](#)) tends to rotate and passive resistance develops above the toe on the opposite face to add to the resistance of the soil near the ground surface. Eventually the rigid pile will fail by rotation when the passive resistance of the soil at the head and toe are exceeded. The short rigid pile restrained at the head by a cap or bracing will fail by translation in a similar manner to an anchor block which fails to restrain the movement of a retaining wall transmitted through a horizontal tied rod ([Figure 6.18b](#)).

The failure mechanism of an infinitely long pile is different. The passive resistance of the lower part of the pile is infinite, and thus rotation of the pile cannot occur, the lower part remaining vertical while the upper part deforms to a shape shown in [Figure 6.19a](#). Failure takes place when the pile fractures at the point of maximum bending moment, and for the purpose of analysis a plastic hinge capable of transmitting shear is assumed to develop at the point of fracture. In the case of a long pile restrained at the head, high bending stresses develop at the point of restraint, *e.g.* just beneath the pile cap, and the pile may fracture at this point ([Figure 6.19b](#)).

The pile head may move horizontally over an appreciable distance before rotation or failure of the pile occurs, to such an extent that the movement of the structure supported by the pile or pile group exceeds tolerable limits. Therefore, having calculated the ultimate load and divided it by the appropriate safety factor, it is still necessary to check that the permissible deflection of the pile is not exceeded.

Quite extensive research has been undertaken on the resistance of piles to lateral loading but this research has not yielded any simple design method which can be universally applied to any soil or type of pile. There are many inter-related factors. The dominant one is the pile stiffness, which influences

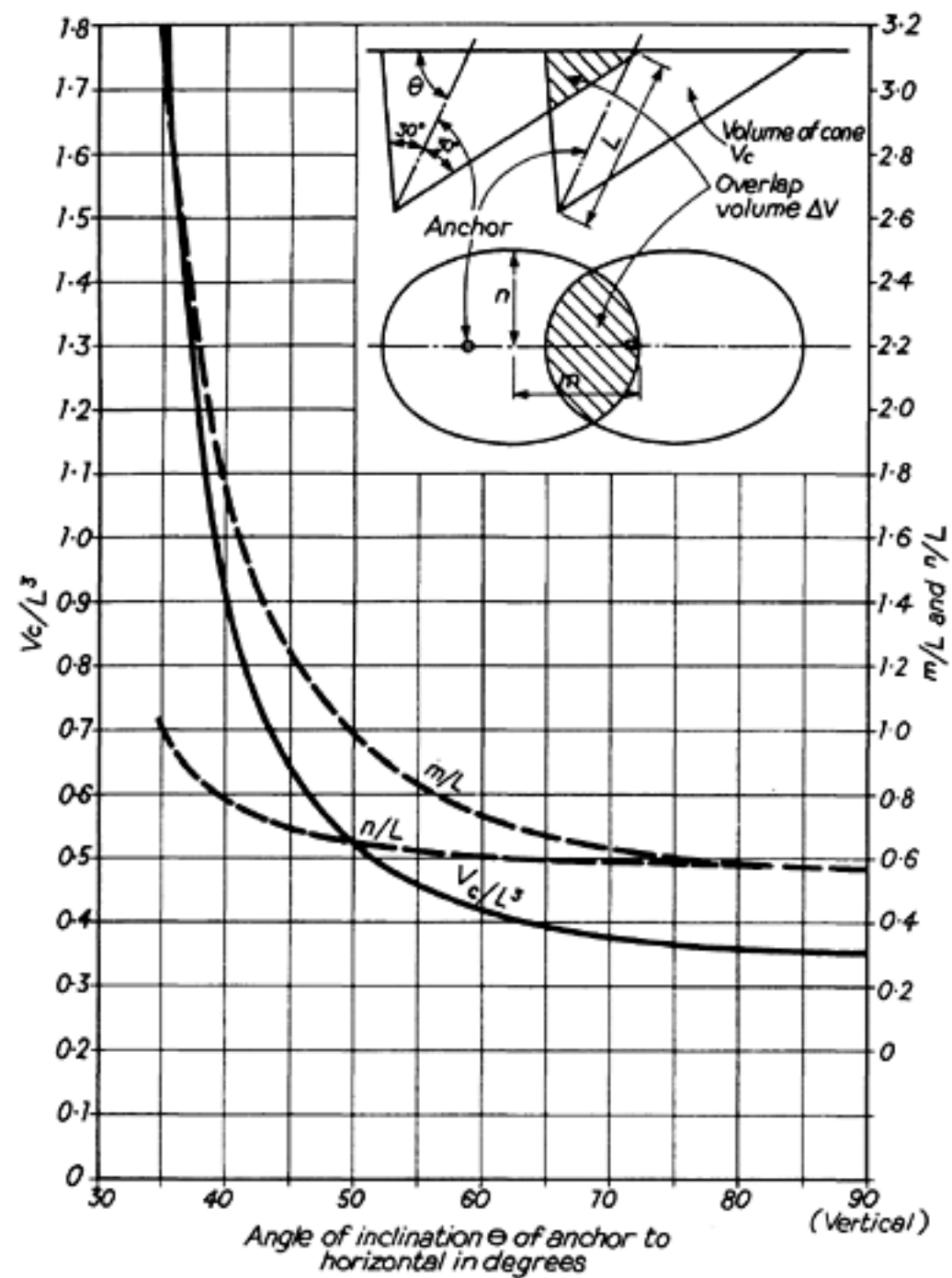


Fig. 6.17 (a) Chart for calculating volumes of single or overlapping cones with vertical or inclined axes

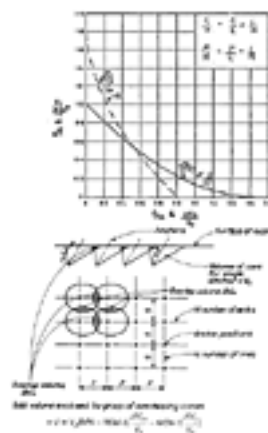


Fig. 6.17 (b) Charts for calculating total volume of rock enclosed by groups of anchors arranged on rectangular grid pattern

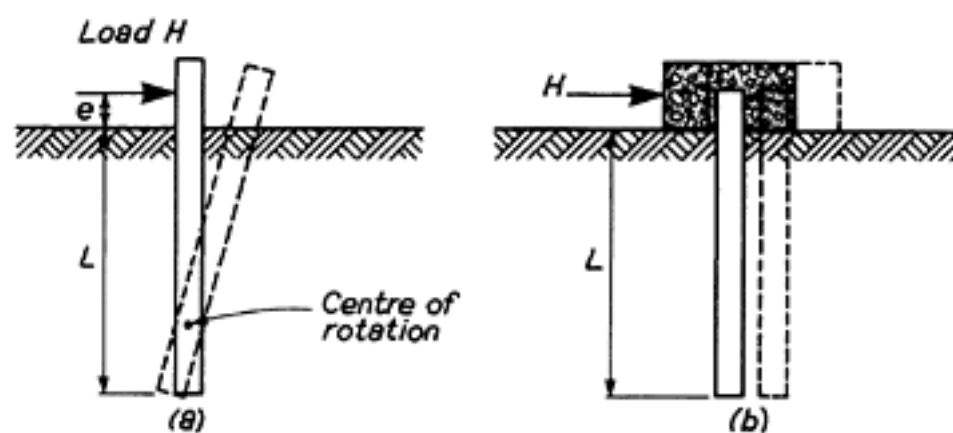


Fig. 6.18 Short vertical pile under horizontal load

the deflection and determines whether the failure mechanism is one of the rotation of a short rigid element, or is due to flexure followed by the failure in bending of a long flexible pile. The type of loading, whether sustained (as in the case of earth pressure transmitted by a retaining wall) or alternating (say, from reciprocating machinery) or pulsating (as from the traffic loading on a bridge pier), influences the degree of yielding of the soil. External influences such as scouring around piles at sea-bed level, or the seasonal shrinkage of clay soils away from the upper part of the pile shaft, affect the resistance of the soil at a shallow depth.

The simpler methods of calculating ultimate resistance and deflection under lateral loads are presented in the following sections of this chapter. No attempt is made to give the complete theoretical basis

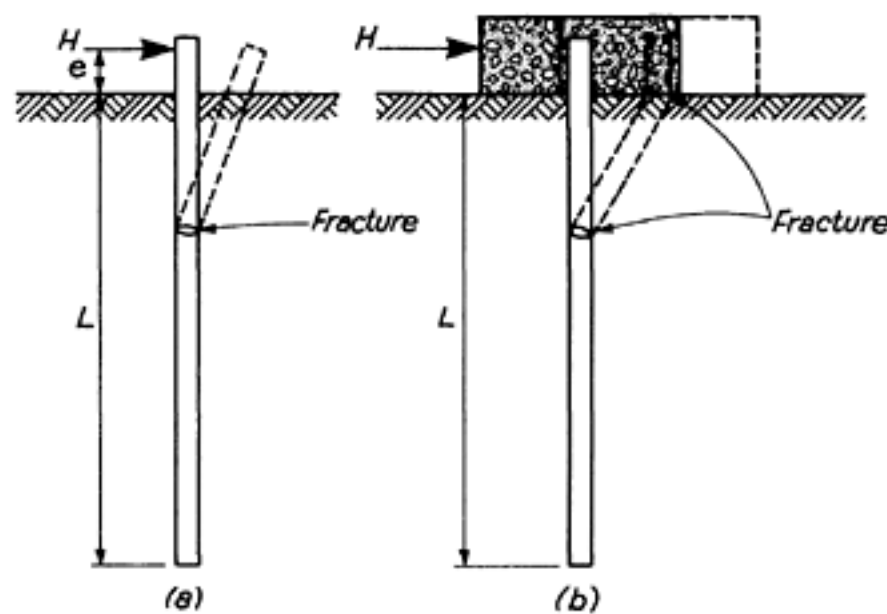


Fig. 6.19 Long vertical pile under horizontal load

- (a) Free head  
(b) Fixed head

of these methods, which have been reviewed by Broms<sup>(6.13, 6.14)</sup> and Elson<sup>(6.15)</sup>. Various simplifications have been necessary in order to provide simple solutions to complex problems of soil-structure interaction, and the limitations of the methods are stated where these are particularly relevant. Most practical calculations are processes of trial and adjustment, starting with a very simple approach to obtain an approximate measure of the required stiffness and embedment depth of the pile. The process can then be elaborated to some degree to narrow the margin of error, and to provide the essential data for calculating bending moments, shearing forces and deflections at the working load. Very elaborate calculation processes are not justified, because of the non-homogeneity of most natural soil deposits and the disturbance to the soil caused by installing piles. None of these significant factors can be reproduced in the calculation methods.

Eurocode [Section 7.8](#) requires the design of piles to resist transverse loading to be carried out using the same rules as specified for compression loading in [Section 7.6](#) of code (as discussed in [Chapter 4](#) of this book). Where the transverse capacity is calculated from soil test results and pile strength parameters, the capacity is required to be calculated using a compatible set of structural bending moments, shear forces, soil reactions and displacements. The pile strength is required to be determined in accordance with the relevant material Eurocodes for structural design.

For any important foundation structure which has to carry high or sustained lateral loading it is advisable to make field loading tests on trial piles having at least three different shaft lengths, in order to assess the effects of embedment depth and structural stiffness. For less-important structures, or where there is previous experience of pile behaviour to guide the engineer, it may be sufficient to make lateral loading tests on pairs of working piles by jacking or pulling them apart. These tests are rapid and economical to perform (see [Section 11.4.4](#)) and provide a reliable check that the design requirements have been met.

### 6.3.1 Calculating the ultimate resistance to lateral loads

The first step is to determine whether the pile will behave as a short rigid unit or as an infinitely long flexible member. This is done by calculating the stiffness factors  $R$  and  $T$  for the particular combination of pile and soil. The stiffness factors are governed by the stiffness ( $EI$  value) of the pile and the compressibility of the soil. The latter is expressed in terms of a 'soil modulus', which is not constant for any soil type but depends on the width of the pile  $B$  and the depth of the particular loaded area of soil being considered. The soil modulus  $k$  has been related to Terzaghi's concept of a modulus of horizontal subgrade reaction<sup>(6.16)</sup>. In the case of a stiff over-consolidated clay, the soil modulus is generally assumed to be constant with depth. For this case

$$\text{stiffness factor } R = \sqrt[4]{\frac{EI}{kB}} \text{ (in units of length)} \quad \dots(6.8)$$

Relationships between  $k$  and the Terzaghi modulus  $k_1$ <sup>(16.16)</sup> and other methods of determining  $k$  are given in [Section 6.3.3](#). The modulus  $k_1$ , is obtained from load-deflection measurements on a 305mm square plate. It is related to the undrained shearing strength of the clay, as shown in [Table 6.2](#).

Table 6.2 Relationship of modulus of subgrade reaction (k1) to untrained shearing strength of stiff overconsolidated clay

Consistency	Firm to stiff	Stiff to very stiff	Hard
Undrained shear strength ( $c_u$ ) kN/m <sup>2</sup>	50–100	100–200	>200
Range of $k_1$ MN/m <sup>3</sup>	15–30	30–60	> 60

For most normally-consolidated clays and for granular soils the soil modulus is assumed to increase linearly with depth, for which

stiffness factor  $T = \sqrt[5]{\frac{EI}{n_h}}$  (in units of length)

...(6.9)

where soil modulus  $K = n_h \times x/B$

...(6.10)

Values of the coefficient of modulus variation  $n_h$  were obtained directly from lateral loading tests on instrumented piles in submerged sand at Mustang Island, Texas. The tests were made for both static and cyclic loading conditions and the values obtained, as quoted by Reese *et al.*(6.17) were considerably higher than those of Terzaghi(6.16). The investigators recommend that the Mustang Island values should be used for pile design and these are shown together with the Terzaghi values in [Figure 6.20](#).

Other observed values of  $n_h$  are as follows:

Soft normally-consolidated clays: 350 to 700kN/m<sup>3</sup>.

Soft organic silts: 150kN/m<sup>3</sup>.

Having calculated the stiffness factors  $R$  or  $T$ , the criteria for behaviour as a short rigid pile or as a long elastic pile are related to the embedded length  $L$  as follows:

Pile type	Soil modulus	
	Linearly-increasing	Constant
Rigid (free head)	$L \leq 2T$	$L \leq 2R$
Elastic (free head)	$L \geq 4T$	$L \geq 3.5R$

Ultimate resistance of short rigid piles

Brinch Hansen’s method(6.19) can be used to calculate the ultimate lateral resistance of short rigid piles. The method is a simple one which can be applied both to uniform and layered soils. The resistance

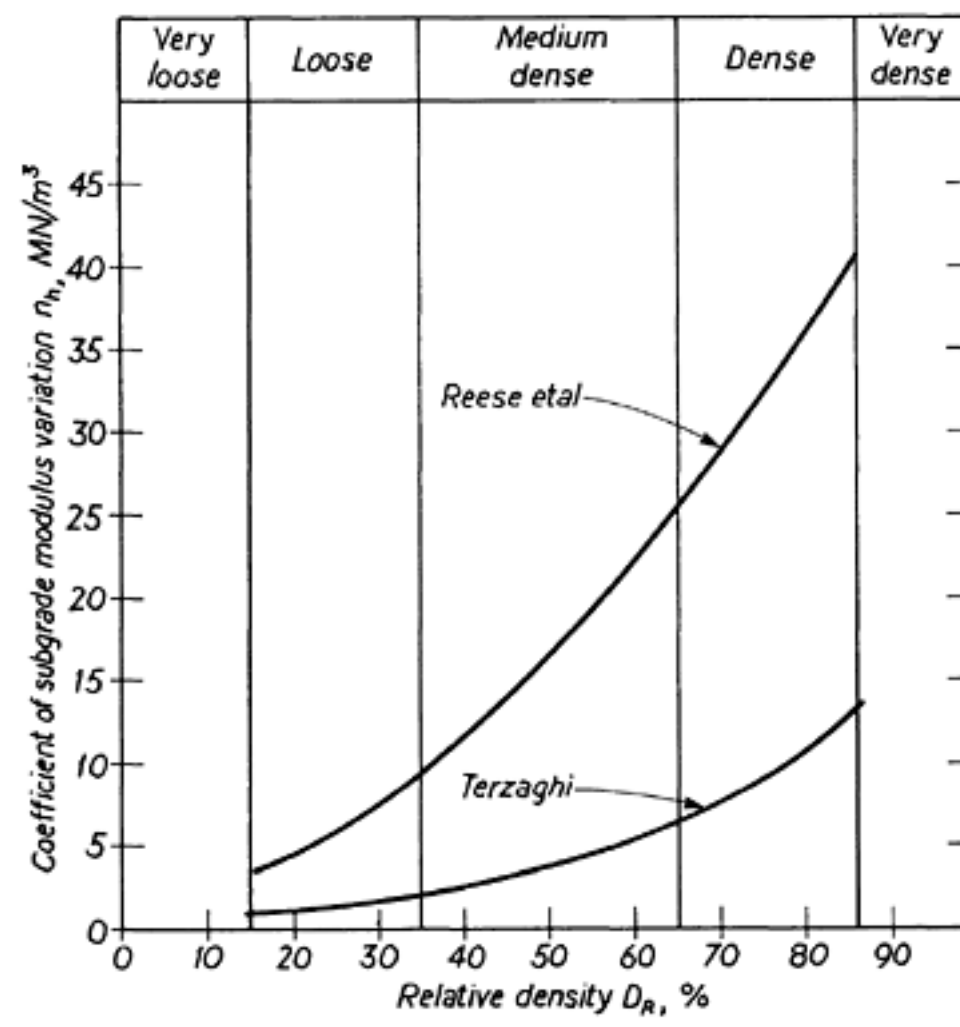


Fig. 6.20 Relationship between coefficient of modulus variation and relative density of sands  
(after Garassino et al<sup>(6-18)</sup>)

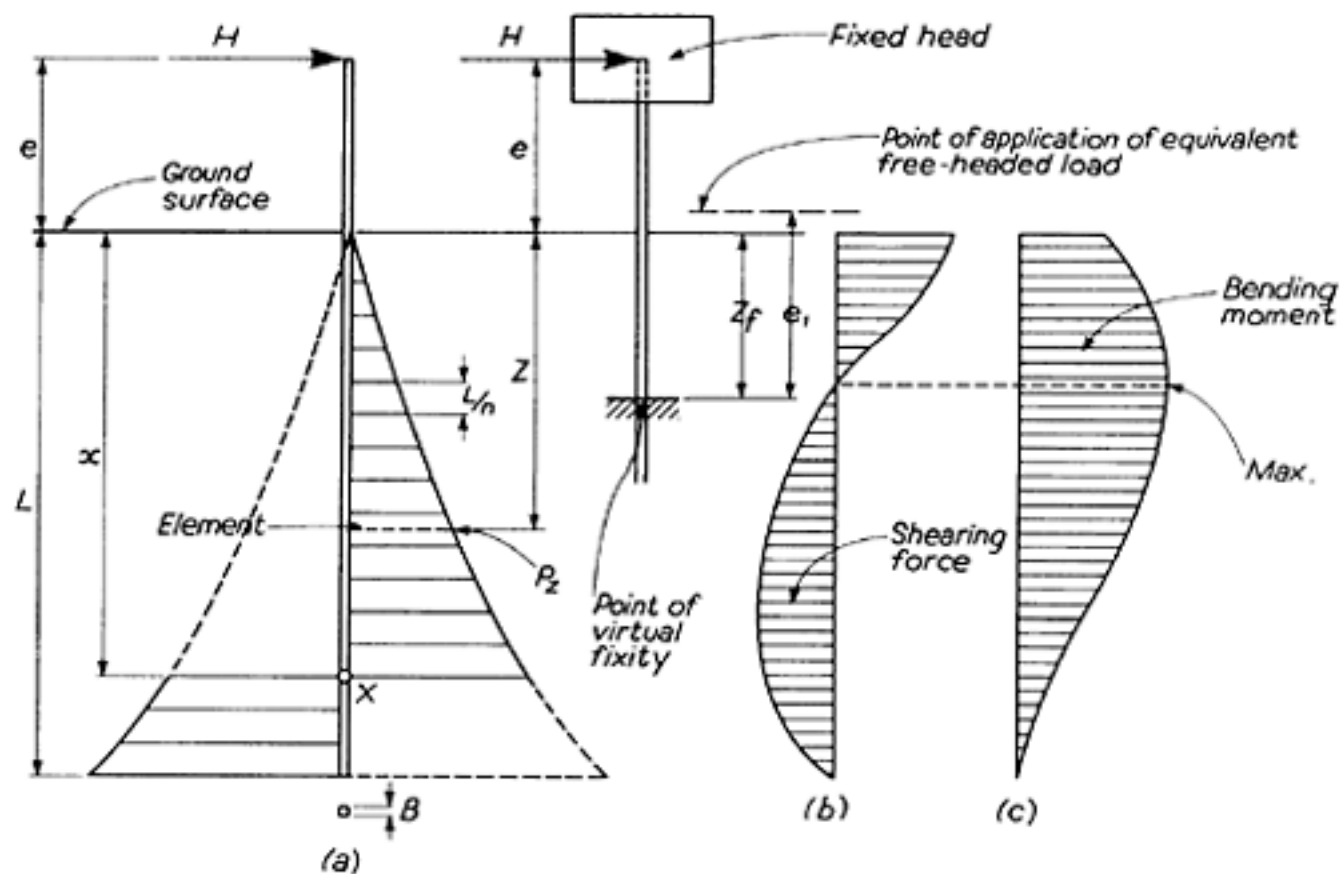


Fig. 6.21 Brinch Hansen's method for calculating ultimate lateral resistance of short piles

- (a) Soil reactions
- (b) Shearing force diagram
- (c) Bending moment diagram

of the rigid unit to rotation about point X in [Figure 6.21](#) a is given by the sum of the moments of the soil resistance above and below this point. The passive resistance diagram is divided into a convenient number  $n$  of horizontal elements of depth  $L/n$ . The unit passive resistance of an element at a depth  $z$  below the ground surface is then given by

$$p_z = p_{oz} K_{qz} + cK_{cz} \quad \dots(6.11)$$

where  $p_{oz}$  is the effective overburden pressure at depth  $z$ ,  $c$  is the cohesion of the soil at depth  $z$ , and  $K_{qz}$  and  $K_{cz}$  are the passive pressure coefficients for the frictional and cohesive components respectively at depth  $z$ . Brinch Hansen<sup>(6.19)</sup> has established values of  $K_q$  and  $K_c$  in relation to the depth  $z$  and the width of the pile  $B$  in the direction of rotation, as shown in [Figure 6.22](#).

The total passive resistance on each horizontal element is  $p_z \times \frac{L}{n} \times B$  and, by taking moments about the point of application of the horizontal load,

$$\sum M = \sum_{z=0}^{z=x} p_z \frac{L}{n} (e+z) B - \sum_{z=x}^{z=L} p_z \frac{L}{n} (e+z) B \quad \dots(6.12)$$

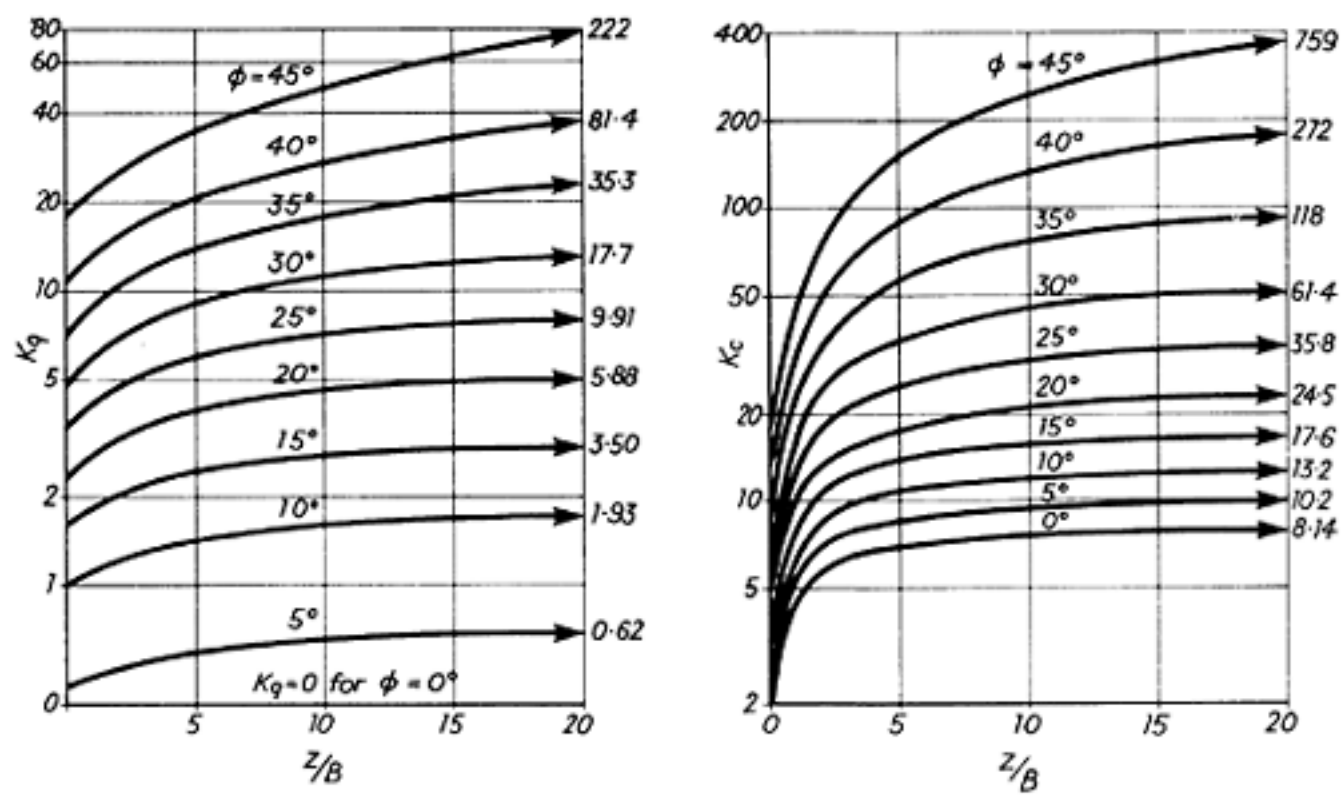


Fig. 6.22 Brinch Hansen's coefficients  $K_q$  and  $K_c$

The point of rotation at depth  $x$  is correctly chosen when  $\Sigma M = 0$ , i.e. when the passive resistance of the soil above the point of rotation balances that below it. Point X is thus determined by a process of trial and adjustment. If the head of the pile carries a moment  $M$  instead of a horizontal force, the moment can be replaced by a horizontal force  $H$  at a distance  $e$  above the ground surface where  $M$  is equal to  $H \times e$ .

Where the head of the pile is fixed against rotation, the equivalent height  $e_1$  above ground level of a force  $H$  acting on a pile with a free head is given by

$$e_1 = \frac{1}{2}(e + z_f) \quad \dots(6.13)$$

where  $e$  is the height from the ground surface to the point of application of the load at the fixed head of the pile ([Figure 6.21a](#)), and  $z_f$  is the depth from the ground surface to the point of virtual fixity. The depth  $z_f$  is not known at this stage but for practical design purposes it can be taken as 1.5m for a compact granular soil or stiff clay (below the zone of soil shrinkage in the latter case), and 3m for a soft clay or silt. The American Concrete Institute<sup>(2.9)</sup> recommend that  $z_f$  should be taken as  $1.4R$  for stiff, over-consolidated clays and  $1.8T$  for normally consolidated clays, granular soils and silt, and peat (see equations 6.8 and 6.9).

Having obtained the depth to the centre of rotation from equation 6.12, the ultimate lateral resistance of the pile to the horizontal force  $H_u$  can be obtained by taking moments about the point of rotation, when

$$H_u(e + x) = \sum_0^x p_z \frac{L}{n} B(x - z) + \sum_x^{x+L} p_z \frac{L}{n} + B(z - x) \quad \dots(6.14)$$

The final steps in Brinch Hansen's method are to construct the shearing force and bending moment diagrams ([Figure 6.21b](#) and [6.21c](#)). The ultimate bending moment, which occurs at the point of zero shear, should not exceed the ultimate moment of resistance  $M_u$  of the pile shaft. The appropriate load factors are applied to the horizontal design force to obtain the ultimate force  $H_u$ .

When applying the method to layered soils, assumptions must be made concerning the depth  $z$  to obtain  $K_q$  and  $K_c$  for the soft clay layer, but  $z$  is measured from the top of the stiff clay stratum to obtain  $K_c$  for this layer, as shown in [Figure 6.23](#).

The undrained shearing strength  $c_u$  is used in equation 6.11 for short-term loadings such as wave or ship-berthing forces on a

jetty, but the drained effective shearing strength values ( $c'$  and)  $\phi'$  are used for long-term sustained loadings such as those on retaining walls. A check should be made to ensure that there is an adequate safety factor for undrained conditions in the early stages of loading. The step-by-step procedure using the Brinch Hansen method is illustrated in worked examples 6.4 and 6.5.

For short-term loading in uniform cohesive ( $\phi = 0$ ) soils the method of Broms<sup>(6.13)</sup> is quick and convenient to use. For such soils Broms has assumed that the reaction of the soil on the pile is represented by the simplified diagrams for piles with free and fixed heads shown in [Figure 6.24](#). The zone of zero pressure over the depth  $1.5B$  represents the effect of soil shrinkage away from the pile. The ultimate lateral resistance  $H_u$  is obtained from [Figure 6.25](#), in which it is related to the undrained

shearing strength  $c_u$ , the pile width  $B$  and the ratio  $L/B$  of embedded length to width. From the diagram in [Figure 6.24](#)

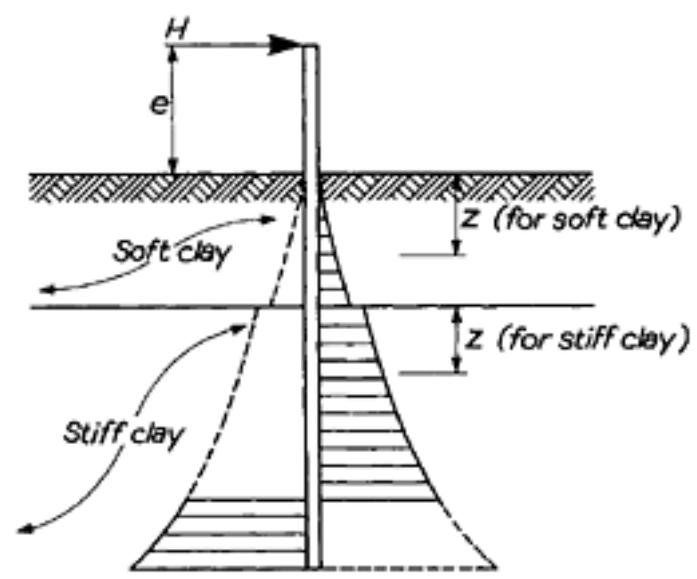


Fig. 6.23 Reactions in layered soil on vertical pile under horizontal load

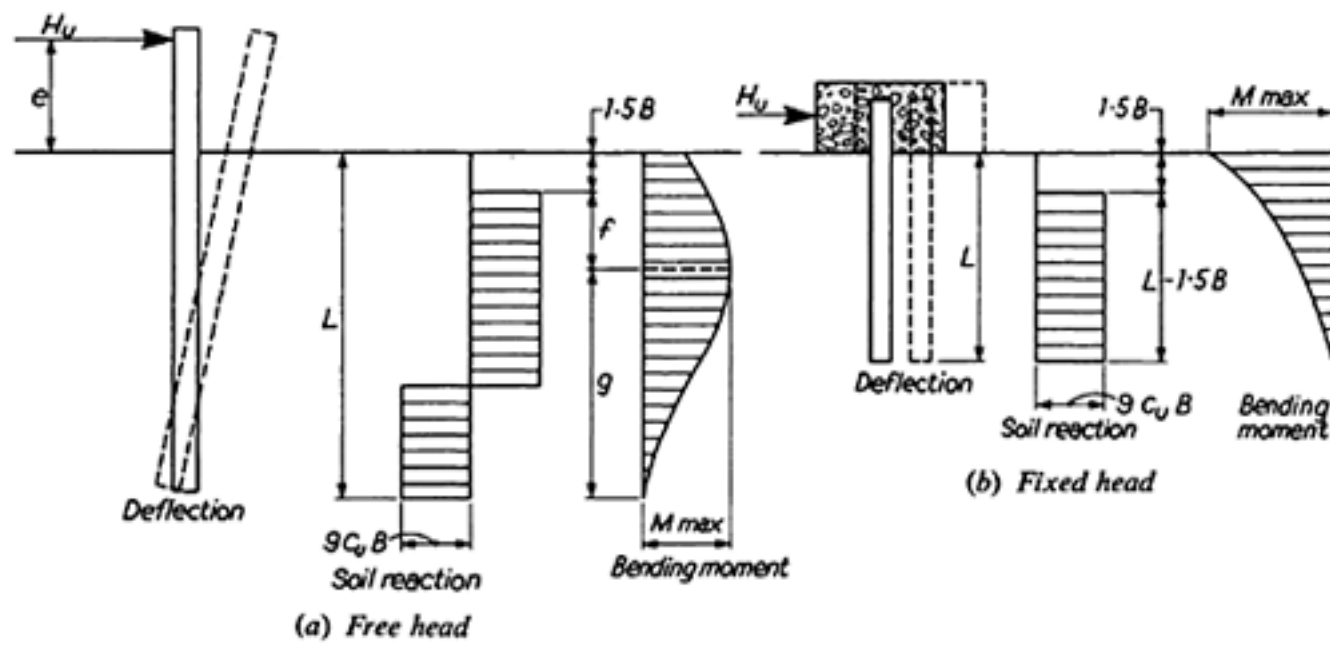


Fig. 6.24 Soil reactions and bending moments for short pile under horizontal load in cohesive soil (after Broms<sup>(6.13)</sup>)

the depth to the point of zero shear  $f$ , and from this the maximum bending moment for free-headed conditions, can be calculated from

$$f = H/9c_u B \quad \dots (6.15)$$

and

$$M_{max} = H(e + 1.5B + 0.5f) \quad \dots (6.16)$$

The part of the pile of length  $g$  resists the bending moment  $M_{max}$ , and from equilibrium in short rigid piles,

$$M_{max} = 2.25c_u Bg^2 \quad \dots (6.17)$$

The short rigid fixed-headed pile (Figure 6.24b) behaves as a simple cantilever carrying a load over part of its length, for which

$$M_{max} = \frac{1}{2} \times 9c_u B(L^2 - 2.25B^2) \quad \dots (6.18)$$

The ultimate lateral resistance for the fixed-headed pile can also be determined from Figure 6.25.

For cohesionless ( $c_u=0$ ) soils the distributions of soil reaction and bending moment are as shown in Figure 6.26. At any

depth  $z$  the unit soil reaction on the pile is given by

$$p_z = 3Bp_{oz}K_p \quad \dots (6.19)$$

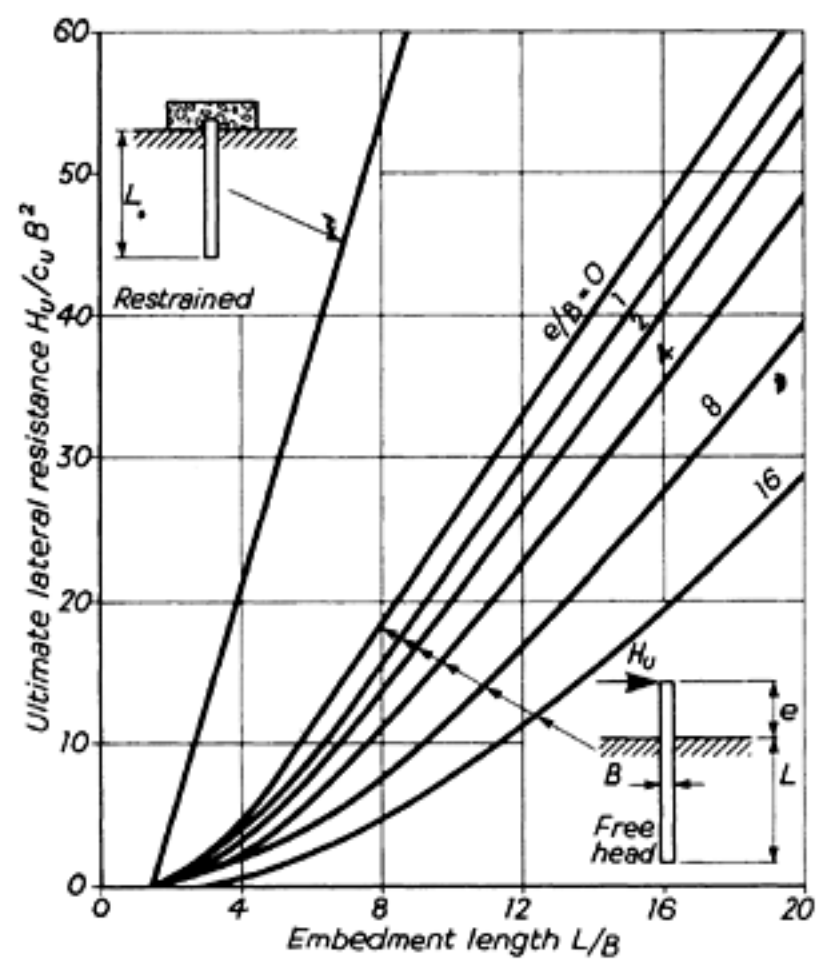


Fig. 6.25 Ultimate lateral resistance of short pile in cohesive soil related to embedded length (after Broms<sup>(6.13)</sup>)

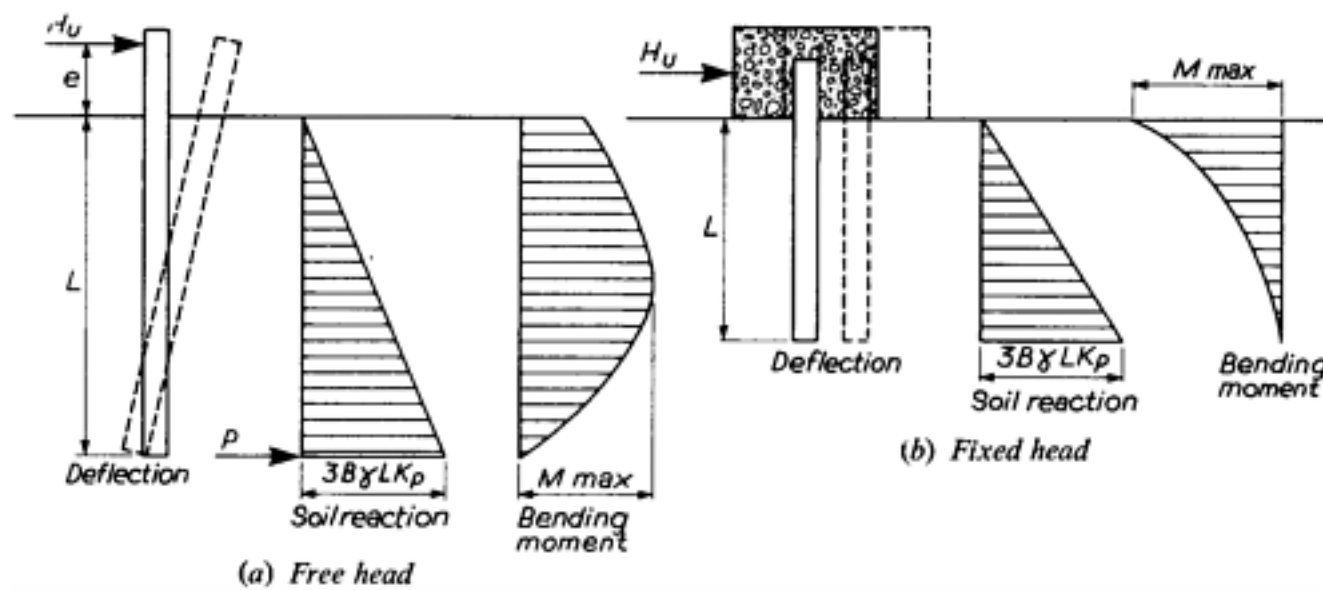


Fig. 6.26 Soil reactions and bending moments for short pile under horizontal load in cohesionless soil (after Broms<sup>(6.14)</sup>)

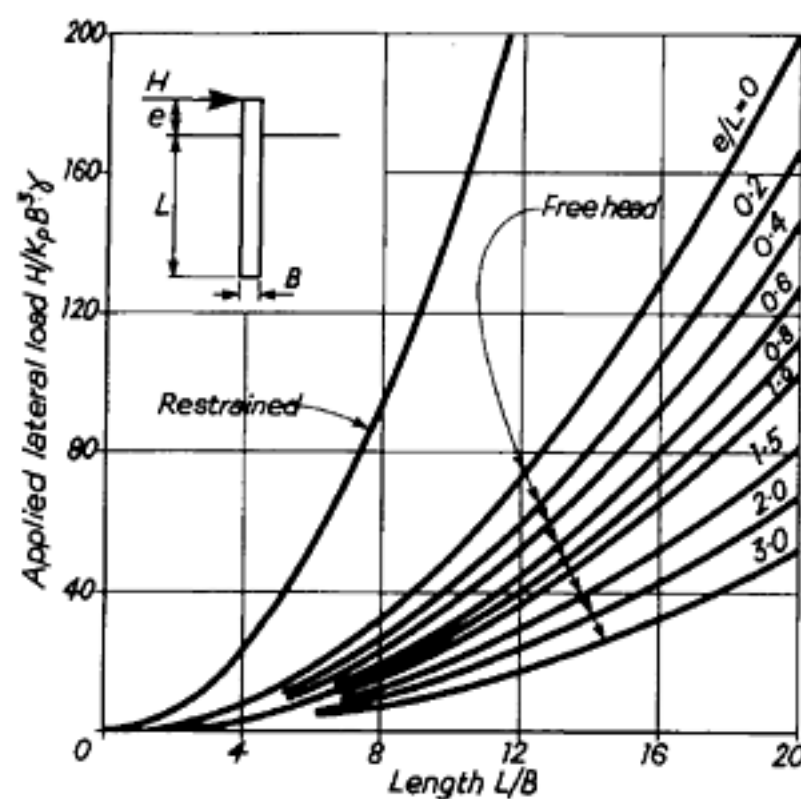


Fig. 6.27 Ultimate lateral resistance of short pile in cohesionless soil related to embedded length (after Broms<sup>(6.14)</sup>)

where  $B$  is the width of the pile perpendicular to the direction of rotation,  $p_{oz}$  is the effective overburden pressure at a depth  $z$  and  $K_p$  is Rankine's coefficient of passive pressure, *i.e.*

The high passive resistance at the toe of a pile in a cohesionless soil  $K_p = (1 + \sin \phi)/(1 - \sin \phi)$  can be replaced by a single concentrated horizontal force  $P$  and, from equation 6.19, the bending moments and shearing forces of the driving and the resistance forces can be equated to obtain the ultimate lateral resistance  $H_u$  in a similar manner to that already described

for Brinch Hansen's method.

For uniform cohesionless soils, Broms<sup>(6.14)</sup> has established the graphical relationships for  $H/k_p B^3 \gamma$  and  $L/B$  shown in [Figure 6.27](#), from which the ultimate lateral resistance  $H_u$  can be determined. It can also be obtained from

$$H_u = 0.5BL^3K_p\gamma/(e + L) \quad \dots(6.20)$$

For short rigid fixed-headed piles in cohesionless soils, failure is due to simple translation ([Figure 6.26b](#)), from which

$$H_u = 1.5B\gamma L^2K_p \quad \dots(6.21)$$

Equation 6.21 is valid only when the maximum negative bending moment which develops at the pile head is less than the ultimate resistance moment  $M_u$  of the pile at this point. The bending moment is given by

$$M_{max(-ve)} = B\gamma L^3 K_p \quad \dots(6.22)$$

### Ultimate resistance of long piles

The passive resistance provided by the soil to the yielding of an infinitely long pile is infinite. Thus the ultimate lateral load which can be carried by the pile is determined solely from the ultimate moment of resistance  $M_u$  of the pile shaft.

A simple method of calculating the ultimate load, which may be sufficiently accurate for cases of light loading on short or long piles of small to medium width, for which the cross-sectional area is governed by considerations of the relatively-higher compressive loading, is to assume an arbitrary depth  $z_f$  to the point of virtual fixity. Then from [Figure 6.28](#),

$$\text{ultimate lateral load on free-headed pile } H_u = M_u / (e + z_f) \quad \dots(6.23)$$

$$\text{ultimate lateral load on fixed-headed pile } H_u = 2M_u / (e + z_f) \quad \dots(6.24)$$

Arbitrary values for  $z_f$  which are commonly used are given in the reference to the Brinch Hansen method.

The ultimate moment of resistance of the pile is also taken by Broms ([6.13](#), [6.14](#)) as the criterion of failure of a long pile. A plastic hinge capable of resisting shear is assumed to form at the point of failure of the shaft. The soil reactions and bending moments for cohesive soils are shown in [Figure 6.29](#).

The maximum (positive) bending moment for a free-headed pile is given by

$$M_{\max(+ve)} = H(e + 1.5B + 0.5f) \quad \dots(6.25)$$

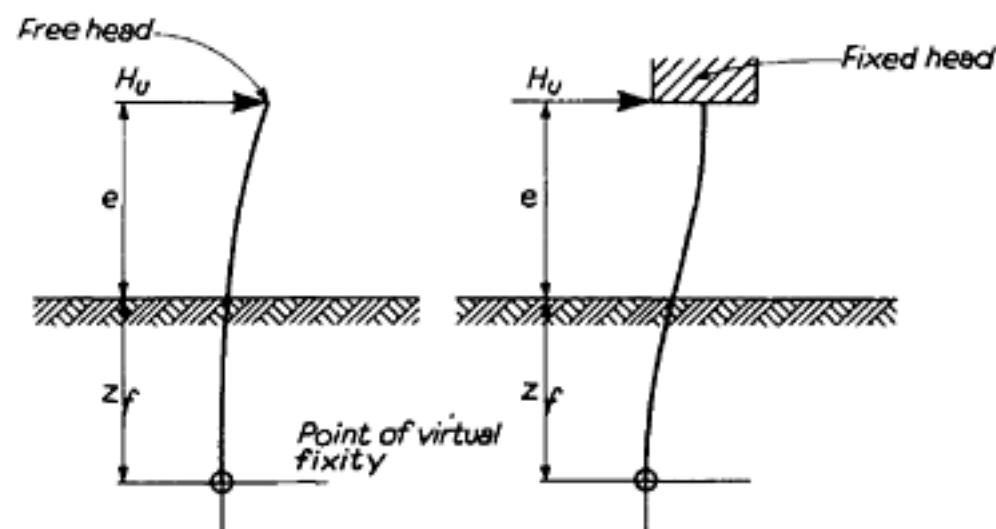
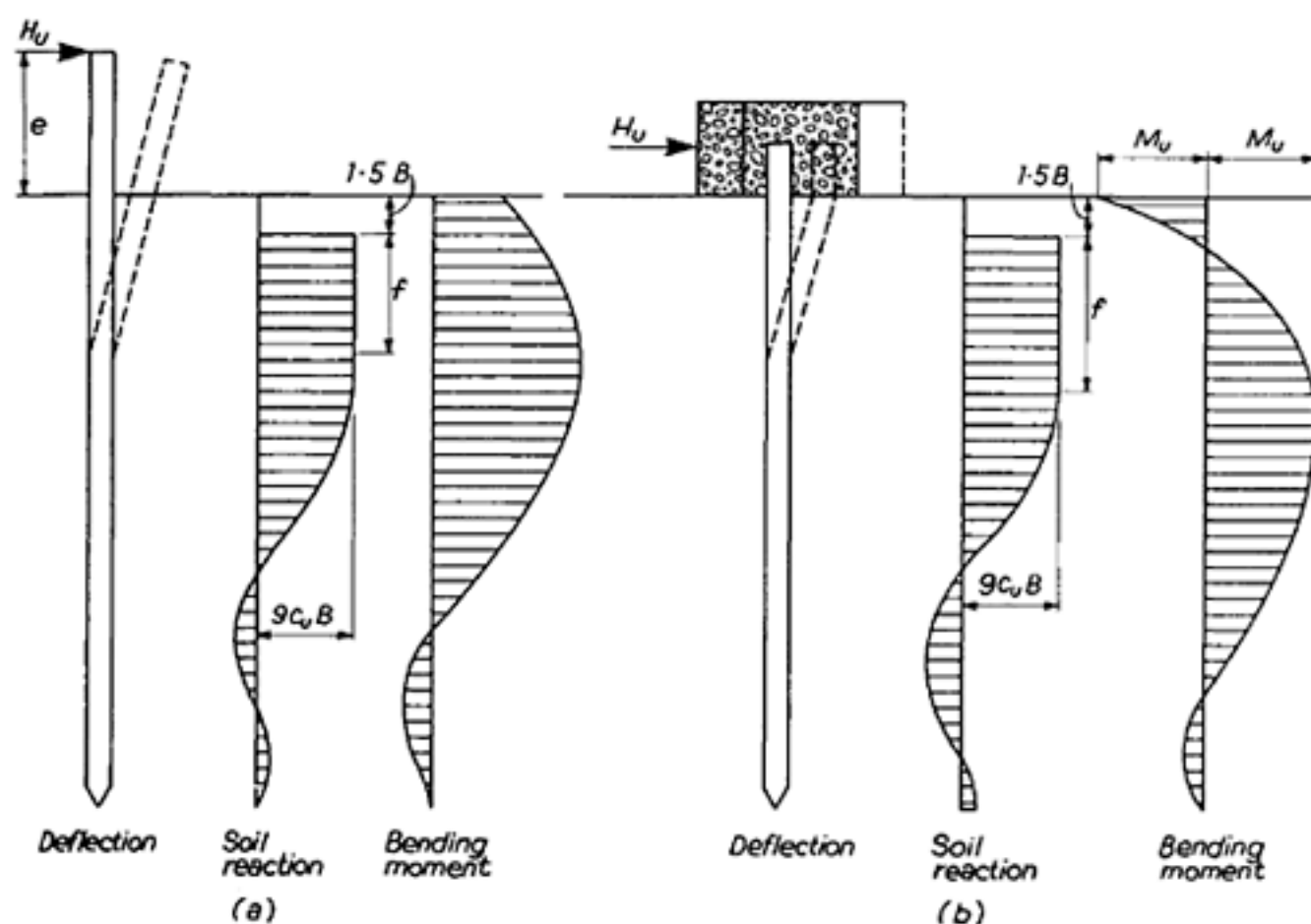


Fig. 6.28 Piles under horizontal load considered as simple cantilevers



*Fig. 6.29 Soil reactions and bending moments for long pile under horizontal load in cohesive soil  
(after Broms<sup>[6.13](#)</sup>)*

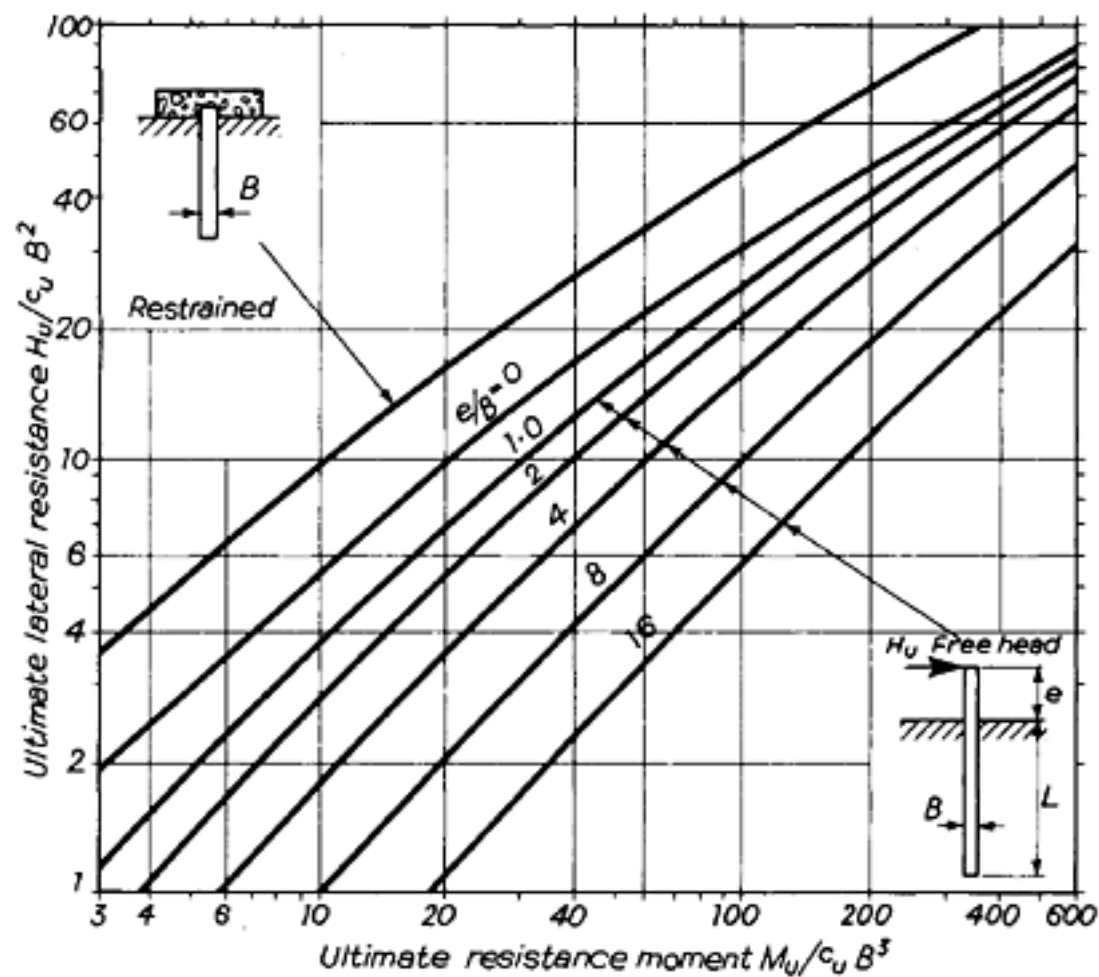


Fig. 6.30 Ultimate lateral resistance of long pile in cohesive soil related to ultimate resistance moment (after Broms<sup>(6.13)</sup>)

where  $f = H/9c_u B$ . The ultimate value of  $H$  is the load at which  $M_{max(+ve)}$  is equal to the ultimate resistance moment of the pile shaft. Thus

$$H_u = \frac{M_u}{(e + 1.5B + 0.5f)} \quad \dots(6.26)$$

For fixed-headed piles in cohesive soils (Figure 6.29b),

$$H_u = \frac{2M_u}{(1.5B + 0.5f)} \quad \dots(6.27)$$

Broms<sup>(6.13)</sup> has established the graphical relationship for  $H_u/c_u B^2$  and  $M_u/c_u B^3$  shown in Figure 6.30 for free- and fixed-headed piles, from which  $H_u$  can be determined.

For long piles in cohesionless soils the soil reactions and bending moments for free-headed piles are shown in Figure 6.31a. The maximum bending moment on the pile shaft occurs at the point where the shearing force is zero. Thus for free-headed piles,

$$f = 0.82 \sqrt{\frac{H}{\gamma B K_p}} \quad \dots(6.28)$$

The corresponding maximum positive bending moment is given by

$$M_{max(+ve)} = H(e + 0.67f) \quad \dots(6.29)$$

For zero bending moment at the pile head, the ultimate lateral load is given by

$$H_u = \frac{M_u}{e + 0.54 \sqrt{\frac{H_u}{\gamma B K_p}}} \quad \dots(6.30)$$

Broms<sup>(6.14)</sup> has established the graphical relationship between  $H/K_p \gamma B^3$  and  $M_u/\gamma K_p B^4$  shown in [Figure 6.32](#). These graphs can be used to determine the ultimate lateral load  $H_u$ .

The soil reactions and bending moments for fixed-headed piles are shown in [Figure 6.31b](#). The maximum negative bending moment occurs at the pile head and at the ultimate load it is equal to the ultimate moment of resistance of the pile shaft. The ultimate lateral load is given by

$$H_u = \frac{M_{u(+ve)} + M_{u(-ve)}}{e + 0.54 \sqrt{\frac{H_u}{\gamma B K_p}}} \quad \dots(6.31)$$

For a pile of uniform cross-section,

$$H_u = \frac{2M_u}{e + 0.54 \sqrt{\frac{H_u}{\gamma B K_p}}} \quad \dots(6.32)$$

Broms' graphical relationships for fixed-headed long piles are also shown in [Figure 6.32](#).

It is evident from the worked examples that the ultimate lateral load for a long pile is much higher than that for a short pile of the same cross-section. Therefore when lateral loads are to be carried by vertical piles the aim should be to provide piles of sufficient length to develop their maximum resistance.

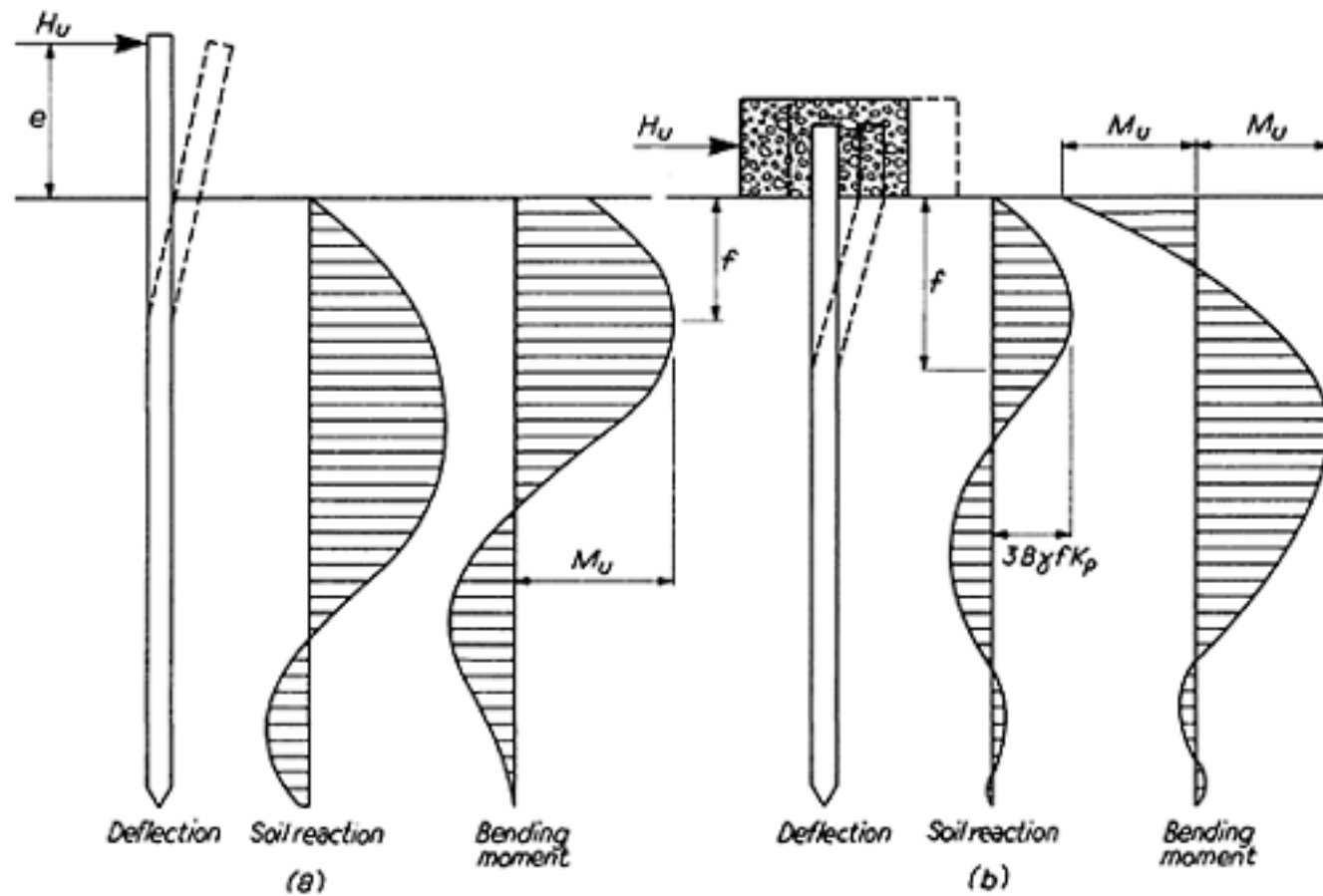


Fig. 6.31 Soil reactions and bending moments for long pile under horizontal load in cohesionless soil (after Broms<sup>(6.14)</sup>)

- (a) Free head  
(b) Fixed head

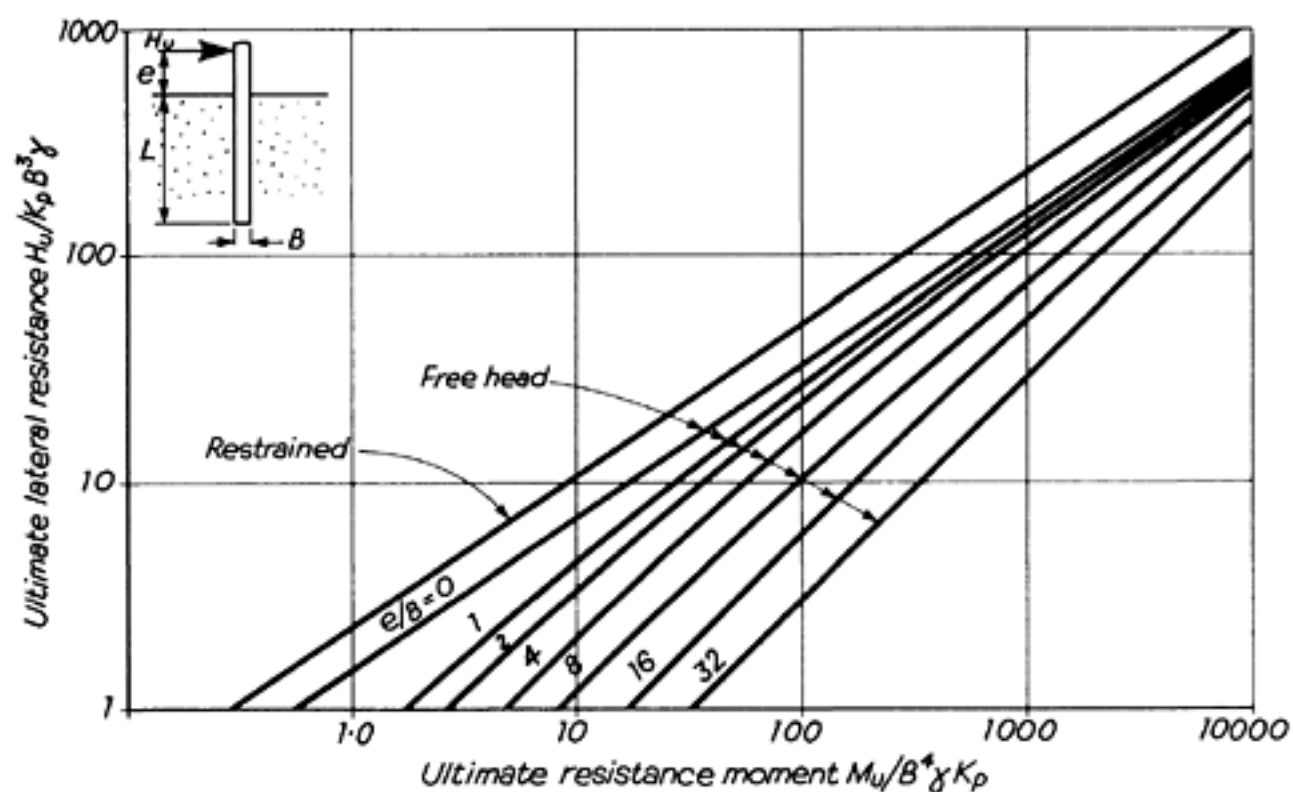


Fig. 6.32 Ultimate lateral resistance of long pile in cohesionless soil related to ultimate resistance moment (after Broms<sup>(6.14)</sup>)

There is, however, no point in taking piles to greater depths than the following.

$$\begin{aligned} L &= 4T && \text{(for linearly increasing soil modulus)} \\ L &= 3.5R && \text{(for constant soil modulus and free head)} \\ L &= 2R && \text{(for constant soil modulus and fixed head)} \end{aligned}$$

Considerations of the resistance to compressive or uplift loads may require greater penetration depths.

It has already been stated that vertical piles offer poor resistance to lateral loads. However, in some circumstances it may be justifiable to add the resistance provided by the passive resistance of the soil at the end of the pile cap and the friction or cohesion on the embedded sides of the cap. The pile cap resistance can be taken into account when the external loads are transient in character, such as wind gusts and traffic loads, but the resulting elastic deformation of the soil must not be so great as to cause excessive deflection and hence overstressing of the piles. The design of pile caps to resist lateral loading is discussed in 7.9.

### 6.3.2 Bending and buckling of partly embedded single vertical piles

A partly-embedded vertical pile may be required to carry a vertical load in addition to a lateral load and a bending moment at its head. The stiffness factors  $R$  and  $T$  as calculated from equations 6.8 and 6.9 have been used by Davisson and Robinson (6.20) to obtain the equivalent length of a free-standing pile with a fixed base, from which the factor of safety against failure due to buckling can be calculated using conventional structural design methods.

A partly-embedded pile carrying a vertical load  $P$ , a horizontal load  $H$ , and a moment  $M$  at a height  $e$  above the ground surface is shown in Figure 6.33a. The equivalent height  $L_e$  of the fixed-base pile is shown in Figure 6.33b.

For soils having a constant modulus:

$$\text{Depth to point of fixity } z_f = 1.4R \quad \dots(6.33)$$

For soils having a linearly-increasing modulus:

$$z_f = 1.8T \quad \dots(6.34)$$

The relationships 6.33 and 6.34 are only approximate, but Davisson and Robinson state that they are valid for structural design purposes provided that  $l_{max}$ , which is equal to  $L/R$ , is greater than 4 for soils having a constant modulus and provided

that  $z_{max}$ , which is equal to  $L/T$ , is greater than 4 for soils having a linearly-increasing modulus. From equations 6.33 and

6.34 the equivalent length  $L_e$  of the fixed-base pile (or column) is equal to  $e + z_f$  and the critical load for buckling

$$P_{cr} = \frac{\pi^2 EI}{4(e + z_f)^2} \text{ for free-headed conditions} \quad \dots(6.35)$$

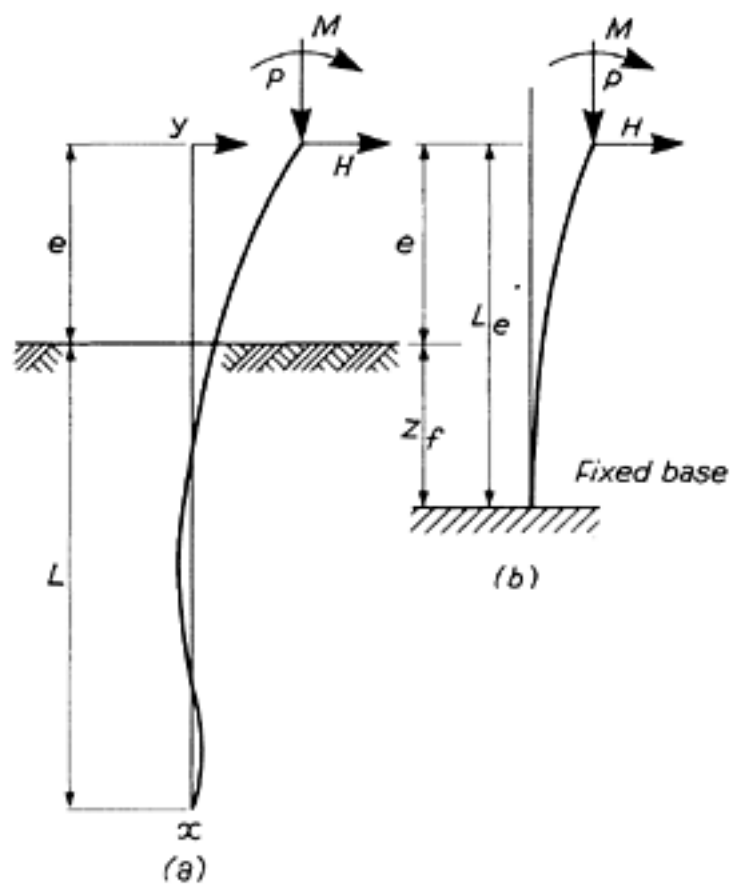


Fig. 6.33 Bending of pile carrying vertical and horizontal loads at head

- (a) Partly-embedded pile
- (b) Equivalent fixed base pile or column

and

$$P_{cr} = \frac{\pi^2 EI}{(e + z_f)^2} \text{ for fixed- (and translating-) headed conditions} \quad \dots(6.36)$$

### 6.3.3 The deflection of vertical piles carrying lateral loads

A simple method which can be used to check that the deflections due to small lateral loads are within tolerable limits and as an approximate check on the more-rigorous methods described below, is to assume that the pile is fixed at an arbitrary depth below the ground surface and then to calculate the deflection as for a simple cantilever either free at the head, or fixed at the head but with freedom to translate. Thus from [Figure 6.28](#),

$$\text{deflection at head of free-headed pile } y = \frac{H(e + z_f)^3}{3EI} \quad \dots(6.37)$$

and

$$\text{deflection at head of fixed-headed pile } y = \frac{H(e + z_f)^3}{12EI} \quad \dots(6.38)$$

where  $E$  is the elastic modulus of the material forming the pile shaft, and  $I$  is the moment of inertia of the cross-section of the pile shaft. Depths which may be arbitrarily assumed for  $z_f$  are noted in [Section 6.3.2](#).

More-rigorous methods of calculating deflections, which are based on the concept of the modulus of horizontal subgrade reaction, have been described by Broms([6.13](#), [6.14](#)) as follows.

Piles in cohesive soils

The deflection behaviour depends primarily on the dimensionless length  $\beta L$  where

$$\beta = \sqrt[4]{\frac{kB}{4E_{pile} I_{pile}}} \quad \dots(6.39)$$

A free-headed pile behaves as a rigid (infinitely stiff) element when  $\beta L$  is less than 1.5. For these conditions,

$$\text{lateral deflection at ground surface } y_0 = \frac{4H\left(1 + \frac{1.5e}{L}\right)}{kBL} \quad \dots(6.40)$$

A fixed-headed pile behaves as a rigid element when  $\beta L$  is less than 0.5, for which

$$y_0 = \frac{H}{kBL} \quad \dots(6.41)$$

where  $k$  is the coefficient of subgrade reaction.

The above equations are not particularly sensitive to the value of  $k$  which is, as has already been stated, a variable quantity depending on the depth below the surface. In the case of short rigid piles it is sufficiently accurate to take  $k$  as  $k_1$  [the modulus for the 300mm square plate] for the case of a soil with a constant modulus, and as the average value of  $k_1$  over a depth equal to  $0.8 \beta L$  for a soil with a linearly-increasing modulus. Values of  $k_1$  for cohesive soils are shown in [Table 6.2](#).

For the case of infinitely long flexible piles, for which  $\beta L$  is greater than 2.5, the lateral deflection at the ground surface of the free-headed pile

$$y_0 = \frac{2H\beta(e\beta + 1)}{k_x B} \quad \dots(6.42)$$

The fixed-headed pile behaves as an infinitely long element when  $\beta L$  is greater than 1.5, and the lateral deflection

$$y_0 = \frac{H\beta}{k_\infty B} \quad \dots(6.43)$$

Broms<sup>(6.13)</sup> has established a graphical relationship between  $y_0 kBL/H$  and  $\beta L$  as shown in [Figure 6.34](#), from which the deflection at the ground surface  $y_0$  can be determined.

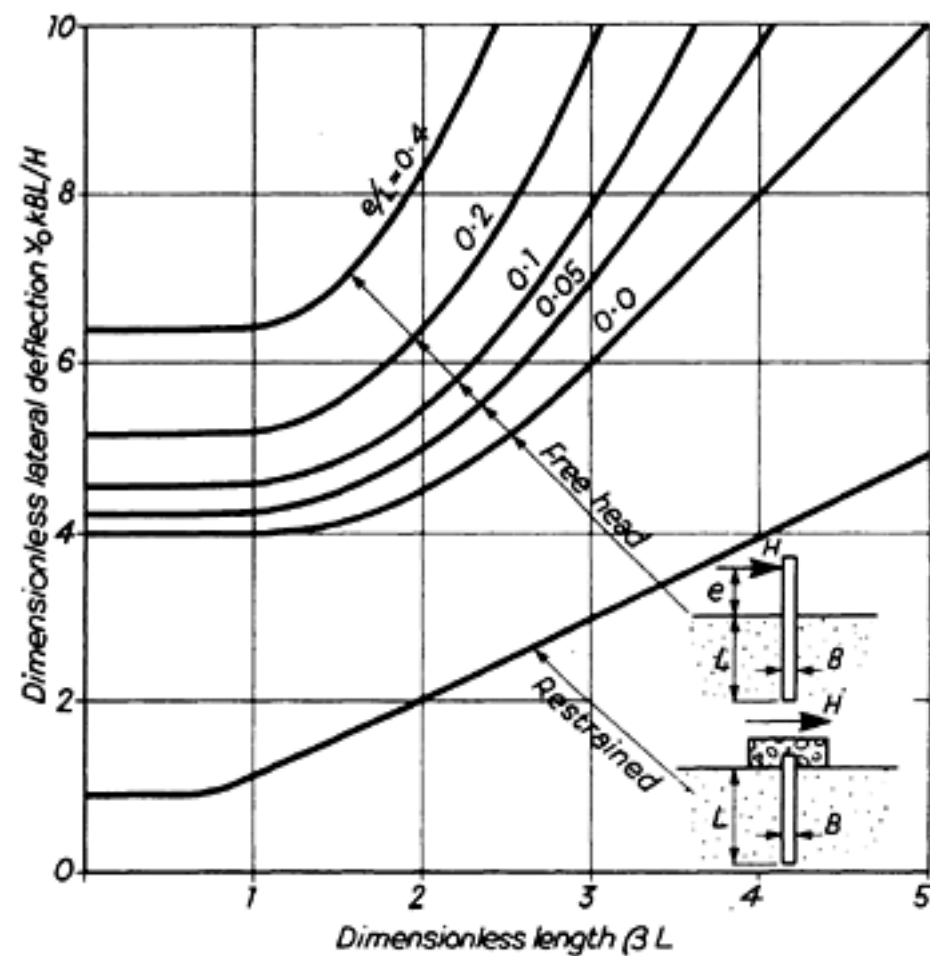


Fig. 6.34 Charts for calculating lateral deflection at ground surface of horizontally loaded pile in cohesive soil (after Broms<sup>(6.13)</sup>)

The coefficient of subgrade reaction  $k$  for the infinitely long pile is calculated from

$$k_{\infty} = \alpha K_0/B \tag{6.44}$$

where  $\alpha$  is equal to  $0.52 \sqrt[12]{K_0 B^4/E_{pile} I_{pile}}$  and  $K_0$  is a coefficient of subgrade reaction. Where the Terzaghi modulus is used  $K_0 = 0.305 k_1$ .

The calculated deflection is not particularly sensitive to the value of  $\alpha$  and Broms<sup>(6.13)</sup> has shown that, for practical purposes, in equation 6.44 can be calculated from

$$\alpha = n_1 \times n_2 \tag{6.45}$$

where the dimensionless factors  $n_1$  and  $n_2$  are related to the compressive strength of the soil and the material forming the pile respectively. Assuming that the shearing strength of a clay is equal to one-half of the unconfined compressive strength, the values of  $n_1$  are shown in Table 6.3. Values of  $n_2$  for various pile materials are shown in Table 6.4.

Table 6.3 Values of coefficient  $n_1$  (after Broms<sup>(6.13)</sup>)

Shearing strength kN/m <sup>2</sup>	Coefficient $n_1$
<27	0.32
27–107	0.36
>107	0.40

Table 6.4 Values of coefficient  $n_2$  (after Broms<sup>(6.13)</sup>)

Material forming pile	Coefficient $n_2$
Steel	1.00

Concrete	1.15
Wood	1.30

Where the results of plate loading tests (made with plates set in a vertical position) are available, the coefficient  $K_0$  in equation 6.44 can be obtained from the secant modulus of the soil. A secant modulus  $E_{50}$ , corresponding to a pressure on the soil of 50% of the pressure at the failure of the plate, is used in equation

$$K_0 = 1.67E_{50} \qquad \dots(6.46)$$

The above equations for calculating deflections are all based on elastic theory and take no account of deflections caused by the long-term consolidation of the soil under sustained lateral loading. This can be done by using methods based on elastic continuum theory described in [Section 6.3.8](#). Methods of calculating pile displacements in conditions of a yielding soil are described in [Section 6.3.5](#).

Piles in cohesionless soils

Broms<sup>(6.14)</sup> has related the deflection of a pile embedded in a cohesionless soil carrying a horizontal load  $H$  to the dimensionless depth of embedment  $\eta L$  where

$$\eta = \sqrt[5]{n_h/EI} \quad \dots(6.47)$$

Values of the relationship between  $y_0(EI)^{3/5}n_h^{2/5}/HL$  and  $\eta L$  for various values of  $e/L$  are shown in [Figure 6.35](#). These curves show that for values of  $\eta L$  of less than 2 the pile behaves as an infinitely stiff member, while when  $\eta L$  is greater than 4 the pile behaves as an infinitely long member for which the deflections at the ground surface are unaffected by the depth of embedment.

The deflections  $y_0$  of the pile at ground level can be calculated directly without recourse to [Figure 6.35](#) from the following equations.

Short pile ( $\eta L < 2$ ) with free head:

$$y_0 = \frac{18H \left(1 + \frac{1.33e}{L}\right)}{L^2 n_h} \quad \dots(6.48)$$

Short pile ( $\eta L < 2$ ) with fixed head:

$$y_0 = \frac{2H}{L^2 n_h} \quad \dots(6.49)$$

Long pile ( $\eta L > 4$ ) with free head:

$$y_0 = \frac{2.4H(1 + 0.67e\eta)}{n_h^{3/5}(EI)^{2/5}} \quad \dots(6.50)$$

Long pile ( $\eta L > 4$ ) with fixed head:

$$y_0 = \frac{0.93H}{n_h^{3/5}(EI)^{2/5}} \quad \dots(6.51)$$

Values of the coefficient of modulus variation  $n_h$  are obtained from [Figure 6.20](#). Broms states that equations 6.49 and 6.50

are not applicable to very short piles, *i.e.* with a length of less than 4 diameters, because of the restraining effect of soil friction beneath the toes of such piles.

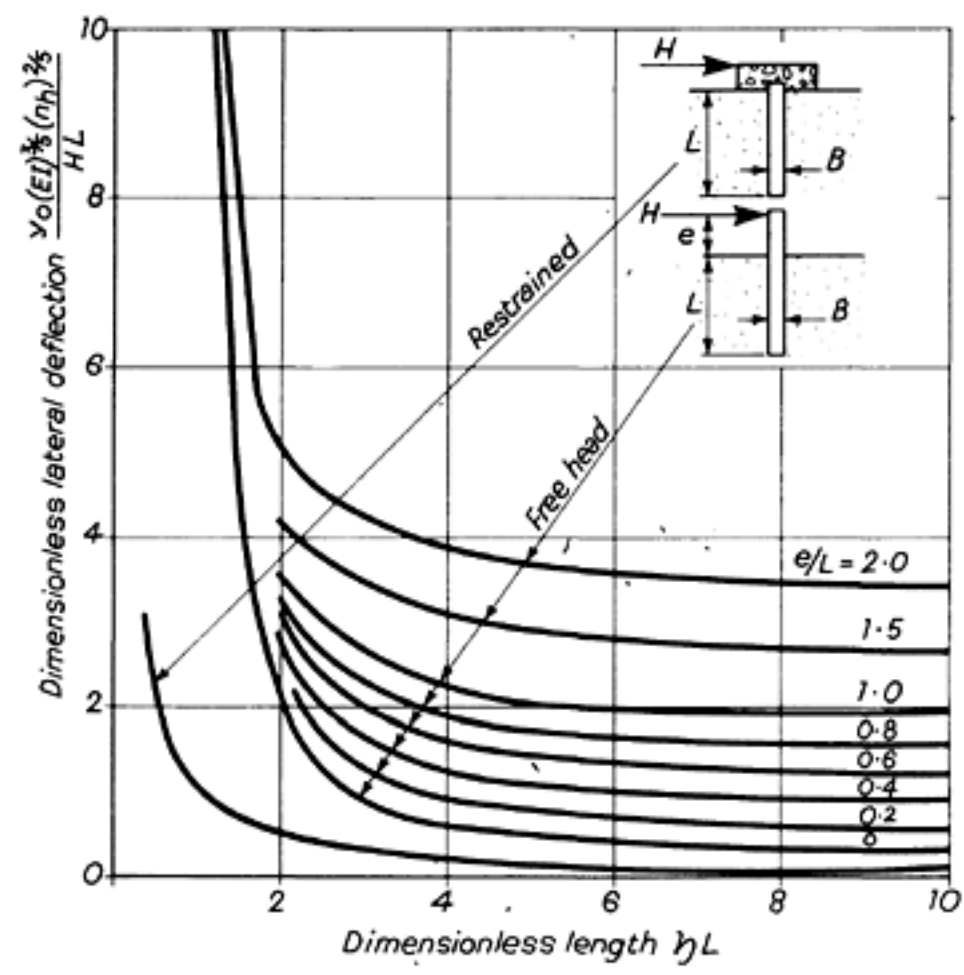


Fig. 6.35 Charts for calculating lateral deflection at ground surface of horizontally loaded pile in cohesionless soil (after Broms<sup>(6.14)</sup>)

### 6.3.4 Elastic analysis of laterally-loaded vertical piles

The suggested procedure for using sections [6.3.2](#) and [6.3.3](#) is first to calculate the ultimate load  $H_u$  for a pile of given cross-section (or to determine the required cross-sections for a given ultimate load) and then to divide  $H_u$  by an arbitrary safety factor to obtain the working load  $H$ . The alternative procedure is to calculate the deflection  $y_0$  at the ground surface for a range of progressively increasing loads  $H$  up to the value of  $H_u$ . The working load is then taken as the load at which  $y_0$  is within the allowable limits. The deflection  $y_0$  can be obtained from the equations or from the graphical relationships shown in [6.3.3](#).

However, it may be necessary to determine the bending moments, shearing forces and deformed shape of a pile over its full depth at a selected working load. These can be obtained for working-load conditions on the assumption that the pile behaves as an elastic beam on a soil which also behaves elastically. Calculations for the bending moments, shearing forces, deflections and slopes of laterally-loaded piles are necessary when considering their behaviour as energy absorbing members resisting the berthing impact of ships (see [8.1.1](#)), or the wave forces in offshore platform structures (see [8.2](#)).

Reese and Matlock([6.21](#)) have established a series of curves for normally consolidated and cohesionless soils for which the elastic modulus of the soil  $E_s$  is assumed to increase from zero at the ground surface in direct proportion to the depth. The

deformed shape of the pile and the corresponding bending moments, shearing forces and soil reactions are shown in [Figure 6.36](#).

Coefficients for obtaining these values are shown for a lateral load  $H$  on a free pile head in [Figures 6.37a to 6.37e](#), and for a moment applied to a pile head in [Figures 6.38a to 6.38e](#). The coefficients for a fixed pile head are shown in [Figures 6.39a to 6.39c](#). For combined lateral loads and applied moments the basic equations for use in conjunction with [Figures 6.37](#) and [6.38](#) are as follows.

$$\text{Deflection } y = y_A + y_B = \frac{A_y H T^3}{EI} + \frac{B_y M_t T^2}{EI} \quad \dots(6.52)$$

$$\text{Slope} = s_A + s_B = \frac{A_s H T^2}{EI} + \frac{B_s M_t T}{EI} \quad \dots(6.53)$$

$$\text{Bending moment} = M_A + M_B = A_m H T + B_m M_t \quad \dots(6.54)$$

$$\text{Shearing force} = V_A + V_B = A_v H + \frac{B_v M_t}{T} \quad \dots(6.55)$$

$$\text{Soil reaction} = P_A + P_B = \frac{A_p H}{T} + \frac{B_p M_t}{T^2} \quad \dots(6.56)$$

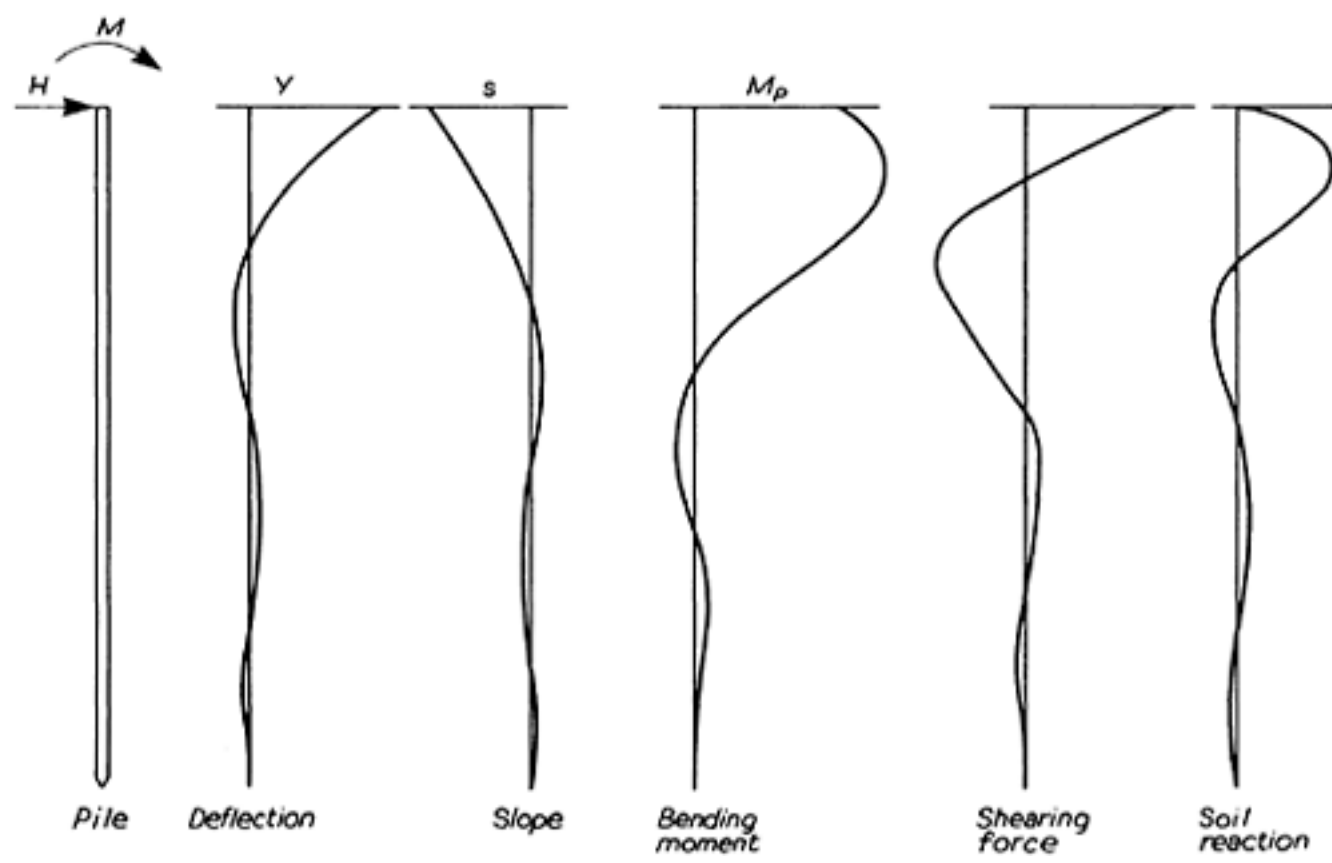


Fig. 6.36 Deflections, slopes, bending moments, shearing forces, and soil reactions for elastic conditions (after Reese and Matlock<sup>(6.21)</sup>)

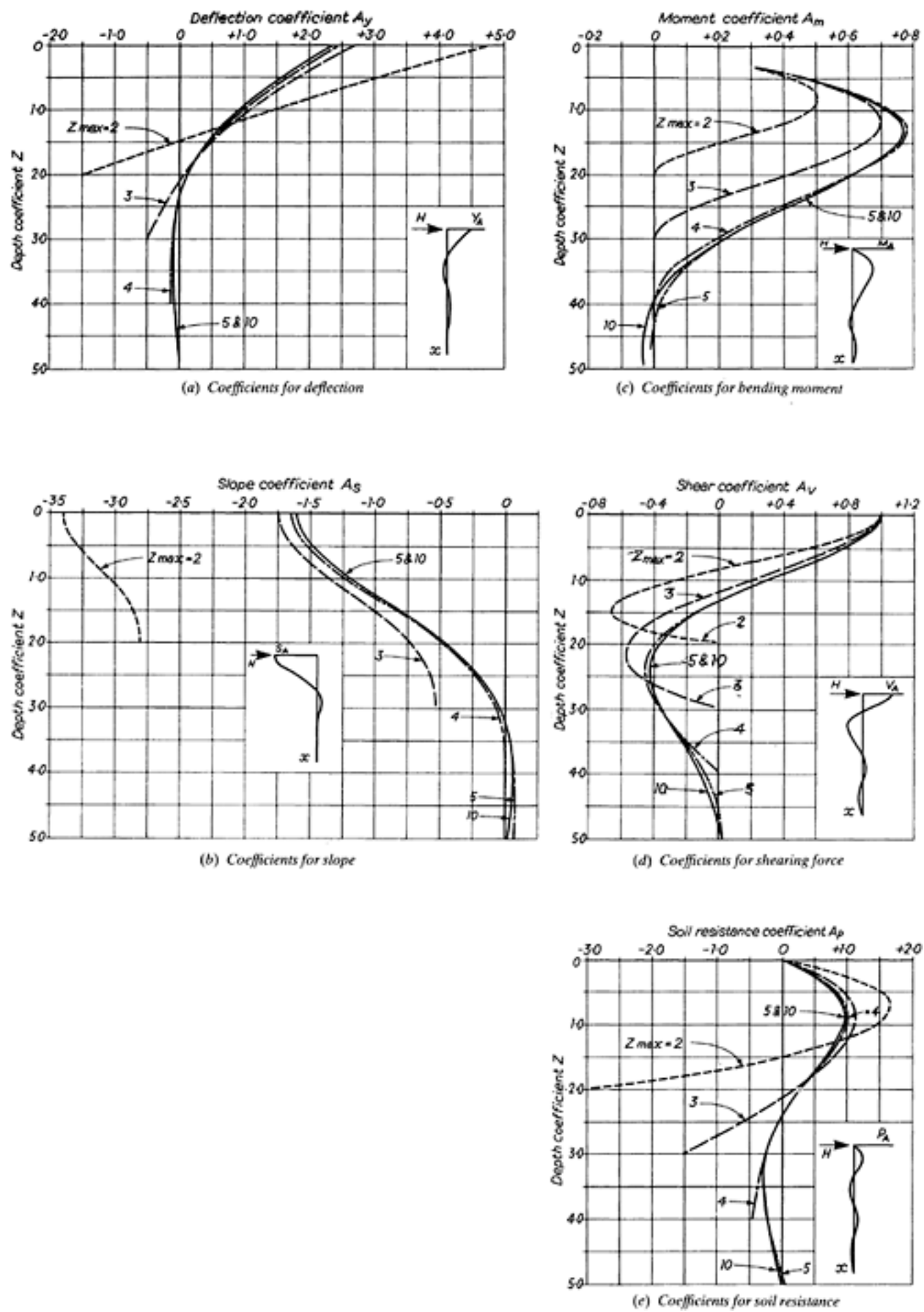


Fig. 6.37 Coefficients for laterally loaded free-headed piles in soil with linearly increasing modules (after Reese and Matlock<sup>(6.21)</sup>)

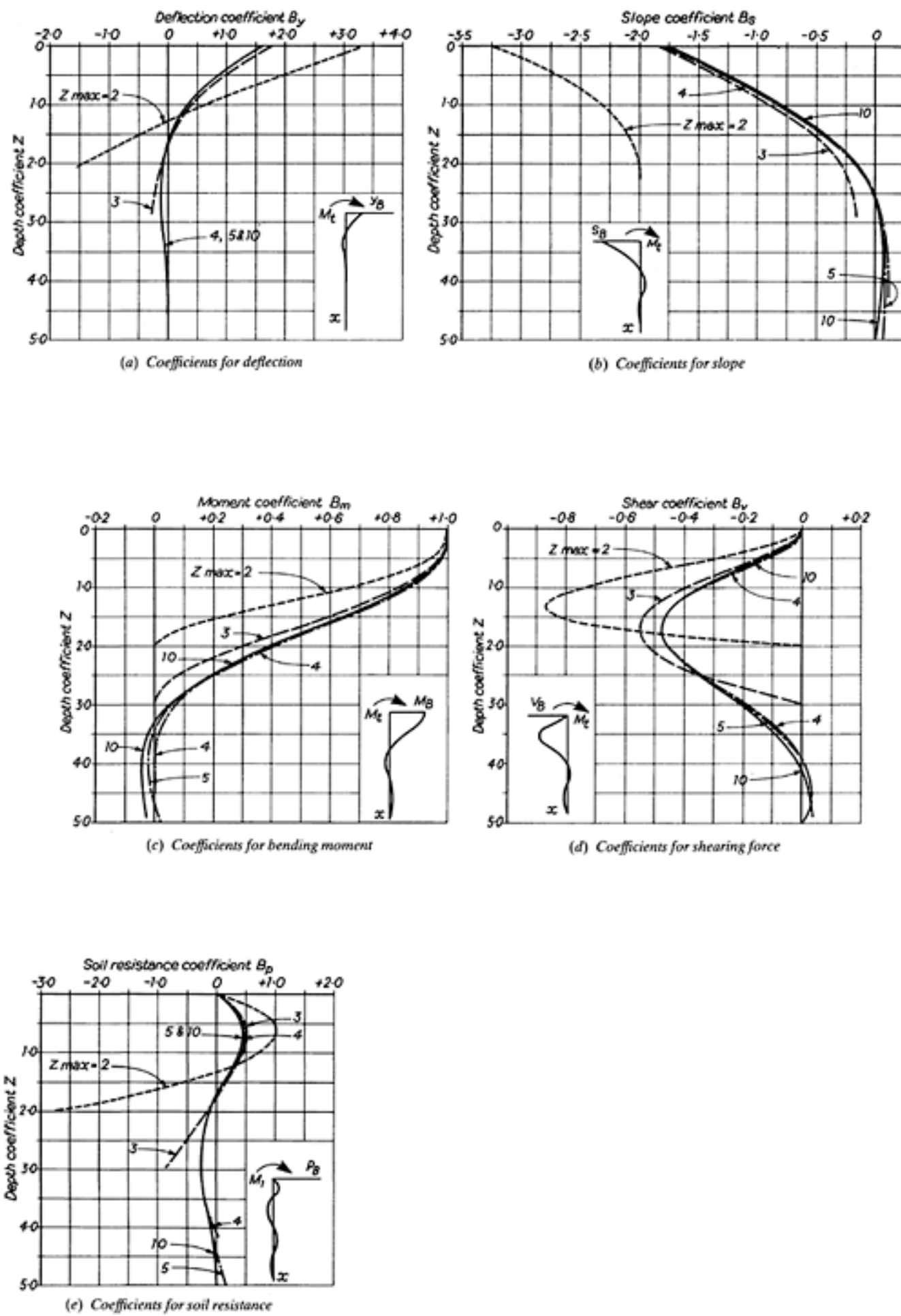


Fig. 6.38 Coefficients for piles with moment at free head in soil with linearly increasing modulus (after Reese and Matlock<sup>(6.21)</sup>)

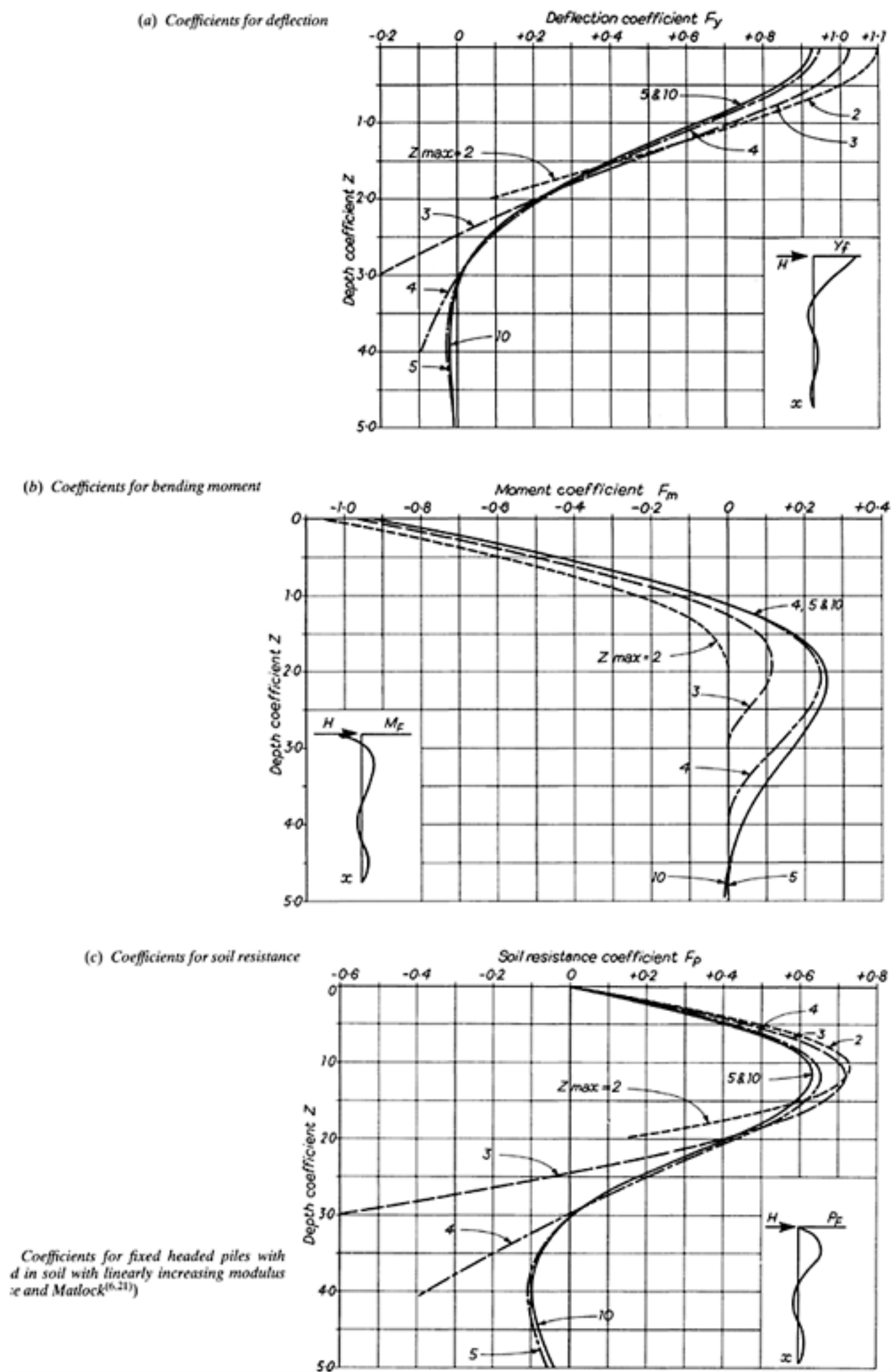


Fig. 6.39 Coefficients for fixed headed piles with lateral load in soil with linearly increasing modulus (after Reese and Matlock<sup>(6,21)</sup>)

For a fixed pile head the basic equations are as follows.

$$\text{Deflection} = y_F = \frac{F_y H T^3}{EI} \quad \dots(6.57)$$

$$\text{Bending moment} = M_F = F_m H T \quad \dots(6.58)$$

$$\text{Soil reaction} = P_F = F_p \frac{H}{T} \quad \dots(6.59)$$

In equations 6.52 to 6.59,  $H$  is the horizontal load applied to the ground surface,  $T$  (a stiffness factor)  $= \sqrt[5]{EI/n_h}$  (as equation 6.9),  $M_t$  is the moment applied to the head of the pile,  $A_y$  and  $B_y$  are deflection coefficients (Figures 6.37a and 6.38a),  $A_s$  and  $B_s$  are slope coefficients (Figures 6.37b and 6.38b),  $A_m$  and  $B_m$  are bending-moment coefficients (Figures 6.37c and 6.38c),  $A_v$  and  $B_v$  are shearing-force coefficients (Figures 6.37d and 6.38d),  $A_p$  and  $B_p$  are soil reaction coefficients (Figures 6.37e and 6.38e),  $F_y$  is the deflection coefficient for a fixed pile head (Figure 6.39a),  $F_m$  is the moment coefficient for a fixed pile head (Figure 6.39b), and  $F_p$  is the soil reaction coefficient for a fixed pile head (Figure 6.39c).

In Figures 6.37 to 6.39 the above coefficients are related to a depth coefficient  $Z$  for various values of  $Z_{max}$ , where  $Z$  is equal to the depth  $x$  at any point divided by  $T$  (i.e.  $Z=x/T$ ) and  $Z_{max}$  is equal to  $L/T$ . The use of curves in Figure 6.39 is illustrated in example 6.6.

The case of a load  $H$  applied at a distance  $e$  above the ground surface can be simulated by assuming this to produce a bending moment  $M_t$  equal to  $H \times e$ , this value of  $M_t$  being used in equations 6.52 to 6.59. The moments  $M_a$  produced by load  $H$  applied at the soil surface are added arithmetically to the moments  $M_b$  produced by moment  $M_t$  applied to the pile at the

ground surface. This yields the relationship between the total moment and the depth below the soil surface over the embedded length of the pile. The deflection of a pile due to a lateral load  $H$  at some distance above the soil surface is calculated in the same manner. The deflections of the pile and the corresponding slopes due to the load  $H$  at the soil surface are calculated and added to the values calculated for moment  $M_t$  applied to the pile at the surface. To obtain the deflection at

the head of the pile, the deflection as for a free-standing cantilever fixed at the soil surface is calculated and added to the deflection produced at the soil surface by load  $H$  and moment  $M_t$ , together with the deflection corresponding to the calculated

slope of the pile at the soil surface. This procedure is illustrated in example 8.2.

Davisson and Gill(6.22) have analysed the case of elastic piles in an elastic soil of constant modulus. The bending moments and deflections are related to the stiffness coefficient  $R$  (equation 6.8) but in this case the value of  $K$  is taken as Terzaghi's subgrade modulus  $k_1$ , using the values shown in Table 6.2. The dimensionless depth coefficient  $Z$  in Figure 6.40 is equal to  $x/R$ . From these curves, deflection and bending moment coefficients are obtained for free-headed piles carrying a moment at the pile head and zero lateral load (Figure 6.40a) and for free-headed piles with zero moment at the pile head and carrying a horizontal load (Figure 6.40b). These curves are valid for piles having an embedded length  $L$  greater than  $2R$  and different moment and deflection curves are shown for values of  $Z_{max} = L/R$  of 2, 3, 4 and 5. Piles longer than  $5R$  should be analysed for  $Z_{max} = 5$ . The equations to be used in conjunction with the curves in Figure 6.40 are as follows.

*Load on pile head For free-headed pile:*

$$\text{Moment } M; \quad \text{Bending moment} = M M_m \quad \dots(6.60)$$

$$\text{Moment } M; \quad \text{Deflection} = M y_m \frac{R^2}{EI} \quad \dots(6.61)$$

$$\text{Horizontal load } H; \quad \text{Bending moment} = H M_h R \quad \dots(6.62)$$

$$\text{Horizontal load } H; \quad \text{Deflection} \quad = Hy_h \frac{R^3}{EI} \quad \dots(6.63)$$

The effect of fixity at the pile head can be allowed for by plotting the deflected shape of the pile from the algebraic sum of the deflections (equations 6.61 and 6.62) and then applying a moment to the head which results in zero slope for complete fixity, or the required angle of slope for a given degree of fixity. The deflection for this moment is then deducted from the calculated value for the free-headed pile. The use of the curves in [Figure 6.40](#) is illustrated in example 8.2. Conditions of partial fixity occur in jacket-type offshore platform structures where the tubular jacket member only offers partial restraint to the pile that extends through it to below sea-bed level.

Where marine structures are supported by long piles ( $L \geq 4T$ ), Matlock and Reese([6.23](#)) have simplified

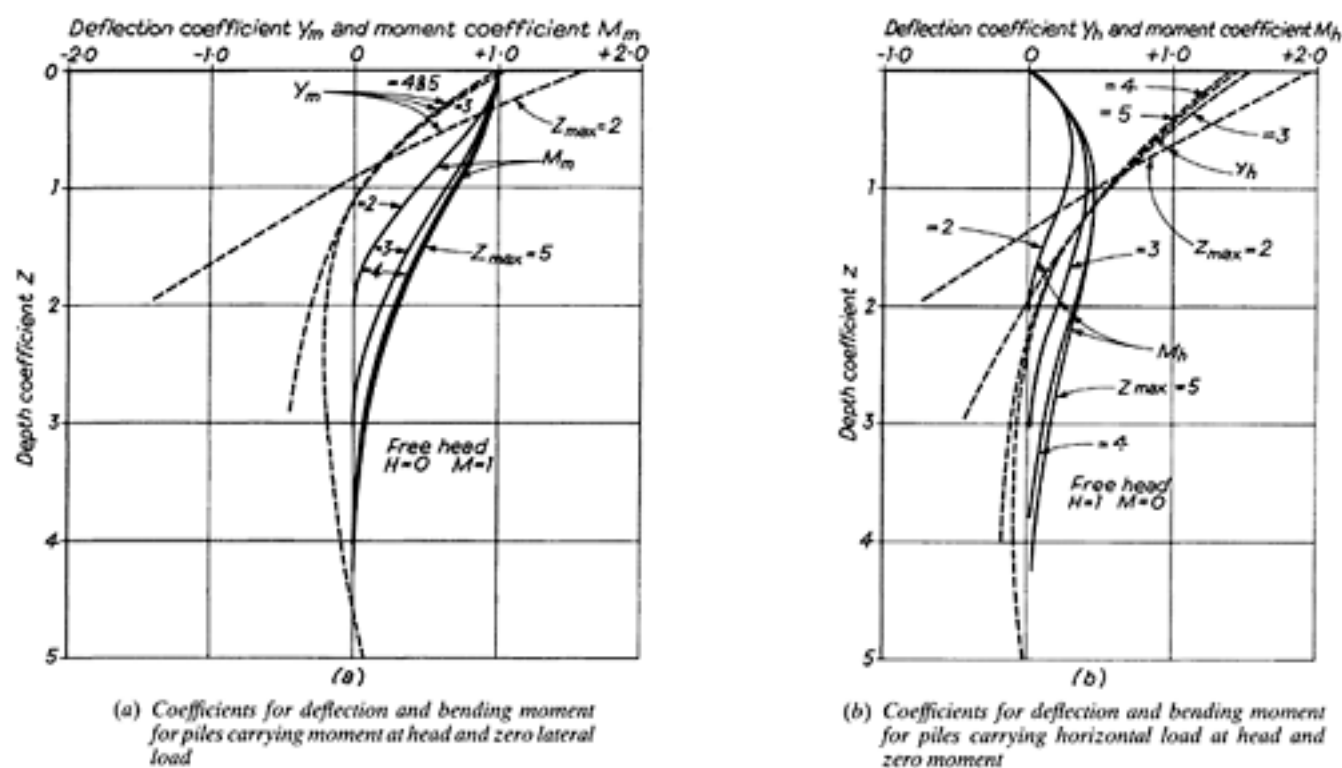


Fig. 6.40 Coefficients for free headed piles carrying lateral load or moment at pile head in soil of constant modulus (after Davisson and Gill<sup>(6.22)</sup>)

the process of calculating deflections by re-arranging equation 6.47 to incorporate a deflection coefficient  $C_y$ . Then

$$y = C_y \frac{HT^3}{EI} \quad \dots(6.64)$$

where

$$C_y = A_y + \frac{M_t B_y}{HT} \quad \dots(6.65)$$

Values of  $C_y$  are plotted in terms of the dimensionless depth factor  $Z(=x/T)$  for various values of  $M_t/HT$  in [Figure 6.41](#).

Included in these curves are the fixed-headed case (*i.e.*  $M_t/HT = -0.93$ ) and the free headed case (*i.e.*  $M_t = 0$ )

The elastic deflections of piles in layered soils, each soil layer having its individual constant modulus, have been analysed by Davisson and Gill<sup>(6.22)</sup> who have produced design charts for this condition.

### 6.3.5 The use of $p$ - $y$ curves

The analytical methods of Reese and Matlock<sup>(6.21)</sup> and Davisson and Gill<sup>(6.22)</sup> that are described in the previous section are applicable only to the deflections of piles which are within the range of the elastic compression of the soil caused by the lateral loading on the piles. However, these analytical methods can be extended beyond the elastic range to analyse movements where the soil yields plastically up to and beyond the stage of shear failure. This can be done by employing the artifice of ' $p$ - $y$ ' curves, which represent the deformation of the soil at any given depth below the soil surface for a range of horizontally applied pressures from zero to the stage of yielding of the soil in ultimate shear, when the deformation increases without any further increase of load. The  $p$ - $y$  curves are independent of the shape and stiffness of the pile and represent the deformation of a discrete vertical area of soil that is unaffected by loading above and below it.

The form of a  $p$ - $y$  curve is shown in [Figure 6.42a](#). The individual curves may be plotted on a common pair of axes to give a family of curves for the selected depths below the soil surface, as shown in [Figure 6.42b](#). Thus for the deformed shape of the pile (and also the induced bending moments and shearing forces) to be predicted correctly using the elastic analytical method described above, the deflections resulting by these analyses must be compatible with those obtained by the  $p$ - $y$  curves for the given

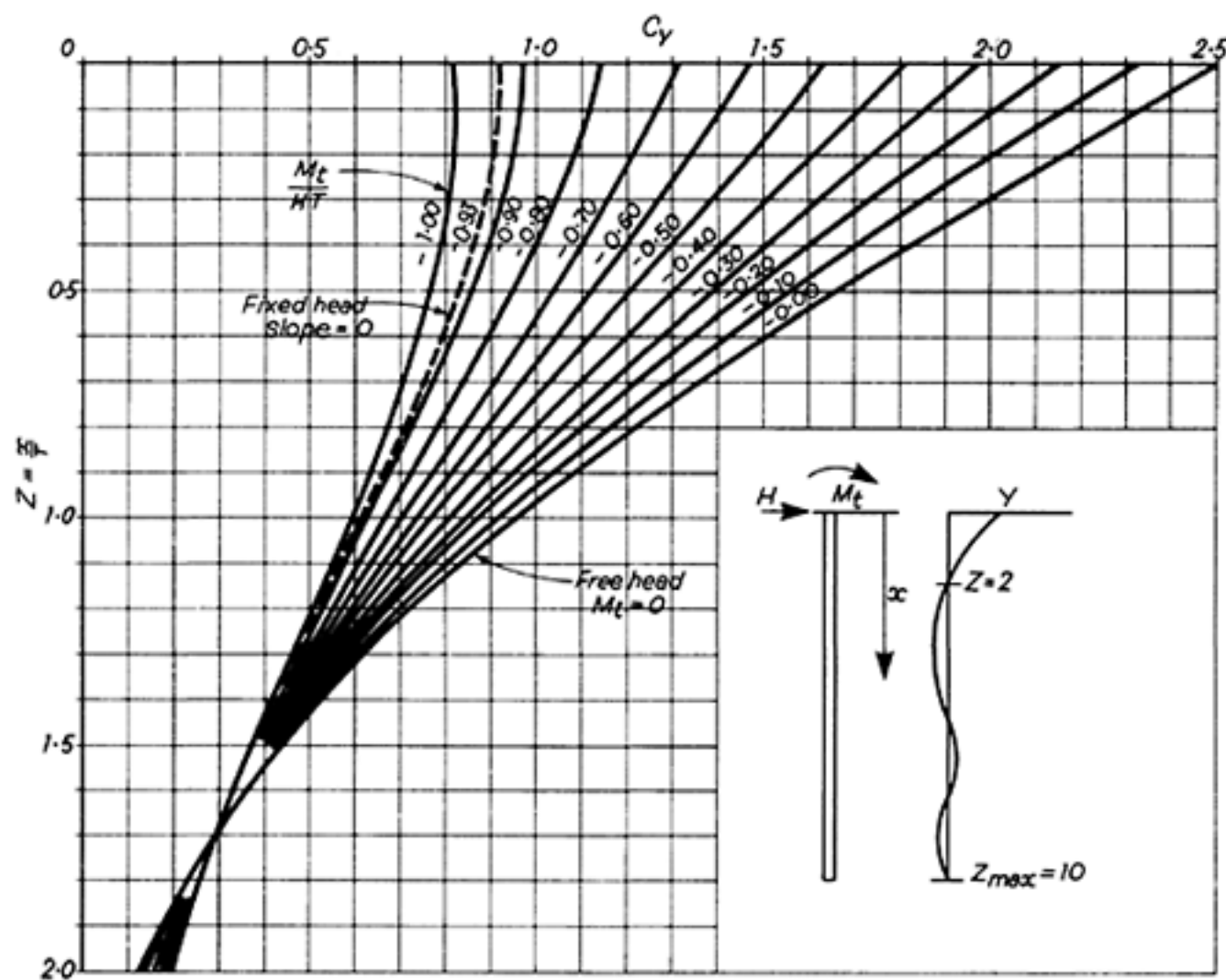


Fig. 6.41 Coefficients for calculating deflection of pile carrying both moment and lateral load (after Matlock and Reese<sup>(6.23)</sup>)

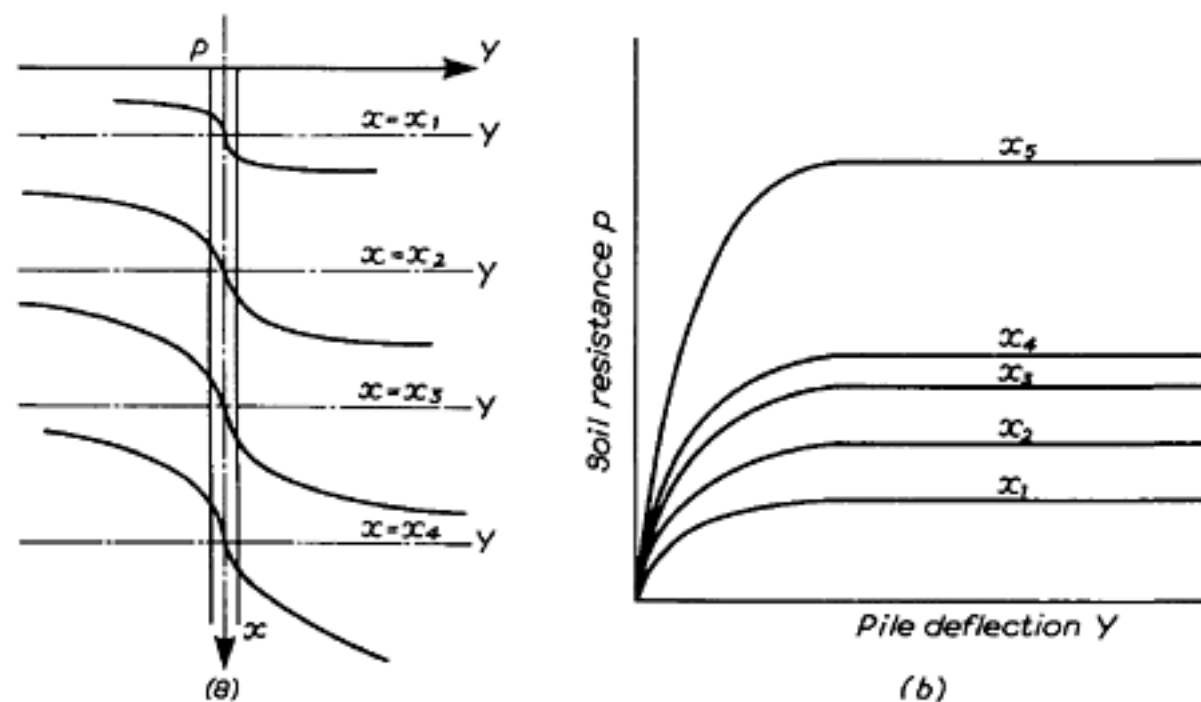


Fig. 6.42  $p$ - $y$  curves for laterally-loaded piles

- (a) Shape of curves at various depths  $x$  below soil surface  
 (b) Curves plotted on common axes

soil conditions. The deflections obtained by the initial elastic analysis are based on an assumed modulus of subgrade reaction  $n_h$  and this must be compared with the modulus obtained from the pressures corresponding to these deflections, as obtained

from the  $p$ - $y$  curve for each particular depth analysed. If the soil moduli, expressed in terms of the stiffness factor  $T$ , do not correspond, the stiffness factor must be modified by making an appropriate adjustment to the soil modulus  $E_s$  and from this

to a new value of  $n_h$  and hence to the new stiffness factor  $T$ . The deflections are then recalculated from the Reese and

Matlock curves, and the corresponding pressures again obtained from the  $p$ - $y$  curves. This procedure results in a new value of the soil modulus which is again compared with the second trial value, and the process repeated until reasonable agreement is obtained.

Methods of drawing sets of  $p$ - $y$  curves have been established for soils which have a linearly-increasing modulus, *i.e.* soft to

firm normally-consolidated clays and granular soils. Empirical factors were obtained

by applying lateral loads to steel tubular piles driven into soft to firm clays and sands. The piles were instrumented to obtain soil reactions and deflections over their full embedded depth.

The method of establishing  $p$ - $y$  curves for soft to firm clays is described by Matlock<sup>(6.24)</sup>. The first step is to calculate the ultimate resistance of the clay to lateral loading. Matlock's method is similar in concept to those described in 6.3.1. but the bearing-capacity factor  $N_c$  is obtained on a somewhat different basis.

Below a critical depth  $x_r$  the coefficient is taken conventionally as 9 as in 6.3.1. Above this depth it is given by equation

$$N_c = 3 + \frac{\gamma x}{c_u} + \frac{Jx}{B} \quad \dots(6.66)$$

where  $\gamma$  is the density of the overburden soil,  $x$  is the depth below ground level,  $c_u$  is the undrained cohesion value of the clay,  $J$  is an empirical factor, and,  $B$  is the width of the pile.

The experimental work of Matlock yielded values of  $J$  of from 0.5 for a soft clay to 0.25 for a stiffer clay. The critical depth is given by the equation

$$x_r = \frac{6B}{\frac{\gamma B}{c_u} + J} \quad \dots(6.67)$$

The ultimate resistance above and below the critical depth is expressed in the  $p$ - $y$  curves as a force  $P_u$  per unit length of pile, where

$$P_u = N_c c_u B \quad \dots(\text{as Figure 6.24a})$$

Up to the point A in [Figure 6.43](#) the shape of the  $p$ - $y$  curve is derived from that of the stress-strain curve obtained by testing a soil specimen in undrained triaxial compression, or from the load-settlement curve in a plate loading test ([Figure 5.12](#)). The shape of the curve is defined by the equation

$$\frac{P}{P_u} = 0.5 \sqrt[3]{\frac{y}{y_c}} \quad \dots(6.68)$$

where  $y_c$  is the deflection corresponding to the strain  $\epsilon_c$  at a stress equal to the maximum stress resulting from the laboratory stress-strain curve. The strain  $\epsilon_c$  can also be obtained from the established relationship between  $c_u$  and the deformation modulus  $E_u$  (see 5.2.2). Matlock<sup>(6.24)</sup> quotes values of  $\epsilon_c$  of 0.005 for 'brittle and sensitive clays' and 0.020 for 'disturbed or remoulded clays or unconsolidated sediments.' These values of  $\epsilon_c$  have been based on the established range of  $E_u/c_u$  of 50 to 200 for most clays, and they can be applied to stiff over-consolidated clays, for example the value of  $E_u/c_u$  for stiff London clay is 400. Matlock<sup>(6.24)</sup> recommends an average value of 0.010 for normally-consolidated clays for use in the equation

$$y_c = 2.5 \epsilon_c B \quad \dots(6.69)$$

The effect of cyclic loading at depths equal to or greater than  $x_r$  can be allowed for by cutting off the  $p$ - $y$  curve by a horizontal line representing the ultimate resistance  $p_b$  of the clay under cyclically applied loads. From the experimental work of Matlock<sup>(6.24)</sup>, the point of intersection of this line with the  $p$ - $y$  curve (shown in [Figure 6.43](#) as point B) is given by

$$p_b/p_u = 0.72$$

...(6.70)

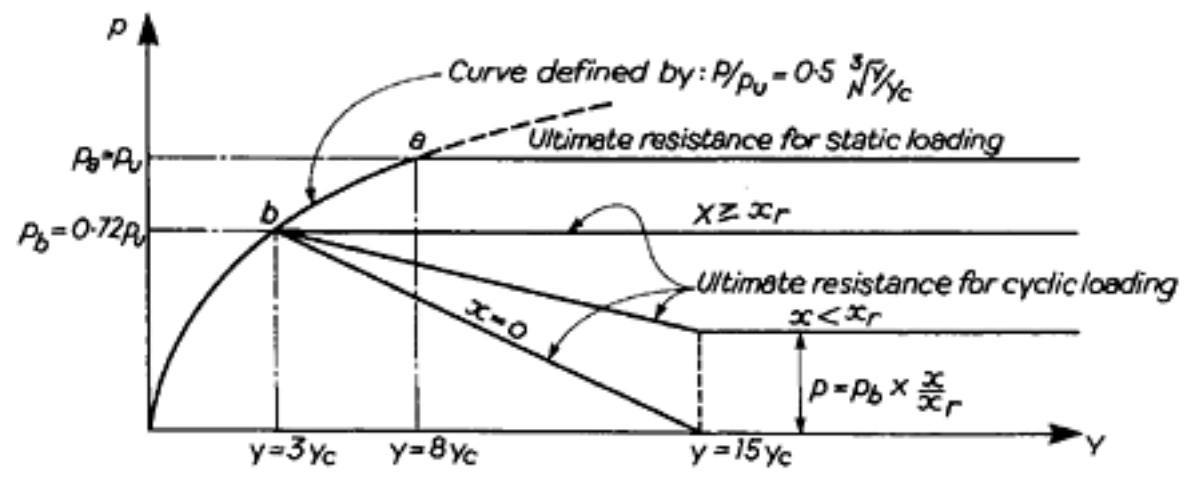


Fig. 6.43 Determining shape of p-y curve in soft to firm clay (after Matlock<sup>(6.24)</sup>)

The  $p$ - $y$  curves for cyclic loading with values of  $y/y_c$  from 3 to 15 and for depths of less than  $x_r$  at  $x=0$  are shown in [Figure 6.43](#).

There are little published data on values of  $p_b$  for various types of clay. The application of a static horizontal load after a period of cyclic loading, say in a deep-sea structure where a berthing ship strikes a dolphin after a period of wave loading, produces a more complex shape in the  $p$ - $y$  curve and a method of establishing the curve for this loading condition has been described by Matlock(6.24).

The procedure for establishing  $p$ - $y$  curves for laterally-loaded piles in sands has been described by Reese, Cox and Koop. (6.17) The steps in the computation are as follows.

1. Obtain the soil parameters  $\phi$  and  $\gamma$  from field or laboratory tests and select the pile width  $B$ .
2. Obtain the factors  $K_0 = 0.4$ ,  $\beta = 45^\circ + \frac{1}{2}\phi$ , and  $\alpha = \frac{1}{2}\phi$ .
3. Calculate the ultimate soil resistance  $p_{cr}$  per unit depth  $K_A = \tan^2(45^\circ - \frac{1}{2}\phi)$  of pile near the ground surface from the equation

$$p_{cr} = \gamma x \left[ \frac{K_0 x \tan \phi \sin \beta}{\tan(\beta - \phi) \cos \alpha} + \frac{\tan \beta}{\tan(\beta - \phi)} (B + x \tan \beta \tan \alpha) + K_0 x \tan \beta (\tan \phi \sin \beta - \tan \alpha) - K_A B \right] \quad \dots(6.71)$$

4. Calculate the ultimate soil resistance  $p_{cd}$  well below the ground surface from the equation

$$p_{cd} = K_A B \gamma x (\tan^8 \beta - 1) + K_0 B \gamma x \tan \phi \tan^4 \beta \quad \dots(6.72)$$

5. Plot curves for  $p_{cr}$  and  $p_{cd}$  against  $x$ , from which the critical depth  $x_r$  is obtained from the point of intersection of the two curves, as shown in [Figure 6.44](#).
6. Select the particular depth at which the  $p$ - $y$  curve is to be drawn.
7. Establish point  $y$ -axis, where  $y$  is equal to  $3B/80$  ([Figure 6.45](#)).
8. Calculate  $p_u$  corresponding to point  $y_u$  from the equation

$$p_u = A' p_c \quad \dots(6.73)$$

Values of the coefficient  $A'$  are obtained from [Figure 6.46a](#) for either static or cyclic applications of load. Values of  $p_c$  are obtained either from equation 6.71 for points above the critical  $x_r$ , or from equation 6.72 for points below the critical depth. Having obtained  $p_u$  and  $y_u$ , plot the point  $u$  ([Figure 6.45](#)).

9. Establish the point  $m$  on the  $p$ - $y$  curve from  $y_m$  equals  $B/60$  and

$$p_m = B' p_c \quad \dots(6.74)$$

Values of coefficient  $B'$  are obtained from [Figure 6.46b](#), and  $p_c$  from either equation 6.71 or 6.72 for points above or below the critical depth  $x_r$  respectively.

10. Establish the slope of the initial portion of the  $p$ - $y$  curve from the origin to point  $k$  by selecting the appropriate value of  $n_h$  from [Figure 6.20](#) (the values of Reese *et al.* (6.17) are applicable to long steel tubular piles in granular soils).

11. Define the parabola to be fitted between the points  $k$  and  $m$  from the equation

$$p = Cy^{1/n}$$

...(6.75)

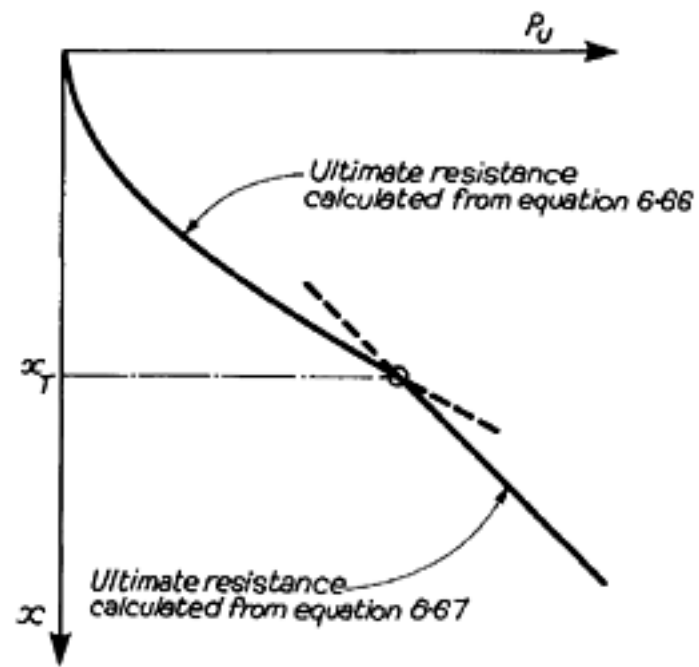


Fig. 6.44 Obtaining critical depth for lateral load on pile in cohesionless soil

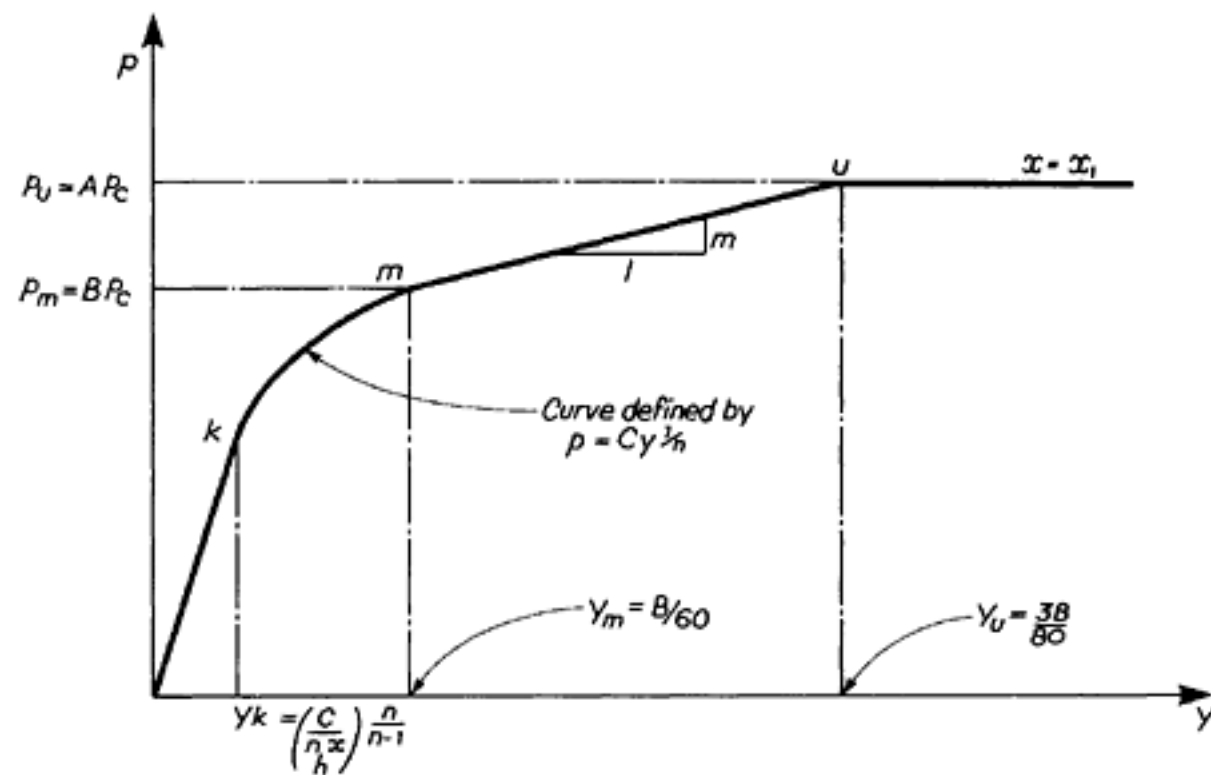


Fig. 6.45 Determining shape of  $p$ - $y$  curve in sand (after Reese, Cox and Coop<sup>(6.17)</sup>)

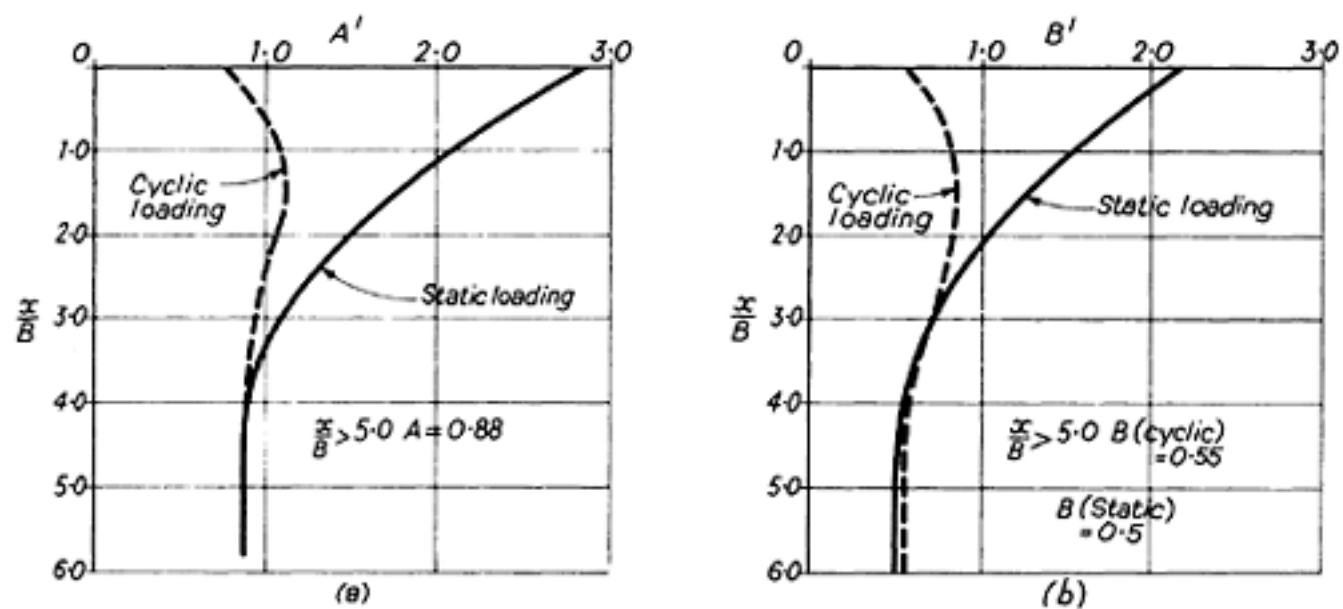


Fig. 6.46 Charts for coefficients  $A'$  and  $B'$

and from the slope of the line between  $m$  and  $u$ , where

$$m = \frac{p_u - p_m}{y_u - y_m} \quad \dots(6.76)$$

In equation 6.75,  $n$  is given by

$$n = \frac{p_m}{m y_m} \quad \dots(6.77)$$

and coefficient  $C$  by

$$C = \frac{p_m}{y_m^{1/n}} \quad \dots(6.78)$$

Point  $k$  is given by

$$y_k = \left( \frac{C}{n_h x} \right)^{n/(n-1)} \quad \dots(6.79)$$

where  $x$  is the depth below the soil surface selected for the particular  $p$ - $y$  curve.

The procedure described below for using the  $p$ - $y$  curves in conjunction with the deflections obtained from the elastic analyses

of Reese and Matlock is common both to piles in soft to firm clays and to granular soils. From the deflections obtained by equations 6.52 or 6.57 for the given horizontal load or bending moment on the pile the corresponding soil pressures are read off the  $p$ - $y$  curves and a soil modulus  $E_s$  is obtained from the linear relationship

$$E_s = -p/y \quad \dots(6.80)$$

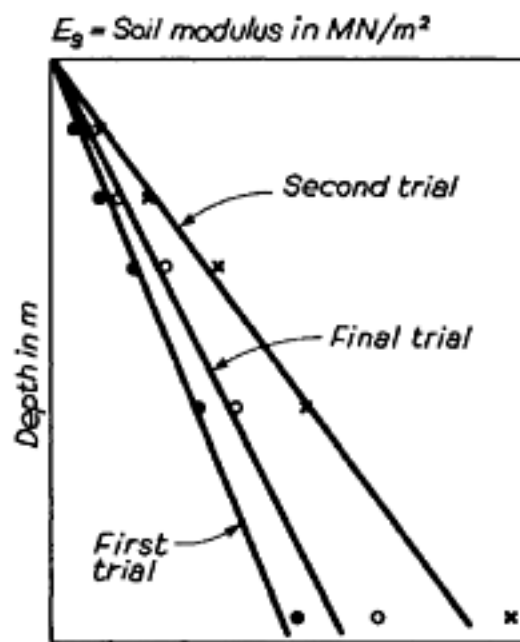


Fig. 6.47 Trial plotting of soil modulus against depth

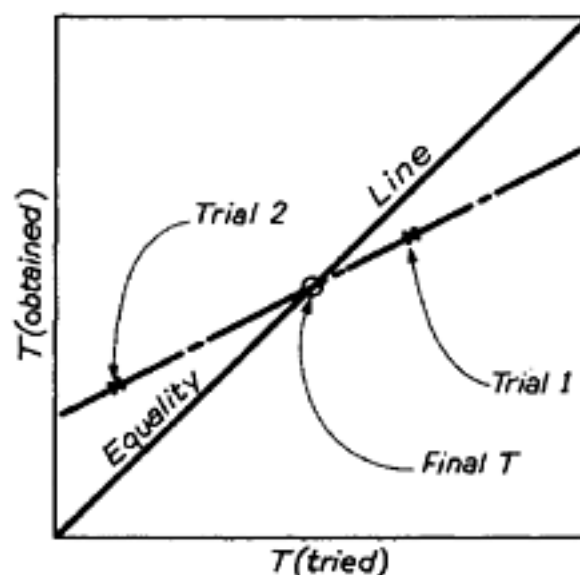


Fig. 6.48 Trial of stiffness factor

The values of  $E_s$  obtained at each point below the ground surface are plotted against the corresponding depth and a straight line passing through the origin is drawn through these points (Figure 6.47). Weight is given to the points near to the soil surface at depths of less than  $0.5T$ , where the deflections are of the most significance. The value of the subgrade modulus  $n_h$  is then obtained from the slope of the trial line through the origin.

The value of the stiffness factor  $T$  can then be calculated from equation 6.9 using the new value of  $n_h$ . If this factor  $T$

obtained is equal to the value first assumed  $T$  tried then the pile deflections are compatible with the  $p$ - $y$  curves and no further trials are necessary. If the two factors differ, however, a new value of  $T$  is assumed. The pile deflections are recalculated and the corresponding value of soil modulus obtained as described for the first trial above. The new value of  $T$  obtained is plotted against the second  $T$  tried. The intersection of a line drawn between the plotted points of the first and second trials and the equality line *i.e.* where  $T$  obtained =  $T$  tried, as shown in Figure 6.48, gives the finally selected value of  $T$  from which the final deflections, bending moments and shearing forces can be calculated.

The above methods are illustrated by examples 8.2 and 8.4.

It is evident from the foregoing that the repetition of the lengthy calculations makes the whole procedure rather laborious, and time and effort can be saved by computerizing the successive steps in the calculations. For example by using the University of Austin in Texas L-PILE program, the computerized process also enables additional factors to be introduced, such as the variation of the value of  $EI$  of the pile below the ground surface. This enables the effect of changing the thickness of the pile wall to be evaluated and the cases of inserted piles with a grouted annulus, and piles within tubular jackets, to be included in the programs available.

The use of the  $p$ - $y$  curves as described above is strictly applicable only to soils with a linearly increasing modulus (*i.e.* granular soils or normally consolidated clays). Correlations with instrumented piles to prove the validity of the procedure have so far been confined to these soils. However, the principle is helpful to the rationalization of design and it is desirable to extend it to stiff over-consolidated clays having a constant soil modulus, where deflections greater than the elastic limit of the soil are anticipated.

In the absence of experimental data, equation 6.66 can be used to obtain values of  $N_c$  above the critical depth. The latter can be calculated from equation 6.67 using a value of 0.25 for coefficient J. Values of  $n_h$  are obtained by plotting the soil modulus  $E_s$  (equation 6.80) against the depth, but the trial line is a vertical one passing through the plotted points, again with weight being given to depths of  $0.5R$  or less. Cyclic loading can be a critical factor in stiff clays. The relationship in equation 6.70 should preferably be established for the particular site by laboratory and field tests, but the factor of 0.72 may be used if results of such studies are not available.

Instead of relating the deflection  $y_c$  to the strain  $\varepsilon_c$  at a stress corresponding to the maximum stress obtained in the laboratory stress-strain curve for use in equation 6.68, Reese and Welch<sup>(6.25)</sup> adopt the following relationship for stiff clays.

$$\frac{p}{p_u} = 0.5 \sqrt[4]{\frac{y}{y_{50}}} \quad \dots(6.80)$$

where  $p$  and  $p_u$  are as previously defined, and  $y_{50}$  is the deflection corresponding to the strain  $\varepsilon_{50}$  at one-half of the maximum principal stress difference in the laboratory stress-strain curve.

If no value of  $\varepsilon_{50}$  is available from laboratory tests a figure of between 0.005 and 0.010 can be used in equation 6.64 but substituting  $y_{50}$  for  $y_c$  and  $\varepsilon_{50}$  for  $\varepsilon_c$ . The larger of these two values is the more conservative. Reese and Welch<sup>(6.25)</sup> have described a method for establishing  $p$ - $y$  curves for cyclic loading on piles in stiff clay.

### 6.3.6 Effect of method of pile installation on behaviour under lateral loads and moments applied to pile head

The method of installing a pile, whether driven, driven-and-cast-in-situ or bored-and-cast-in-situ, has not been considered in sections 6.3.1 to 6.3.4. No research has been undertaken to study the effect of the installation method on the behaviour under lateral load, although it can be allowed for by appropriate adjustments to the soil parameters. For example, when considering the resistance to lateral loads, of piles driven into a soft sensitive clay the remoulded shearing strength should be used in equations 6.15 and 6.18 and in Figures 6.25 and 6.30, to obtain the ultimate resistance over a period of a few days or weeks after driving. If the piles are not to be subjected to loading for a few months after driving, the full ‘undisturbed’ shearing strength can be used. There is unlikely to be much difference between the ultimate lateral resistance of piles driven into stiff over-consolidated clays and bored piles in the same type of soil. The softening effects for bored piles mentioned in Section 4.2.3 occur over a very short radial distance from the pile and the principal resistance to lateral loads is provided by the undisturbed soil beyond the softened zone.

In the case of piles installed in cohesionless soils the effect of loosening due to the installation of bored piles can be allowed for by assuming a low value of  $\phi$  when determining  $K_p$  for use in equations 6.19 to 6.22 and 6.28 to 6.32. The same low values should be used in Figures 6.27 and 6.32. When considering the deflection of bored piles in cohesionless soils the value of the soil modulus  $n_h$  in Figure 6.20 should be appropriate to the degree of loosening which is judged to be caused by the method of installing the piles.

### 6.3.7 The use of the pressuremeter test to establish $p$ - $y$ curves

The pressuremeter test (see 11.1.4) made in a borehole (or in a hole drilled by the pressuremeter device) is particularly suitable for use in establishing  $p$ - $y$  curves for laterally loaded piles. The test produces a curve of the type shown in Figure 6.49a. The initial portion represents a linear relationship between pressure and volume change, that is the radial expansion of the walls of the borehole. At the creep pressure,  $p_f$  the pressure/volume relationship becomes non-linear indicating plastic yielding of the soil; at the limit pressure,  $p_l$  the volume increases rapidly without increase of pressure as represented by the horizontal portion of the  $p$ - $y$  curve.

Menard used a Poisson’s ratio of 0.33 to derive an expression for determining the pressuremeter modulus of the soil from the initial portion of the curve in Figure 6.49a. This equation as given by Baguelin *et al.*<sup>(6.26)</sup> is,

$$E_m = 2.66 V_m \frac{\Delta_p}{\Delta_v} \qquad \dots(6.81)$$

where  $\frac{\Delta_p}{\Delta_v}$  is the slope of the curve between  $V_0$  and  $V_f$

$V_m$  is midpoint volume

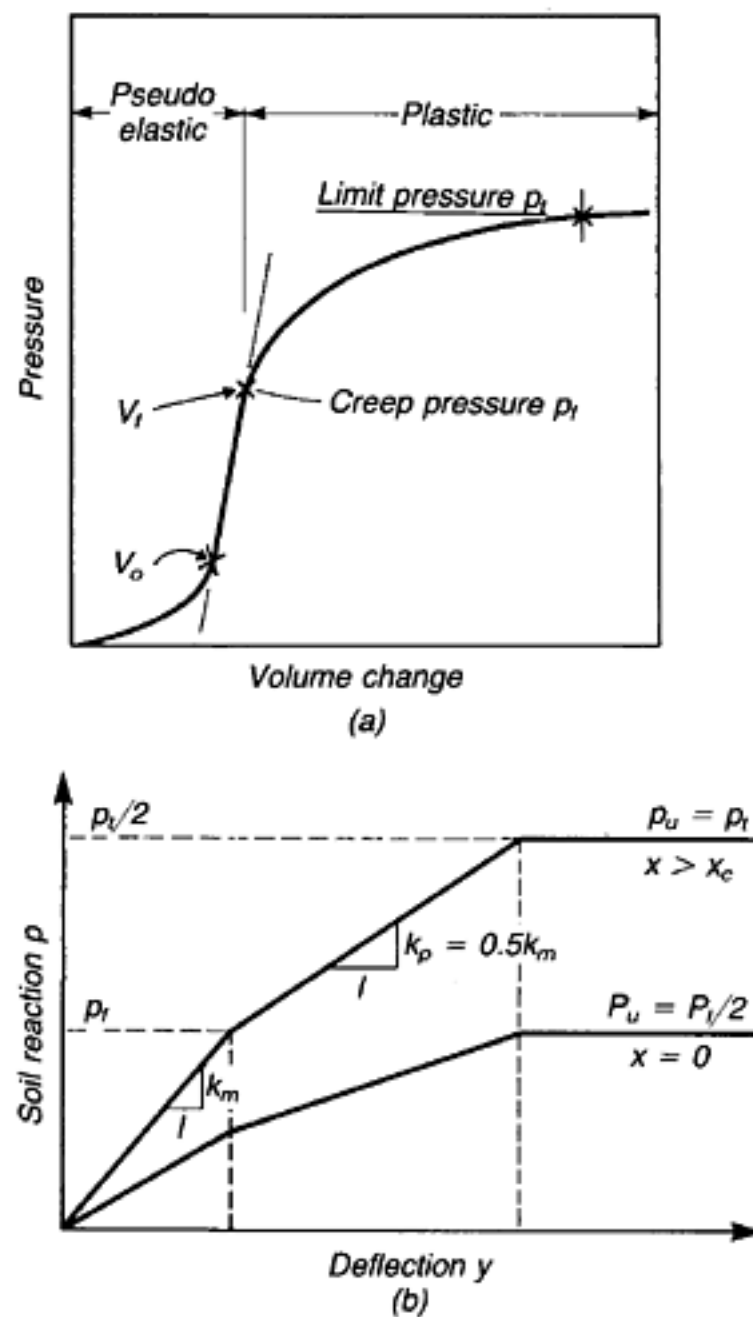


Fig. 6.49 Obtaining soil reaction values from pressuremeter test, (a) pressure/ volume change curve, (b) design reaction curve (after Baguelin *et al.* (6.26))

Baguelin *et al.* (6.26) give two sets of curves relating the response of the soil to lateral loading for the two stages in the pressuremeter tests as shown in Figure 6.49b. The upper curve is for depths below the ground surface equal to or greater than the critical depth,  $x_c$ , at which surface heave affects the validity of the calculation method. In cohesive soils  $x_c$  is taken as twice the pile width and in granular soils it is four times the width. Where there is a pile cap there is no surface heave,  $x_c$  is zero and the lower curve in Figure 6.49b applies. The value of the coefficient of subgrade reaction,  $k_m$  in Figure 6.49b, is

given by,

for pile widths greater than 600mm

$$\frac{1}{k_m} = \frac{2}{9E_m} B_0 \left( \frac{B}{B_0} \cdot 2.65 \right)^\alpha + \frac{\alpha B}{6E_m} \quad \dots(6.82a)$$

for pile widths less than 600mm

$$\frac{1}{k_m} = \frac{B}{E_m} \left( \frac{4(2.65)^\alpha + 3\alpha}{18} \right) \quad \dots(6.82b)$$

where  $E_m$  is the mean value of the pressuremeter modulus over the characteristic length of the pile

$B_0$  is the pile width

$\alpha$  is a rheological factor varying from 1.0 to 0.5 for clays, 0.67 and 0.33 for silts, and 0.5 to 0.33 for sands

Between the ground surface and the critical depth,  $X_c$ , the value of  $k_m$  should be reduced by the coefficient  $\lambda_z$ , given by,

$$\lambda_z = \frac{1 + \frac{X}{X_c}}{2} \quad \dots(6.83)$$

A simplified procedure in a homogeneous soil is to assume that there will be no lateral soil reaction between the ground surface and a depth equivalent to 0.5 to 0.755 then to use the full reaction given by the upper curve in [Figure 6.49b](#).

Baguelin *et al.* (6.26) give the following equations for calculating deflections, bending moments and shears at any depth  $z$  below the ground surface for conditions of a constant value of the pressuremeter modulus with depth

$$\text{Deflection} \quad y(z) = \frac{2H}{Rk_m B} \cdot F_1 + \frac{2M_t}{R^2 k_m B} \cdot F_4 \quad \dots(6.83a)$$

$$\text{Moment} \quad M(z) = \frac{H \cdot R \cdot F_3}{R} + M_t F_2 \quad \dots(6.83b)$$

$$\text{Shear} \quad T(z) = HF_4 - \frac{2M_t}{R} F_3 \quad \dots c)$$

where  $R$  is the stiffness coefficient given by equation 6.8 (Baguelin refers to this as the transfer length,  $l_0$ )

$H$  is the horizontal load applied to the pile head

$M_t$  is the bending moment at the pile head

$z$  is the dimensionless coefficient equal to  $\frac{X}{R}$

Values of the coefficients  $F_1$  to  $F_4$  are given in [Figure 6.50](#).

At the ground surface the deflection becomes

$$y_0 = \frac{2H}{Rk_m B} + \frac{2M_t}{R^2 k_m B} \quad \dots(6.84)$$

$$\text{and slope} = y_0' = \frac{-2H}{R^2 k_m B} - \frac{4M_t}{R^1 k_m B} \text{ radians} \quad \dots(6.85)$$

If the head of the pile is fixed so that it does not rotate ( $y_0 = 0$ ) the equations 6.83 to 6.85 become,

$$y(z) = \frac{H}{Rk_m B} F_2 \quad \dots(6.86a)$$

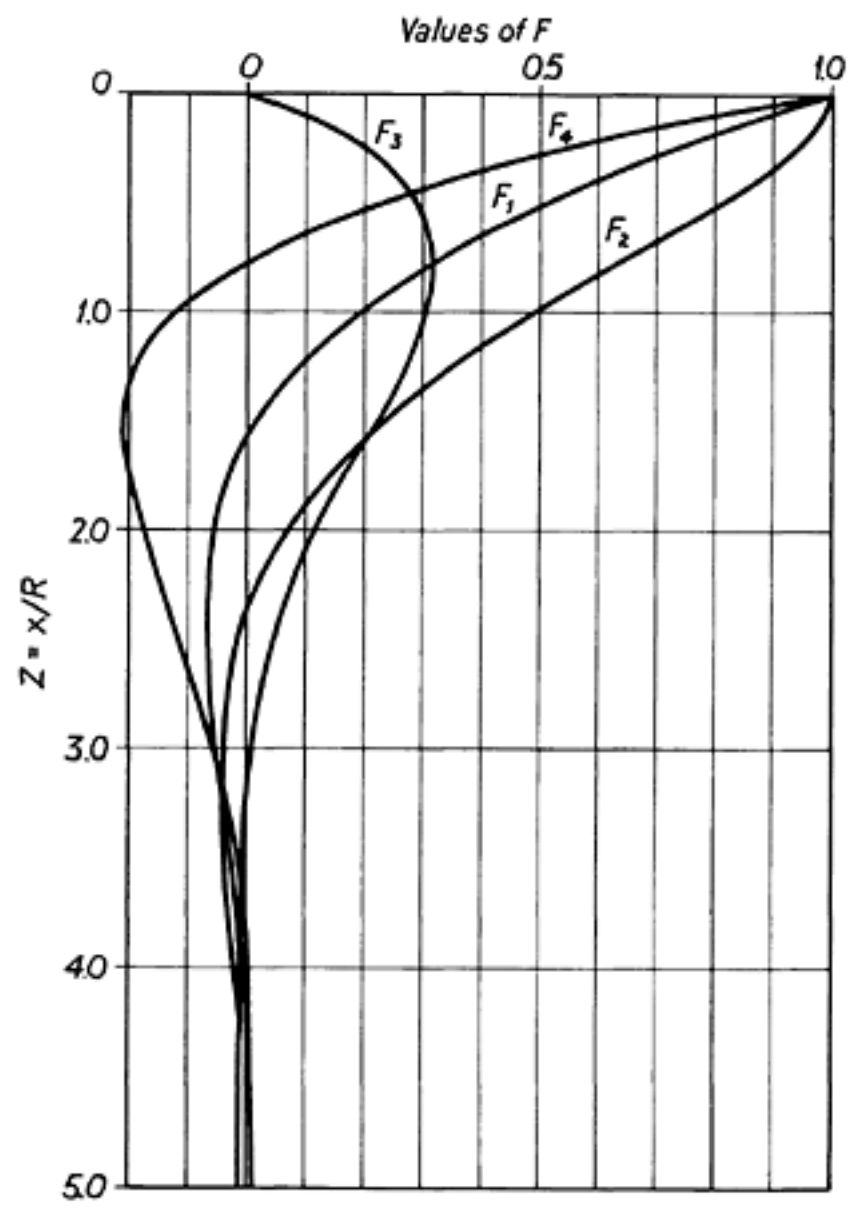


Fig. 6.50 Values of the coefficients  $F_1$  to  $F_4$  (after Baguelin et al. [\(6.26\)](#))

$$M(z) = \frac{-HR}{2} F_4 \quad \dots(6.86b)$$

$$T(z) = H \cdot F_3 \quad \dots(6.86c)$$

$$y_0 = \frac{H}{Rk_m B} \quad \dots(6.86d)$$

$$M_t = \frac{-HR}{2} \quad \dots(6.86e)$$

To draw the pile load/deflection curve, the deflections corresponding to soil reactions equal to the creep pressure,  $p_f$  and the limit pressure,  $p_l$ , are calculated from the relationship  $p=kmy$ . The lateral pile loads then follow from equations 6.83a, 6.84, 6.86a or 6.86d. For soil reactions between the limit pressure and creep pressure the value of  $k_m$  is halved as shown in [Figure 6.50](#). The procedure is illustrated in worked example 6.10 where the pressuremeter tests show a linearly increasing soil modulus. The values of  $n_h$  can be calculated from equation 6.10 taking  $K$  as  $k_{mB}$ . Deflections are calculated from the Matlock and Reese curves.

### 6.3.8 Calculation of lateral deflections and bending moments by elastic continuum methods

The method of preparing  $p$ - $y$  curves described in [Section 6.3.5](#) was based on the assumption that the laterally loaded pile could be modelled as a beam supported by discrete springs. The springs would be considered as possessing linear or non-linear behaviour. In the latter case the method could be used to model pile behaviour in strain conditions beyond the elastic range.

In many cases where lateral forces are relatively low and piles are stiff the pile head movements are within the elastic range and it may be convenient to use the elastic continuum model to calculate deflections and bending moments. In his development of this model Poulos<sup>(6.27)</sup> considered the pile as an infinitely long strip having a width and flexural stiffness equal to those of the full-scale pile. Results of analyses performed by computer were presented in terms of a dimensionless factor  $K_R$  defined as

$$K_R = \frac{E_p I_p}{E_s L^4} \quad \dots(6.87)$$

where  $E_p I_p$  is the flexural stiffness of the pile

$E_s$  is the Young's modulus of the soil

$L$  is the embedded length of the pile.

As originally developed by Poulos<sup>(6.27)</sup> the method could be used for manual calculations only by assuming that  $E$  was constant with depth. It was subsequently developed by Poulos<sup>(6.28)</sup>, Randolph<sup>(6.29)</sup> and others to deal with the case of  $E_s$  increasing linearly with depth. The equations established by Poulos for use with the coefficient  $K_R$  are:

*Free head pile*

$$\text{Deflection of pile head at ground surface} = y_0 = \frac{I_H H_0}{E_s L} + \frac{I_{HM} M_0}{E_s L^2} \quad \dots(6.88)$$

$$\text{Rotation of pile head at ground surface} = \theta_0 = \frac{I_{\theta H} H_0}{E_s L^2} + \frac{I_M M_0}{E_s L^3} \quad \dots(6.89)$$

where  $E_s$  increases linearly with depth, the factors  $I_H$ ,  $I_{HM}$  and  $I_M$  are replaced by  $I'_H$ ,  $I'_{MM}$  and  $I'_M$

*Fixed head pile*

$$y_0 = I_{FH} \frac{H_0}{E_s L} \quad \dots(6.90)$$

$$M_0 = \frac{-I_{HM} \cdot H_0 L}{I_M} \quad \dots(6.91)$$

Note:  $I_{HM} = I_{\theta M}$  and  $I_{FH} = I_H - (I_{HM}^2/I_M)$

Where  $E_s$  increases linearly with depth, the factor  $I_{FH}$  is replaced by  $I'_{FH}$ .

The factors  $I_H$ ,  $I_{HM}$  and  $I_M$  are shown in [Figures 6.51](#)(a to c),  $I'_H$ ,  $I'_{HM}$  and  $I'_M$  in [Figures 6.52](#)(a to c), and  $I_{FH}$  and  $I'_{FH}$  in [Figures 6.53](#)(a and b).

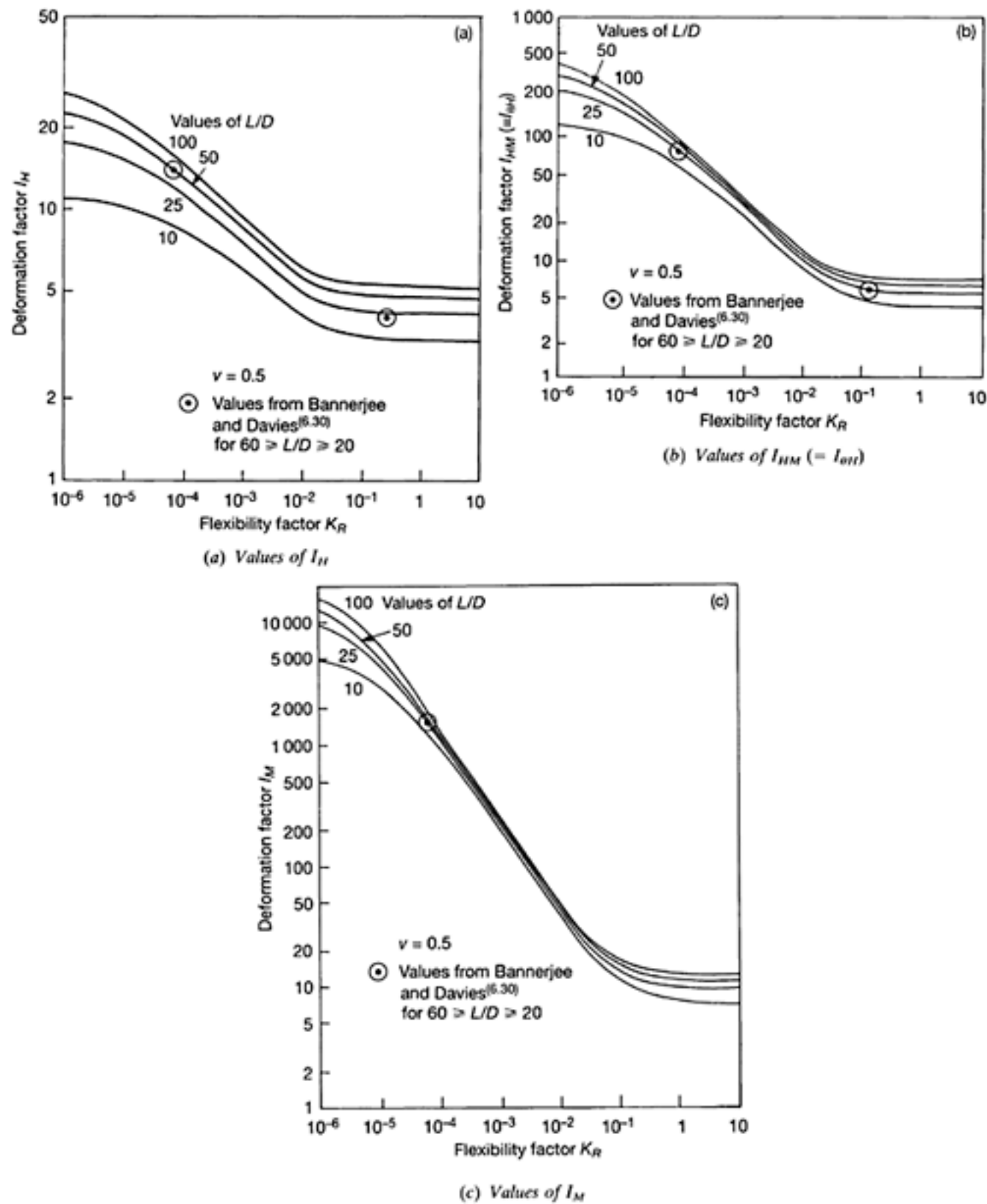


Fig. 6.51 Deformation factors for free-head pile in homogeneous soil (after Poulos<sup>(6.27)</sup>)

Bannerjee and Davies<sup>(6.30)</sup> extended the work of Poulos using an improved mathematical model. Their factors were plotted on the Poulos curves by Elson<sup>(6.15)</sup> for purposes of comparison as shown in Figures 6.51 to 6.53.

Randolph<sup>(6.29)</sup> used finite element analyses to establish relationships between pile deflections and bending moments with depth for lateral force and moment loading as shown in Figure 6.54. The following notation applies to the parameters in Figure 6.54.

- $y_0$  is the lateral displacement at ground surface
- $z$  is the depth below ground level
- $H_0$  is the lateral load applied at ground surface
- $M$  is the bending moment in the pile
- $M_0$  is the bending moment at ground surface

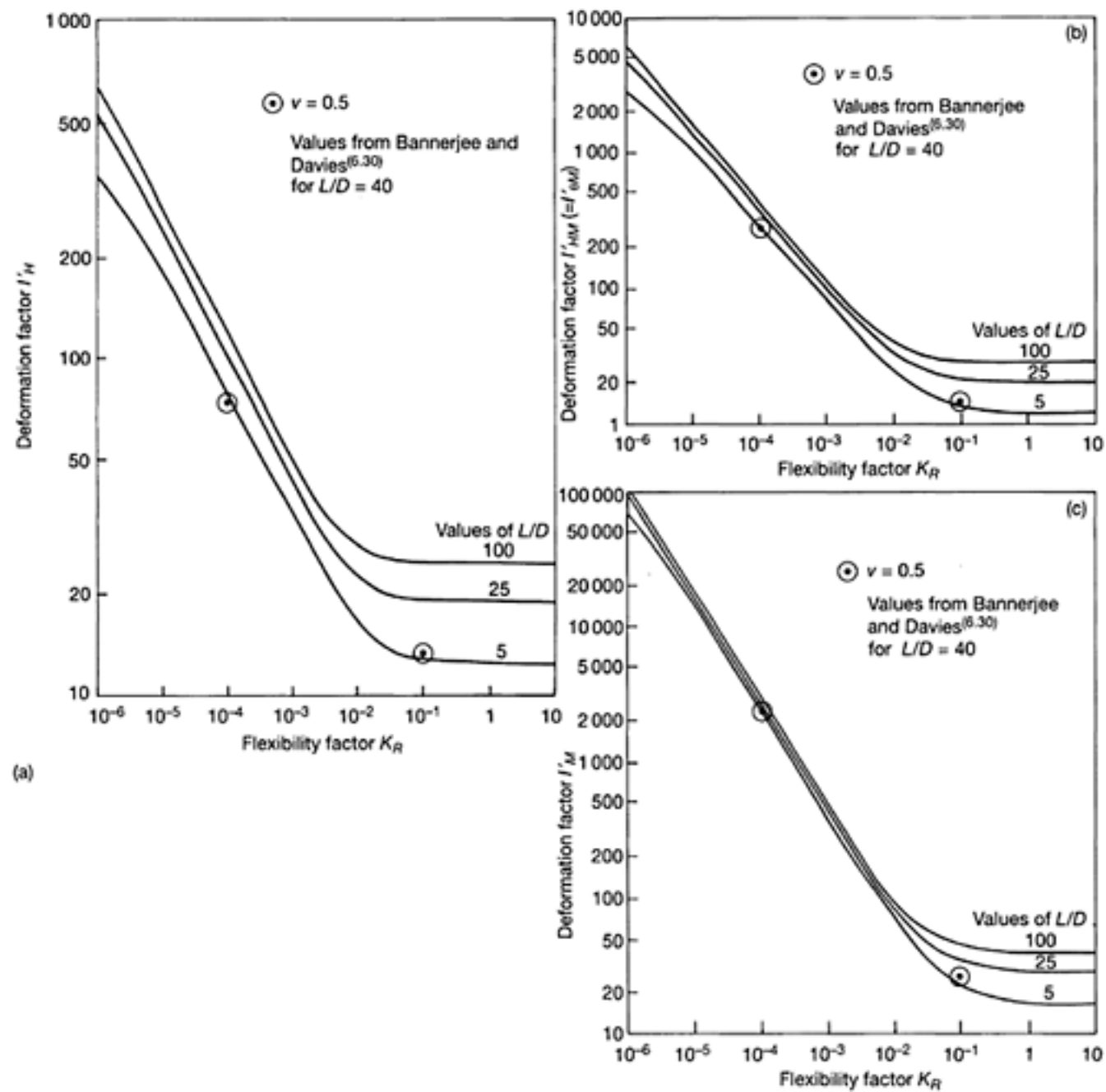


Fig. 6.52 Deformation factors for free-head pile in non-homogeneous soil where soil modulus increases linearly with depth (after Poulos<sup>(6.27)</sup>)

(a) Values of  $I'_H$

(b) Values of  $I'_{HM} (= I'_{\theta H})$

(c) Values of  $I'_M$

$r_0$  is the radius of the pile

$E'_p$  is the effective Young's modulus of a solid circular pile of radius  $r_0$  (i.e.  $4E_p I_p / \pi r_0^4$ )

$G_c$  is the characteristic modulus of the soil, i.e. the average value of  $G^*$  over depths less than  $l_c$

$G^* = G(1 + \frac{3}{4}v)$  where  $G$  is the shear modulus of the soil

$v$  is the Poisson's ratio

$l_c$  is the critical length of the pile

$l_c = 2r_0(E'_p/G^*)^{2/7}$  for homogeneous soil

$= 2r_0(E'_p/m_*r_0)^{2/9}$  for soil increasing linearly in stiffness with depth

$m^* = m \left(1 + \frac{3v}{4}\right)$

$m = \frac{G}{z}$  where  $G$  varies with depth as  $G = mz$

$\rho_c$  is a homogeneity factor where  $\rho_c = \frac{G^* \text{ at } l_c/4}{G^* \text{ at } l_c/2}$

The use of the Randolph curves is illustrated in worked example 8.4.  
 The Randolph method is useful where the shear modulus is obtained directly in the field using the

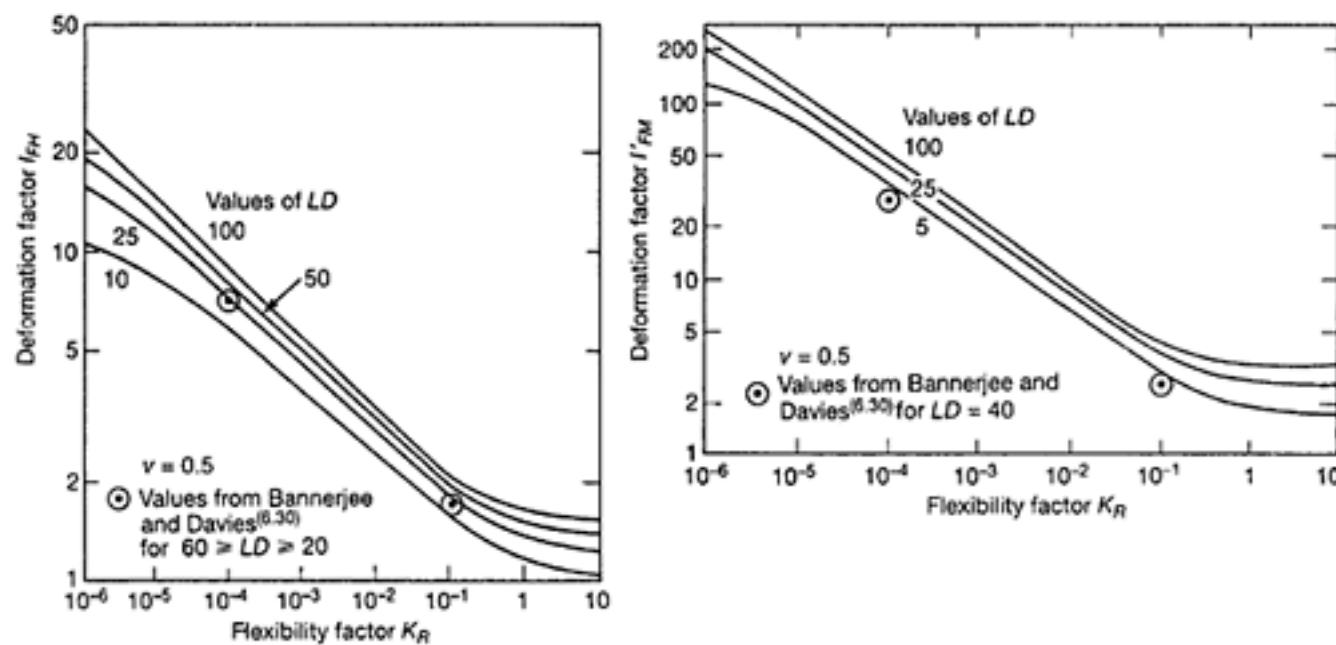


Fig. 6.53 Deformation factors for fixed-head piles (after Poulos<sup>(6.27)</sup>)

(a) Values of  $I_{FH}$  for fixed-head pile in homogeneous soil

(b) Values of  $I'_{FH}$  for fixed-head pile in non-homogeneous soil where soil modulus increases linearly with depth

pressuremeter. If Young's modulus values only are available the shear modulus for an isotropic soil can be obtained from the equations

$$E_u = 2G(1 + \nu_u) \quad \text{and} \quad E' = 2G(1 + \nu') \quad \dots(6.92)$$

where  $\nu_u$  and  $\nu'$  are the undrained and drained Poisson's ratios respectively.

## 6.4 Lateral loads on raking piles

The resistance of vertical piles to lateral loads is very small even in good soil. Tests at Pickering Generating Station in Ontario reported by Hanna<sup>(6.31)</sup> showed that H-piles driven through very stiff to hard clays having ultimate compression loads of more than 2000kN could be allowed to carry only 50kN in horizontal loading.

The most effective way of arranging piles to resist lateral loads is to have pairs of piles raking in opposite directions as shown in Figure 6.5. The simple graphical method of determining the compressive and tensile forces in the piles by a triangle of forces assumes that the piles are hinged at their point of intersection and that the lateral loads are carried only in an axial direction by the piles. The tension pile will develop its maximum pull-out resistance with negligible movement, and the yielding of properly designed compression pile of small to medium diameter is unlikely to exceed 10mm at the working load. Thus the horizontal deflections of the pile cap will be quite small.

For economy, the raking piles should be installed at the largest possible angle from the vertical. This depends on the type of pile used (see 3.4.10). Where raking piles are embedded in fill which is settling under its own weight (Figure 6.55a) or in a compressible clay subjected to a surcharge load or to superimposed fill (Figure 6.55b) the vertical loading on the upper surface of the rakers may induce high bending moments in the pile shaft. Because of this, raking piles may not be an appropriate form of construction in deep fill or compressible layers.

## 6.5 Lateral loads on groups of piles

Loads on individual piles forming a group of vertical piles that is subject to horizontal loading or to combined vertical and horizontal loading can be determined quite simply (for cases where the resultant cuts the underside of the pile cap) by taking moments about the neutral axis of the pile group. Thus in Figure 6.56 the vertical component  $V$  of the load on any pile produced by an inclined thrust  $R$ , where  $R$  is the resultant of a horizontal load  $H$  and a vertical load  $W$  is given by

$$V = \frac{W}{n} + \frac{We\bar{x}}{\sum \bar{x}^2} \quad \dots(6.93)$$

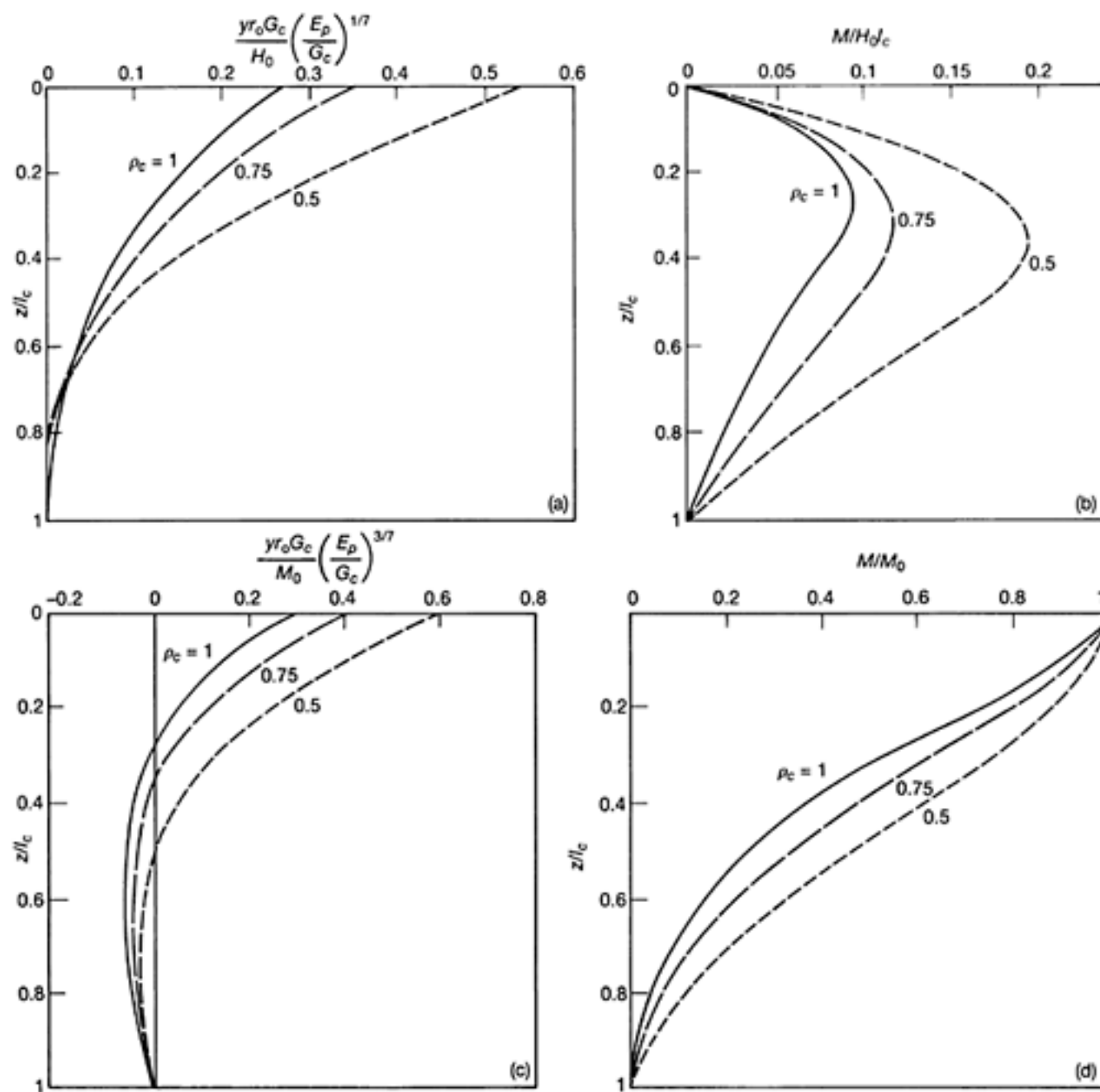


Fig. 6.54 Generalized curves giving deflected pile shape and bending moment profile for lateral force and bending moment applied to pile head (after Randolph<sup>(6.29)</sup>)

- (a) Deflected pile shape for lateral force loading
- (b) Bending moment profile for lateral force loading
- (c) Deflected pile shape for moment loading
- (d) Bending moment profile for moment loading

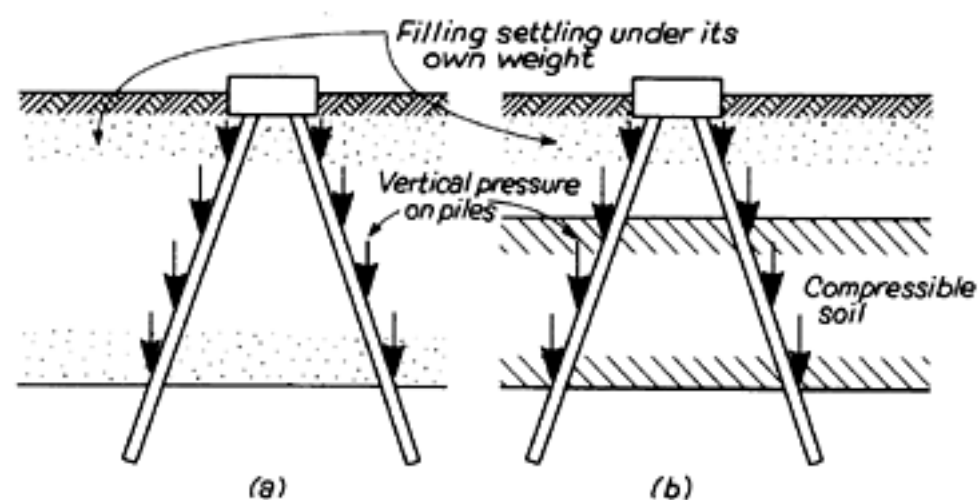


Fig. 6.55 Bending of slender raking piles due to loading from soil subsidence

- (a) Fill settling under own weight
- (b) Fill overlying compressible soil

where  $W$  is the total vertical load on the pile group,  $n$  is the number of piles in the group,  $e$  is the distance between the point of intersection of  $R$  with the underside of the pile cap and the neutral axis of the pile group, and  $\bar{x}$  is the distance between the pile and the neutral axis of the pile group ( $\bar{x}$  is positive when measured in same direction as  $e$  and negative when the opposite direction).

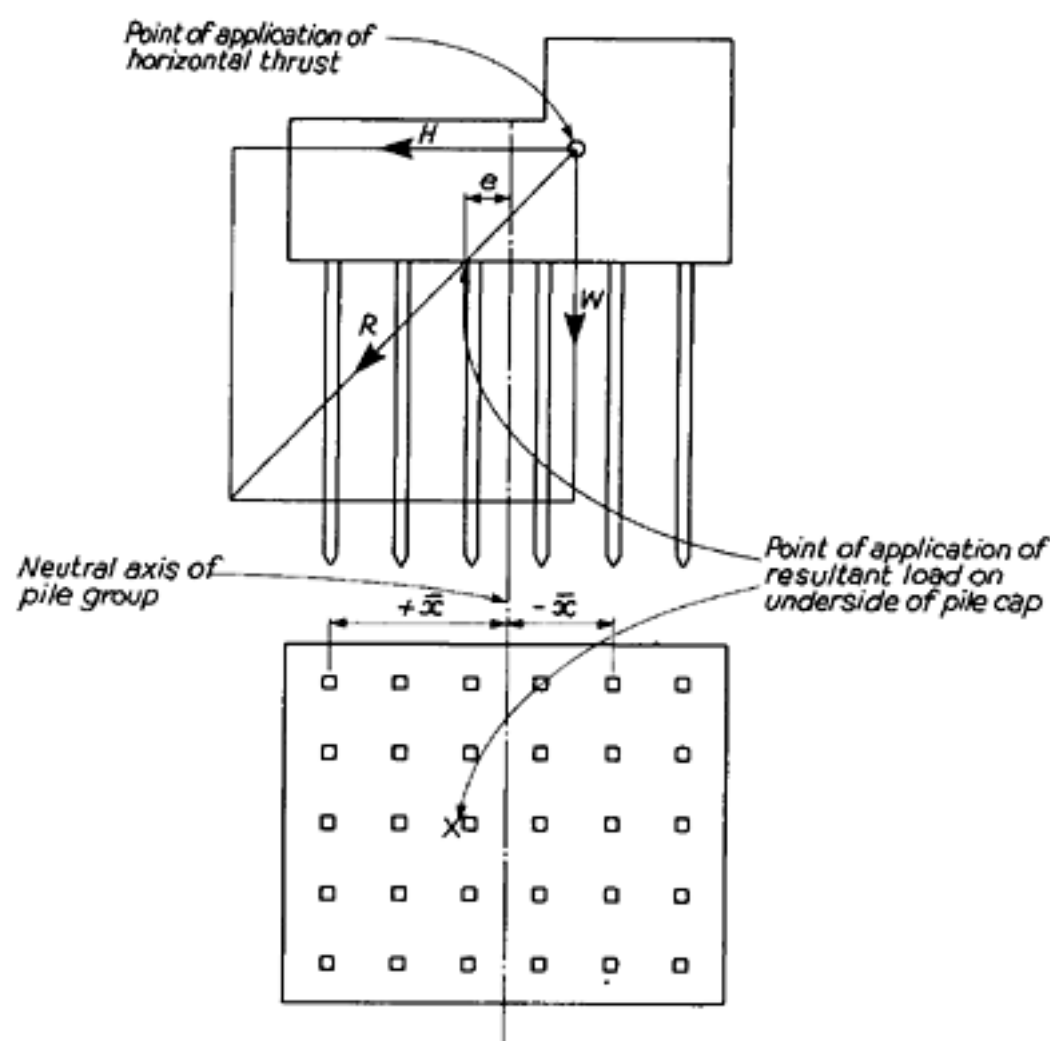


Fig. 6.56 Calculating load distribution on group of piles carrying vertical and horizontal loading

Determination of the individual loads on groups of raking or combined raking and vertical piles is a complex matter if there are more than three rows of piles in the group. The latter case can be analysed by static methods if it is assumed that the piles are hinged at their upper ends, that horizontal loads are carried only by axial forces in the inclined piles and that vertical piles do not carry any horizontal loading. The forces in the piles are resolved graphically as shown in [Figure 6.57](#). The same method can be used if pairs of piles or individual groups of three closely-spaced piles are arranged in not more than three rows, as shown in [Figure 6.58](#). To produce the parallelogram of forces the line of action of the forces in the piles is taken as the centre-line of each individual group.

The determination of the individual loads on piles installed in groups comprising multiple rows of raking or combined raking and vertical piles is a highly complex process which involves the analysis of movements in three dimensions; that is movements in vertical and horizontal translation and in rotational modes. The analysis of loadings on piles subjected to these movements requires the solution of six simultaneous equations, necessitating the use of a computer for practical design problems.

Elson([6.15](#)) has reviewed analytical methods and the computer programs based on them. He reproduces design charts derived from the Poulos method([6.28](#)). Available computer programs are:

Program	Source	Availability
SWPile	Midland Road Construction Unit, Warwickshire County Council	Ove Arup and Partners, Warwick
Minipoint	Department of Transport, Highway Engineering Computer Branch	Various bureaux
PGROUP310	Department of Transport, Highway Engineering Computer Branch	Various bureaux
M-PILE	Department of Transport, Highway Engineering Computer Branch	Various bureaux
LAWPILE	Wood	London United Computing Systems
DEFPIG	Poulos	University of Sydney Engineering Laboratory

The case of closely-spaced groups of piles acting as a single unit when subject to lateral loads must also be considered. Prakash([6.32](#)) showed that piles behave as individual units if they are spaced at more than three pile widths in a direction normal to the direction of loading and at more than 6 to 8 diameters parallel to this direction. Piles at a closer spacing can be considered to act as a single unit in order

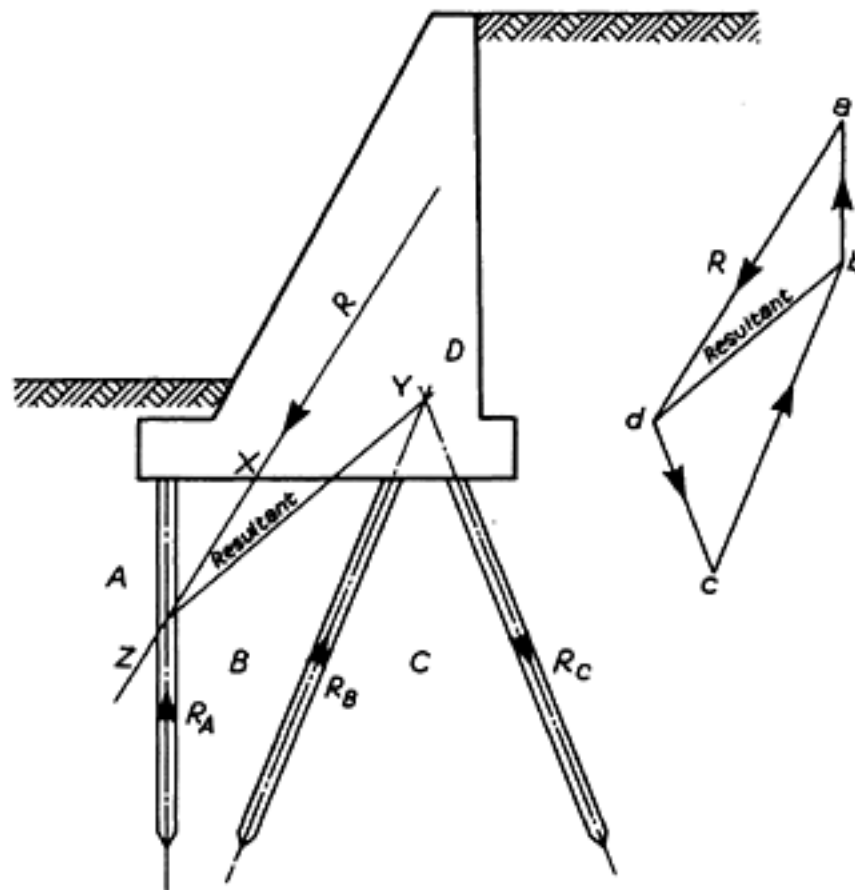


Fig. 6.57 Graphical method for determining forces on groups of vertical and raking piles under inclined loading

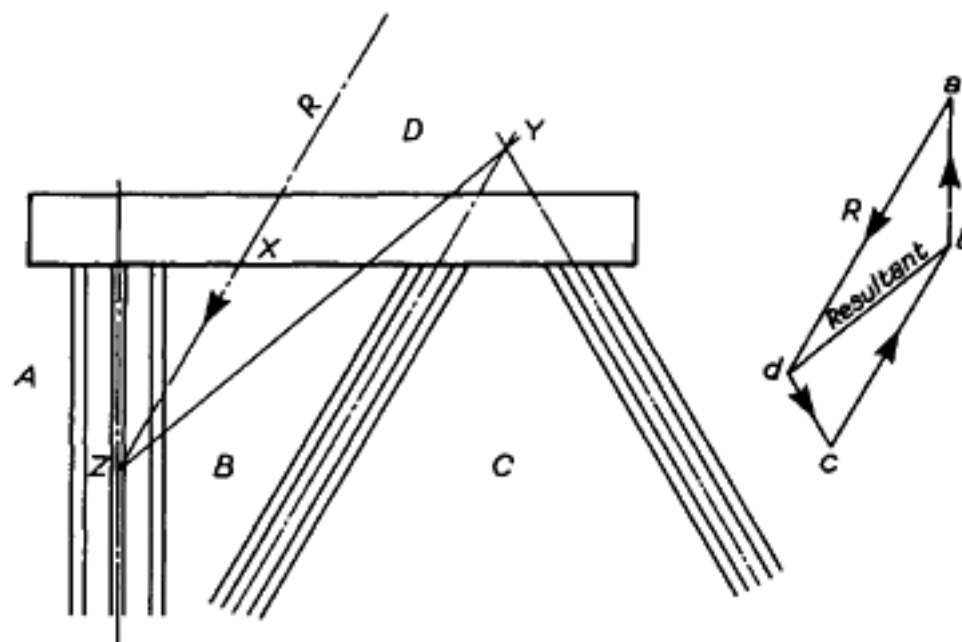


Fig. 6.58 Graphical method for determining forces on groups of closely-spaced vertical and raking piles under inclined loading

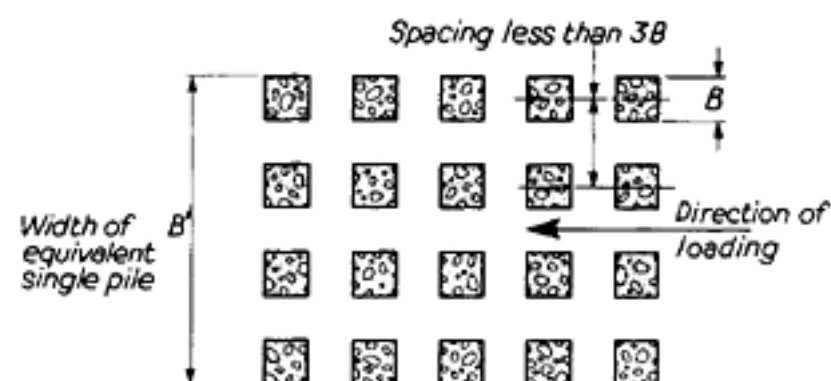


Fig. 6.59 Piles at close spacing considered as single unit (after Prakash<sup>(6.32)</sup>)

to calculate the ultimate resistance and deflections under lateral loads (Figure 6.59). The various methods of calculating the deflections are reviewed by Elson<sup>(6.15)</sup>.

Meyerhof<sup>(6.33)</sup> reported the results of inclined loading tests on model pile groups with eccentricities of loading of up to one-half of the width of the group. These tests showed that the reduced vertical base resistance due to the inclined loading was offset by the increased skin friction, such that the total bearing capacity of the group was practically unaffected by the eccentricity of loading. Thus the ultimate bearing capacity of the group could be calculated on the assumption of a centrally applied vertical load. However, when the width of a pile group is of the same order of magnitude or is greater than the

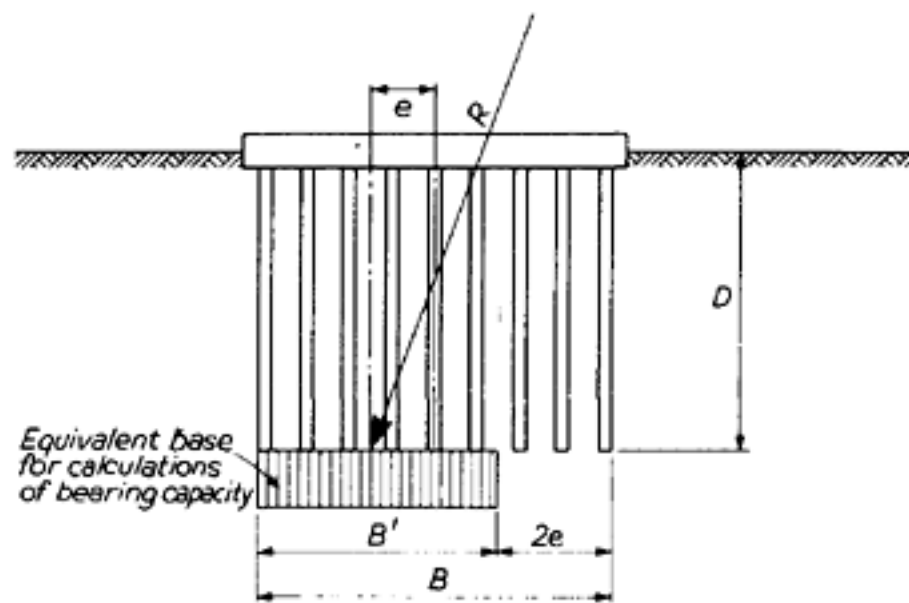


Fig. 6.60 Inclined loading on group of closely spaced vertical piles

penetration length of the piles the effects of lateral earth pressure are relatively small, and the ultimate bearing capacity must be calculated in the same manner as a shallow foundation under inclined loading. Brinch Hansen<sup>(6.34)</sup> has established a general formula for bearing capacity which takes into account inclined loading with various foundation depth/width ratios, but the simpler method of Meyerhof<sup>(6.33)</sup> is satisfactory for most practical cases. This assumes that only the piles on the compression side of the group carry the loading. The tension piles are ignored and an equivalent base width  $B'$  is adopted where

$$B' = B - 2e \quad \dots(6.94)$$

where  $e$  is the eccentricity of the loading (Figure 6.60).

For the case of pile groups in cohesive soils the second term in equation 5.1 is multiplied by an inclination factor  $i_c$  where

$$i_c = (1 - \alpha/90)^2 \quad \dots(6.95)$$

and  $\alpha$  is the angle of inclination of the loading to the vertical in Figure 6.60. As already stated in 5.3 pile groups in cohesionless soils carrying vertical loading will not fail as a unit provided that the individual piles have an adequate factor of safety against compressive loading. This is equally true for pile groups carrying inclined loading, where it is only necessary to check the safety factors of the piles carrying the maximum loading on the compression side of the group and on the tension side if uplift occurs on part of the group. Settlements of pile groups carrying inclined loading are calculated by the methods described in 5.2.2 and 5.3 on the assumption that uniform vertical loading occurs over the effective base width  $B'$  obtained from equation 6.94. For calculating settlements the depth  $D$  in Figure 6.60 is taken as the depth to the equivalent raft foundation (Figure 5.3).

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## 6.7 Worked examples

### Example 6.1

A bored-and-cast-in-situ pile having a shaft diameter of 0.6m and a 2m diameter enlarged base is installed to a depth of 11m in a clay with an undrained cohesion of 40kN/m<sup>2</sup> and an angle of shearing resistance of 25°. The ground-water level is below 11m. Calculate the ultimate resistance in uplift of the pile for sustained loading conditions. In [Figure 6.9](#) the failure surface does not extend up to the ground surface.

For  $\phi = 25^\circ$ ,  $H/B = 3$ , giving  $H = 3B = 3 \times 2 = 6\text{m}$ . Also from [Figure 6.9](#), with  $\phi = 25^\circ$  and the value of the shape factor is the maximum of 1.3.  $K_u = 0.88$

The weight  $W$  can be taken conservatively as the weight of a cylinder of soil 2m in diameter extending up to ground level (*i.e.* the weight of the soil displaced by the pile is assumed to be equal to the weight of the concrete). Then from equation 6.2,

$$Q_u = (\pi \times 40 \times 2 \times 6) + \{1.3 \times \frac{1}{2}\pi \times 2.1 \times 9.81 \times 2[(2 \times 11) - 6] \times 6 \times 0.88 \times \tan 25^\circ\} + (\frac{1}{4}\pi \times 2^2 \times 11 \times 2.1 \times 9.81) = 5534\text{kN}.$$

This value of the uplift resistance must not be greater than that given by the uplift resistance  $Q_{ub}$  of the projecting part of the enlarged base plus the skin friction  $Q_{us}$  on the pile shaft. The latter value is calculated on the overall depth minus the depth of the enlarged base (*i.e.* 1.5m) and a zone 1.5m deep at the ground surface where the soil can shrink away from the shaft. From [Figure 4.24](#) Terzaghi and Peck's coefficients for  $\phi = 25^\circ$  are  $N_c = 25$ ,  $N_q = 12.5$  and  $N_\gamma = 10$ . Then from equation 6.3a,

$$Q_{ub} = \frac{1}{4}\pi(2^2 - 0.6^2) [(1.3 \times 40 \times 25) + (2.1 \times 9.81 \times 11) (12.5 - 1) + (0.4 \times 2.1 \times 9.81 \times 2 \times 10)] = 11\,638\text{kN}.$$

Calculating the skin friction on the cohesion component only of the soil, from equation 4.5,

$$Q_{us} = 0.3 \times 40 \times \pi \times 0.6 [11 - (2 \times 1.5)] = 181\text{kN}.$$

Therefore,

$$\text{total uplift resistance} = Q_{ub} + Q_{us} = 11\,638 + 181 = 11\,819\text{kN}.$$

Although the length/diameter ratio is 5.5 it is advisable to take only one-half of this value as the ultimate resistance; *i.e.* a value of 5910kN. Therefore the ultimate resistance is governed by equation 6.2; *i.e.* has a value of 5 534kN.

### Example 6.2

The floor of a shipbuilding dock covers an area of 210×60m. The 0.8mm floor is restrained against uplift by precast concrete shell piles having an overall diameter of 450mm which are driven through 8m of soft clay ( $\bar{c}_u = 164\text{N/m}^2$ ) on to a strong shale ( $\gamma = 2.3\text{Mg/m}^3$ ). The piles are spaced on a 3m square grid and each pile carries an uplift load of 1100kN. Design a suitable anchorage system for the dock floor using stressed cable anchors. From equation 4.5, and adopting an adhesion factor of 1 (Figure 4.7),

$$Q_s = 1 \times 16 \times \pi \times 0.45 \times 8 = 181\text{kN}.$$

For a safety factor of 2.5,

$$\text{allowable uplift resistance of the pile in soft clay} = 181/2.5 = 72\text{kN}.$$

Thus the load to be carried by anchorage in shale =  $1\,100 - 72 = 1\,028\text{kN}$ . This load can be resisted by an anchor cable made up from seven 15.2mm 7-wire Dyform strands. The breaking load of each strand is 300kN. Therefore,

$$\text{working load} = \frac{1\,028}{7 \times 300} = 0.49 \times \text{breaking load of strand},$$

which is satisfactory. The approximate overall diameter of the 7-strand cable is 45mm. Therefore for an *allowable* bond stress between steel and grout of  $1.0\text{N/mm}^2$ ,

$$\text{required bond length of cable} = \frac{1\,028 \times 1\,000}{\pi \times 45 \times 1.0 \times 1\,000} = 7.3\text{m}.$$

Drill the cable hole to 9m and provide an unwrapped and cleaned bond length of 7m with compression fittings swaged on to the lower end. The cable can be fed down a 150mm borehole for which,

$$\text{working bond stress between rock and grout} = \frac{1\,028 \times 1\,000}{\pi \times 150 \times 7.0 \times 1\,000} = 0.31\text{N/mm}^2,$$

which is satisfactory for a strong shale (Table 6.1). The stress is not excessive if the anchors are stressed to 1.5 times the working load during installation.

From Figure 6.17a, the volume of a rock cone with a  $30^\circ$  half angle lifted by single anchor cable is  $0.35 \times 9^3 = 255\text{m}^3$ . The submerged weight of the rock cone =  $1.3 \times 9.81 \times 255/1\,000 = 3.25\text{MN}$ .

$$\text{Factor of safety against uplift} = \frac{3.25}{1.03} = 3.1, \text{ (which is satisfactory).}$$

The anchorage of the whole dock floor requires 70 lines of anchors (at right-angles to the centre-line of the dock) and 20 ranks of anchors (parallel to the centre-line of the dock) to form the 3m square grid. Therefore in Figure 6.17b,  $N = 70$ ,

$M = 20$ , and  $P = S = 3\text{m}$ . From Figure 6.17a,  $m/L = n/L = 0.57$ , and therefore,  $m = n = 0.57 \times 9 = 5.1\text{m}$ . Then  $P/n = s/m = 0.59$  so that, from Figure 6.17b,  $\Delta V_n/V_c = \Delta V_m/V_c = 0.45$ . Because

$(P/n)^2 + (S/m)^2 = 2 \times 0.59^2 = 0.7$  is less than 4, there is composite overlapping of the rock cones, and the charts are not valid. The intersecting cones represent a rock volume roughly estimated to be  $69 \times 3 \times 19 \times 3 \times 6 = 70\,794\text{m}^3$ .

The sum giving the total force resisting uplift is thus as follows.

$$= \frac{210 \times 60 \times 0.8 \times 2.4 \times 9.81}{1\,000} = 237.3\text{MN}$$

$$= \frac{210 \times 60 \times 8.0 \times 0.8 \times 9.81}{1000}$$

$$= 791.1 \text{ MN}$$

$$= \frac{70794 \times 1.3 \times 9.81}{1000}$$

$$= 902.8 \text{ MN}$$

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$$\text{Total} = 1931.2 \text{ MN}$$

$$\text{Total uplift on underside of floor} = \frac{70 \times 20 \times 1100}{1000} = 1540 \text{ MN.}$$

Therefore,

factor of safety against uplift  $= \frac{1931.2}{1540} = 1.25$  (which is satisfactory and a closer estimate of the rock volume is not needed).

Therefore the anchorage should consist of a 3m square grid of 450mm shell piles driven on to rockhead, each pile being anchored 9m below rockhead with a seven 15.2mm/7-wire Dyform strand cable having the bottom 7m bonded to the rock.

### Example 6.3

A piled dolphin carrying a horizontal pull of 1800kN consists of a pair of compression piles and a pair of tension piles, raked at angles of 1 horizontal to 3 vertical. Design 'dead' anchors for the tension piles, which are driven through 3m of weak weathered chalk to near-refusal on strong rock chalk (having an average submerged density of 0.5Mg/m<sup>3</sup>).

From the triangle of forces ([Figure 6.61](#)) the uplift load on a pair of tension piles is 2800kN. The load to be carried by a single pile is thus

From [Table 6.1](#), the ultimate skin friction  $0.5 \times 2800 = 1400\text{kN}$  in weathered chalk is 0.2N/mm<sup>2</sup>. For a safety factor of 2.5 in uplift, in weathered chalk on a 600mm steel tubular pile,

$$\text{allowable load} = \frac{\pi \times 0.6 \times \sqrt{(3^2 + 1^2)} \times 0.2 \times 1000}{2.5} = 480\text{kN}.$$

The load to be carried by the dead anchor is thus  $1400 - 480 = 920\text{kN}$ . Use a steel tube having an overall diameter of 6.5in (168.3mm). A wall thickness of 5/16in (16mm) provides a cross-sectional area of 7600mm<sup>2</sup>. Thus

$$\text{working tensile stress in anchor} = \frac{920 \times 1000}{7600} = 120\text{N/mm}^2.$$

If high-tensile steel (which has a yield stress of 347N/mm<sup>2</sup>) is used,

$$\text{factor of safety at yield} = \frac{347}{120} = 2.9 \text{ (which is satisfactory).}$$

The anchor will be installed in a 215mm diameter drill-hole. Thus for an ultimate bond stress between grout and strong rock of 0.8N/mm<sup>2</sup> ([Table 6.1](#)), allowable bond stress  $= 0.8/2.5 = 0.3\text{MN/m}^2$ .

$$\text{required bond length} = \frac{920 \times 1000}{\pi \times 215 \times 0.3 \times 1000} = 4.5\text{m}.$$

Drill the hole to say 5m below the toe of the pile and grout the annulus fully. Then

$$\text{working stress between steel and grout} = \frac{920 \times 1000}{\pi \times 168.3 \times 5 \times 1000} = 0.35\text{N/mm}^2 \text{ (which is satisfactory).}$$

Checking the uplift resistance of the cone of rock, the weathered chalk and hard chalk will act together. Therefore in [Figure 6.17a](#),  $L = 5 + \sqrt{3^2 + 1^2} = 8.16\text{m}$ . If  $\theta$  is 71.5°, the volume of rock cone for a single anchor is  $0.38 \times 8.16^3 = 206.5\text{m}^3$  and the weight of the cone is  $0.5 \times 9.81 \times 206.5 = 1013\text{kN}$

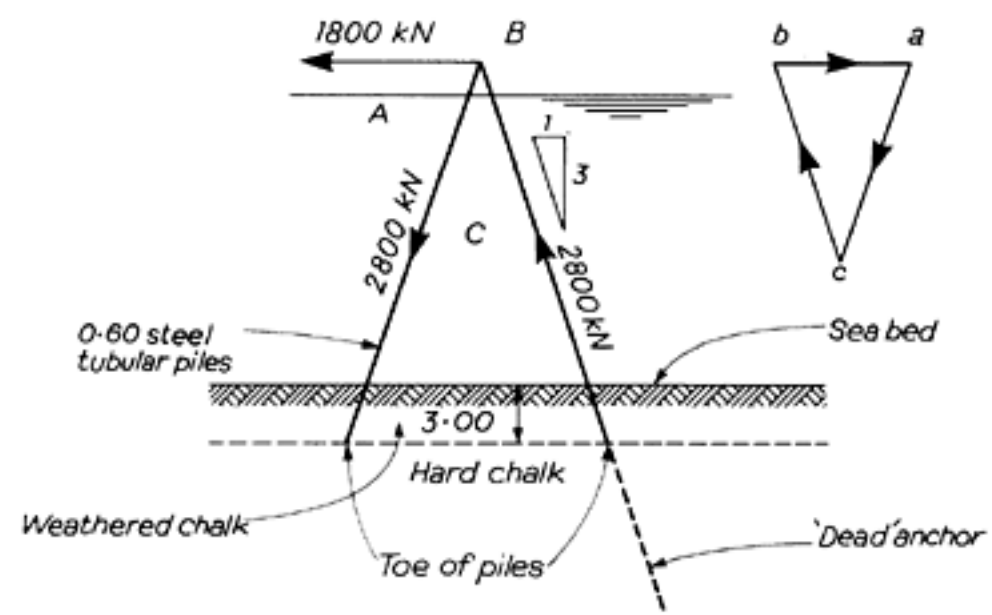


Fig. 6.61

This is inadequate for an uplift load of 2800kN, even if the piles are so spaced that the cones do not intersect. Increase the anchor length to give a total value of  $L$  of 10m below the surface of the weathered chalk and space the piles at 4m centres. Then in [Figure 6.17a](#),  $V_c = 0.35 \times 10^3 = 350\text{m}^3$ . Since  $m/L = 0.61$ ,  $m = 10 \times 0.61 = 6.1\text{m}$ . In [Figure 6.17b](#),  $S = 4\text{m}$ , so that  $S/m = 4/6.1 = 0.66$ , and thus  $\Delta V_m/V_c = 0.42$ .  $M = 2$ ,  $N = 1$ , and  $P = 0$ , and therefore  $\Delta V_n/V_c = 0$

Rock volume anchored by pair of anchors  $= 350[(2 \times 1) - (1 \times 0.40)] = 560\text{m}^3$

Weight of rock resisting uplift  $= 560 \times 0.5 \times 9.81 = 2747\text{kN}$

Factor of safety against uplift  $= 2747/2800 = 0.98$ . This is satisfactory since the shear around the cone is not taken into account and the weight of the piles and cap gives added safety. In addition it will be necessary to extend the bonded length by 2.5m so that the base of the rock cone (the critical factor) can be taken at half the bonded length ([Figure 6.14](#)).

If shear connectors are to be provided equation 6.4 can be used to calculate the required bond length. It is not strictly valid for the geometry of the connection but this example will illustrate the use of the equations. Assume a characteristic grout compression strength of  $25\text{N/mm}^2$  at 3 days and a modular ratio of 18. For a shear key upstand height of 10mm and a spacing of 150mm, the ratio  $h/s$  of 0.067 gives  $C_s = 1.0$ . Assume a bond length to anchor tube diameter greater than 12 to give  $C_L = 0.7$

From equation 6.5, stiffness factor

$$K = \frac{1}{18} \left( \frac{568}{200} \right)^{-1} + \left( \frac{168}{16} + \frac{600}{16} \right)^{-1}$$

$$= 0.04$$

From equation 6.4, characteristic bond strength

$$= 0.04 \times 0.7 (9 \times 1 + 1100 \times \frac{10}{150}) \sqrt{25}$$

$$= 11.5\text{N/mm}^2$$

For work of this kind special grout monitoring would not be used. Hence a safety factor of 6 should be used giving,

$$\text{Allowable bond strength} = \frac{11.5}{6} = 1.9\text{N/mm}^2$$

$$\text{Required bond length over anchor} = \frac{920 \times 1000}{\pi \times 168.3 \times 1.9} = 916\text{mm}$$

Therefore provide  $916/150 = 6.1$ , say 7 shear keys spaced at 150mm centre over a distance of 1.05m over anchor tube and pile.

The bond length required from considerations of uplift of a cone of rock, *i.e.* a length of 12.5m can be checked for compliance with the Eurocode regulations. The working load on a single anchor pile of 1400kN is increased by a factor of 1.3 to allow for cycles of berthing impact loading to give required tensile capacity  $= Q_t = 1820\text{kN}$ . The ultimate rock to grout bond stress of  $0.8\text{MN/m}^2$  is reduced by 50% to allow for cyclic loading.

$$\text{Characteristic tensile capacity} = Q_{tk} = \frac{0.5 \times 0.8 \times \pi \times 0.215 \times 12.5}{1.5} = 2.25\text{MN}$$

$$\text{Design tensile capacity} = Q_{tk}/\gamma_s = 2.25/1.3 = 1.7\text{MN}.$$

This is slightly lower than the required value but will be satisfactory if the bond resistance of the pile in the weathered chalk is added.

### Example 6.4

A vertical bored-and-cast-in-situ pile 900mm in diameter carrying a centrally-applied vertical compressive load of 1500kN is installed to a depth of 6m in a stiff over-consolidated clay ( $\phi' = 25^\circ$ ). Find the ultimate sustained horizontal load which can

be applied ( $\bar{c}_u = 120\text{kN/m}^2$ ,  $c' = 10\text{kN/m}^2$ ), at a point 4m above ground level. Also find the maximum working load if the lateral deflection of the pile at ground level is limited to not more than 25mm.

Considering first the ultimate horizontal load. For conditions of immediate application, *i.e.* using the undrained shearing strength, from [Table 6.2](#) for  $\bar{c}_u = 120\text{kN/m}^2$ , the soil modulus  $k$  is  $7.5\text{MN/m}^2$ . If the elastic modulus of concrete is  $26 \times 10^3\text{MN/m}^2$  and the moment of inertia of the pile is  $0.0491 \times (900)^4\text{mm}^4$ , from equation 6.8,

$$R = \sqrt[4]{\frac{26 \times 10^3 \times 0.0491 \times 0.9^4}{7.5 \times 0.9}} = 3.3\text{m}$$

L is 6m which is less than 2R, therefore the pile will behave as a short rigid unit, and Brinch Hansen's method can be used. Brinch Hansen's coefficients, as shown in [Figure 6.21](#) with  $c = c_u = 120\text{kN/m}^2$  and  $\phi = 0$  are as tabulated below.

$z$ (m)	0	1	2	3	4	5	6
$\frac{z}{B}$	0	1.1	2.2	3.3	4.4	5.5	6.6
$K_c$	2.2	5.5	6.2	6.7	7.0	7.2	7.3
$c_u K_c$	264	660	744	804	840	864	876

The soil resistance of each element 1m wide by 1m deep is plotted in [Figure 6.62a](#). As a trial assume the point of rotation X is at 4.0m below ground level. Then taking moments about point of application of  $H_u$ ,

$$\begin{aligned} \Sigma M &= (462 \times 1 \times 4.5) + (702 \times 1 \times 5.5) + (774 \times 1 \times 6.5) + (822 \times 1 \times 7.5) \\ &- [(852 \times 1 \times 8.5) + (870 \times 1 \times 9.5)] = + 1629\text{Nm per metre width of pile.} \end{aligned}$$

If the point of rotation is raised to 3.9m below ground level,  $\Sigma M = + 297\text{kNm}$ , which is sufficiently close to zero for the purpose of this example. Taking moments about the centre of rotation,

$$\begin{aligned} H_u \times 7.9 &= (462 \times 3.4) + (702 \times 2.4) + (774 \times 1.4) + (820.2 \times 0.9 \times 0.4) \\ &+ (838.2 \times 0.1 \times 0.05) + (852 \times 0.6) + (870 \times 1.6), \end{aligned}$$

and thus  $H_u = 828\text{kN}$ . For a pile 0.9m wide,  $H_u = 0.9 \times 828 = 745\text{kN}$

This value of  $H_u$  can be checked by using Broms's curves for a short rigid pile ([Figure 6.25](#)). For  $L/B = 6/0.9 = 6.7$  and  $e/b = 4/0.9 = 4.4$ ,  $H_u/c_u B^2 = 8$  giving  $H_u = 8 \times 120 \times 0.9^2 = 778\text{kN}$

Now consider the long-term stability under sustained loading, when the drained shearing strength parameters  $c' = 10\text{kN/m}^2$  and  $\phi' = 25^\circ$  apply- From [Figure 6.22](#), Brinch Hansen's coefficients for  $K_c$  and  $K_q$  are tabulated below.

$z$ (m)	0	1	2	3	4	5	6
$\frac{z}{B}$	0	1.1	2.2	3.3	4.4	5.5	6.6
$K_c$	5.8	16	20	23	26	27	28
$c'K_e$	58	160	200	230	260	270	280
$K_q$	3.3	5.0	5.5	5.9	6.3	6.7	6.9
$p_0(\text{kN/m}^2)$	0	18.6	39.3	58.8	78.5	98.2	118
$p_0K_q(\text{kN/m}^2)$	0	93	216	347	495	658	814
$c'K_x + p_0K_q(\text{kN/m}^2)$	58	253	416	577	755	928	1094

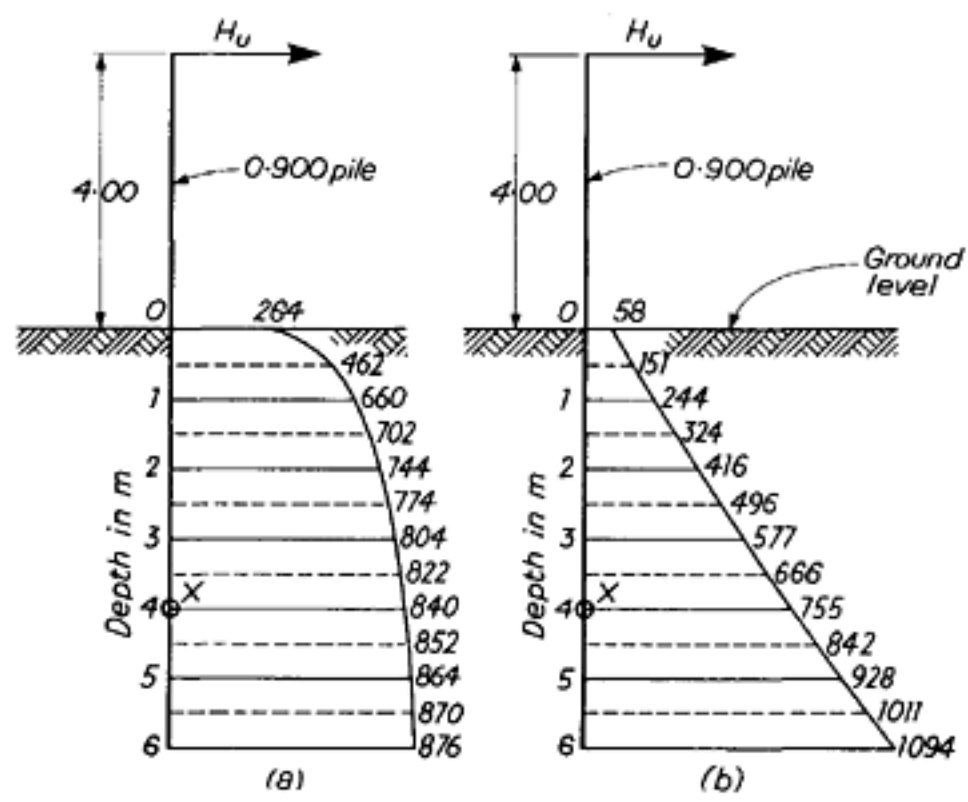


Fig. 6.62

The soil resistance of each element 1m deep for a pile 1m wide is plotted in [Figure 6.62b](#). As a trial, consider the point of rotation X to be 4.0m below ground level. Taking moments about the point of application of  $H_u$ ,

$$\Sigma M = (155 \times 1 \times 4.5) + (335 \times 1 \times 5.5) + (496 \times 1 \times 6.5) + (666 \times 1 \times 7.5) - [(842 \times 1 \times 8.5) + (1011 \times 1 \times 9.5)] = -6002 \text{ kNm.}$$

If the centre of rotation is lowered to 4.5m, then

$$\Sigma M = 10759 + (798 \times 0.5 \times 8.25) - [(885 \times 0.5 \times 8.75) + 9604] = 14051 - 13476 + 575 \text{ kN,}$$

which is sufficiently close to zero for the purpose of this example. Then taking moments about the centre of rotation,

$$H_u \times 8.5 = (155 \times 4.0) + (335 \times 3.0) + (496 \times 2.0) + (666 \times 1.0) + (798 \times 0.5 \times 0.25) + (885 \times 0.3 \times 0.25) + (1011 \times 1 \times 1).$$

Thus  $H_u = 530 \text{ kN}$  per metre width. Therefore the lowest value of the ultimate load results from drained shearing strength conditions. For a 900mm pile, the ultimate horizontal load  $= 0.9 \times 530 = 477 \text{ kN}$

Calculating the allowable horizontal load which limits the lateral deflection at ground level to 25mm, from equation 6.39,

$$\beta = \sqrt[4]{\frac{7.5 \times 0.9}{4 \times 26 \times 10^3 \times 0.0491 \times 0.9^4}} = 0.21.$$

From [Figure 6.34](#),  $y_0 kBL/H$  for  $e/L = 4/6 = 0.66$  and  $\beta L = 1.26$  is approximately equal to 9. Therefore if  $y_0$  is limited to 25mm, then

$$H = \frac{25 \times 10^{-3} \times 7.5 \times 10^3 \times 9 \times 10^{-1} \times 6}{9} = 112 \text{ kN.}$$

The factor of safety on the ultimate load of the soil resistance

The deflection at the pile head can be checked approximately  $= 477/112 = 4.2$  from equation 6.37. Thus

$$y = \frac{112(4 + 1.5)^3}{3 \times 26 \times 10^6 \times 0.0491 \times 0.9^4} = 0.0074 \text{ m} = 7 \text{ mm}$$

This represents a deflection at ground level of about 3mm.

The pile head deflection for the lateral force of 112kN can be checked by the Poulos method ([Section 6.3.8](#)). For

$\bar{C}_u = 120 \text{ kN/m}^2$ , take  $E_s = 200 \times 120 \times 10^{-3} = 24 \text{ MN/m}^2$

giving  $K_R = \frac{26 \times 10^3 \times 0.0491 \times 0.9^4}{24 \times 6^4} = 0.027$

For a free-head pile and  $L/D = 6.7$  the deformation factors from [Figure 6.51](#) are  $I_H = 3.5$ ,  $I_{HM} = I_{\theta H} = 6$  and  $I_M = 20$

Then for  $H_0 = 112 \text{ kN}$  and  $M_0 = 112 \times 4 = 448 \text{ kNm}$ , equations 6.88 and 6.89 give

$$y_0 = \frac{3.5 \times 0.112}{24 \times 6} + \frac{6 \times 0.448}{24 \times 36} = 0.0058 \text{ m} = 6 \text{ mm}$$

$$\theta_0 = \frac{6 \times 0.112}{24 \times 36} + \frac{20 \times 0.448}{24 \times 216} = 0.0023 \text{ radians}$$

Therefore pile head deflection  $= 6 + 4000 \times 0.0025 = 16 \text{ mm}$

which is somewhat larger than the figure given by the approximate method, but less than the required limiting deflection of 25mm. This is the serviceability limit state of the pile which determines the design transverse limit load  $Q_{tr} L$ .

### Example 6.5

A mooring dolphin structure ([Figure 6.63a](#)) consists of four reinforced concrete piles 350mm square rigidly braced by a pile cap at a point 6.9m above the sea bed. The piles are driven through 2.5m of soft clay ( $\bar{c}_u = 12\text{kN/m}^2$ ) to a depth of 6.5m into stiff clay ( $\bar{c}_u = 120\text{kN/m}^2$ ). Find the allowable load (with a factor of safety of 3) which can be applied to the pile cap and calculate the corresponding deflections and bending moments in each pile.

From equation 6.13 the effect of the rigid pile cap and virtual fixity of the pile at the surface of

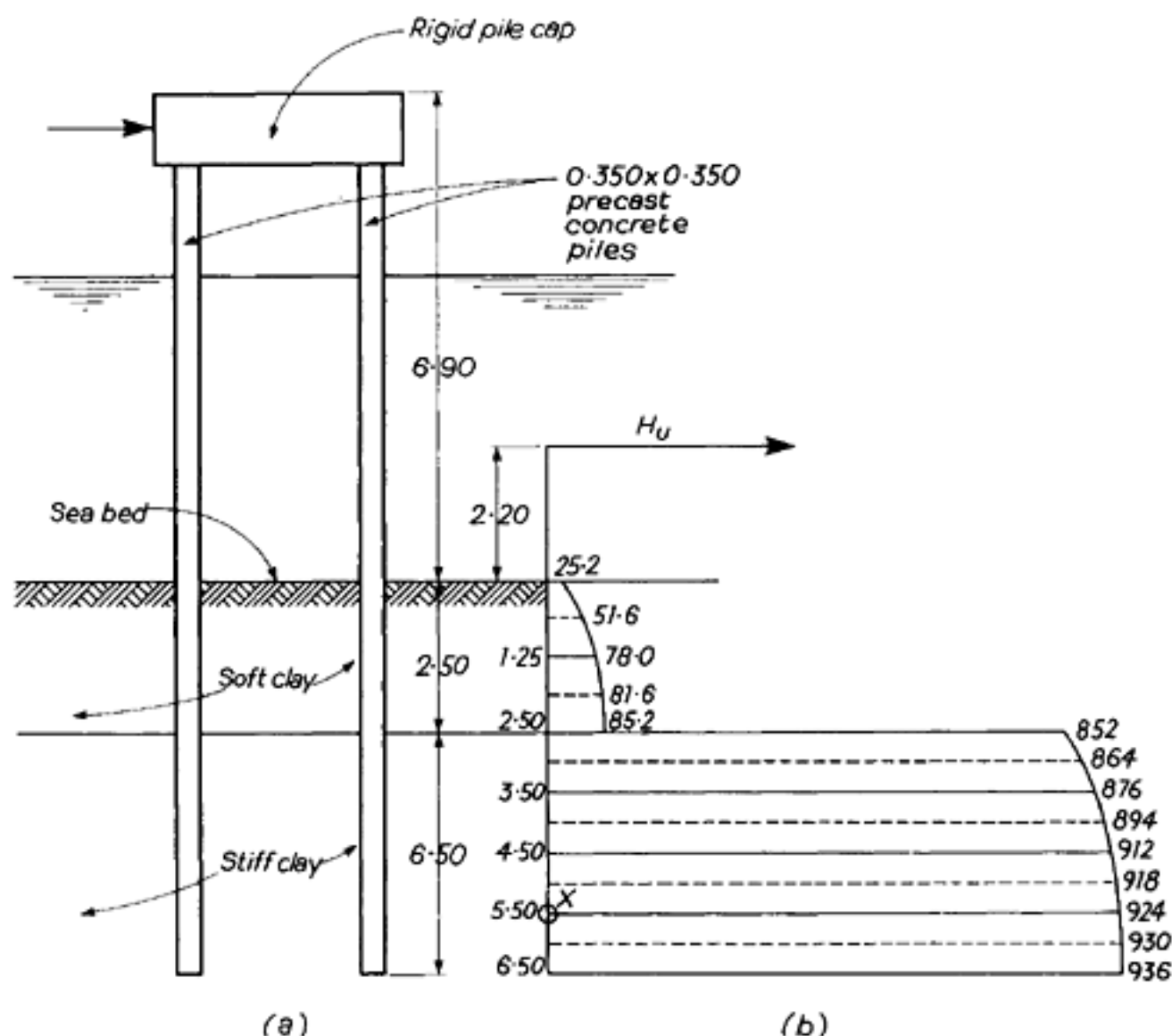


Fig. 6.63

the stiff clay is to provide an equivalent free-headed pile with an effective height of  $\frac{1}{2} \times 9.40 = 4.70\text{m}$  above the top of the stiff clay or 2.20m above the sea bed.

Calculating the stiffness factor in relation to the stiff clay, from Table 6.2, the Terzaghi modulus  $k_1$  is  $30\text{MN/m}^3$ . Therefore

from equation 6.8,  $R$  is  $1.3 \times 10^3\text{mm}$  and therefore  $L$  is less than  $3.5R$  and the pile behaves as a rigid unit. The Brinch Hansen method is used to obtain the ultimate horizontal resistance  $H_u$ . From Figure 6.22, Brinch Hansen's values of  $K_c$  and

the calculated soil resistances for a 1m width of pile are tabulated below.

$z$ (m)	0	1.25	2.50 (soft)	2.50 (stiff)	3.50	4.50	5.50	6.50
$\frac{z}{B}$ (m)	0	3.6	7.1	7.1	10	12.8	15.7	18.6
$K_c$	2.1	6.5	7.0	7.0	7.3	7.6	7.7	7.8
$c_u K_c$ (kN/m <sup>2</sup> )	25.2	78	85.2	852	876	912	924	936

The soil resistances are plotted in Figure 6.63b. Assume a trial centre of rotation X 5.5m below the sea bed. Then taking moments about the point of application of the load at the effective height,

$$\begin{aligned}\Sigma M &= (51.6 \times 1.25 \times 2.82) + (81.6 \times 1.25 \times 4.07) + 1[(864 \times 5.2) + \\ &\quad (894 \times 6.2) + (918 \times 7.2)] - (930 \times 8.2) \\ &= 17\,242 - 7\,626 = 9\,616\text{kNm per metre width.}\end{aligned}$$

If the centre of rotation is raised to 4.8m below sea bed,

$$\begin{aligned}\Sigma M &= (51.6 \times 1.25 \times 2.82) + (81.6 \times 1.25 \times 4.07) + 1[(864 \times 5.2) + (894 \times 6.2)] \\ &\quad + (0.3 \times 913.8 \times 6.85) - [(0.2 \times 916.8 \times 7.1) + (0.5 \times 921 \times 7.45) \\ &\quad + (930 \times 8.2)] = 12\,510 - 12\,358 = +152\text{kNm, per metre width,}\end{aligned}$$

which is sufficiently close to zero for the purpose of this calculation.

Taking moments about the centre of rotation,

$$\begin{aligned} H(2.2 + 4.8) = & (51.6 \times 1.25 \times 4.175) + (81.6 \times 1.25 \times 2.925) + 1[(864 \times 1.8) \\ & + (894 \times 0.8)] + (0.3 \times 913.6 \times 0.15) + (0.2 \times 916.6 \times 0.1) \\ & + (0.5 \times 921 \times 0.45) + (930 \times 1 \times 1.2). \end{aligned}$$

Thus  $H = 603\text{kN}$  per metre width of pile.

This is a large figure for a 350mm pile and  $H_u$  will be governed by the ultimate moment of resistance of the pile. For a

section 350mm square reinforced by one 32mm high-yield steel bar in each corner, with 50mm cover to 8mm link steel, a characteristic strength  $f_{cu}$  of  $25\text{N/mm}^2$  and steel with a yield strength of  $410\text{N/mm}^2$ ,

$d = 350 - (50 + 8 + 16) = 276\text{mm}$ . Then from Chart 22 of CP 110: Part 2, with  $100A_s/bd = 100 \times 2 \times \frac{1}{4}\pi \times 32^2 / (350 \times 276) = 1.66$ ,  $M_u/bd^2 = 4.7$ , so that  $10^6\text{Nmm}$ . Taking as a trial the point of zero shear at 3.0m below the  $M_u = 4.7 \times 350 \times 276^2 = 125 \times$  sea bed and resolving the horizontal forces above this point,

$$603 - [(51.6 \times 1.25) + (81.6 \times 1.25) + (858 \times 0.5)] = +7.5\text{kN},$$

which is sufficiently close to zero to give the point of zero shear, and hence the point of maximum bending moment.

Equating the maximum bending moment with the ultimate resistance moment of the pile,  $H_u(2.20 + 3.00) = 125$  and thus  $H_u = 24.0\text{kN}$ . With a factor of safety of 2.5, the allowable load on a four-pile dolphin is  $24.0 \times 4 / 2.5 = 38\text{kN}$

The ultimate horizontal load on the pile can be checked by using Broms's curves (Figure 6.30). Ignoring the soft clay and taking the equivalent free-headed pile as giving  $e/B = 4.7/0.35 = 13.4$ , and  $M_u/c_u B^3 = 125/(120 \times 0.35^3) = 24.3$ .

Therefore from Figure 6.30,  $H_u = 1.5 \times 120 \times 0.35^2 = 22.0\text{kN}$ . This is reasonably close to the value calculated by Brinch Hansen's method.

For a working load on a single pile of  $\frac{1}{4} \times 38 = 9\text{kN}$  (approx) the deflection at the surface of the stiff clay (ignoring the resistance of the soft clay) can be calculated from equation 6.42, where

$$\beta = \sqrt[4]{\frac{30 \times 10^3 \times 0.35 \times 12}{4 \times 26 \times 10^6 \times 0.35^4}} = 0.53\text{m},$$

and  $k_\infty$  for a concrete pile in a stiff clay (equation 6.44)  $= 0.4 \times 1.15K_0/0.35 = 1.31K_0 = 1.31 \times 0.305$ . Therefore from equation 6.42,  $\times 30 = 12$

$$\text{deflection of pile at stiff clay surface} = \frac{2 \times 9 \times 0.53[(4.70 \times 0.53) + 1] \times 10^3}{12 \times 10^3 \times 0.35} = 7.9\text{mm},$$

This deflection corresponds to a slope in the pile at sea-bed level of  $2 \times 7.9 / (2.5 \times 1000) = 0.0063\text{mm}$  per metre. Thus,

$$\text{corresponding deflection at pile head} = 0.0061 \times 6.9 \times 1000 = 42\text{mm}.$$

To this must be added the deflection of the fixed-headed pile head considered as a cantilever fixed at sea-bed level and carrying a horizontal load of 9kN. Then from equation 6.38,

$$\text{deflection at pile head} = \frac{9 \times 6.9^3 \times 12}{12 \times 26 \times 10^6 \times 0.35^4} = 8 \times 10^{-3}\text{m}.$$

Thus total deflection at pile-cap level  $= 8 + 43 = 51\text{mm}$ .

### Example 6.6

A group of thirty-six Larssen BP5 box piles are spaced at 1.25m centres in both directions to form six rows of six piles surmounted at ground level by a rigid cap. The piles are driven to a depth of 9m into a medium-dense water-bearing sand. Find the ultimate horizontal load which can be applied to a single pile and calculate the bending moments, deflections and soil-resistance values at various points below the ground surface for a working load of one-half of the ultimate value. Calculate the horizontal deflection of the pile cap if the horizontal load is applied in the direction resisted by the maximum resistance moment of the piles. Moment of inertia of BP5 pile in direction of maximum resistance moment  $= 58\,064\text{cm}^4$  and elastic modulus of steel  $= 20\text{MN/cm}^2$ .

From Figure 6.20, Terzaghi's value of  $n_h$  for a medium dense sand is  $5\text{MN/m}^3$ . Then from equation 6.5,

$$T = \sqrt[5]{\frac{20 \times 58064}{5 \times 10^{-6}}} = 188\text{cm.}$$

Because the embedded length of 9m is more than  $4T$  the pile behaves as a long elastic fixed-headed element and the ultimate resistance can be obtained from Broms's curve ([Figure 6.32](#)). The ultimate resistance of a Larssen BP5 pile

(modulus of section =  $2950\text{cm}^3$ ) =  $2950 \times 0.0247 = 72.86\text{MNcm}$  for a working stress on BS4360 Grade 40A steel of  $247\text{MN/m}^2$ . Submerged density of soil =  $1.3\text{Mg/m}^3$ .  $K_p = 1 + \sin 35^\circ / (1 - \sin 35^\circ) = 3.69$ . Breadth of pile

$B = 46.7\text{cm}$ . From [Figure 6.32](#) for

$$\frac{M}{B^4 \gamma K_p} = \frac{72.86 \times 1000 \times 10^6}{46.7^4 \times 1.3 \times 9.81 \times 3.69} = 326,$$

$$\frac{H_u}{K_p B^3 \gamma} = 100 \text{ for restrained head.}$$

Therefore  $H_u = 100 \times 3.69 \times 46.7^3 \times 1.3 \times 9.81 \times 10^{-6} = 479 \text{ kN}$ , For the specified safety factor, the working load is  $\frac{1}{2} \times 479 = 239.5 \text{ kN}$ , say **240 kN**

The deflections, bending moments and soil-resistance values for the single pile at the working load can be calculated from the curves in [Figure 6.39](#). From equation 6.57,

$$\text{deflection } y_F = \frac{240 \times 188^3}{20 \times 10^3 \times 58064} F_y = 1.373 F_y \text{ cm} = 13.73 F_y \text{ mm.}$$

From equation 6.58,

$$\text{bending moment } M_F = 240 \times 188 F_m = 45120 F_m \text{ kNcm} = 451.2 F_m \text{ kNm.}$$

From equation 6.59,

$$\text{soil reaction } P_F = 240 F_p / 188 = 1.28 F_p \text{ kN per cm depth} = 128 F_p \text{ kN per m depth.}$$

$Z_{max} = L/T = 9.0/1.88 = 4.8$ . Tabulated values of  $y_F$ ,  $M_F$  and  $P_F$  are as follows.

$x \text{ (m)}$	0	0.5	1.0	1.5	2.0	2.5	3.0	4.0	5.0
$Z = \frac{x}{T}$	0	0.27	0.53	0.80	1.06	1.33	1.60	2.13	2.66
$F_y$	+0.92	+0.90	+0.82	+0.71	+0.61	+0.50	+0.37	+0.18	+0.04
$y_F \text{ (mm)}$	+12.6	+12.4	+11.3	+9.7	+8.4	+6.9	+5.1	+2.5	—
$F_m$	−0.91	−0.65	−0.40	−0.18	−0.03	+0.10	+0.19	+0.25	+0.21
$M_F \text{ (kN/m)}$	−411	−293	−180	−81	−14	+4.5	+86	+113	+95
$F_p$	0	+0.25	+0.45	+0.57	+0.62	+0.62	+0.57	+0.38	+0.13
$P_F \text{ (kN/m)}$	0	+32.0	+57.6	+73.0	+79.4	+79.4	+73.0	+48.6	+16.6

Because the piles are spaced at  $125/46.7 = 2.67$  diameters the group will act as a single unit equivalent to a block foundation having a width of  $5 \times 1.25 \text{ m} = 6.25 \text{ m}$  and a depth below the ground surface of 9m. The ultimate passive resistance to the horizontal thrust from a block foundation is

$$P_u = \frac{1}{2} B \gamma D^2 K_p = \frac{1}{2} \times 6.25 \times 1.3 \times 9.81 \times 9^2 \times 3.69 = 11912 \text{ kN.}$$

For a safety factor of 1.5 on the passive resistance, the total horizontal load on the pile group must be limited to  $11912/1.5 = 7941 \text{ kN}$ . Thus the load on each pile must be limited to  $7941/36 = 220 \text{ kN}$ , which is fairly close to the value estimated for the single pile.

It is also necessary to calculate the horizontal deflection of the pile group under the working load of 220 kN per pile. The values of  $P_F$  above show that the horizontal load is effectively distributed over a depth of 4m below the ground surface. Thus

the load on the group can be simulated by a block foundation having a ‘width’  $B$  of 4m, a ‘length’ of 6.25m and a depth of 6.25m. The elastic modulus of a medium-dense sand can be taken as  $20 \text{ MN/m}^2$ . From equation 5.3, with  $H/B = 1000$ ,  $L/B = 6.25/4 = 1.55$ ,  $D/B = 6.25/4 = 1.55$ ,  $\mu_1 = 0.85$  and  $\mu_0 = 0.91$

$$\text{elastic settlement } \rho_i = \frac{0.85 \times 0.91 \times \frac{220 \times 36}{4 \times 6.25} \times 4 \times 1\,000}{20 \times 1\,000} = 49\text{mm}$$

This is within safe limits and, as would be expected, it is more than ten times greater than the deflection of the single pile.

### Example 6.7

In the previous example the horizontal working load of  $36 \times 220 = 7\,920\text{kN}$  is applied at the level of the centroid of a 7.25m square by 2.0m deep pile cap. If a centrally-applied vertical load of 8000kN also acts at the centroid, find the resultant vertical component of the load on each pile of the group due to the combined horizontal and vertical loading.

Weight of pile cap  $= 7.25^2 \times 2.0 \times 9.81 \times 2.4 = 2\,475\text{kN}$ . Thus the total vertical load

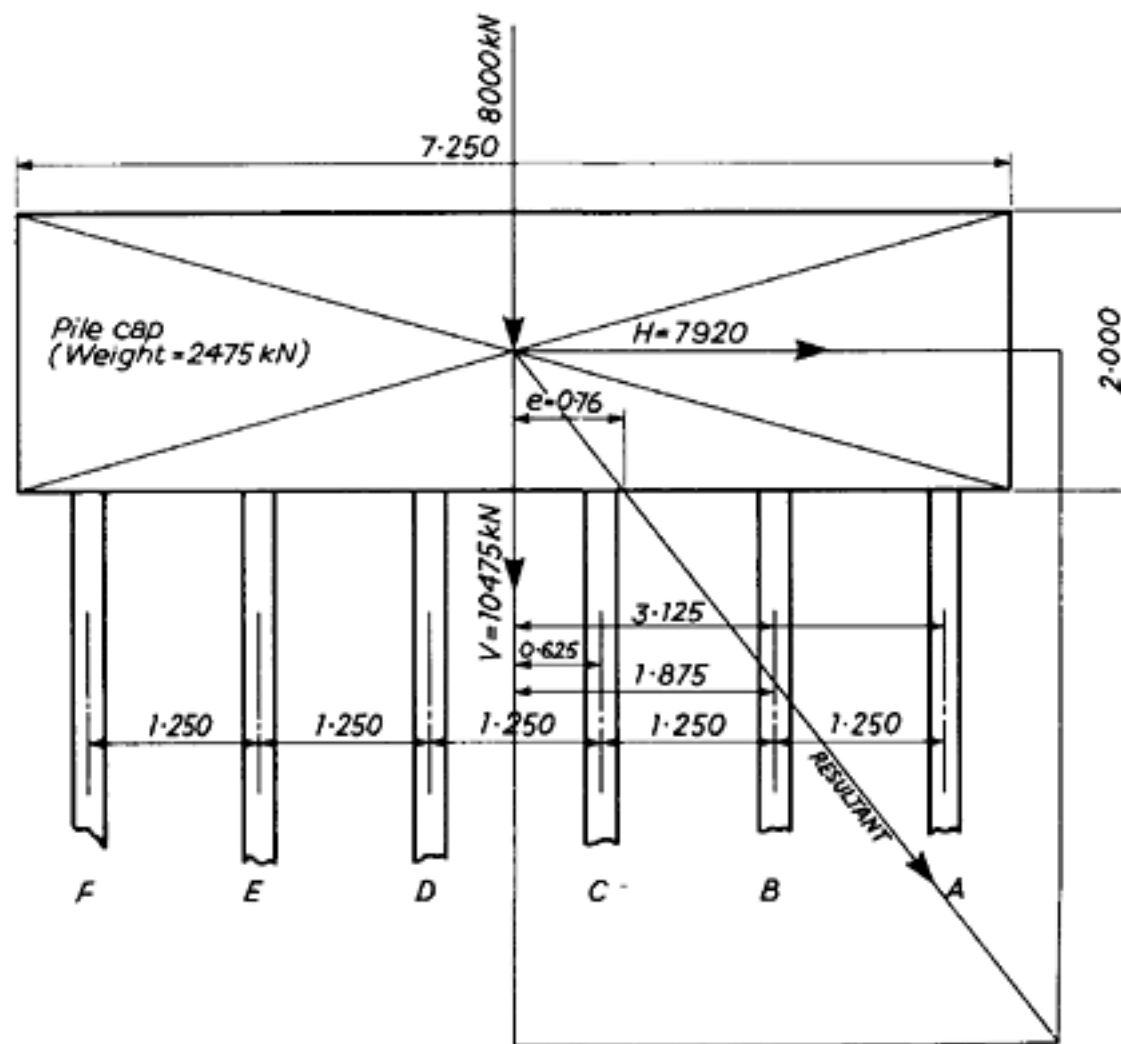


Fig. 6.64

= 10 475kN. From [Figure 6.64](#) the resultant of the horizontal and vertical loads cuts the underside of the pile cap at a point 0.76m from the centroid. From equation 6.93.

$$\begin{aligned}
 &= \frac{10\,475}{36} + \frac{10\,475 \times 0.76 \times 3.125}{12(3.125^2 + 1.875^2 + 0.625^2)} \\
 &= 291.0 + \frac{24\,878}{164.1} \\
 &= 291.0 + 151.6 = 442.6\text{kN} \\
 &= 291.0 + \frac{10\,475 \times 0.76 \times 1.875}{164.1} = 291.0 + 91.0 = 382.0\text{kN} \\
 &= 291.0 + \frac{10\,475 \times 0.76 \times 0.625}{164.1} = 291.0 + 30.3 = 321.3\text{kN} \\
 &D = 291.0 - 30.3 = 260.7\text{kN} \\
 &E = 291.0 - 91.0 = 200.0\text{kN} \\
 &F = 291.0 - 151.7 = 139.3\text{kN}
 \end{aligned}$$

Checking:  $6(442.6 + 382.0 + 321.3 + 260.7 + 200.0 + 139.3) = 10\,475\text{kN}$  and the calculation is thus correct.

### Example 6.8

A tower is to be constructed on a site where 6m of very soft clay overlies a very stiff glacial clay (undrained shearing strength=190kN/m<sup>2</sup>). The tower and its base slab weigh 30000kN, and the tower is subject to a maximum horizontal wind force of 1500kN with a centre of pressure 35m above ground level. The base of the tower is 12m in diameter. Design the foundations and estimate the settlements under the dead load and wind loading.

Because of the presence of the soft clay layer, piled foundations are required and the heavy vertical load favours the use of large bored-and-cast-in-situ piles. A suitable arrangement of piles to withstand the eccentric loading caused by the wind force is twenty-two piles in the staggered pattern shown in [Figure 6.65](#).

The resultant of the vertical and horizontal forces has an eccentricity of  $1\,500 \times 35 / 30\,000 = 1.75\text{m}$  at ground level. From equation 6.93, the load on each of the outer four piles due to wind loading from an east-west direction is given by

$$V = \frac{30\,000}{22} \pm \frac{30\,000 \times 1.75 \times 6}{(4 \times 6^2) + (6 \times 4.5^2) + (4 \times 3^2) + (6 \times 1.5^2)}$$

$$= 1\,364 \pm 1\,000\text{kN}.$$

Therefore uplift does not occur on the windward side and the maximum pile load is 2364kN. Checking the maximum pile load for wind in a north-south direction,

$$V = 1\,364 \pm \frac{30\,000 \times 1.75 \times 5.20}{(8 \times 5.20^2) + (10 \times 2.60^2)} = 1\,364 \pm 962\text{kN}.$$

Therefore, maximum pile load=2 326kN.

For piles with a shaft diameter of 1m,

$$\text{working stress on concrete} = \frac{2\,364 \times 1\,000}{\frac{1}{4}\pi \times 1\,000^2} = 3\text{N/mm}^2,$$

which is within safe limits.

Adopting an under-reamed base to a diameter of 1.8m, from equation 4.2,

$$\text{ultimate base resistance } Q_b = 9 \times 190 \times \frac{1}{4}\pi \times 1.8^2 = 4\,351\text{kN}.$$

For a safety factor of two on the combined base resistance and skin friction, the required ultimate skin friction  
 $= (2 \times 2\,364) - 4\,351 = 377\text{kN}$

If the required depth of penetration into the glacial clay to mobilize the required ultimate resistance is  $L$ m, ignoring the small skin friction in the very soft clay and adopting an adhesion factor of 0.3 (to allow for delays in under-reaming), then from equation 4.5,

$$Q_s = 0.3 \times 190 \times \pi \times 1 \times L,$$

and if  $Q_s = 377$ ,  $L = 2.10\text{m}$ . Thus the allowable pile load for a factor of safety of 3 in base resistance and unity in skin friction is  $\frac{1}{3}Q_b + Q_s = (\frac{1}{3} \times 4\,351) + 377 = 1\,827\text{kN}$ , which is insufficient. Taking  $L$  as 4.9m,

$$Q_s = 0.3 \times 190 \times \pi \times 1 \times 4.9 = 877\text{kN},$$

and the allowable pile load is  $\frac{1}{3} \times 4\,351 + 877 = 2\,328\text{kN}$ , which is satisfactory.

It is necessary to add two shaft diameters and the depth of the under-ream to arrive at the total penetration of the piles below ground level. Thus

$$D = 6\text{m (soft clay)} + 4.9 + 2.0 + 0.8 + = 13.7\text{m}.$$

An adhesion factor of 0.5 is used for straight-shafted piles in a glacial clay ([Figure 4.11](#)). Therefore the allowable load on a straight-shafted pile drilled to the same depth as the under-reamed piles and adopting a safety factor of 2 on combined end bearing and shaft friction is given by

$$Q_a = \frac{(9 \times 190 \times \frac{1}{4}\pi \times 1^2) + (0.5 \times 190 \times \pi \times 1 \times 7.7)}{2} = 1\,820\text{kN}.$$

Therefore straight-shafted piles can be used for the eight inner piles as shown in [Figure 6.65](#). The maximum working load on these is one-half or less than one-half of the outer piles.

The overall depth to the base of the pile group of 13.7m is only a little greater than the overall width of the group, *i.e.* 13m to the outsides of the pile shafts. Therefore it is necessary to check that block failure will not occur due to eccentric loading.

$$\text{Eccentricity of loading with respect to base of pile group} = \frac{1\,500 \times (35 + 13.7)}{30\,000} = 2.43\text{m}.$$

From equation 6.94, the width of an equivalent block foundation for winds in a north-south direction

$= 10.40 - (2 \times 2.43) = 5.54\text{m}$ . The overall dimensions of this block foundation are thus  $13 \times 5.54\text{m}$ . The tangent of the angle of inclination of the resultant force  $= \tan \alpha = 1\,500/30\,000 = 0.05$ , and thus

From equation 6.95, the inclination factor  $i_c = (1 - 2.87/90)^2 = 0.94$ . From equation 5.1, and  $\alpha = 2.87^\circ$  calculating the ultimate resistance of the block foundation from its base resistance only (ignoring skin friction around the periphery of the group), in [Figure 5.6](#),  $L/B = 13.0/5.54 = 2.34$ , therefore shape factor  $s = 1.1$ . Depth factor  $D/B = 7.7/5.54 = 1.4$ , therefore  $N_c = 7.5$

The base resistance is then given by

$$Q_b = i_c c_b s N_c B L = 0.94 \times 190 \times 1.1 \times 7.5 \times 5.54 \times 13 = 106\,120\text{kN}.$$

The safety factor for the vertical load  $= 106\,120/30\,000 = 3.5$  (which is satisfactory).

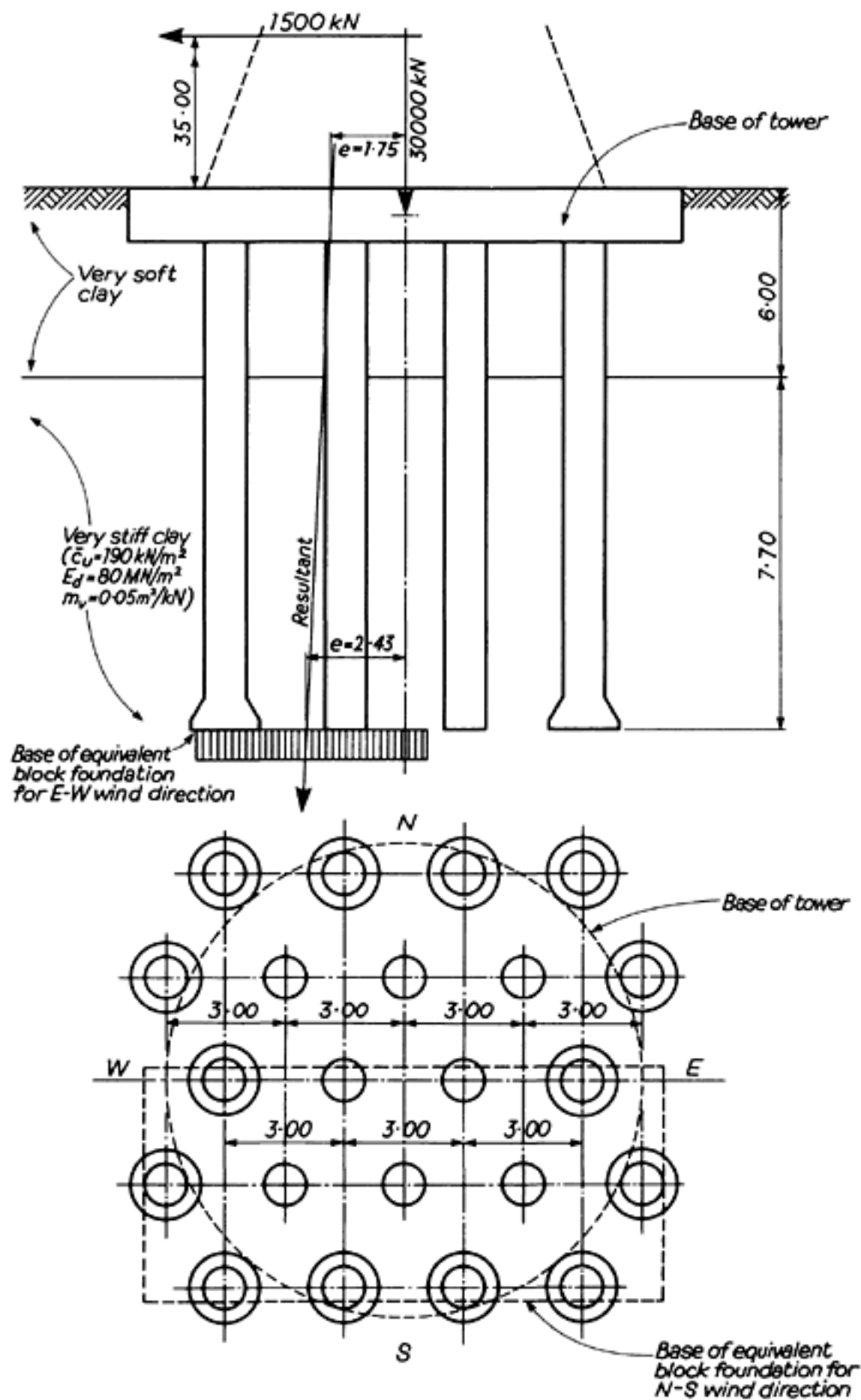


Fig. 6.65

It is also necessary to confirm that the total settlements and tilting of the structure are within safe limits. Calculating first the immediate and consolidation settlements under dead and imposed load, but excluding the wind load. Because the piles have under-reamed bases which carry the major proportion of the load, the base of the equivalent raft will be close to pile base level. The approximate overall dimensions of the equivalent raft outside the toes of the pile bases are  $13.8 \times 11.2$  m. Therefore

$$\text{overall base pressure beneath raft} = \frac{30000}{13.8 \times 11.2} = 194 \text{ kN/m}^2.$$

Assume a value of  $E$  for the glacial clay of  $80 \text{ MN/m}^2$  and a value of  $m_v$  of  $0.05 \text{ m}^2/\text{kN}$ . From Figure 5.10 for

$L/B = 13.8/11.2 = 1.2$ ,  $H/B = \infty$  and  $D/B = 7.7/11.2 = 0.69$ , (ignoring the soft clay), and  $\mu_0 = 0.92$ . Therefore

$$\text{immediate settlement} = \frac{0.75 \times 0.92 \times 194 \times 11.2 \times 1000}{80 \times 1000} = 19 \text{ mm}.$$

From [Figure 5.17](#) the average vertical stress at the centre of a layer of thickness  $2B$  is  $0.3 \times 194 = 58 \text{ kN/m}^2$

The depth factor for  $\frac{D}{\sqrt{LB}} = 0.62$  is 0.81 and the geological factor is 0.5. Therefore, from equations 5.5 and 5.6,

$$\rho_c = \frac{0.5 \times 0.62 \times 0.05 \times 58 \times 2 \times 11.2 \times 1,000}{1,000} = 20\text{mm}$$

Part of the imposed loading will not be sustained and will not contribute to the long-term settlement. Thus the total settlement under the vertical load of 30000kN will probably not exceed 30mm. It is necessary to estimate the amount of tilting which would occur under sustained wind pressure, *i.e.* the immediate settlement induced by the horizontal  $\frac{30,000}{13 \times 5.54} = 416\text{kN/m}^2$  wind force of 1500kN producing a pressure under the combined vertical and horizontal loading of on the equivalent raft caused by the eccentric loading. For  $L/B = 13/5.54 = 2.3$ ,  $H/B = \infty$  and  $D/B = 7.7/5.54 = 1.4$ ,  $\mu_1 = 1.0$  and  $\mu_0 = 0.9$  giving

$$\text{immediate settlement} = \frac{1.0 \times 0.9 \times 416 \times 5.54 \times 1,000}{80 \times 1,000} = 26\text{mm}.$$

Of this amount 16mm is due to vertical loading only, giving a tilt of 10mm due to wind loading. A movement of this order would have a negligible effect on the stability of the tower.

The horizontal force on each pile if no wind load is carried by the pile cap is  $1,500/22 = 68\text{kN}$ . A pile 1 metre in diameter can carry this load without excessive deflection (see example 6.4).

### Example 6.9

Pressuremeter tests made at intervals of depth in a highly weathered weak broken siltstone gave the following parameters

Pressuremeter modulus	$= E_m = 30\text{MN/m}^2$
Limit pressure	$= p_l = 1.8\text{MN/m}^2$
Creep pressure	$= p_f = 0.8\text{MN/m}^2$

The above values were reasonably constant with depth. Draw the deflection curve for a horizontal load applied to the head of a 750mm pile at the ground surface up to the ultimate load, and obtain the deflection for a horizontal load of half the ultimate.

$$\text{Moment of inertia of uncracked pile} = \frac{\pi \times 0.75^4}{64} = 0.0155\text{m}^4$$

Modulus of elasticity of pile  $= 26 \times 10^3\text{MN/m}^2$   
Take a rheological factor of 0.8

$$\text{From } \frac{1}{k_m} = \frac{2 \times 0.6}{9 \times 30} \left( \frac{0.75}{0.6} \times 2.65 \right)^{0.8} + \frac{0.8 \times 0.75}{6 \times 30}$$

$$k_m = 67\text{MN/m}^2$$

Over elastic range from  $p = 0$  to  $p = p_f$

$$\text{Stiffness factor} = R = \sqrt[4]{\frac{26 \times 10^3 \times 0.0155}{67 \times 0.75}} = 1.68\text{m}$$

To allow for surface heave assume no soil reaction from ground surface to assumed surface at  $0.5 \times 0.75 = 0.375\text{m}$

$$= \frac{0.8 \times 0.75}{67} = 0.0090\text{m}$$

At creep pressure of  $0.8\text{MN/m}^2$ , corresponding deflection

From equation 6.84 corresponding lateral load applied at assumed ground surface

$$= H = \frac{0.0090 \times 1.68 \times 67 \times 0.75}{2} = 0.380 \text{ MN}$$

From equation 6.85, slope at assumed ground surface

$$= - \frac{2 \times 0.380}{1.68^2 \times 67 \times 0.75} = - 0.0054 \text{ radians}$$

Deflection at real ground surface

$$\begin{aligned}
 &= 0.009 - \frac{0.380 \times 0.375^2}{6 \times 26 \times 10^3 \times 0.0155} - (-0.0054 \times 0.375) \\
 &= 0.0110\text{m} \\
 &= 11\text{mm}
 \end{aligned}$$

Between  $p=p_f$  and  $p=p_i$  the upper curve in Figure 6.49b gives  $k_m = \frac{67}{2} = 33.5\text{MN/m}^2$  and

$$R = \sqrt[4]{\frac{26 \times 10^3 \times 0.0155}{33.5 \times 0.75}} = 2.00\text{m}$$

From upper curve in Figure 6.49b

$$\begin{aligned}
 \text{At limit pressure of } 1.8\text{MN/m}^2 \text{ corresponding deflection} &= \frac{1.8 \times 0.75}{33.5} = 0.0403\text{m} \\
 \text{Corresponding lateral load at assumed ground surface} &= \frac{0.0403 \times 2.00 \times 33.5 \times 0.75}{2} = 1.012\text{MN}
 \end{aligned}$$

$$\text{Slope at assumed ground surface} = -\frac{2 \times 1.012}{2^2 \times 33.5 \times 0.75} = -0.0201 \text{ radians}$$

Total deflection at real ground surface

$$\begin{aligned}
 &= 0.0110 + 0.0403 - \frac{1.012 \times 0.375^2}{6 \times 26 \times 10^3 \times 0.0155} - (-0.0201 \times 0.375) \\
 &= 0.0588 \\
 &= 59\text{mm}
 \end{aligned}$$

The load/deflection curve is shown in Figure 6.66. The deflection corresponding to an applied load of half the ultimate load of 1012kN is 20mm.

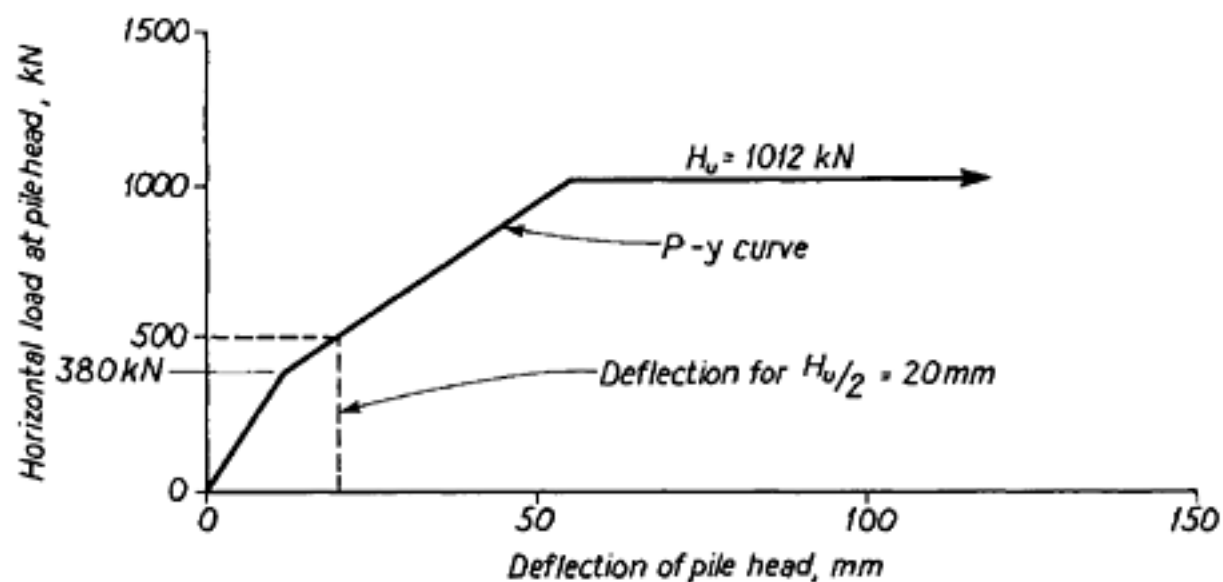


Fig. 6.66

## CHAPTER 7

# The structural design of piles and pile groups

### 7.1 General design requirements

Piles must be designed to withstand stresses caused during their installation, and subsequently when they function as supporting members in a foundation structure. Stresses due to installation occur only in the case of piles driven as preformed elements. Such piles must be capable of withstanding bending stresses when they are lifted from their fabrication bed and pitched in the piling frame. They are then subjected to compressive, and sometimes to tensile, stresses as they are being driven into the ground, and may also suffer bending stresses if they deviate from their true alignment. Piles of all types may be subjected to bending stresses caused by eccentric loading, either as a designed loading condition or as a result of the pile heads deviating from their intended positions. Differential settlement between adjacent piles or pile groups can induce bending moments near the pile heads as a result of distortion of the pile caps or connecting beams.

The working stresses adopted for piles should take into account the effects of unseen breakage caused during driving, possible imperfections in concrete cast in situ, and the long-term effects of corrosion or biological decay.

Pile caps, capping beams and ground beams are designed to transfer loading from the superstructure to the heads of the piles, and to withstand pressures from the soil beneath and on the sides of the capping members. These soil pressure can be caused by settlement of the piles, by swelling of the soil, and by the passive pressures resulting from lateral loads transmitted to the pile caps from the superstructure.

Eurocode No. 7, [Chapter 7](#), requires design actions to comply with the relevant material Eurocodes for structural design.

### 7.2 Designing reinforced concrete piles for lifting after fabrication

The reinforcement of piles to withstand bending stresses caused by lifting has to be considered only in the case of precast reinforced (including prestressed) concrete piles. Bending takes place when the piles are lifted from their horizontal position on the casting bed for transportation to the stacking area. The most severe stresses thus occur at the time when the concrete is immature. Timber piles in commercially available lengths which have a cross-sectional area sufficiently large to withstand driving stresses will not be overstressed if they are lifted at the normal picking-up points. Splitting could occur if attempts were to be made to lift very long piles fabricated by splicing together lengths of timber, but there is no difficulty in designing spliced joints so that the units can be assembled and bolted together while the pile is standing vertically in the leaders of the piling frame. Again, steel piles with a cross-sectional area capable of withstanding driving stresses and of sufficient thickness to allow for corrosion losses will not fracture when lifted in long lengths from the horizontal position in the fabrication yard. Reinforced concrete and prestressed concrete piles have a comparatively low resistance to bending, and the stresses caused during lifting may govern the amount of longitudinal reinforcing steel needed. The static bending moments induced by lifting and pitching piles at various points on their length are shown in [Table 7.1](#)

The design charts in Figures [7.2a](#) to [7.2d](#) show the bending moments due to self-weight which are

Table 7.1 Bending moments induced by lifting and pitching piles

Condition	Maximum static bending moment
Lifting by two points at $L/5$ from each end	$WL/40$ ( <a href="#">Figure 7.1a</a> )
Lifting by two points at $L/4$ from each end	$WL/32$ ( <a href="#">Figure 7.1b</a> )
Pitching by one point $3L/10$ from head	$WL/22$ ( <a href="#">Figure 7.1c</a> )
Pitching by one point $L/3$ from head	$WL/18$ ( <a href="#">Figure 7.1d</a> )
Pitching by one point $L/4$ from head	$WL/18$ ( <a href="#">Figure 7.1e</a> )
Pitching by one point $L/5$ from head	$WL/14$ ( <a href="#">Figure 7.1f</a> )
Pitching from head	$WL/8$ ( <a href="#">Figure 7.1g</a> )
Lifting from centre	$WL/8$ ( <a href="#">Figure 7.1h</a> )

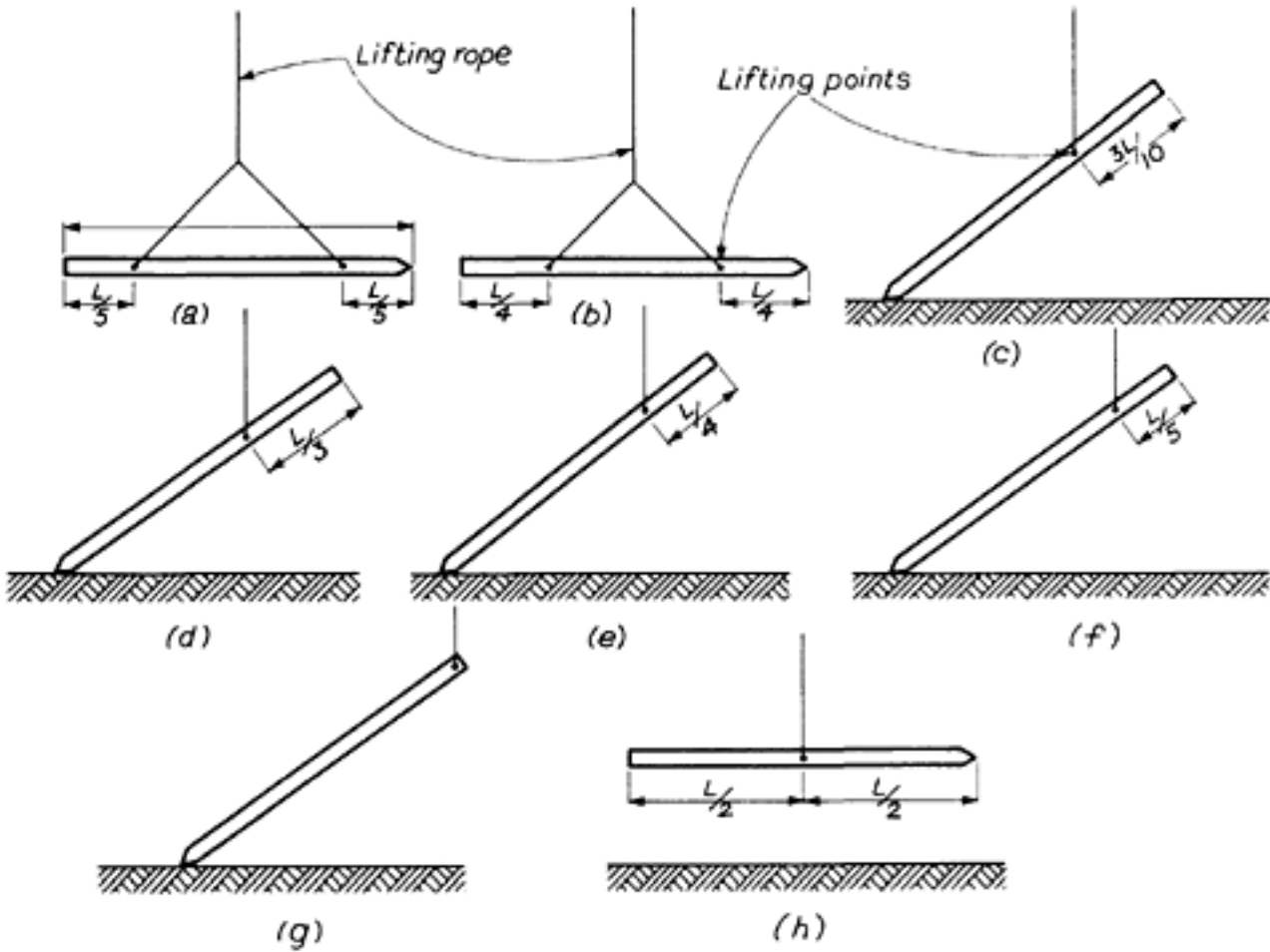


Fig. 7.1 Methods of lifting reinforced concrete piles

induced when square piles of various cross-sectional dimensions are lifted from the head or centre (pick-up point A as in Figures [7.1g](#) and [7.1h](#)), from a point one-third of the length from the head (pick-up point B as in [Figure 7.1d](#)), and from points one-fifth of the length from the head and toe (pick-up point C as [Figure 7.1a](#)).

Also shown on the charts are horizontal lines representing the ultimate moment of resistance of each pile section using four main reinforcing bars of various sizes. Concrete with a characteristic strength of  $40\text{N/mm}^2$  has been used with a characteristic strength for the reinforcement of  $250\text{N/mm}^2$ , and 40mm cover to the link steel.

It is desirable to limit the steel stresses to  $250\text{N/mm}^2$  when determining ultimate resistance moments. This will ensure that a 4 bar reinforcement arrangement complies with Table 3.30 of BS8110: Part 1 regarding maximum spacing requirements, *i.e.*, 300mm assuming no redistribution. If a characteristic strength of  $460\text{N/mm}^2$  were to be used, the maximum allowable bar spacing becomes 160mm, thus requiring an 8 bar arrangement.

However, the situation can arise where a moment is induced in a pile due to the pile being driven just within the permitted tolerance of 1 in 75 to the vertical. In such a case the pile can be checked as a column with an axial load and moment, by use of the design charts in BS 8110: Part 3. These charts relate only to high yield steel with a characteristic strength of  $460\text{N/mm}^2$ . It is therefore advisable to use high yield steel for pile reinforcement, but to limit the characteristic strength to  $250\text{N/mm}^2$  when designing for lifting from the casting bed.

The charts can be used for a concrete having a higher characteristic strength but the lifting lengths will be somewhat conservative. The concrete strength is usually governed by the strength required to resist driving stresses (see [2.2.2](#)), or to give durability in an aggressive environment (see [10.3](#) and Tables [3.2](#) and [3.4](#) of BS 8110: Part 1).

The length and cross-sectional dimensions of the pile are obtained from considerations of the resistance of the soil or rock to

axial or lateral loading (Chapters [4](#) and [6](#)). Then for the given length and cross-section the pick-up point is selected, having regard to the type of piling plant and craneage to be employed,

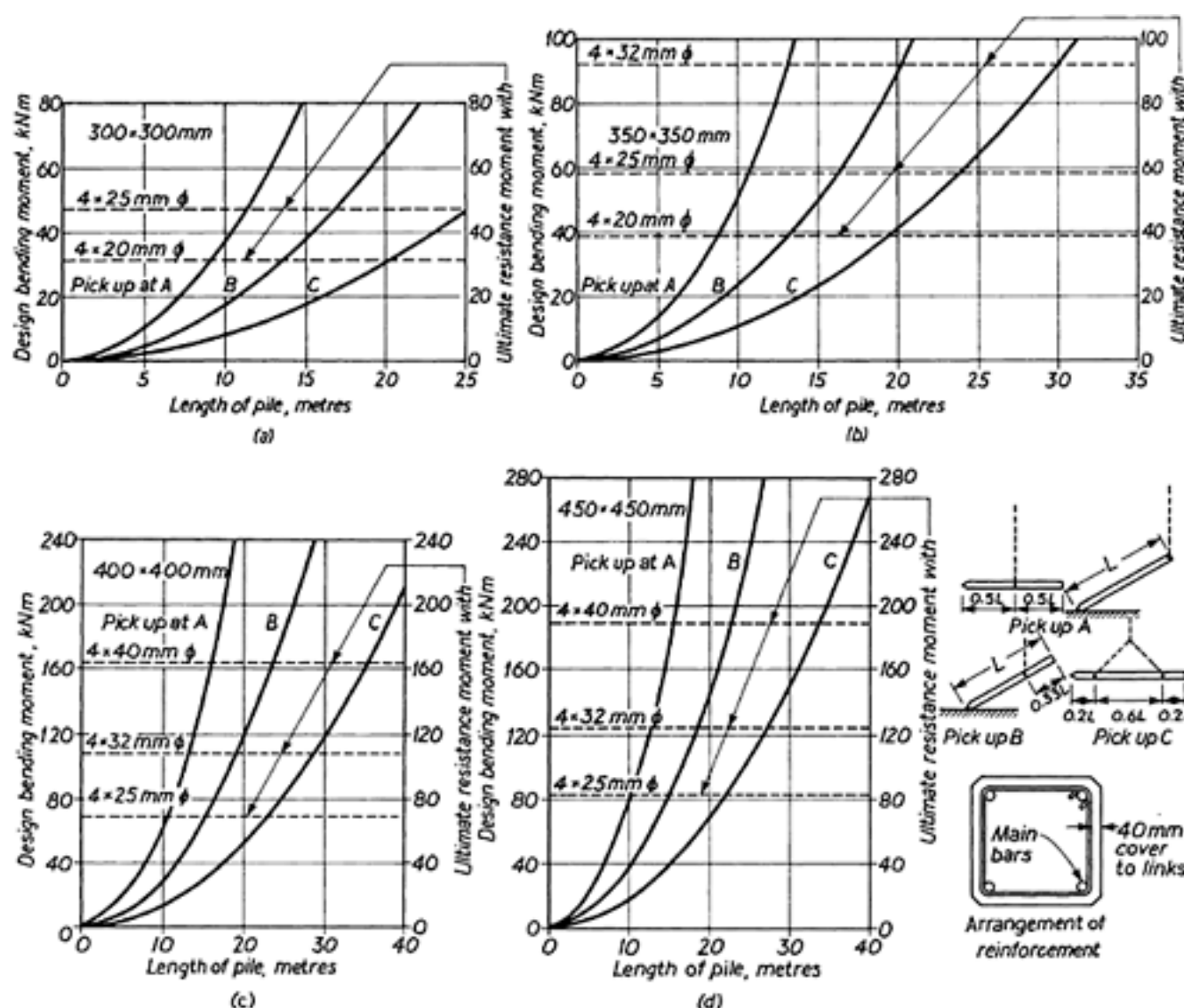


Fig. 7.2 Diagrams showing required lifting points for reinforced concrete piles of various cross-sections (see also opposite page)

and the economies which may be achieved by lifting the piles from points other than at the ends or the centre. The bending moment due to the *ultimate* dead load corresponding to the selected pick-up point is then read off from the appropriate curve. Next the ultimate dead load bending moment is compared with the ultimate moments of resistance for various sizes of reinforcement as shown on the charts and the appropriate reinforcement is selected. A partial safety factor of 1.4 on the bending moment due to dead load is suitable for normal designs, and this value has been used in the derivation of [Figure 7.2](#) and [Table 7.2](#).

It is evident that there is a maximum lifting length for a pile of given cross-sectional dimensions and size of main reinforcement. The design curves shown in [Figure 7.2](#) have been used to obtain the maximum lifting lengths for the three pick-up conditions for square piles in [Table 7.2](#). Also shown in these tables are the requirements for the link steel over a length equal to three times the pile width at the head and toe, and in the body of the pile, in accordance with BS 8004.

The reinforcement stress and bar spacing previously referred to ensure that, from considerations of serviceability, a surface crack width of 0.3mm is not exceeded. This is permissible for concrete in contact with a non-aggressive soil. In severely aggressive conditions, such as exposure to sea water, the surface crack width should not exceed 0.004 times the nominal cover to the main reinforcement. Reference should be made to Clause 3.8.3 of BS 8110: Part 2.

The use of the charts in [Figure 7.2](#) and of [Table 7.2](#) is illustrated in example 7.1.

In the case of prestressed concrete piles a wide range of designs is possible by varying the numbers of prestressing wires or strands in a given cross-section, and by varying the prestressing conditions. Consequently, design charts have not been prepared for these pile types, but an example of a calculation in accordance with the former CP 110 and the Concrete Society data sheet CSG1 ([2.2](#)) is given in example 7.3.

Table 7.2 Maximum lengths of square section precast concrete piles for given reinforcement

Pile size (mm)	Main reinforcement	Maximum length in metres for pick up at			Transverse reinforcement	
		Head and toe	0.33×length from head	0.2×length from head and toe	Head and toe	Body of pile
300×300	4×20mm	9.0	13.5	20.5	6mm at	6mm at
	4×25mm	11.0	16.5	25.0	40mm crs	130mm crs
350×350	4×20mm	8.5	13.0	19.5	8mm at	8mm at
	4×25mm	10.5	16.0	24.0	70mm crs	175mm crs
	4×32mm	13.0	20.0	30.0		
400×400	4×25mm	10.0	15.0	22.5	8mm at	8mm at
	4×32mm	12.5	19.0	28.0	60mm crs	200mm crs
	4×40mm	15.5	23.0	34.5	or 10mm at 100mm crs	
450×450	4×25mm*	9.5	14.5	22.0	8mm at	8mm at
	4×32mm*	12.0	18.0	27.0	60mm crs	180mm crs
	4×40mm*	15.0	22.5	33.5	or 10mm at 90mm crs	or 10mm at 225mm crs

Notes: Piles designed in accordance with BS 8110 and BS 8004

Characteristic strength of reinforcement limited to 250N/mm<sup>2</sup>

Cover to link steel=40mm

Characteristic strength of concrete=40N/mm<sup>2</sup>

\* Alternatively use a larger number of smaller diameter bars.

### 7.3 Designing piles to resisting driving stresses

It is necessary to check the adequacy of the designed strength of a pile to resist the stresses caused by the impact of the piling hammer. Much useful data to aid the estimation of driving stresses came from the research of Glanville, Grime and Davies (7.1). They embedded stress recorders in piles to measure the magnitude and velocity of the stress wave induced in the pile by blows from the hammer. The tests showed that the stress wave travels from the head to the toe of the pile and is partly reflected from there to return to the head. If the pile is driven onto a hard rock, the sharp reflection of the wave at the toe can cause a compressive stress at the point which is twice that at the head, but when long piles are driven into soil of low resistance, the tensile stress wave is reflected, causing tension to develop in the pile. It can be shown from simple impact theory that the *magnitude* of the stress wave depends mainly on the height of the drop. This is true for a perfectly elastic pile rebounding from an elastic material at the toe. In practice there is plastic yielding of the soil beneath the toe, and the pile penetrates the soil by the amount described as the ‘permanent set’. The weight of the hammer is then important in governing the length of the stress wave and hence the efficiency of the blow in maintaining the downward movement of the pile. The simplest approach to ensuring that driving stresses are within safe limits is to adopt working stresses under static loading such that heavy driving is not required to achieve the depth of penetration required for the calculated ultimate bearing capacity. For this reason many codes of practice limit the working stresses or limit the load which can be carried by a pile of a given cross-section. Some of these code requirements are listed in Tables 2.3, 2.6, and 2.9. However, the practice of calculating driving stresses by methods based on the stress-wave theory is being adopted to an increasing extent. The usual practice is to assume that the dynamic resistance of pile to its penetration into the soil is equal to its ultimate static load-bearing capacity, and then to calculate the ‘permanent set’ in terms of blows per unit penetration distance to develop this resistance, using a hammer of given rated energy or weight and height of drop. The driving stress is assumed to be the ultimate driving resistance divided by the cross-sectional area of the pile, and this must not exceed the safe working stress on the pile material. As already stated in 1.4, the dynamic resistance is not necessarily equal to the static load-bearing capacity. However, if soil mechanics calculations as described in Chapter 4 have been made to determine the required size and penetration depth necessary to develop the ultimate bearing capacity, then either a simple dynamic pile driving formula or, preferably, stress-wave theories can be used to check that a hammer of a given weight and drop (or rated energy) will not overstress the pile in driving it to the required penetration depth. If at any stage of penetration the stresses are excessive a heavier hammer must be used, but if greater hammer weights and lesser drops still cause overstressing then other

measures, such as drilling below the pile toe or using an insert pile having a smaller diameter, must be adopted.

It is important to note that in many instances the soil resistance to driving will be higher than the value of ultimate bearing capacity as calculated for the purpose of determining the allowable pile load. This is because calculations for ultimate bearing capacity are normally based on average soil parameters. Where soil strength data are scanty it is necessary to assume conservative parameters. However when considering resistance to driving the possible presence of soil layers stronger than the assumed average must be taken into account. Hence, when assessing driving stresses it is advisable to make a separate calculation of ultimate bearing capacity based on soil strength values higher than the average. Also, in cases where negative skin friction is added to the working load, the soil strata within which the downdrag is developed will provide resistance to driving at the installation stage.

The simple piling-driving formulae such as the Hiley formula (see [Section 11.3.1](#)) have useful applications as a means of site control but they are not helpful in predicting driving stresses because of the uncertainty of values to be adopted for set and temporary compression which have an important influence on the calculated driving resistance. This aspect will be discussed further in [Section 11.3.1](#).

A widely-adopted method of using stress-wave theory to calculate driving stresses involves the wave equation developed by Smith<sup>(7.2)</sup>. This method divides the length of the pile into a number of elements. Each element is represented by a weight joined to the adjacent element by a spring, and the hammer, the helmet, and the packing are also represented by separate elements joined to each other and to the pile by springs. The toe resistance of the ground is assumed to act as a resisting force to the downward motion of the pile when struck by the hammer. In the case of friction piles the soil at the sides acts as a damping force to the stress wave. For each blow of the hammer, and each element in the hammer-pile system, calculations are made to determine the displacement of the element, the spring compression of the element, the force exerted by the spring, the accelerating force and the velocity of the element in a given interval of time. This time interval is selected in relation to the velocity of the stress wave and a computer is used to calculate the successive action of the weights and springs as the stress wave progresses from the head to the toe of the pile. The output of the computer is the compressive or tensile force in the pile at any required point between the head and toe.

The input to the computer comprises the following data.

1. Length of pile.
2. Material of pile.
3. Weight per unit length of pile.
4. Weight and fall of hammer (or rated energy per hammer blow).
5. Efficiency of hammer.
6. Weight of helmet, packing, and any dolly or follower used.
7. Elastic modulus of packing, and of any dolly used.
8. Elastic modulus of pile.
9. Coefficient of restitution of packing (and dolly if used).
10. Elastic compression (quake) of soil.
11. Damping properties of soil
12. Required ultimate driving resistance.

Of the above input data, items 1, 2, 3, 4, 6 and 11 are factual. The efficiency of the hammer is obtained from the manufacturer's rating, but this can decrease as the working parts become worn. The elastic modulus and coefficient of restitution of the packing may also change from the commencement to the end of driving. The elastic compression of the ground is usually taken as the elastic modulus under static loading, and this again will change as the soil is compacted or is displaced by the pile. Thus the wave equation can never give exact values throughout all stages of driving, and its continuing usefulness depends on amassing data on correlations between calculated stress values and observations of driving stresses in instrumented piles. Further refinements of the calculation procedure may also be made to allow for the changing dynamic characteristics of the hammer-pile-soil system during driving. Smith<sup>(7.2)</sup> states that the commonly accepted values for the soil compression (quake), and the damping constants for the toe and sides of the pile are not particularly 'sensitive' in the calculations, *i.e.* small changes in these values produce a smaller change in the final calculated results.

The basic Smith idealization represents a pile being driven by a drop hammer or a single-acting hammer. Diesel hammers have to be considered in a different manner because the energy transmitted to the pile varies with the resistance of the pile as it is being driven down. At low resistances there are low energies per blow at a high rate of striking. As the pile resistance increases the energy per blow increases and the striking rate decreases. Manufacturers of diesel hammers provide charts of energy versus rate of striking. When predictions are being made of the ability of a particular diesel hammer to drive a

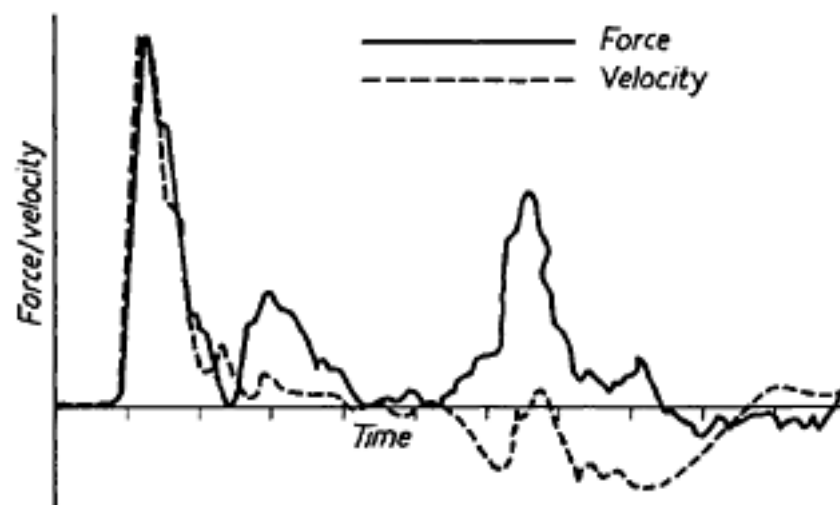


Fig. 7.3 Force and velocity vs. time for hammer blow on pile

pile to a given resistance consideration should be given to the range of energy over which the hammer may operate. Goble and Rausche<sup>(7.3)</sup> have published details of a computer programme which models diesel hammer behaviour realistically. The programme proceeds by iterations until compatibility is obtained between the pile/soil system and the energy/blows per minute performance of the hammer.

Pile driving resistance can be computed from field measurements of acceleration and strain at the time of driving<sup>(7.4)</sup>. Pairs of accelerometers and strain transducers are mounted near the pile head and the output of these instruments is processed to give plots of force and velocity versus time for selected hammer blows as shown in [Figure 7.3](#). The second stage of the method is to run a wave equation analysis with the pile only modelled from the instrument location downwards. Values of soil resistance, quake and damping are assigned and the measured time varying velocity is applied as the boundary condition at the top of the pile model. The analysis generates a force versus time plot for the instrument location and this is compared with the measured force versus time plot. Adjustments are made to the values of resistance, quake and damping until an acceptable agreement is reached between computed and measured values. At this stage the total soil resistance assigned in the analysis is taken as the resistance at the time of driving. The latter is a reliable assessment of the static resistance in soils and rocks where time effects are negligible. Examples of the use of the equipment and the data processing to predict static pile resistance and driving stresses can be found in the proceedings of the specialist conferences on pile dynamics<sup>(7.5a)</sup>. The instrumentation described above including the field processing equipment which produces the force/time plot is a useful method of field control of pile driving in difficult ground conditions. The shape of the plot ([Figure 7.3](#)) can give an indication of a broken pile and a check can be made of the stresses in the pile induced by driving.

Too much reliance should not be placed on immediate readings of the output from the field processing unit. Rigorous analysis of the data by experienced engineers is required in conjunction with the appropriate computer programs. Wheeled<sup>(7.5)</sup> described experiences of a field trial competition in Holland when a number of firms specializing in dynamic pile testing were invited to predict the ultimate bearing capacity of four instrumented precast concrete piles driven through sands and silts to penetrations between 11.5 and 19m. A wide range of predicted capacities was obtained. In the case of one pile the range was 90 to 510kN compared with a failure load of 340kN obtained by static testing. Reliable estimates of ultimate bearing capacity may not be possible if the available hammer has insufficient energy to overcome the resistance mobilized by the soil against penetration of the pile. A downward movement of the toe of piles up to about 1m in diameter of 2.5mm or preferably more is required to mobilize sufficient soil resistance to obtain reliable results. A heavier hammer can be used to achieve the necessary penetration but this may involve a risk of overstressing the pile.

#### 7.4 The effects on bending of piles below ground level

Slender steel tubular piles and H-piles may deviate appreciably off line during driving. These effects were described in 2.2.4 where it was noted that, whereas the ill-effects of bending or buckling of tubular piles below ground level could be overcome by inserting a reinforcing cage and filling the pile with concrete, such a procedure could not be adopted with H-piles. Therefore, where long H-piles are to be driven in ground conditions giving rise to bending or buckling, a limiting value must be placed on their curvature.

It is not usual to take any special precautions against the deviation of reinforced concrete piles other

than to ensure that the joints between elements of jointed pile systems (see [2.2.3](#)) are capable of developing the same bending strength as the adjacent un jointed sections. Reinforced concrete piles without joints cannot in any case be driven to very long lengths in soil conditions which give rise to excessive curvature. It is, of course, possible to inspect hollow prestressed concrete piles internally and to adopt the necessary strengthening by placing in-situ concrete if they are buckled.

It is impossible to drive a pile with a sufficient control of the alignment that the pile is truly vertical (or at the intended rake) and that the head finishes exactly at the designed position. Tolerances specified in various codes of practice are given in 3.4.11. If the specified deviations are exceeded, to an extent detrimental to the performance of the piles under working conditions, the misaligned piles must be pulled out for redriving or additional piles driven. Calculations may show that minor excesses from the specified tolerances do not cause excessive bending stresses as a result of the eccentric loading. In the case of driven-and-cast-in-place or bored-and-cast-in-place piles it may be possible to provide extra reinforcement in the upper part of the pile to withstand these bending stresses. For this reason Fleming and Lane<sup>(7.6)</sup> recommend that checks on the positional accuracy of in-situ forms of piling should be made before the concrete is placed. The methods described in [Section 6.3.2](#) can be used to calculate the bending stresses caused by eccentric loading. The effect of the deviation is expressed as a bending moment  $Pe$ , where the load  $P$  deviates by a distance  $e$  from the vertical axis of the pile.

## 7.5 The design of axially-loaded piles as columns

Buckling of axially-loaded piles terminating at ground level in a pile cap or ground beam cannot occur if the piles are loaded to within the permissible working stresses on the pile material. Thus such piles need not be considered as long columns for the purpose of structural design. However, it is necessary to consider the column strength of piles projecting above the soil line, as in jetties or piled trestles.

BS 8004 recommends that the depth below ground surface to the point of contraflexure should be taken as 1m in firm ground and as much as one-half of the penetration depth but not necessarily more than 3m in a weak ground such as soft clay or silt. A stratum of liquid mud should be treated as if it were water. The column strength of the pile is then calculated as for a short column and a reduction factor is applied to the calculated ultimate load to allow for the slenderness of the column, where the slenderness is defined as the ratio of the effective length to the breadth or radius of gyration. The BS 8004 criteria are rather crude and applicable only to small-diameter piles. They are inapplicable to large and stiff piles, for which the methods described in 6.3.1 and 6.3.2 should be followed.

Effective lengths for timber piles when regarded as columns are defined by BS 5268 as follows.

Restrained at both ends in position and direction:	0.7L
Restrained at both ends in position and one end in direction:	0.85L
Restrained at both ends in position but not in direction:	1.0L
Restrained at one end in position and direction and at the other end in direction but not in position:	1.5L
Restrained at one end in position and direction and free at the other end:	2.0L

Where steel and reinforced concrete piles are considered as columns the effective lengths for various conditions of end restraint are given in BS 5950 and BS 8110 respectively.

A pile embedded in the soil can be regarded as properly restrained in position and direction at the point of virtual fixity in the soil. The restraint at the upper end depends on the design of the pile cap and the extent to which the pile cap is restrained against movement by its connection with adjacent pile caps or structures. Some typical cases of the restraint of piles are shown in Figures [7.4a](#) to [7.4e](#).

The appropriate codes of practice should be consulted to obtain the reduction factors on the working stresses to allow for the slenderness of the piles.

## 7.6 Lengthening piles

Precast (including prestressed) concrete piles can be lengthened by cutting away the concrete to expose the main reinforcement or by splicing bars for a distance of forty bar diameters. The reinforcement of the new length is then spliced to the projecting steel, formwork is set up, and the extension is concreted. It is usual to lengthen a prestressed concrete pile by this technique in ordinary reinforced concrete. The disadvantage of using the method is the time required for the new length to gain sufficient strength to allow further driving.

A rapid method of lengthening which can be used where the piles carry compressive loads or only

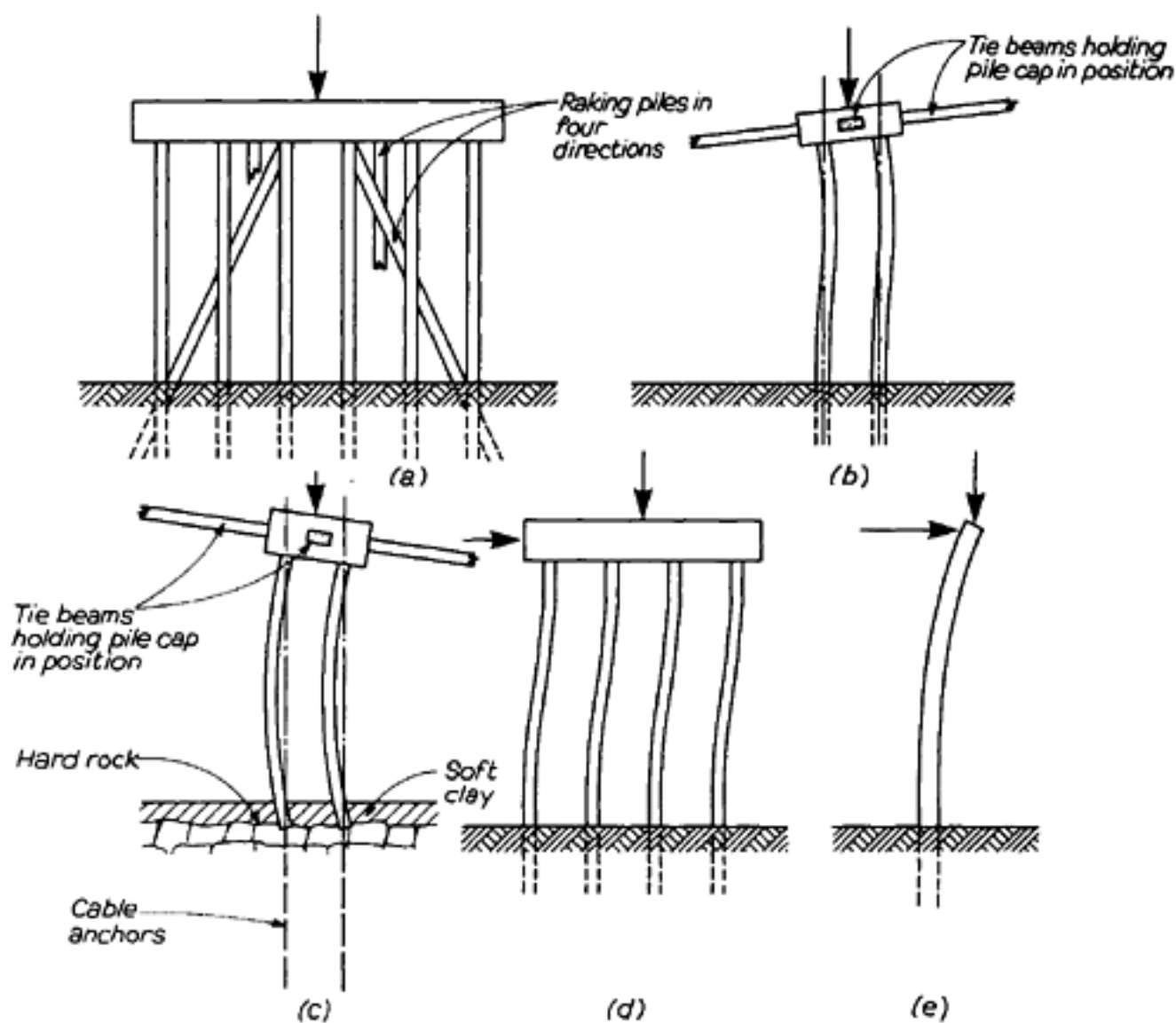


Fig. 7.4 Conditions of restraint for vertical piles

- (a) Restrained at top and bottom in position and direction
- (b) Restrained at bottom in position and direction; restrained at top in direction but not in position
- (c) Restrained at top and bottom in position but not in direction
- (d) Restrained at bottom in position and direction; restrained at top in direction but not in position
- (e) Restrained at bottom in position and direction; unrestrained at top in position or direction

small bending moments is to place a mild steel sleeve with a length of four times the pile width over the head of the pile to be extended. The sleeve is made from 10mm plates and incorporates a central diaphragm which is bedded down on a 10 to 15mm layer of earth dry sand-cement mortar trowelled onto the pile head. After setting the sleeve a similar layer of mortar is placed on the upper surface of the diaphragm and rammed down by a square timber. The extension pile with a square end is then dropped down into the sleeve and driving commences without waiting for the mortar to set. An epoxyresin-sand mortar can be used instead of sand-cement mortar. An epoxy-resin joint can take considerable tensile or bending forces, but the length of time over which the adhesion of the resin to the concrete is effective is indeterminate. The bond may be of rather short duration in warm damp conditions.

Another method of lengthening piles is to drill holes into the pile head. Then bars projecting from the extension piece are grouted into these holes using a cement grout or an epoxy-resin mortar.

Timber piles are lengthened by splicing as shown in Figures 2.3 and 2.4, and steel piles are butt-welded to lengthen them (Figures 7.5a and 7.5b). Backing plates or rings are provided to position the two parts of the pile while the butt weld is made but the backing plates for the H-piles (Figure 7.5a) may not be needed if both sides of the pile are accessible to the welder. The backing ring for the tubular pile shown in Figure 7.5b is deliberately made thin so that it can be 'sprung' against the inside face of the pile. When lengthening piles in marine structures, the position of the weld should be pre-determined so that, if possible, it will be situated below sea-bed level, and thus be less susceptible to corrosion than it would if located at a higher elevation.

The specification adopted for making welded splices in steel piles should take into account the conditions of loading and driving. For example, piles carrying only compressive loading and driven in easy to moderate conditions would not require a stringent specification with non-destructive testing for welding below the soil line. On the other hand, piles carrying substantial bending moments in marine structures would require a specification similar to that used for welding boilers or pressure vessels. Advice on specifications suitable for given conditions of loading and driving should be sought from the manufacturers of the piles.

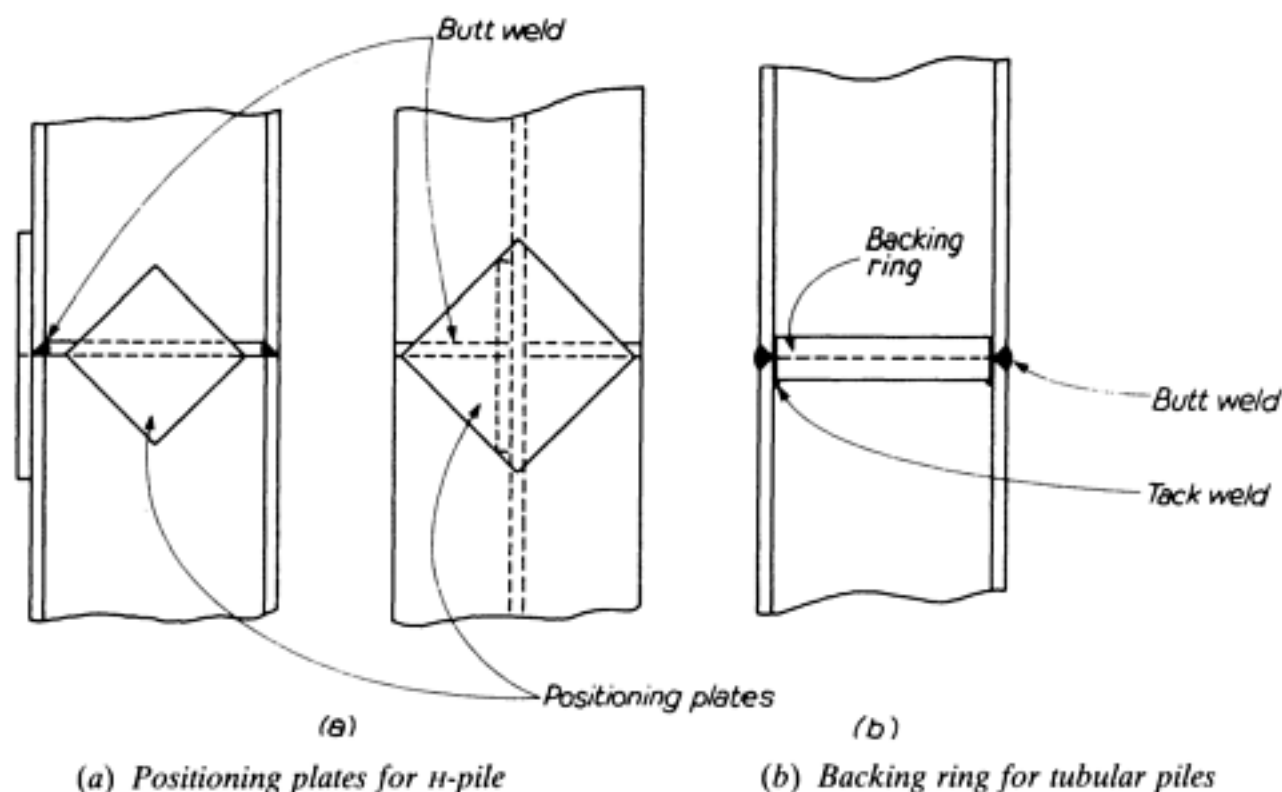


Fig. 7.5 Splicing steel piles

### 7.7 Bonding piles with caps and ground beams

Where simple compressive loads without bending or without alternate compressive and uplift loading are carried by precast or cast-in-situ concrete piles it is satisfactory to trim off the pile square so that the head without any projecting reinforcement is set some 75 to 100mm into the cap ([Figure 7.6a](#)). Some uplift (but not bending) can be carried if the sides of the pile are roughened over a distance of about 300mm and cast into the cap ([Figure 7.6b](#)). Where bending moments are to be transferred from the cap to the piles (or vice versa) the concrete must be cut away to expose the reinforcing steel or prestressing tendons, which are then bonded into the cap ([Figure 7.6c](#)). It is sometimes the practice to provide special mild steel splicing bars in the heads of pre-stressed concrete piles, which are exposed by cutting away the concrete after driving is complete ([Figure 2.6](#)). Alternatively, couplers can be set flush with the pile head to which further tendons or bars are attached for bonding into the cap. Splicing bars or couplers are satisfactory if the depth of penetration of the pile can be predicted accurately. If the upper part of the pile has to be cut away they no longer have any useful function but they can serve as a means of lengthening a pile should this be necessary.

Excess lengths of pile are cut off by cutting a groove around the pile at the desired level to expose the main steel which is then severed by sawing or burning. A tractor or winch is used to break off the surplus length of pile.

Steel box, tubular, or H-section piles carrying only compressive loads, can be terminated at about 100 to 150mm into the pile cap without requiring any special modifications to the pile to provide for bonding ([Figure 7.7a](#)). There must, however, be a sufficient thickness of concrete in the pile cap over the head of the pile to prevent failure in punching shear. Research by the Ohio Department of Highways([7.7](#))

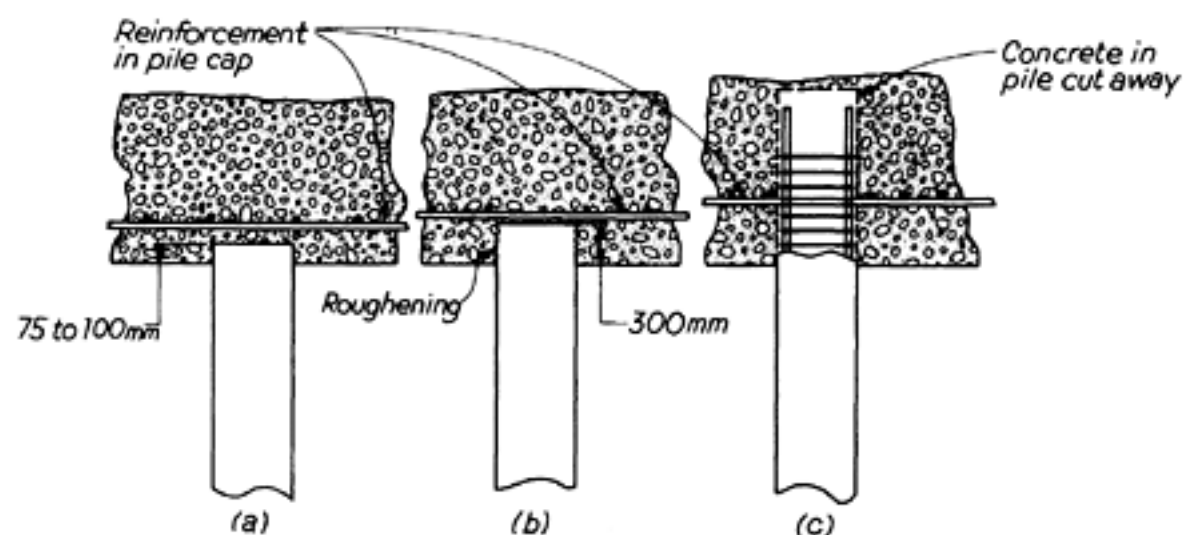


Fig. 7.6 Bonding reinforced concrete piles into pile caps

(a) Compressive loading only on piles

(b) Compressive loading alternating with light to moderate uplift loading on piles

(c) Bending moments or heavy uplift loads on piles

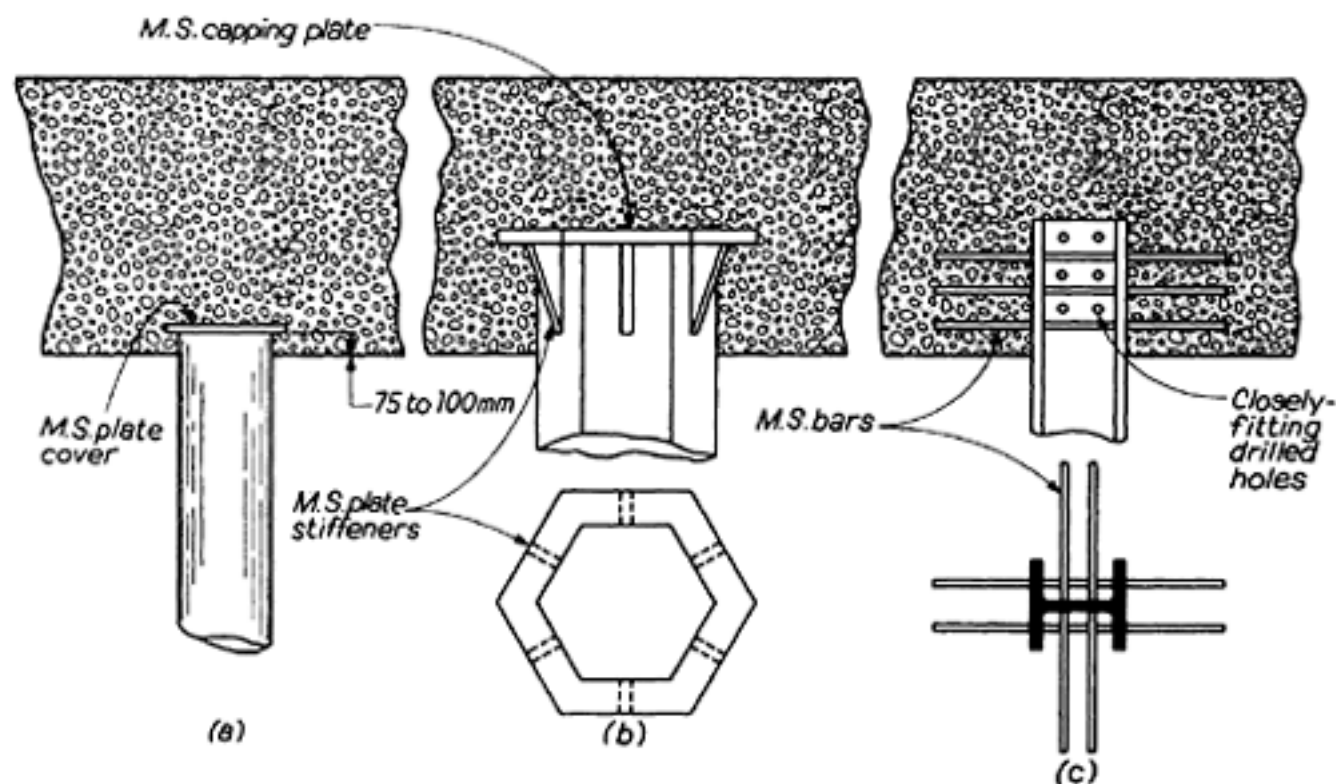


Fig. 7.7 Bonding steel piles into pile caps

- (a) Compressive loads only on steel tubular piles  
 (b) Hexagonal box pile carrying heavy compressive loads or uplift loads  
 (c) H-pile carrying uplift loading or bending moments

has shown that if the concrete forming the pile cap is of adequate thickness and if the reinforcement is correctly disposed to withstand shearing and bending forces there is no need to provide a bearing plate or other device for transferring load at the head of an H-pile. However where steel piles are carrying the maximum working load permitted by the material in cross-section, the thickness of concrete in the pile cap to resist punching shear may be uneconomically large. In such cases the head of the pile should be enlarged by welding on a capping plate (Figure 7.7b) or by threading steel bars through close-fitting holes drilled in the pile (Figure 7.7c). The capping arrangements shown in the latter two figures can be used to bond the pile to the cap when uplift loads or bending moments are carried by the pile, or alternatively bonding bars can be welded to the pile. Load transfer from large diameter tubular piles to pile caps can be achieved by welding rectangular plates around the periphery of the pile at its head.

## 7.8 The design of pile caps

A pile cap has the function of spreading the load from a compression or tension member onto a group of piles so that, as far as possible, the load is shared equally between the piles. The pile cap also accommodates deviations from the intended positions of piles, and by rigidly connecting all the piles in one group by a massive block of concrete, the ill-effects of one or more defective piles are overcome by redistributing the loads. The minimum number of small diameter piles which is permitted in an *isolated* pile cap is three. Caps for single piles must be interconnected by ground beams in two directions, and for twin piles by ground beams in a line transverse to the common axis of the pair.

A single large-diameter pile carrying a column does not necessarily require a cap. Any weak concrete or laitance at the pile head can be cut away and the projecting reinforcing bars bonded to the starter bars of the column reinforcement. Where a steel column is carried by a single large-diameter pile, the concrete is cut down and roughened to key to the pedestal beneath the column base. The heads of large-diameter piles are cast into the ground floor or basement floor concrete in order to distribute the horizontal wind forces on the superstructure to all the supporting piles.

To facilitate construction, ground beams should be arranged, where possible, to pass across the tops of the pile caps and not to frame into the sides of the caps, the connection between the cap and the ground beam being provided by column starter bars and by the friction and bond between cap and beam. The concrete forming the caps may then be placed in one operation and without the inconvenience and potential weakness that result from the formation of pockets to receive the ground beams. If the beams must frame into the cap sides, an alternative to providing pockets is to place the concrete in the caps in two operations, a horizontal construction joint being formed in each cap at the level of the underside of the ground beams.

Site setting out is also simplified by locating the ground beams on top of the pile caps. The caps (and column starter bars, if required) can be constructed with reference to the minimum number of

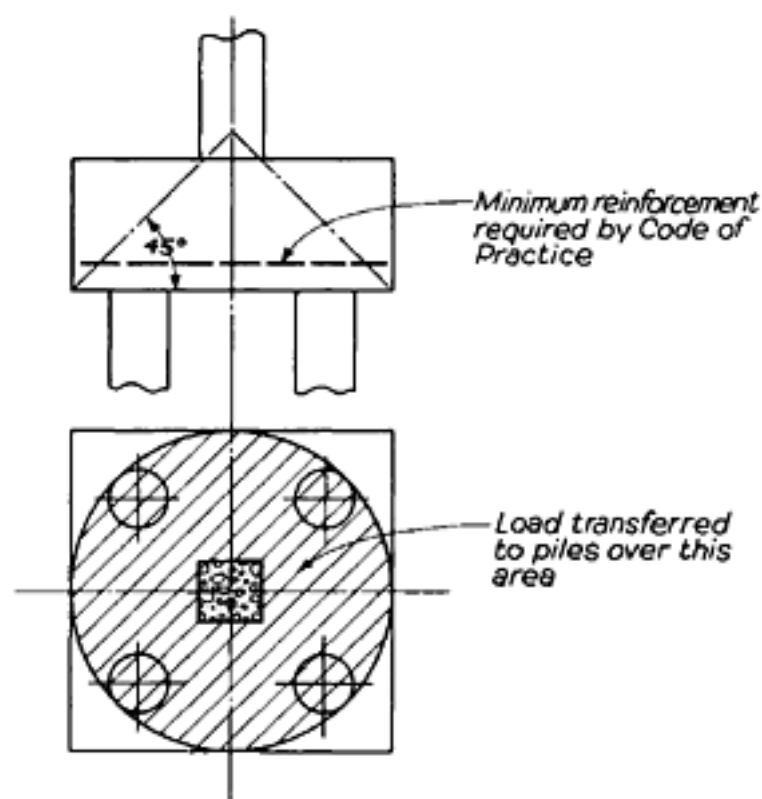


Fig. 7.8 Load transfer from column to deep four-pile cap

Table 7.3 Minimum reinforcement in solid slabs

Code	Mild steel	High-yield steel
BS 8110: Part 1: Table 3.2.7	0.24% of gross cross-section	0.13% of gross cross-section
CP 114 (Clause 3.11)	0.15% of gross cross-section	0.12% of gross cross-section

drawings, and fixed points on the site are then available for setting out the formwork for the ground beams. Provision often has to be made for services to pass through a foundation. If the ground beams are all situated on top of the pile caps, the routes of the services are not obstructed by any pile caps, since the services may pass over the cap through holes or sleeves left in the ground beams. The apparent economy in materials and excavation gained by framing ground beams into the sides of pile caps can easily be lost by the inconvenience it causes to other operations.

A deep cap is suitable for four piles, as shown in [Figure 7.8](#). By adopting this arrangement the column load is transferred directly into the pile heads in compression. The bending and shearing forces are negligible, requiring only the minimum proportion of steel in two directions at the bottom of the cap as required by the relevant code of practice. The minimum proportions of main and secondary reinforcement for solid slabs required by British Standard codes of practice are given in [Table 7.3](#).

Where large numbers of piles support a heavily loaded column the cap is designed as a solid slab carrying concentrated loads from the piles, as shown in [Figure 7.9](#). The slab may be designed either by simple bending theory or by the truss analogy in which the axial thrust is taken by the concrete and the tensile forces by the reinforcing steel (BS 8110: Part 1: Clause 3.11.4). The required thickness of the slab is usually governed by considerations of the allowable shear stresses. Shear should be checked at the critical section, extending across the full width of the cap. The critical section for shear should be assumed to be located 20% of the diameter of the pile inside the face of the pile, as indicated in [Figure 7.10](#). The whole of the force from the piles with centres lying outside this line should be considered to be applied outside this line. A check should be made to ensure that the design shear stress calculated at the perimeter of the column does not exceed  $0.8\sqrt{f_{cu}} \text{ N/mm}^2$  or  $5 \text{ N/mm}^2$ , whichever is the lesser. In addition, if the pile spacing is greater than three diameters, punching shear should be checked in accordance with Clause 3.7.7 of BS 8110: Part 1.

The bending moments on the pile cap are assumed to act from the centre of the pile to the face of the nearest column or column stem ([Figure 7.10](#)). When calculating bending moments an allowance should be made for deviations in the positions of the pile heads, up to the specified maximum tolerance (see [3.4.11](#)).

Arrangements of reinforcement in pile caps of various shapes are shown in [Figure 7.11](#). In the case of caps for two piles ([Figure 7.11 a](#)) the proportion of steel provided in the transverse direction should

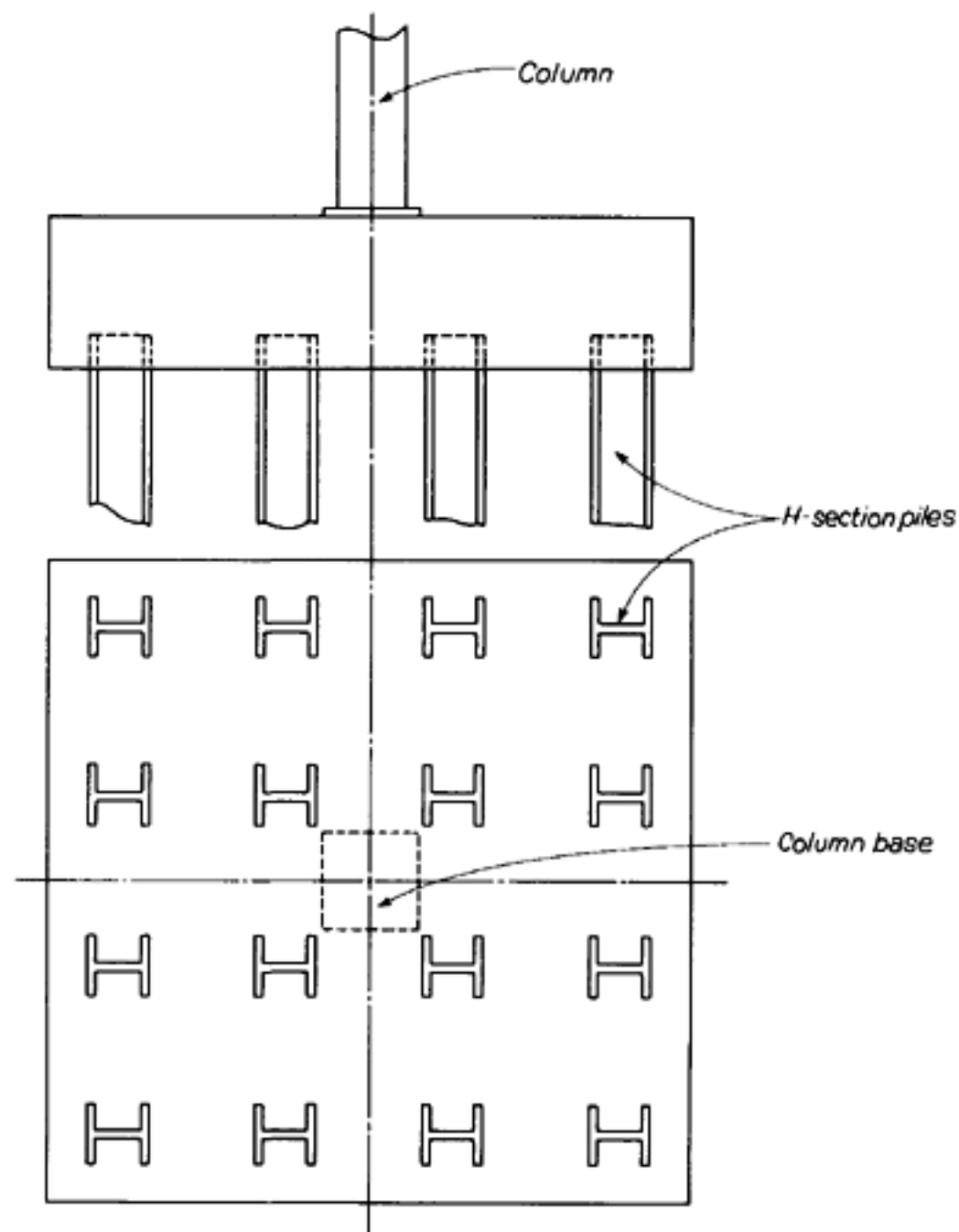


Fig. 7.9 Solid slab cap for 16-pile group

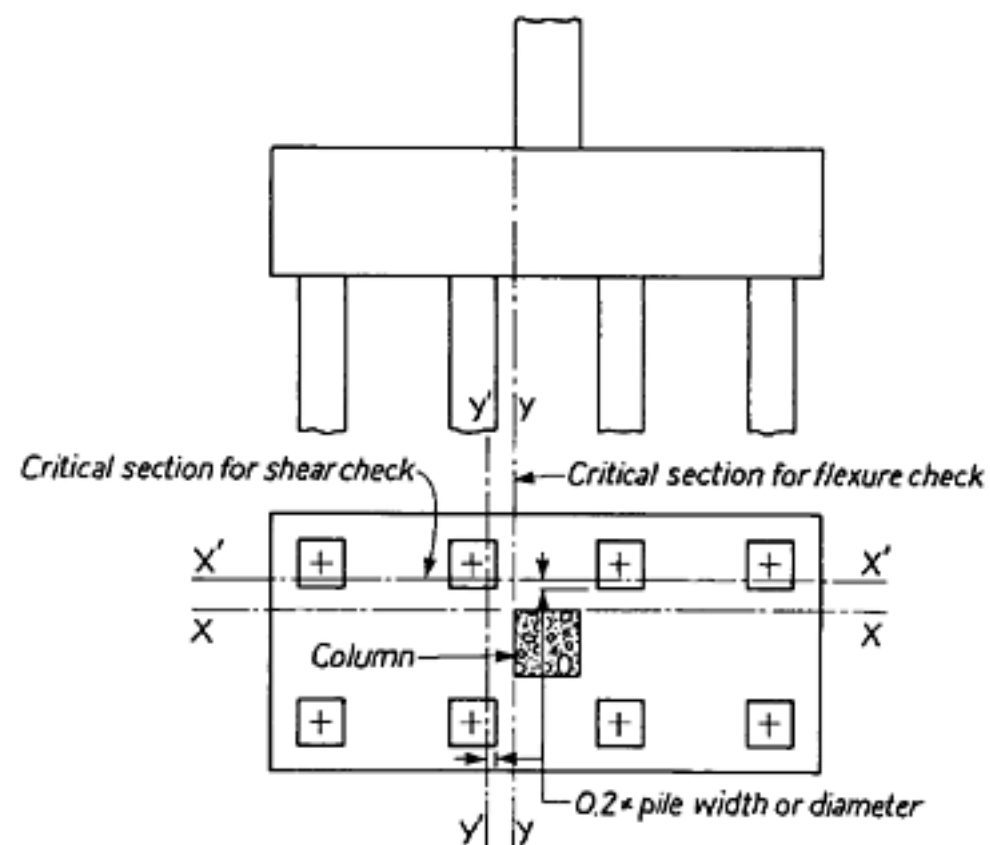


Fig. 7.10 Calculation of bending moments and shearing forces on rectangular pile cap

be the minimum required by the relevant code of practice. Reinforcement in square caps is generally spaced equally across the cap in both directions, except when the truss analogy is used with widely spaced piles (spacing exceeding three diameters), when only the reinforcement within 1.5 times the pile diameter from the centre of the pile should be considered to constitute a tension member of the truss. Thus in such cases reinforcement may need to be concentrated over the pile heads.

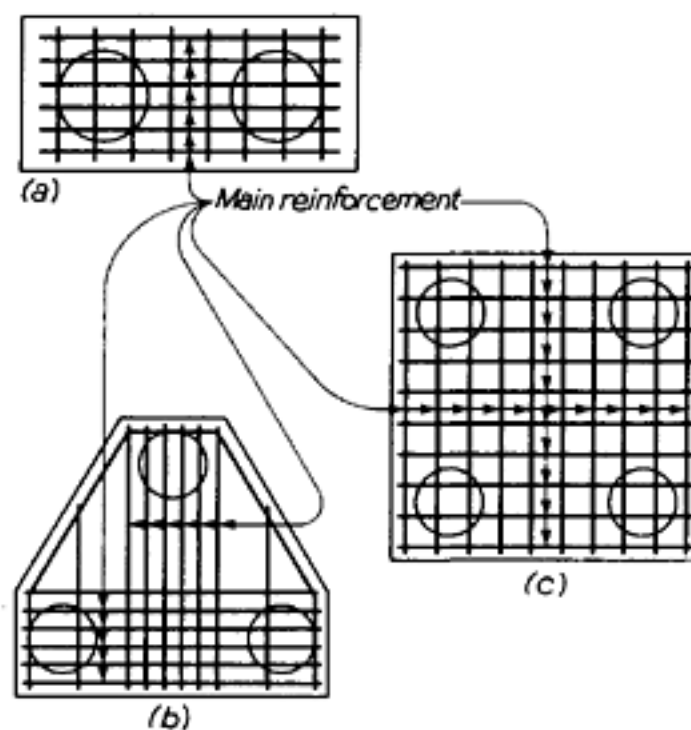


Fig. 7.11 Arrangement of reinforcement in pile caps

Tension reinforcement should be continued past the piles and bent up vertically to provide full anchorage past the centreline of each pile. In addition, fully lapped, circumferential reinforcement consisting of 12mm diameter bars should be provided at 250mm vertical spacing.

The cover to all reinforcement depends on the exposure condition, and the grade of concrete being used in the pile cap, and reference should be made to Clause 3.3 of BS 8110: Part 1. In particular, where concrete is cast directly against the earth, the cover should not be less than 75mm.

Where columns carry a compressive load combined with a unidirectional bending moment, the line of action of the column load should be made to coincide with the centroid of the pile group in order to obtain a uniform distribution of load on the piles.

Whittle and Beattie<sup>(7.8)</sup> have described the setting-up of a computer program to calculate the required reinforcement of pile caps. The program relates the overall dimensions of the pile cap to a standard spacing of the piles. The dimensions of a number of standardized types of cap are shown in [Figure 7.12](#).

Deep pile caps are desirable for providing the stiffness necessary to distribute heavy concentrated column loads on to a large pile cluster. However this can sometimes cause construction difficulties in unstable soils where the ground water level is at a shallow depth below the ground surface. It is desirable, on the grounds of cost, to avoid construction expedients such as a wellpoint ground-water lowering system to enable the pile cap to be constructed in dry conditions. Consideration should therefore be given to raising the level of the pile cap to bring it above ground-water level or to such a level that sump pumping from an open excavation will not cause instability by upward seepage.

The design and construction of pile caps at over-water locations is discussed in [Section 9.6.3](#).

## 7.9 The design of pile capping beams and connecting ground beams

Pile capping beams have the function of distributing the load from walls or closely-spaced columns onto rows of piles. For heavy wall loading in conjunction with transverse bending moments the piles are placed in transverse rows surmounted by a wide capping beam ([Figure 7.13a](#)). The piles may be placed in a staggered row for walls carrying a compressive loading with little or no transverse bending moments ([Figure 7.13b](#)). A lightly-loaded wall can be supported by a single row of piles beneath the centre-line provided that the beam capping the piles is restrained by tying it to transverse capping beams carrying cross walls in the structure.

A design method was proposed by the Building Research Establishment<sup>(7.9)</sup> which allows the bending moments and shearing forces in a beam to be reduced if the beam can be shown to be acting compositely with the brick wall built upon it. The method may be applied to the design of pile capping beams for house foundations, and is applicable to walls having a height of not less than 0.6 of the span of the beam. The walls must not have door or window openings near to the supports, as this would interfere with the arching action of the brickwork. The bending moment produced by a uniformly distributed load on a freely supported beam is  $WL/8$ . With full composite action between beam and wall, this moment may be reduced to a minimum of  $WL/100$  for light loading, where  $W$  is the total distributed load on the brickwork (including self weight) and  $L$  is the span of the beam. A property of composite

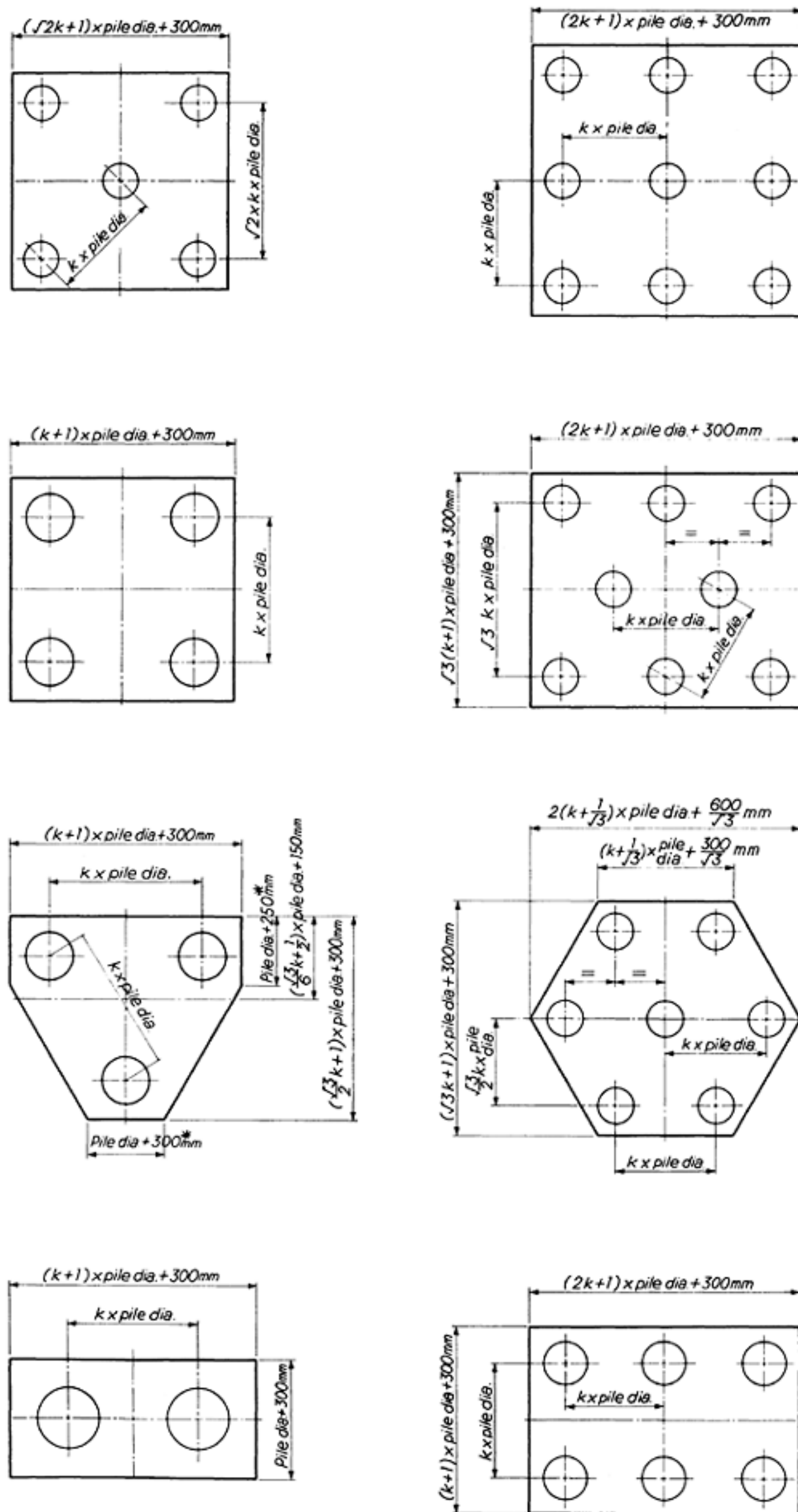


Fig. 7.12 Standard pile caps (after Whittle and Beattie<sup>(7.8)</sup>)

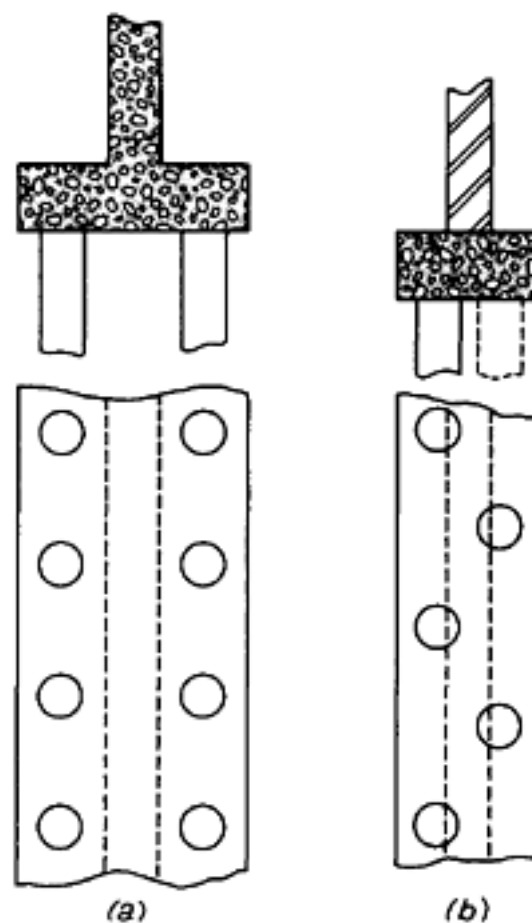


Fig. 7.13 Arrangement of piles in capping beams

(a) Heavy wall loading with transverse bending moments

(b) Light wall loading with little or no transverse bending

action is that the compression in the arch within the brickwork is directed radially towards the nearest firm supports; therefore shearing reinforcement of the beam is theoretically unnecessary if the loaded lengths at the supports can be shown to be not greater than the depth of the beam. However the BRE recommended that all beams designed for bending moments of greater than  $WL/60$  should be designed to resist the shearing forces produced by the full dead and imposed loading. It is recommended that the depth of the beam should be between  $L/15$  and  $L/20$ . However, with a heavily-loaded wall only a small degree of composite action is allowed and it may be necessary to use a beam deeper than  $L/15$ . When considerable composite action is present, *i.e.* when the bending moment is less than  $WL/40$ , the reinforcement should be calculated for a beam having a depth equal to  $L/15$  even if a deeper beam is to be provided for other reasons. This is to ensure there is sufficient steel in the beam to act as a tie across the springing of the arch within the brickwork.

When designing pile capping beams by limit-state principles it is seldom necessary to consider the serviceability limit-state. However, an examination of the limit-state of cracking is necessary if the beam is to be exposed to soil or ground water which can be expected to be corrosive. The limit-state of deflection should be checked if the beam is to support a wall faced with a material such as mosaic tiles, which are particularly susceptible to cracking due to small movements.

The BRE design method assumed that the ground floor slab is carried by the soil and is not connected to the capping beam. *If a suspended floor slab is provided the capping beam must be designed by conventional methods.*

It is a good practice to provide a suspended ground floor slab in cases where piles are provided to restrain a structure from lifting due to a *swelling* soil. A ground floor slab cast directly onto a swelling soil will lift and will, in turn, cause the lifting of internal partitions, with the consequent distortion of any floors carried by them, and the cracking of plaster finishes. Uplift pressures due to soil swelling against the underside of a pile capping beam must be considered. In clay soils where mature trees or hedges have been removed the clay may swell up to 100mm over a long period of years. Swelling of pyritic mudstones and shales can occur due to the growth of gypsum crystals within the laminations of these rocks. Gypsum growth can be caused by chemical and micro-biological changes consequent on changed environmental conditions<sup>(7.10)</sup>. Swelling pressures, if the upward movement of the soils is resisted by a reinforced concrete capping beam, can be of a magnitude which will cause the piles to fail as tension members, or which will lift the piles out of the soil.

It is essential to insert a layer of compressible material such as Clayboard or special low density polystyrene or to provide a void between the soil and underside of the capping beam to reduce the uplift forces transferred to the piles (Figure 7.14).

Load-deflection tests should be made on specimens of the compressible material to ensure that the amount of compression required by the predicted degree of soil swelling does not generate a pressure on the ground beam that is sufficient to cause structural

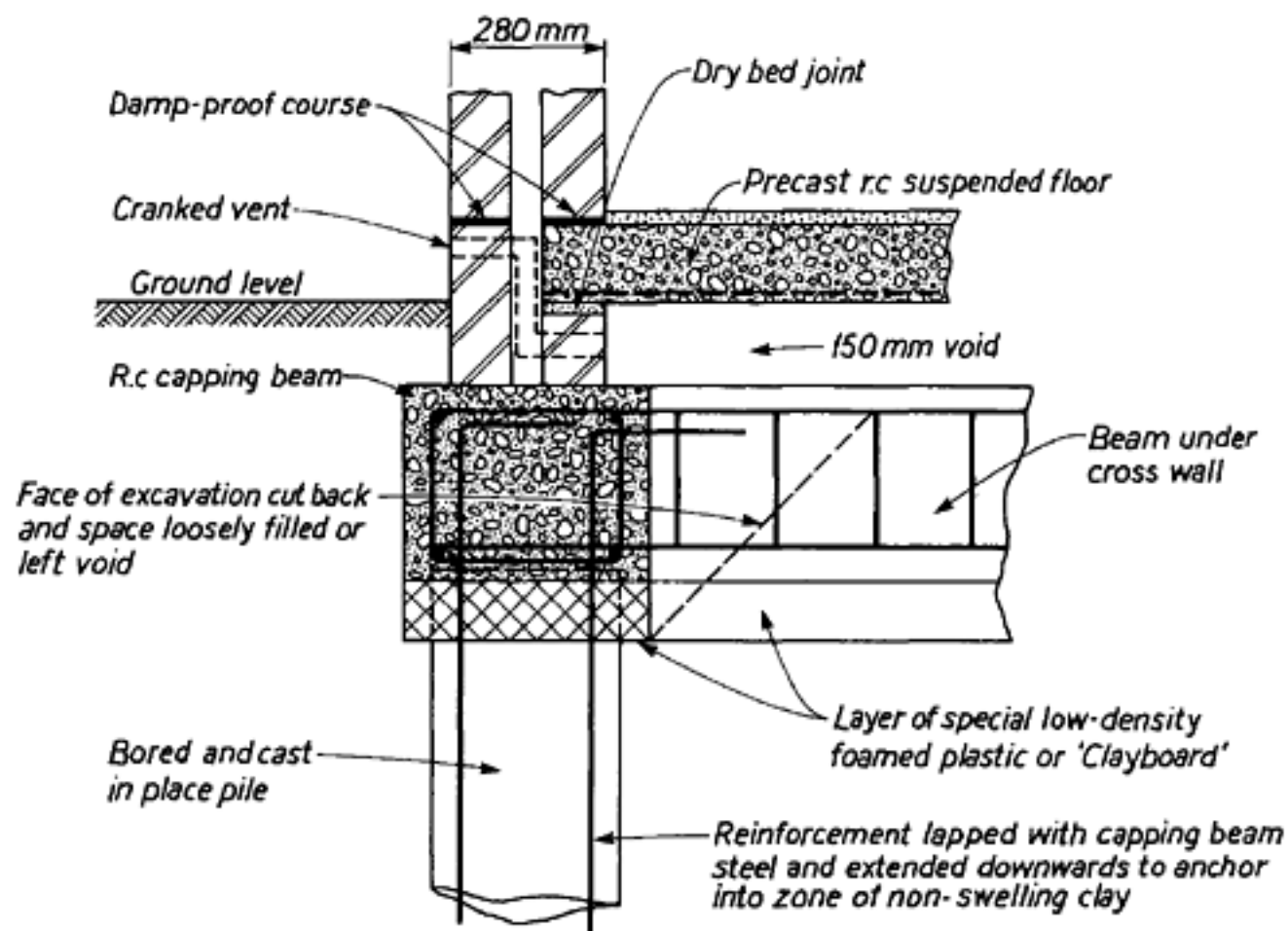


Fig. 7.14 Design of pile capping beam for swelling clay soils

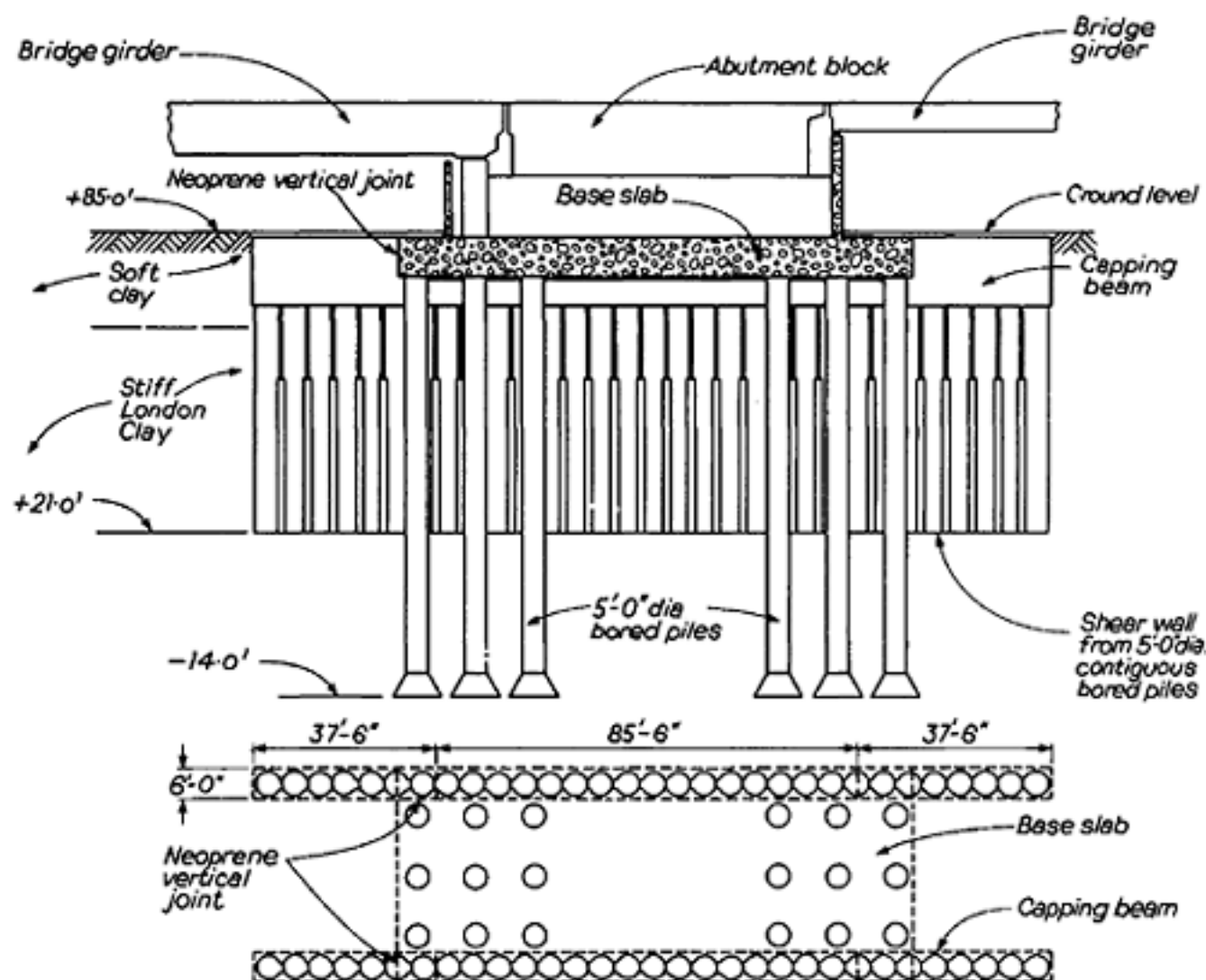


Fig. 7.15 Pile capping arrangements to take horizontal loading for bridge abutments in Westway viaduct

failure of the beams or piles, or lifting of the building. There have been a number of cases of failure and cracking of piles, ground beams and superstructures to low-rise buildings constructed on swelling clays in recent years. These have been caused mainly by deficiencies in design such as inadequate tension reinforcement and lack of proper provision for uplift on ground beams. The latter should be of generous depth to provide for differential uplift forces caused by local tension failure in piles in unpredictable conditions of soil swelling.

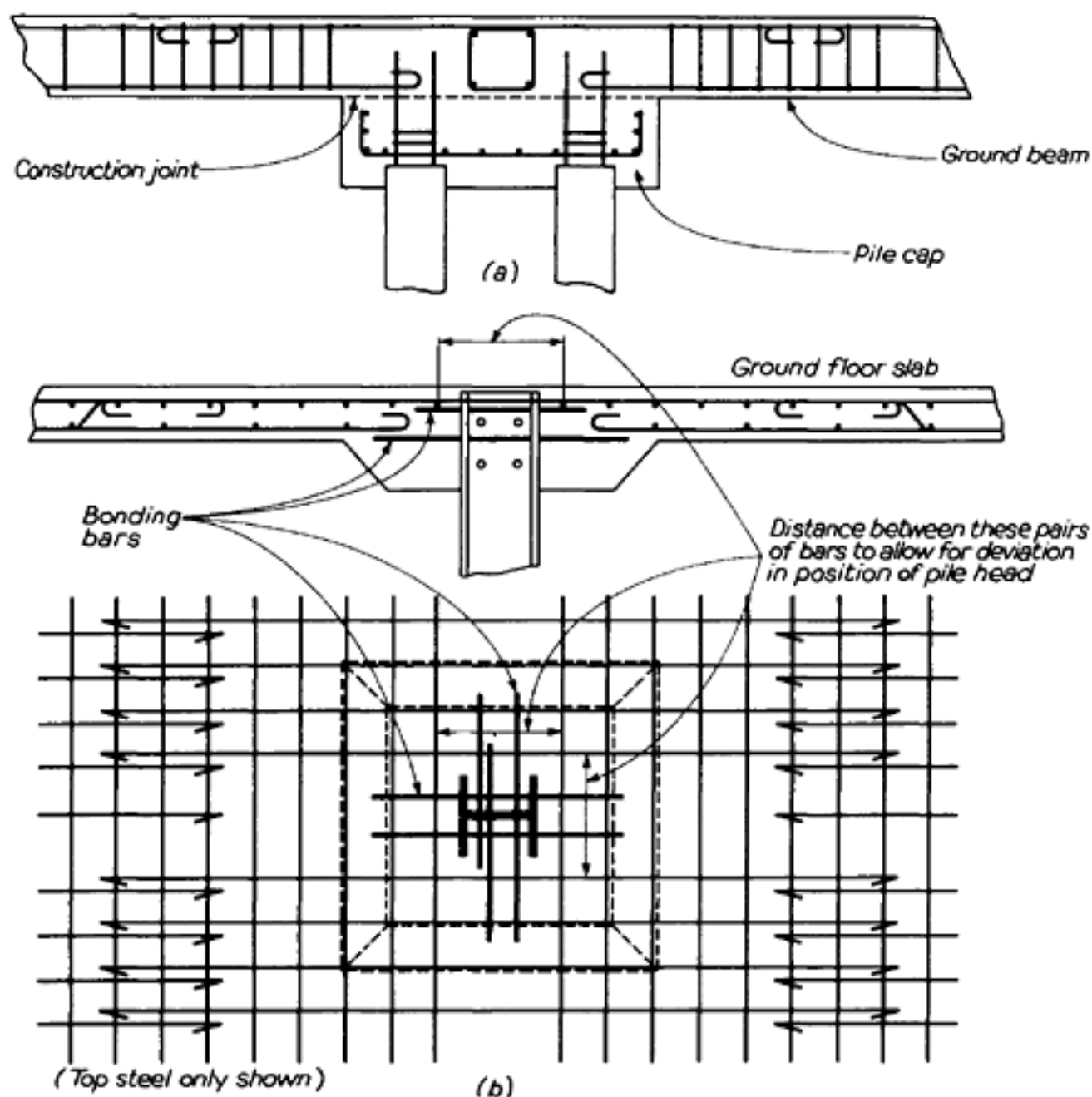


Fig. 7.16 Arrangement of reinforcing steel in ground beams and ground floor slabs

Horizontal swelling forces can also impose loads on pile capping beams due to the restraint provided by the beam to the expansion of the mass of the soil. To avoid excessive swelling forces on the inner sides of beams they should not be left in contact with the clay. After casting the concrete has been completed the clay should be cut back as shown in [Figure 7.14](#). The space between the side of the beam and the cut-back of the excavation is left empty, or is only loosely backfilled. Ground beams are provided to act as ties or compression members between adjacent pile caps, so providing the required restraint against sidesway or buckling of the piles under lateral or eccentric loading (see [Section 7.5](#)). Ground beams and pile capping beams may have to withstand horizontal loading from the soil due to the tendency to movement of vertical piles under lateral loading. They may also be subjected to bending in a vertical direction due to differential settlement between adjacent groups of piles.

It may be permissible to allow the passive resistance of the soil against the sides of pile caps and ground beams to supplement the resistance of the piles to lateral loading. However, in clay soils the ground will shrink away from the sides of shallow members in dry weather conditions. Trenching for building services alongside pile caps must also be considered a possibility. In any case quite appreciable yielding of the soil must take place before its passive resistance is fully mobilized. This movement may be sufficient to cause the failure in bending of vertical piles. The problem of the resistance of pile capping members to lateral loading was an important consideration in the design of an abutment of one section of the Westway viaduct in London ([7.11](#)). The abutment was carried on large vertical bored piles, but raking piles in sufficient numbers to carry a reversible horizontal thrust of 40MN were regarded as too costly. It was therefore decided to carry the horizontal thrust in adhesion on the sides of two longitudinal shear walls constructed from contiguous bored piles ([Figure 7.15](#)). Because the full adhesion between the concrete of the shear walls and the clay should be mobilized under a very small movement, the lateral movement of the shear walls at the design horizontal load was negligible. The base slab carrying the load from the bridge girders and abutment block was rigidly connected to the main bearing piles, but the only connection to the shear wall capping beam was at each end of the base slab, where a neoprene joint allowed relative vertical movement between the shear wall and abutment.

The superimposed loading on the ground beams or pile capping beams is transferred to the piles

by bonding the longitudinal reinforcing steel to the beams into the pile caps. However, it is not a good practice to carry the longitudinal steel through holes burned in the projecting parts of steel piles. It is quite likely that the pile head will have deviated from the correct position and it may be impossible to bend the beam reinforcing bars over a sufficient horizontal distance to pass through the holes in the steel pile without causing complications to the stirrups in the beams. If the main steel in ground beams or ground floor slabs must extend across the piles it should be carried above the pile heads as shown in [Figure 7.16a](#), or at the sides of the projecting piles with ample spaces to allow for misalignment ([Figure 7.16b](#)).

## 7.10 References

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## 7.11 Worked examples

### Example 7.1

Design a precast concrete pile 15m long suitable for moderately hard driving conditions for a structure on land. The pile is to be a square in section and designed to be lifted from the third point.

[Table 7.2](#) indicates that a 350mm square section will be satisfactory. Employ concrete with a characteristic strength of 40N/mm<sup>2</sup> and use a reinforcement with a characteristic strength of 250N/mm<sup>2</sup>. Then from [Figure 7.2b](#), for lifting at the third point the bending moment for ultimate load is 50kN m.

[Figure 7.2b](#) also shows that four 25mm bars give an ultimate moment of resistance of 58kN, which is satisfactory. The cross-section of the pile is shown in [Figure 7.17](#).

The design can be checked from Chart 1 of BS 8110: Part 1. With  $A_s/bd = 100 \times 2 \times 490.9 / (350 \times 289.5) = 0.97$  this chart gives  $M_u/bd^2 = 2.0$ . Therefore

The serviceability limit state can be checked  $M_u = 2.0 \times 350 \times 289.5^2 \times 10^{-6} = 58.7\text{kN m}$  by reference to Clause 3.12.11.2.3 of BS 8110: Part 1. From Table 3.30, if redistribution is not considered, the maximum clear distance between bars should not exceed 300mm for a characteristic steel strength of 250N/m<sup>2</sup>. In the case of the 350mm square pile, the clear distance between bars is  $350 - 2(40 + 8 + 25) = 204\text{mm}$ . This ensures that a crack width of 0.3mm is not exceeded (Clause 3.12.11.12).

The required spacing of link reinforcement at the head and toe is 0.6% of the gross volume, *i.e.*

$0.6 \times 350^2 \times 1000 / 100 = 735 \times 10^3 \text{mm}^3/\text{m}$ . A single 8mm link has a volume of 52700mm<sup>3</sup>. Therefore there should be  $735000 / 52700 = 14$  links per metre, *i.e.* links at a nominal spacing of 70mm. The required spacing of links in body of pile at 0.2% of the gross volume is  $3 \times 70 = 210\text{mm}$ . Therefore space the links at  $\frac{1}{4} \times \text{pile width} = 175\text{mm}$  centres.

The arrangement of the reinforcement is as shown in [Figure 2.5a](#).

### **Example 7.2**

A pretensioned prestressed concrete pile 300mm square by 14.5m long is to be driven by diesel hammer. The pile is to be lifted and pitched by slings placed at 0.2L from each end. Determine the arrangement of tendons and links, and calculate the ultimate axial load the pile will support, assuming that the soil conditions are adequate to support this load. Use concrete having a characteristic strength of 50N/mm<sup>2</sup>.

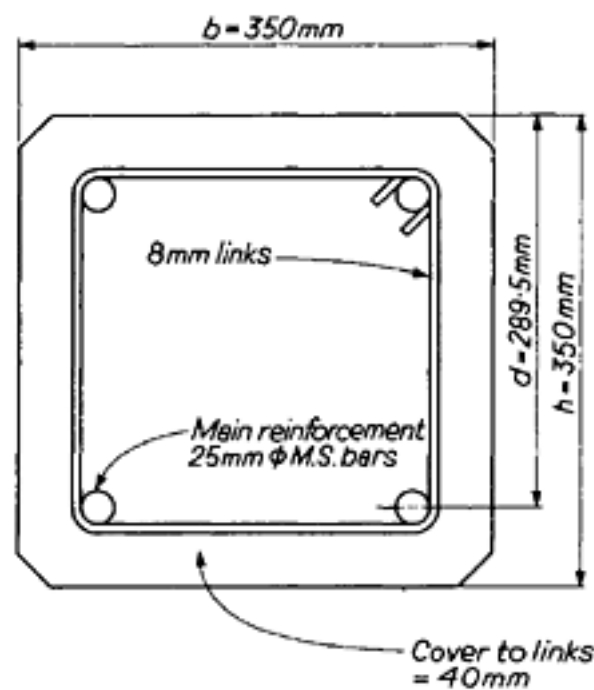


Fig. 7.17

The piles are to be considered as being Class 3 according to Clause 4.1.3. of BS 8110. No loss of prestress will be considered. Design charts for prestressed members are not presently available in BS 8110. Therefore this design is made in accordance with its predecessor, CP 110.

Density of reinforced concrete =  $24 \text{ kN/m}^3$ .

Cross-sectional area of pile =  $0.09 \text{ m}^2$  (ignoring chamfers).

For lifting at  $0.2L$  from each end, maximum design bending moment =  $WL^2/40$ .

For dead load at ultimate limit-state partial factor of safety  $\gamma_f = 1.4$ .

Therefore

$$\text{design bending moment} = \frac{0.09 \times 24 \times 14.5^2}{40} \times 1.4 = 15.9 \text{ kNm}.$$

From BS 8004 for driving with a diesel hammer a minimum uniform prestress in the concrete of  $5 \text{ N/mm}^2$  is required. Therefore

$$\text{force in prestressing tendons} > 5 \times 10^3 \times 0.09 = 450 \text{ kN}.$$

Try using four 7-wire high-tensile steel strands (*i.e.* one in each corner). From CP 110, clause 4.8.1, stress in strand  $\nless 70\%$  of characteristic strength of strand. Therefore

$$\text{strength of single strand} \nless \frac{450}{4} \times \frac{100}{70} = 160.7 \text{ kN}.$$

From Table 30 of CP 110, use 7-wire strand of 12.5mm nominal size, for which,  $A_{ps}f_{pu} = 165 \text{ kN}$  and  $A_{ps} = 94.2 \text{ mm}^2$ , giving  $f_{pu} = 165 \times 10^3 / 94.2 = 1751 \text{ N/mm}^2$ .

Determine depth to neutral axis  $x$ , allowing 40mm nominal cover and 6mm links, from Table 37 of CP 110: Part 3. With  $f_{pu}A_{ps}/f_{cu}bd = 2 \times 165 \times 10^3 / (50 \times 300 \times 251) = 0.088$ , by interpolating between the values obtained from Table 37,

$$\frac{x}{d} = 0.109 + \left(0.108 \times \frac{38}{50}\right) = 0.191.$$

Effective prestress in tendons to provide  $5 \text{ N/mm}^2$  prestress in concrete

$$= f_{pc} = \frac{5}{4} \times \frac{90000}{94.2} = 1194 \text{ N/mm}^2,$$

which is less than  $70\%$  of  $f_{pu}$ .

Thus  $100f_{pc}/f_{pu} = 100 \times 1194/1751 = 68.1$  and  $100A_{ps}/bd = 100 \times 2 \times 94.2/(300 \times 251) = 0.25$ . Obtain  $M_u$  from

Charts 163 and 171 in CP 110: Part 3 for  $f_{pu} = 1700$  which respectively, and interpolate for  $f_{pu} = 1751\text{N/mm}^2$ . From Chart 163,  $M_u/bd^2 = 3.35\text{N/mm}^2$  and from Chart 171,  $M_u/bd^2 = 3.65\text{N/mm}^2$ . Thus for  $f_{pu} = 1751\text{N/mm}^2$

$$\frac{M_u}{bd^2} = 3.35 + 0.3 \times \frac{51}{150} = 3.45\text{N/mm}^2,$$

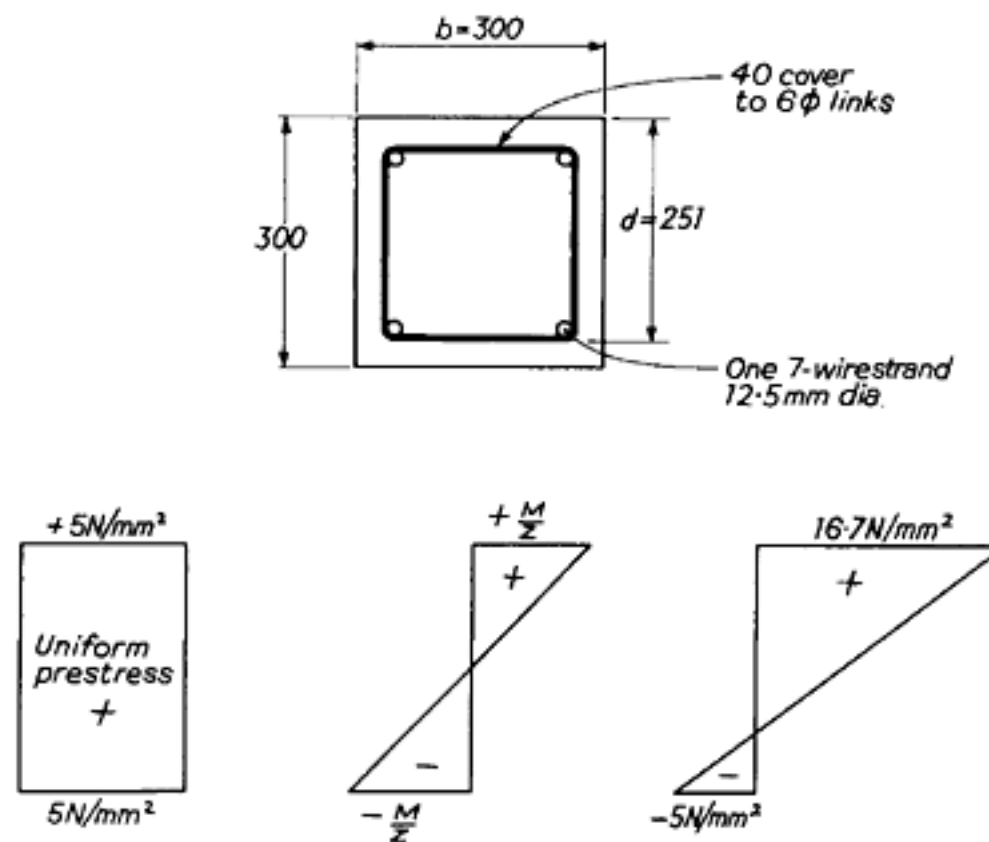


Fig. 7.18

so that  $M_u = 3.45 \times 300 \times 251^2 \times 10^{-6} = 65.2 \text{ kNm}$ . This is greater than the design bending moment of 15.9 kNm, and therefore the minimum prestress required for driving the pile with a diesel hammer is also adequate when the pile is lifted at points located 0.2L from each end.

#### Checking the limit-state of serviceability

From Clause 4.3.3.3.1 of CP 110, allowable compressive stress at transfer. From  $= 0.4f_{cu} = 20 \text{ N/mm}^2$  Clause 4.3.3.2.1, allowable compressive stress in bending  $= 0.33f_{cu} = 16.7 \text{ N/mm}^2$ . From Table 34, allowable flexural tensile stress  $= 5.8 \text{ N/mm}^2$ . Since the uniform prestress was  $5.0 \text{ N/mm}^2$ , the allowable compressive stress at transfer was not a limiting factor and need not be considered.

The elastic analysis of the pile section is shown in [Figure 7.18](#).

Since  $5 + \frac{M}{Z} \leq 16.7 \text{ N/mm}^2$ , therefore  $\frac{M}{Z} \leq 11.7 \text{ N/mm}^2$

Also since  $5 - \frac{M}{Z} > -5.8 \text{ N/mm}^2$ ; therefore  $\frac{M}{Z} \leq 10.8 \text{ N/mm}^2$

The limiting inequality is thus  $M/Z \leq 10.8 \text{ N/mm}^2$ . Now  $Z = I/y = \frac{1}{12} \times 300^4 / \frac{1}{2} \times 300 = 45 \times 10^5 \text{ mm}^3$ , Thus with a partial safety factor  $\gamma_f = 1.4$ ,  $M = 15.9 / 1.4$ . Therefore

$$\frac{M}{Z} = \frac{15.9 \times 10^6}{1.4 \times 45 \times 10^5} = 2.6 \text{ N/mm}^2,$$

which is less than  $10.8 \text{ N/mm}^2$ . Therefore the serviceability limit-state is not reached.

#### Design of lateral reinforcement

Volume of pile  $= 0.9 \text{ m}^3/\text{m}$  run and length of 6mm link  $= 4 \times (300 - 80 - 6) = 4 \times 214 = 856 \text{ mm}$

At ends of pile from clause 7.4.2.3.2 of BS 8004,

$$\text{area of links required} = \frac{90000}{856} \times \frac{0.6}{100} \times 10^3 = 630.8 \text{ mm}^2/\text{m}.$$

Provide 6mm links at 40mm centres ( $708 \text{ mm}^2/\text{m}$ ).

In body of pile, use 6mm links at 120mm centres.

#### Ultimate axial load

From clause 3.5.3 of CP 110, making allowance for any eccentricity of load due to constructional tolerances,

$$\text{ultimate axial load} = [(0.4 \times 50) - 5] \times 90\,000 \times 10^{-3} = 1\,350 \text{ kN}.$$

The ultimate axial load comprises the ultimate dead and imposed loads; *i.e.* the ultimate axial load  $= \gamma_f G_k + \gamma_f Q_k$ , where  $\gamma_f$  is the partial safety factor,  $G_k$  is the characteristic dead load, and  $Q_k$  is the characteristic imposed load. Since ultimate dead load of pile  $= 1.4 \times 24 \times 0.3^2 \times 14.5 = 44\text{kN}$ , therefore

$$\text{ultimate axial load which may be applied to top of pile} = 1\,350 - 44 = 1\,306\text{kN}.$$

Thus the ultimate axial load that the pile will support  $= 1\,306\text{kN}$ .

The arrangement of the reinforcement for this pile is shown in [Figure 2.6](#).

### Example 7.3

Reinforced concrete piles 450mm square and 24m long are used to support a bulk handling plant constructed over the sea. The depth of water at mean high tide is 12m and the underside of the pile cap is 4m above the water. Each pile supports a dead load of 140kN and an imposed load of 90kN: wind loads may be ignored. The piles are driven to just within the permissible maximum deviation of 1 in 75 from the vertical. Calculate the ultimate bending moment on pile, and compare this with the ultimate moment of resistance of a pile.

The piles are to be driven into a medium-dense sand, and therefore the point of virtual fixity can be taken as 1.5m below the soil line. Scour around the pile must be allowed for, say to a depth of 1.5m below the original sea-bed level.

The length of the pile from the underside of the pile cap to the point of fixity is

Pile cap to water level	=4.00m
Water level to original sea bed	=12.00m
Scour depth	=1.50m
Scoured sea bed to point of fixity	=1.50m
Total pile length	=19.00m

Both the pile cap and soil may be regarded as providing full restraint in direction for the ends of the pile, which is unbraced.

Reference to Table 3.22 of BS 8110: Part 1 shows the effective length of pile is  $1.2 \times 19 = 22.8\text{m}$

For a permitted deviation from the vertical of 1 in 75 the eccentricity of the pile head with respect to the point of fixity is  $22.8 \times 1\,000/75 = 304\text{ mm}$

The ultimate loads carried by each pile are as follows.

Dead load	$= 1.4 \times 140 = 196.0\text{kN}$
Imposed load	$= 1.6 \times 90 = 144.0\text{kN}$
Total ultimate load	$= 340.0\text{kN}$

$$\begin{aligned} \text{Ultimate bending moment produced by deviation of pile from vertical} \\ = 340.0 \times 304 \times 10^3 = 103.4 \times 10^6\text{N mm}. \end{aligned}$$

The overall length of the pile, allowing for 1m to be cut away for bonding the reinforcement into the pile cap, is 24m. For this length of pile, reinforcement consisting of four 40mm mild steel bars is required to avoid breakage when lifting from the casting bed at points 0.2L from the head and toe. As mentioned in 7.2, high yield steel is to be used with lifting stress limited to  $250\text{N/mm}^2$ . From [Table 3.4](#) of BS 8110: Part 1, taking condition of exposure as very severe, use 50mm cover with C40 concrete.

Reference to Chart I of BS 8110: Part 3 gives

$$\frac{100A_s}{bd} = \frac{100 \times 2\,513}{450 \times (450 - 50 - 10 - 16)} = 1.49$$

$$\frac{M_u}{bd^2} = 2.9, \text{ thus } M_u = 2.9 \times 450 \times 374^2 = 185 \times 10^6\text{Nmm}.$$

From [Figure 7.2d](#) the design bending moment given by curve C is 95kNm, thus the pile is satisfactorily designed for lifting.

The behaviour of the top 19m of the pile acting as a column must be considered

$$\frac{l_{ex}}{h} = \frac{l_{ey}}{b} = \frac{19 \times 1.2}{0.45} = 50.7 > 10$$

Thus according to BS 8110: Part 1: Clause 3.8.1.3, the column must be considered as slender, and designed to support an additional moment induced in the column by its deflection (see Clause 3.8.3.4)

$$\begin{aligned} a_u &= \beta_e k h \\ &= 1.28 \times k \times 0.45 \\ &= 0.576 \text{ taking } k = 1 \end{aligned}$$

$$M_{add} = N \cdot a_u = 340 \times 0.576 = 195.8 \text{ kN m}$$

Thus total moment =  $103.4 + 195.8 = 299.2 \text{ kN m}$ .

For checking this condition use the characteristic strength of  $460 \text{ N/mm}^2$  for the steel

$$\frac{d}{h} = 0.83, f_{cu} = 40 \text{ N/mm}^2$$

Therefore use Chart 38 of BS 8110: Part 3.

$$\frac{N}{bh} = \frac{340\,000}{450^2} = 1.68$$

$$\frac{100A_{sc}}{bh} = \frac{100 \times 5\,027}{450} = 2.48$$

$$\frac{M_u}{bd^2} = 4.1, \text{ thus}$$

$$M_u = 4.1 \times 450^3 \times 10^{-6} = 373.6 \text{ kN m}$$

This value exceeds the design ultimate moment of  $299.2 \text{ kNm}$ , thus the pile is satisfactory. However, it can be shown by repeating the above analysis that provision of four  $32 \text{ mm}$  bars, although adequate to resist lifting stresses, would not be satisfactory to resist the induced moments due to deviation in the installed condition.

### Example 7.4

A group of six  $350 \text{ mm}$  bored-and-cast-in-place piles are spaced at  $1100 \text{ mm}$  centres. Each pile carries an ultimate load of  $500 \text{ kN}$ . Design a pile cap to support a column having a base  $550 \text{ mm}$  square.

From [Figure 7.12](#), spacing factor  $= k = \frac{1\,100}{350} = 3.14$

Adopting a standard arrangement

$$\text{Length of pile cap} = (2 \times 3.14 + 1) \times 350 + 300 = 2850 \text{ mm}$$

$$\text{Width of pile cap} = (3.14 + 1) \times 350 + 300 = 1\,750 \text{ mm}$$

$$\text{Assume maximum error in plan position of piles} = 115 \text{ mm}$$

Try for overall pile cap depth of  $800 \text{ mm}$

$$\text{Effective depth to bottom layer of steel} = 800 - 75 - \frac{25}{2} = 710 \text{ mm say}$$

$$\text{Characteristic strength of concrete} = f_{cu} = 35 \text{ N/mm}^2$$

$$\text{Characteristic strength of high yield reinforcement} = f_y = 460 \text{ N/mm}^2$$

Design the pile cap by bending theory, with bending moments and reinforcement calculated on critical sections at the column face.

For  $M_{AA'}$ , lever arm

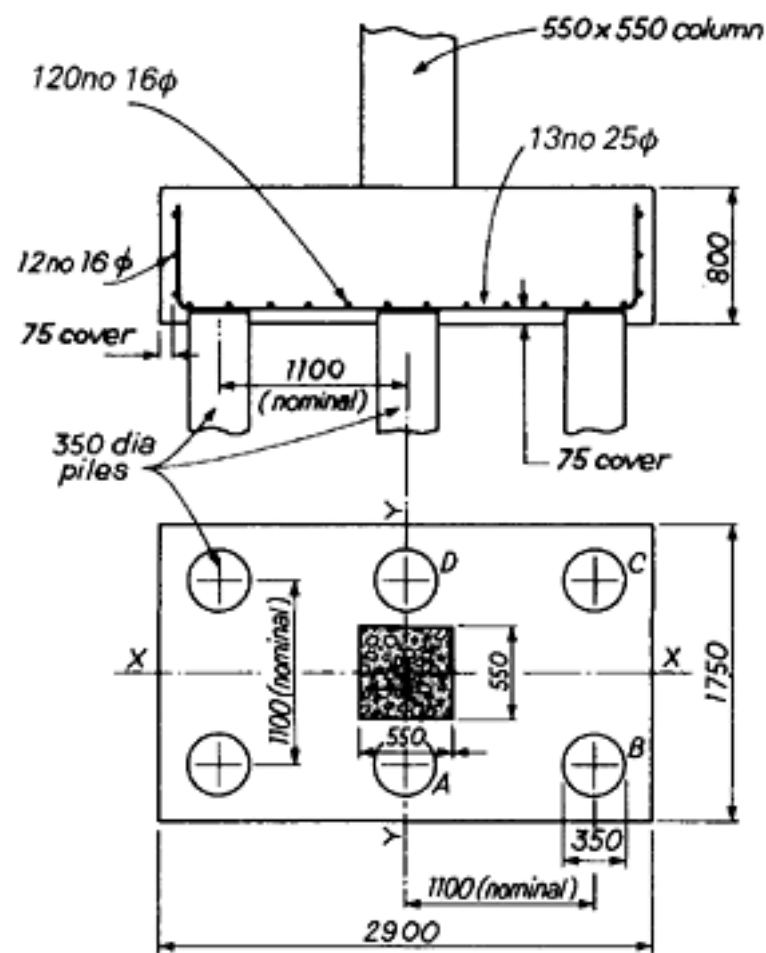
$$= 110 - \frac{550}{2} + 115 = 940\text{mm}$$

$$M_{AA'} = 500 \times 2 \times 0.94 = 940\text{kN m}$$

From Chart 1 of BS 8110: Part 3

$$\frac{M}{bd^2} = \frac{40 \times 10^6}{1750 \times 710^2} = 106$$

$$\frac{100A_s}{bd} = 0.5$$



*Fig. 7.79*

$$A_s = 0.5 \times 1750 \times \frac{710}{100} = 6213 \text{ mm}^2$$

Provide 13 no. 25mm  $\phi$  bars.

Provide 13 no. 25mm  $\phi$  bars.

For  $M_{BB'}$  lever arm  $= \frac{1100}{2} \times \frac{550}{2} + 115 = 390\text{mm}$

$$\text{effective depth} = 710 - 25 = 685\text{mm}$$

$$M = 500 \times 3 \times 0.39 = 585 \text{ kN m}$$

From Chart 2 of BS 8110: Part 3

$$\frac{M}{bd^2} = \frac{585 \times 10^6}{2900 \times 685^2} = 0.43$$

$$\frac{100A_s}{bd} = 0.12$$

$$A_s = 0.12 \times 2\,900 \times \frac{685}{100} = 2\,384 \text{ mm}^2$$

Provide 12 no. 16mm  $\phi$  bars.

Check shear at critical section in accordance with BS 8110: Part 1: Clause 3.11.4.

For critical section,  $b = 1750\text{mm}$

$$v = \frac{V}{bd} = \frac{500 \times 2 \times 10^3}{1750 \times 710} = 0.8 \text{ N/m}^2$$

For design shear resistance,  $a_v = 720\text{mm}$

Pile spacing is less than three pile diameters, thus enhancement may be applied over the whole of the critical section.

From Table 3.9

$$\frac{100A_s}{b_v d} = \frac{100 \times 6381}{1750 \times 710} = 0.51$$

$$V_c = 0.5\text{N/mm}^2$$

$$= \frac{2d}{a_v} = \frac{2 \times 710}{720} = 197$$

Thus modified  $V_c = 0.5 \times 1.97 = 0.99 \text{ N/mm}^2$ . This is greater than  $V = 0.8 \text{ N/mm}^2$ , thus no reinforcement is required. From Clause 3.11.4.5, to check punching shear, the design shear stress at the perimeter of the column

$$= \frac{6 \times 500 \times 10^3}{2 \times (550 + 550) \times 695} = 1.9 \text{ N/mm}^2$$

This is less than  $0.8\sqrt{f_{cu}} = 0.8 \times \sqrt{35} = 4.7 \text{ N/mm}^2$

Thus punching shear requirements are satisfied.

### Example 7.5

A column terminates on a pedestal 2.50m square and is supported on the pile cap shown in Figure 7.20. The total ultimate load on the column inclusive of dead, imposed and wind loading is 80000kN. Design the pile cap for the ultimate loading on each row of piles as shown in Figure 7.20. Assume a characteristic strength in the concrete of  $40 \text{ N/mm}^2$  and the use of high yield reinforcement with a characteristic strength of  $460 \text{ N/mm}^2$ .

In calculating the bending moments and shear forces in the example no allowance will be made for piles out of position. It is highly improbable that most or all of the piles would be simultaneously out of position in an unfavourable direction.

Ultimate bending moment about Section X—X

$$= 6[510.4(1.250 + 0.625) + 422.7 \times 0.625]$$

$$= 7327 \text{ kN-m, or } 7327/7.25 = 1011 \text{ kNm per metre width.}$$

The ultimate resistance moment of the concrete is

$$0.156 \times 40 \times 10^3 \times 1850^2 \times 10^{-6} = 21356 \text{ kNm per metre width.}$$

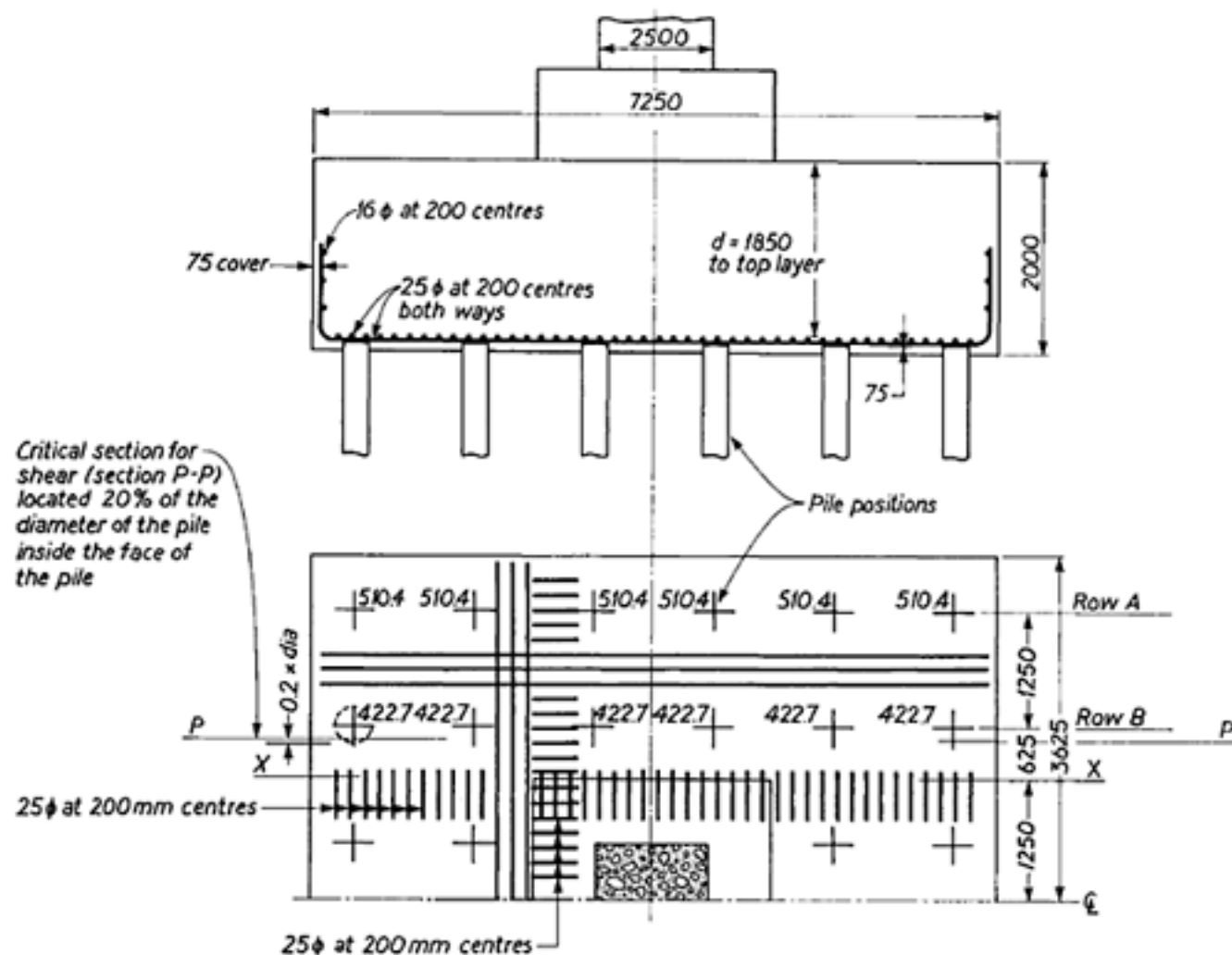


Fig. 7.20

For  $M/bd^2 = 1011 \times 10^6 / 1000 \times 1850^2 = 0.30 \text{ N/mm}^2$ , Chart 2 of BS 8110: Part 3 gives  $100A_s/bd = 0.09\%$ , but minimum steel to be provided  $= 0.13\%bd$  (Table 3.27).

$$\text{Therefore } A_s \text{ required} = \frac{0.13}{100} \times 1000 \times 1850 = 2405 \text{ mm}^2/\text{m}$$

Provide 25mm bars at 200mm centres (2454mm<sup>2</sup>)

For checking shear, the critical section for shear is at  $P-P$  in [Figure 7.20](#) (Clause 3.11.4.3)

$$a_v = 490 \text{ mm}$$

Enhancement factor

$$= \frac{2d}{a_v} = \frac{2 \times 1850}{490} = 7.55$$

$$\frac{100A_s}{bd} = \frac{100 \times 2454}{10 \times 1850} = 0.13$$

From Table 3.9,  $V_c$

$$= 0.34 \text{ N/mm}^2$$

Enhanced value

$$= 0.34 \times 7.55 = 2.57 \text{ N/mm}^2$$

Design shear stress

$$= \frac{(510.4 + 422.7) \times 6 \times 10^3}{7250 \times 1830} = 0.42 \text{ N/mm}^2$$

This is less than the enhanced  $V_c$  value of  $2.57 \text{ N/mm}^2$ , thus shear is acceptable. Checking anchorage bond stress, the maximum force in bars occurs at section  $X-X$ . Design ultimate bond stress

$$= f_{bu} = \beta \sqrt{f_{cu}} = 0.4 \sqrt{40} = 2.5 \text{ N/mm}^2$$

Required anchorage length  $= F_s / \pi \phi f_{bu}$

$$F_s = 460 \times \frac{2405}{2454} \times 490.9 = 221 \times 10^3 \text{ N}$$

$$l = \frac{221 \times 10^3}{\pi \times 25 \times 2.5} = 1125 \text{ mm}$$

This is less than the actual anchorage length provided.

Checking punching shear

$$v = \frac{V}{bd} = \frac{8000 \times 10^3}{4 \times 2500 \times 1850} = 0.43 \text{ N/mm}^2$$

This is less than  $0.8 \sqrt{f_{cu}}$  or  $5 \text{ N/mm}^2$ . Therefore satisfactory.

### Example 7.6

The gable end wall of a house is constructed in cavity brickwork 280mm thick. The combined ultimate dead and imposed load of the 6m long wall averages 40kN per metre run. Design the capping beam.

Divide the gable wall into two equal spans of  $\frac{1}{2}(6.000 - 0.280) = 2.86 \text{ m}$ . Because the load is relatively light and transverse bending moments are comparatively small it is suitable to adopt a single row of three piles beneath the wall.

$$\text{Ultimate load on central pile} = 2.86 \times 40 = 115 \text{ kN,}$$

to which must be added the self-weight of the capping multiplied by 1.4 (BS 8110: Part 1, [Table 2.1](#)). The width of the beam should allow for a projection of about 50mm on each side beyond the face of the wall to allow for deviations in the positions of the piles.

Assume an overall width of 400mm and initially a depth of 400mm. This depth may require adjustment after the required effective depth has been determined, on the basis of singly-reinforced design, that is, no compression steel. Because a suspended floor is provided the imposed load is relatively insignificant compared to the dead load, therefore the beam design will be carried out based on both spans fully loaded.

If each pair of piles deviates from its intended position by 75mm in opposite directions, the span of the beam becomes  
 $2.86 + 0.15 = 3.01\text{m}$

**Then ultimate dead load on beam =  $1.4 \times 0.4 \times 0.4 \times 24 = 5.38\text{kN}$**

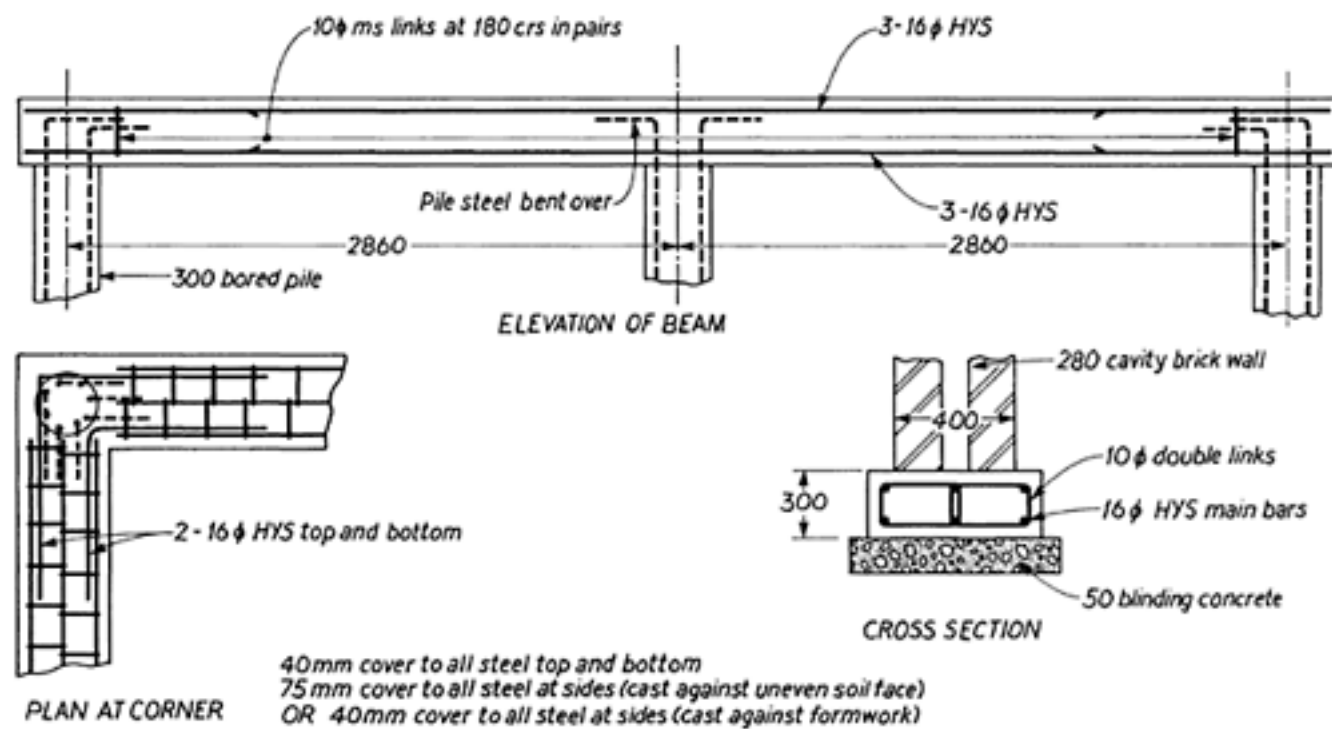


Fig. 7.21

Therefore total ultimate load on beam  $= 40 + 5.38 = 45.38 \text{ kN m}$

$$\text{Take } f_y = 460 \text{ N/mm}^2 \quad f_{cu} = 35 \text{ N/mm}^2$$

Maximum ultimate bending moment at support  $= \frac{45.38 \times 3.01^2}{8} = 51.39 \text{ kN m}$   
 From BS 8110: Part 1: Clause 3.4.4.4

$$d = \sqrt{\frac{M}{0.156bf_{cu}}} = \sqrt{\frac{51.39 \times 10^6}{0.156 \times 400 \times 35}} = 153 \text{ mm}$$

$=$  minimum effective depth for single reinforced section.

Thus a beam of total depth 300mm will be satisfactory, with 40mm cover to all steel and 10mm links. The effective depth  $= 300 - 40 - 10 - 10 = 240 \text{ mm}$  (allowing 10mm for  $\frac{1}{2}$  depth of tension steel). Using Chart 2 in BS 8110: Part 3

$$\frac{M}{bd^2} = \frac{51.39 \times 10^6}{400 \times 240^2} = 2.2$$

$$\frac{100A_s}{bd} = 0.6$$

$$A_s = 0.6 \times 400 \times 240 \times 10^{-2} = 576 \text{ mm}^2$$

Use 3 no. 16mm  $\phi$  bars ( $603 \text{ mm}^2$ )

$$= \frac{9 \times 45.48 \times 3.01^2}{128} = 28.91 \text{ kN m}$$

Span bending moment

Again, using Chart 2,

$$A_s = 317 \text{ mm}^2$$

Provision of 2 no. 16mm  $\phi$  bars would be suitable for reinforcement requirements, but would not comply with the bar spacing requirement of BS 8110: Part 1: Clause 3.12.11.2.3. Therefore use 3 no. 16mm  $\phi$  bars in span.

For shear steel, 
$$v = \frac{45.38 \times 3.01 \times 10^3}{400 \times 240} = 1.42 \text{ N/mm}^2$$

$$v_c = 0.6$$

$$A_{sv} = \frac{b.s.(v - v_c)}{0.87f_{yv}}$$

As horizontal spacing of links at right angles to the span should not exceed  $d$ , use double stirrups.

Using 10mm mild steel,  $A_{sv} = 341 \text{ mm}^2$  (area of four legs)

Then spacing along the beam

$$S = \frac{0.87 \times 250 \times 314}{400 \times (1.42 - 0.6)} = 208 \text{ mm}$$

But maximum pitch allowed by BS 8110 is  $0.75d$   $0.75 \times 240 = 180 \text{ mm}$  or . Thus use 10mm links in pairs at 180mm centres throughout the beam.

Nominal steel should be provided in the top of the beam in the spans as anchorage for the links. For simplicity 3 no. 16mm  $\phi$  bars are detailed throughout.

Although an overall depth of 300mm has been shown by calculation to be satisfactory, a depth of 400mm is preferred for the reasons stated in 7.9.

## CHAPTER 8

### Piling for marine structures

#### 8.1 Berthing structures and jetties

Cargo jetties consist of a berthing head at which the ships are moored to receive or discharge their cargo and an approach structure connecting the berthing head to the shore and carrying the road or rail vehicles used to transport the cargo. Where minerals are handled in bulk the approach structure may carry a belt conveyor or an aerial ropeway. In addition to its function in providing a secure mooring for ships, the berthing head carries cargo-handling cranes or special equipment for loading and unloading dry bulk cargo and containers.

Berthing structures or jetties used exclusively for handling crude petroleum and its products are different in layout and equipment from cargo jetties. The tankers using the berths can be very much larger than the cargo vessels. On the other hand, the hose-handling equipment and its associated pipework are likely to be much lighter than the craneage or dry bulk-loading equipment installed on cargo jetties serving large vessels. The approach from the shore to a petroleum loading jetty consists only of a trestle for pipework and an access roadway. Where the deep water required by large tankers commences at a considerable distance from the shore-line, it is the usual practice to provide an island berthing structure connected to the shore by pipelines laid on the sea bed.

In spite of the considerable differences between the two types of structure piling is an economical form of construction for cargo jetties as well as for berthing structures and pipe trestles for oil tankers. The berthing head of a cargo jetty is likely to consist of a heavy deck slab designed to carry fixed or travelling cranes and the imposed loading from vehicles and stored cargo. The berthing and mooring forces from the ships using the berths can be absorbed by fenders sited in front of and unconnected to the deck structure ([Figure 8.1a](#)) but it is more usual for the fenders to transfer the berthing impact force to the deck and in turn to the rows of supporting piles. The impact forces may be large and because the resistance of a vertical pile to lateral loading is small the deck is supported by a combination

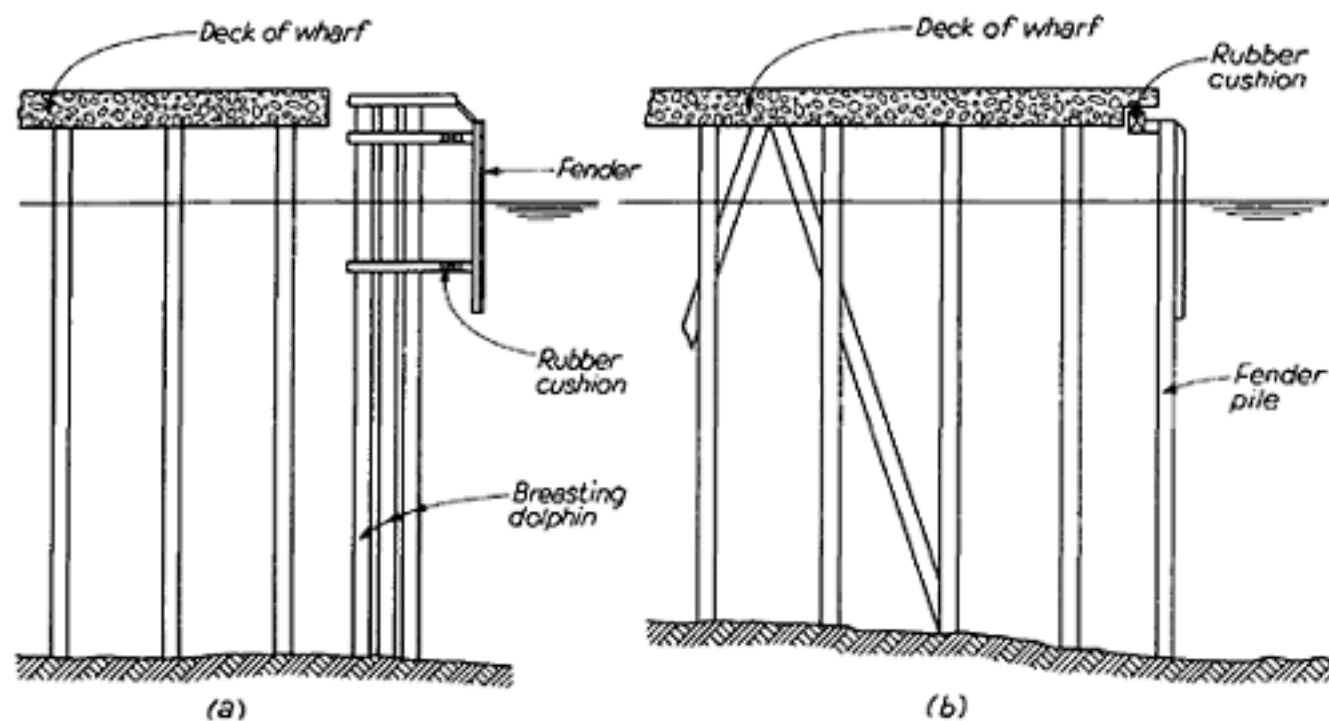


Fig. 8.1 Fender piles for cargo jetties

(a) *In independent breasting dolphin*

(b) *Attached to main deck structure*



*Fig. 8.2 Raking and vertical piles used to restrain berthing forces in bulk handling jetty*

of vertical and raking piles ([Figure 8.1b](#)). These combinations can also be used in structures of the open trestle type such as a jetty head carrying a coal conveyor ([Figure 8.2](#)).

The piles in the berthing head of a cargo jetty are required to carry the following loadings.

1. Lateral loads from berthing forces transmitted through fendering.
2. Lateral loads from the pull of mooring ropes.
3. Lateral loads from wave forces on the piles.
4. Current drag on the piles and moored ships.
5. Lateral loads from wind forces on the berthing head, moored ships, stacked cargo, and cargo handling facilities.
6. Compressive loads from the dead weight of the structure, cargo handling equipment, and from imposed loading on the deck slab.
7. Compressive and uplift forces induced by overturning movements due to loads 1 to 5 above.
8. In some parts of the world piles may also have to carry vertical and lateral loads from floating ice, and loading from earthquakes.

The above forces are not necessarily cumulative. Whereas wind, wave and current forces can occur simultaneously and in the same direction, the forces due to berthing impact and mooring rope pull occur in opposite directions. Berthing would not take place at times of maximum wave height, nor would the thrust from ice sheets coincide with the most severe wave action. Where containers are stored on the deck slab the possibility of stacking them in tiers above a nominal permitted height must be considered.

### 8.1.1 Loading on piles from berthing impact forces

The basic equation used in calculating the force on a jetty or independent berthing structure due to the impact of a ship as it is brought to rest by the structure is

$$\text{kinetic energy } E_k = \frac{m_s V^2}{2g} \quad \dots(8.1)$$

where  $m_s$  is the displacement of the ship and the mass of water moving with the ship, and  $V$  is the velocity of approach to the structure.

The whole of the energy as represented by equation 8.1 is not imparted directly to the jetty piles. Kinetic energy is also absorbed by the deformation of the hull of the ship and by the compression of the fenders and of the cushioning between the fenders and their supporting structure. Ships normally approach the jetty at a narrow angle to the berthing line and the kinetic energy in the direction parallel to this line is generally retained in kinetic form but a part may be lost in overcoming the resistance of the water ahead of the ship's bows, in friction against the fenders, and in the pull on the mooring ropes if these are used to restrain longitudinal movement. A full consideration of the complexities involved in calculating the magnitude and direction of berthing forces cannot be dealt with adequately in this book, and the reader is referred to the British Standard Draft Code of Practice for maritime structures<sup>(8.1)</sup> for guidance on these problems. The design of berthing structures for large tankers has been described by Dent<sup>(8.2)</sup> and helpful design information can be found in the handbooks of the manufacturers of proprietary forms of fendering.

On the assumption that the kinetic energy of the ship transverse and parallel to the berthing line has been correctly calculated the problem is then to assess the manner in which the energy is absorbed by the fenders and their supporting piles. Taking the case of a vertical pile acting as a simple cantilever from the point of virtual fixity below the sea bed, and receiving a blow from the ship with a force  $H$  applied at a point A (Figure 8.3a), the distance moved by the point A can then be calculated by the simple method shown in equation 6.37 and repeated here for convenience, namely

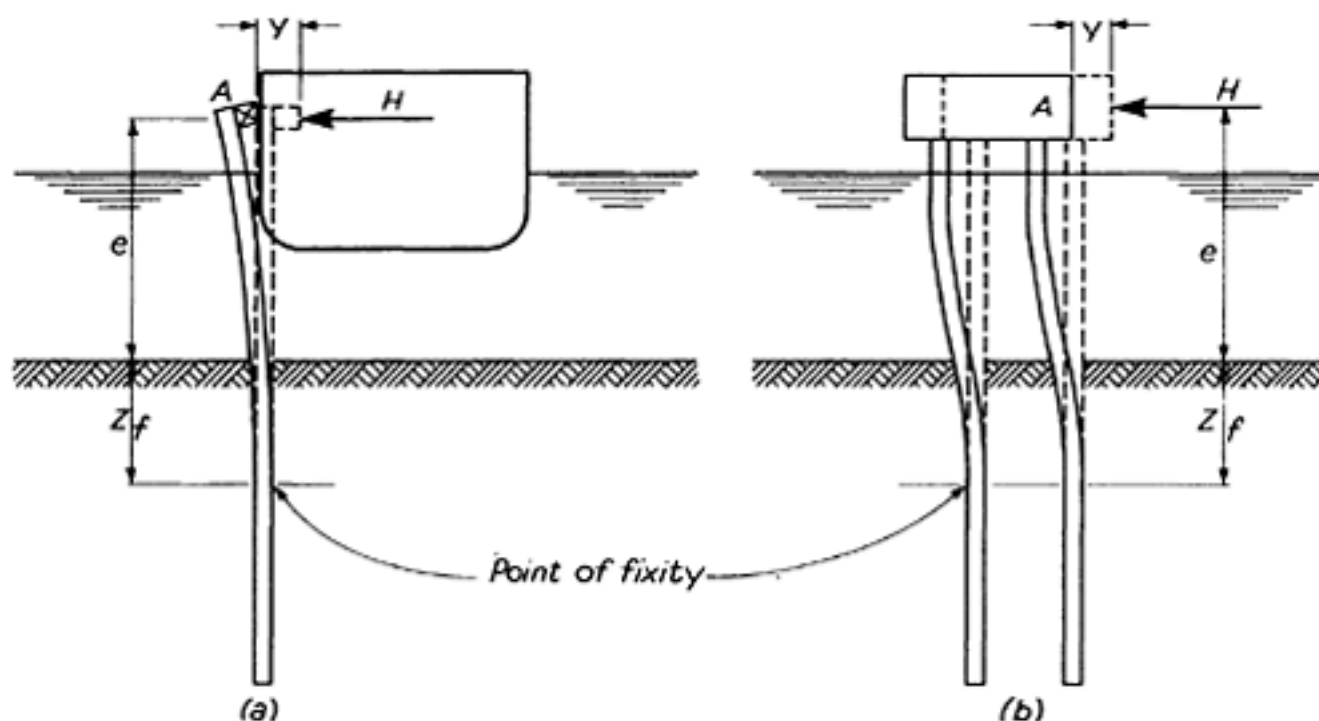
$$\text{distance moved } y = \frac{H(e + z_f)^3}{3EI} \quad \dots(8.2)$$

If the ship is brought to rest by the vertical pile as it moves the pile head over the distance  $y$ , then the work done by the force  $H$  over this distance is given by

$$\text{work done} = \frac{1}{2} Hy = \frac{H^2(e + z_f)^3}{6EI} \quad \dots(8.3)$$

The bending moment  $M$  on the pile is equal to  $H(e + z_f)$  therefore

$$\text{work done} = \frac{M^2(e + z_f)}{6EI} \quad \dots(8.4)$$



*Fig. 8.3 Lateral movement of fender piles due to impact force from berthing ship*

*(a) Single free-headed pile*

*(b) Group of fixed-headed piles*

If required the more-rigorous methods described in Sections 6.3.3 and 6.3.4 can be used to calculate the deflection of the pile head and hence the work done in bringing the ship to rest.

The bending moment which can be applied to a pile is limited by the allowable working stress on the extreme fibres of the material forming the pile for normal berthing impacts, or by the yield stress with abnormal berthing velocities. Thus if the allowable resistance moment  $M_a$  is used in equation 8.4 the capacity of the pile to absorb kinetic energy can be calculated

and compared to the kinetic energy of the moving ship which must be brought to rest. If the capacity of the pile is inadequate the blow from the ship must be absorbed by more than a single pile. In practice, vertical piles are grouped together and linked at the head and at some intermediate point (Figure 8.1a) to form a single berthing dolphin, or are spaced in rows or 'bents' in the berthing head of a jetty structure. In the latter case the kinetic energy of the ship may be absorbed by a large number of piles. In the case of a pile fixed against rotation by the deck slab of a structure (Figure 8.3b) it was shown in equation 6.38 that

$$\text{distance } y \text{ moved by point A} = \frac{H(e + z_f)^3}{12EI} \quad \dots(8.5)$$

The bending moment caused by a load at the fixed head of a pile is equal to  $\frac{1}{2}H(e + z_f)$ , and thus the work done is the same as shown in equation 8.4.

Where medium to large vessels are accommodated the berthing impact is not absorbed directly by a pile or by a deck structure supported by piles. Means are provided to cushion the blow, thus reducing the risk of damaging the ship, and limiting the horizontal movement of the jetty. It is also more economical during design to provide cushioning devices than to absorb forces directly on the structure. It must be noted that whereas independent berthing dolphins can be allowed to deflect over a considerable distance (and large deflections are the most efficient means of absorbing kinetic energy), the deck slab of a cargo jetty cannot be permitted to move to an extent which would cause instability in travelling cranes, stacked containers or mechanical elevators. This limitation restricts the allowable movement of such cargo jetties to a very small distance.

Where energy absorbing fenders are provided, the work equation 8.4 is modified. Taking the simplified case shown in Figure 8.4 of a fender pile backed by a cushion block transmitting the impact to a bent of piles transverse to the berthing line, the work equation becomes

$$\begin{aligned} \text{kinetic energy of moving ship absorbed by system in Figure 8.4} \\ = \frac{1}{2} \times H \times \Delta = \frac{1}{2} \times H (\Delta_1 + \Delta_2) \end{aligned} \quad \dots(8.6)$$

where  $H$  is the impact force of the first blow on the fender;  $\Delta$  is the distance moved in bringing the ship to rest after the first impact,  $\Delta_1$  is the distance moved by the compression of the cushion block, and  $\Delta_2$  is the distance moved by the pile bent.

In a practical design case a limit is placed on  $\Delta_2$  by the operating conditions on the jetty. Then if the cushion block is to be fully compressed by the ship moving at the maximum design approach velocity,  $\Delta_1$  is known and  $\Delta$  is the sum of  $\Delta_1$  and  $\Delta_2$ .

Hence, knowing the kinetic energy of the moving ship, the impact force  $H$  can be calculated. This force is the sum of the force in the cushion block and the shearing force at the head of the pile. The bending moment induced in the fender pile by the action of force  $H$  over distance  $\Delta$  is compared with the moment of resistance of the selected pile, and the energy absorbing capacity of the cushion block is checked to ensure that the force required for full compression is not exceeded by the force  $H$ . The condition shown in Figure 8.4, of a single fender pile transmitting the full force of a moving ship to a single pile bent, does not occur in practice. In

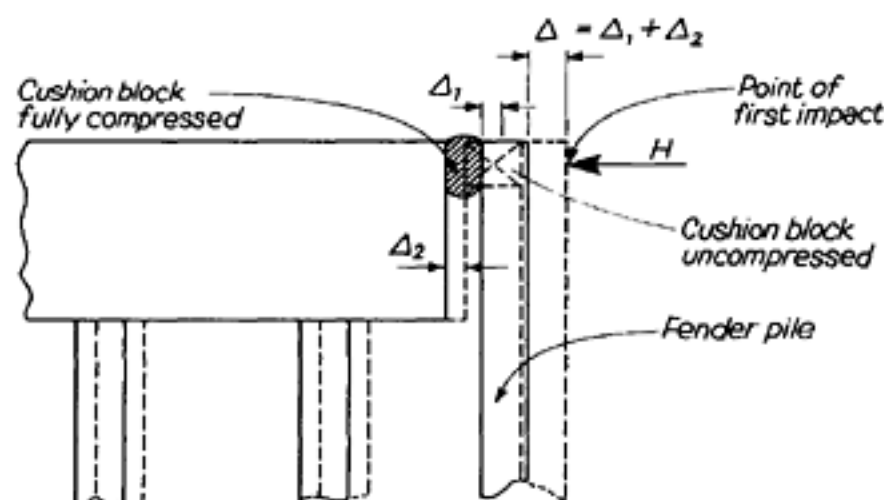


Fig. 8.4 Energy absorption offender pile cushioned at head

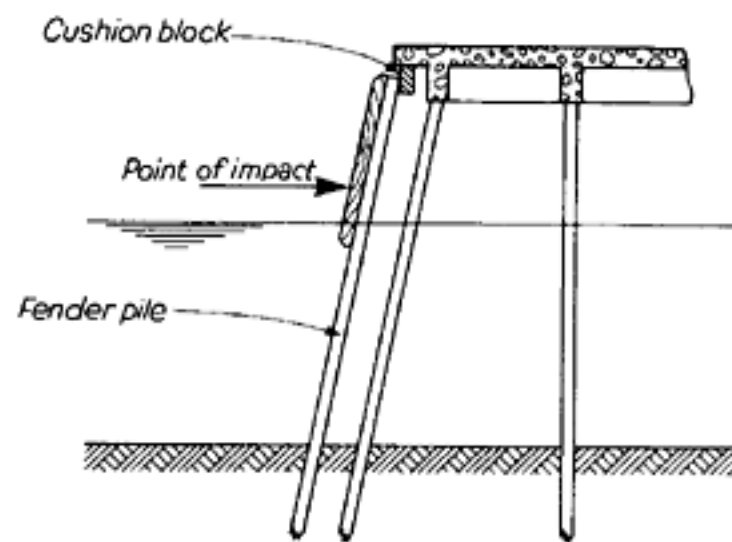


Fig. 8.5 Impact force below head of raking fender pile

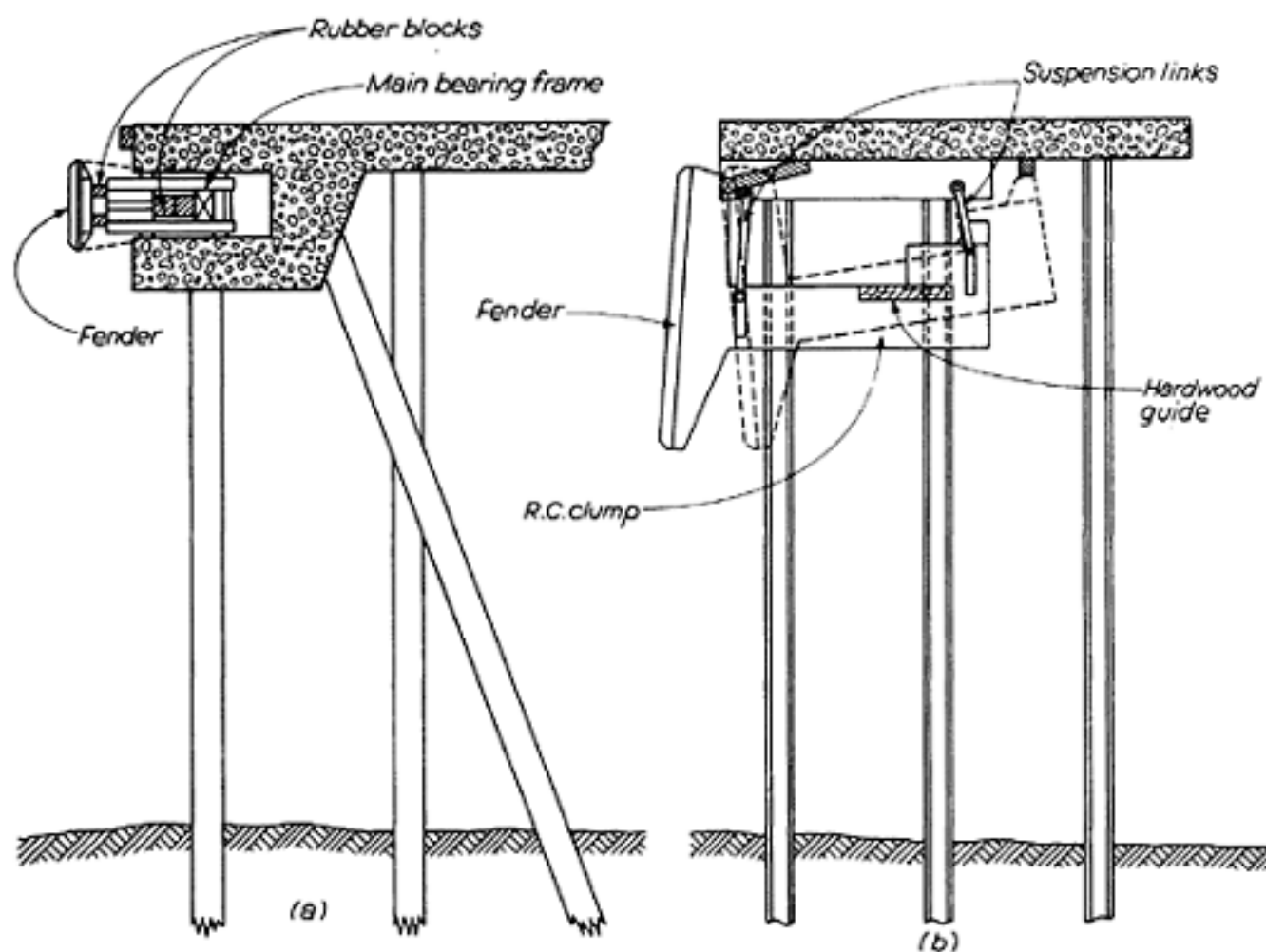


Fig. 8.6 Pile-supported fendering systems

- (a) Rubber cushion fender  
(b) Link-suspended clump fender

a cargo jetty the fender piles are spaced at equal distances along the berthing face and the impact is absorbed by a number of piles, depending on the closeness of their spacing and the extent to which they are tied together by intercostal beams or by a longitudinal berthing beam. An approximate rule is to assume that the blow is absorbed over a length of berthing face equal to twice the width of the jetty.

The design process is one of trial and adjustment to determine the most economical combination of vertical fender piles with rubber or spring cushion blocks that will limit the movement of the protected jetty structure to the desired value. If the impact is delivered at a point below the head ([Figure 8.5](#)) some of the energy is absorbed by the soil, some by the deflection of the pile considered as a beam fixed at the lower end and with a yielding prop at the upper end, and some by the yielding at the prop position (*i.e.* the yielding of the cushion block).

As alternatives to the system of fender piles, each backed by a cushion block as shown in [Figure 8.4](#), a group of piles can carry a rubber fender ([Figure 8.6a](#)) or a link-suspended clump fender ([Figure 8.6b](#)). For these designs the energy transmitted to the supporting piles is equal to the kinetic energy of the moving ship, less the energy expended in compressing, displacing and raising the fender from its neutral position.

Forces act in a direction parallel to as well as normal to the berthing line. Assuming that there are no objects projecting beyond the side of the ship, the force acting parallel to the berthing line is equal

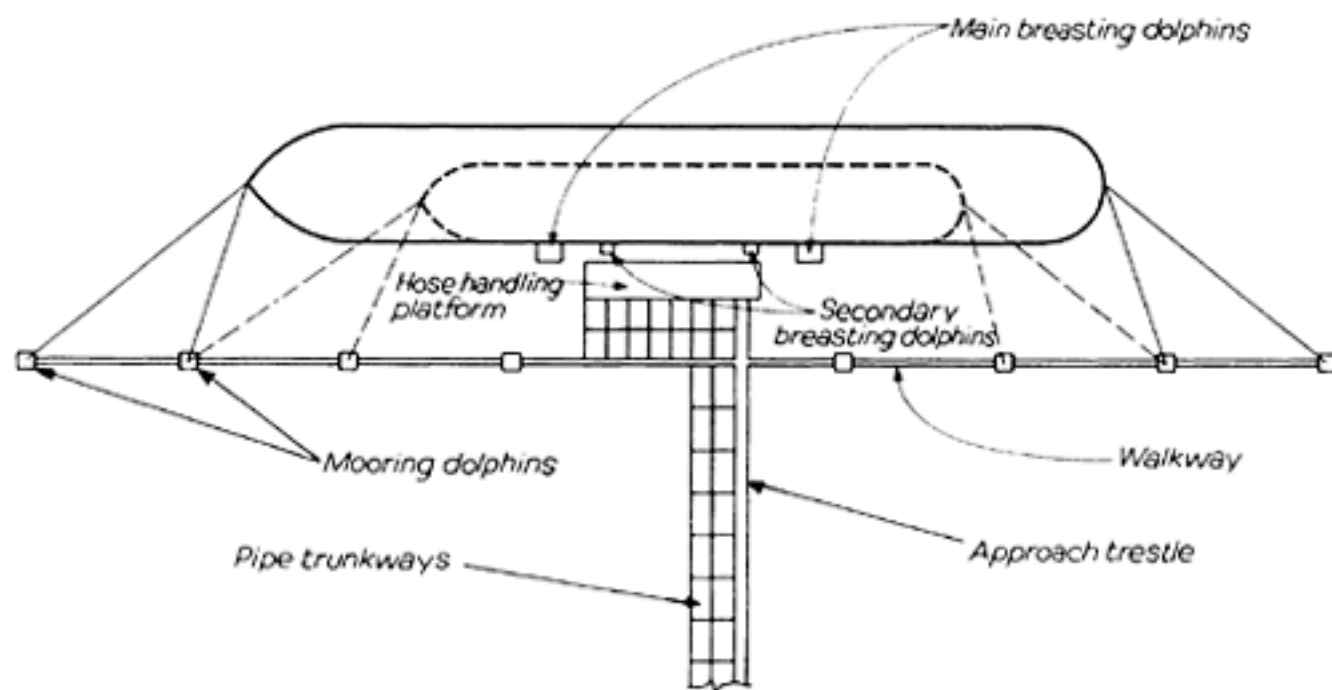


Fig. 8.7 Layout at berthing head of oil jetty

to the coefficient of friction between ship and fender times the reaction normal to the berthing line. The longitudinal force tends to cause the rotation of fender piles and of pile bents set transversely to the berthing line. The rotational force on the pile bents is a maximum when the ship makes contact near the end of the jetty, and it is desirable to provide piles raking in a longitudinal direction at the two ends of the structure. The end piles in a jetty head are vulnerable to impact below the water-line from the bulbous bows of vessels provided with bow-thrust propellers.

Damage to fender piles or their connections to the main structure by longitudinal forces can be avoided by spiking timber rubbing strips onto the faces of the fenders. These will be torn off by a severe impact but the pile will remain relatively undamaged.

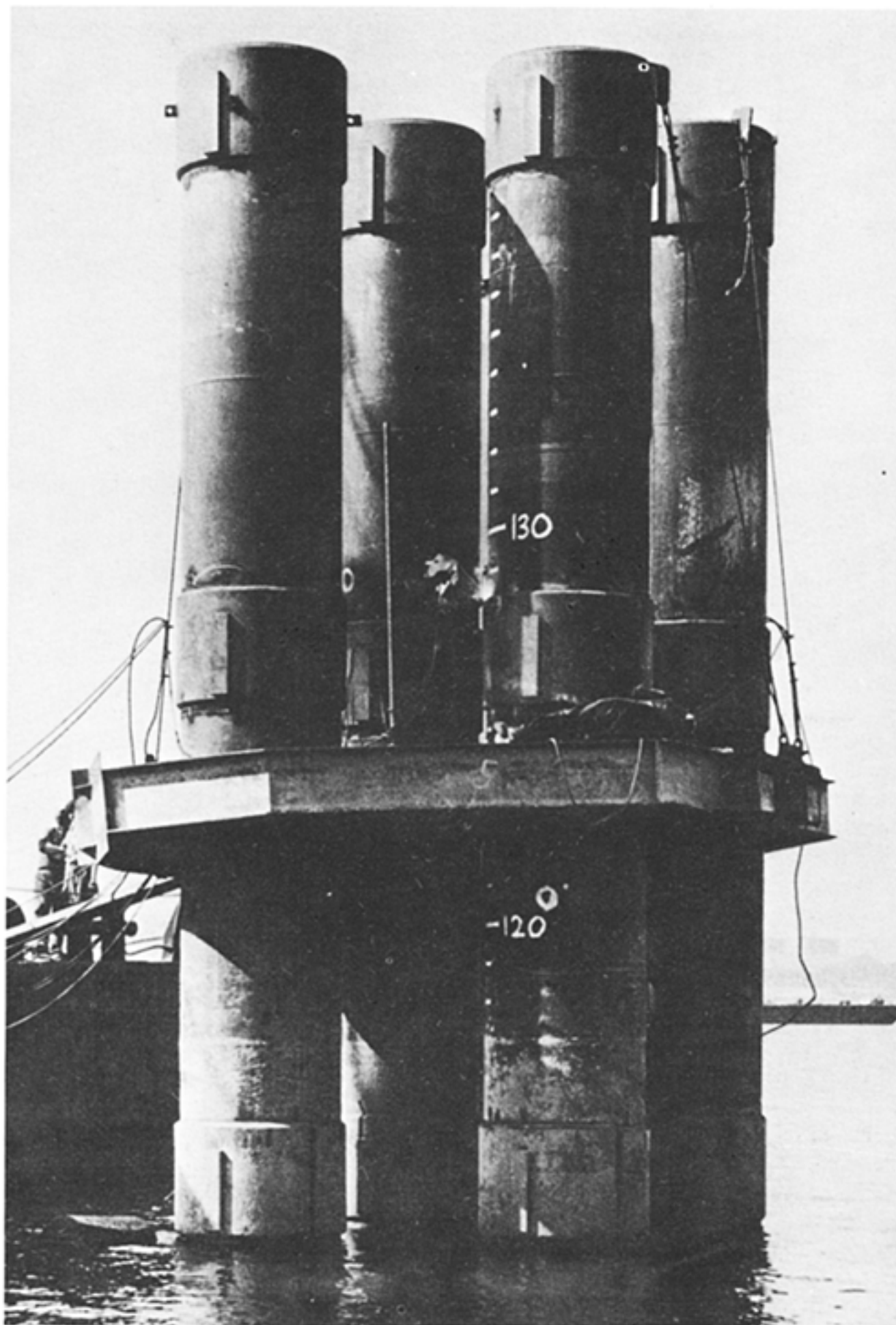
Rubber fenders are designed to deflect in a longitudinal as well as a transverse direction and are thus capable of absorbing impact energy from both directions. Suspended fenders are given a degree of freedom to swing in a longitudinal direction and they fall clear as the ship sheers off after the first impact. Fenders can also be provided with rollers mounted on vertical axles to reduce the longitudinal frictional force on the structure.

As already noted, the facilities provided at the berthing head of an oil jetty or island berthing structure are limited to hose-handling gear and pipework. A relatively small deck area is required and the berthing structure can take the form of two main fenders spaced at a distance equal to about 0.3 times the length of the largest tanker using the berth, with two or more secondary fenders having a lower energy-absorbing capacity sited between them to accommodate smaller vessels ([Figure 8.7](#)). Frequently the main and secondary fenders are sited in front of the hose-handling platform and pile trestles to allow them to take the full impact of the tanker without transmitting any thrust to these structures. The independent breasting dolphins, as shown in [Figure 8.7](#), are designed so that their collapse load is not exceeded by the thrust due to the maximum berthing velocity expected.

The type of piling required for independent breasting dolphins depends on the soil conditions. Where rock, stiff clay or granular soils offering a good resistance to lateral loads are present at or at a short distance below the sea bed, the dolphin can consist of a group of large-diameter circular or box-section vertical steel piles, linked together by horizontal diaphragms ([Figure 8.8](#)) and carrying a timber fender with rubber cushion blocks on the front face of the group. The face area of the fender should be large enough to prevent concentrated loading from damaging the hull of the ship. The horizontal bracing members are not rigidly connected to the pile group. This is to allow the piles to deflect freely to the maximum possible extent while performing their function of bringing the ship to rest.

The layout shown in [Figure 8.7](#) can sometimes restrict the size and numbers of vessels using the berth. It can be more economical to adopt a berthing structure of the type used for cargo handling ([Figure 8.1b](#)). The berthing forces are transmitted directly to the deck so permitting vessels to berth in any position along the face. Pairs of rakers resisting the ship impact are spaced at intervals along the deck or are grouped to form 'strong points' with the deck slab acting as a horizontal beam.

Breasting dolphins for the oil loading terminal of Abu Dhabi Marine Areas Ltd., at Das Island, were designed by the British Petroleum Company to consist of groups of vertical steel tubular piles. The main outer dolphins were formed from a group of seven piles and the inner secondary dolphins were in three-pile groups. The conditions at sea-bed level, which consisted of a layer of shelly limestone cap-rock underlain by a stiff calcareous marl and then a dense detrital limestone, favoured the adoption



*Fig. 8.8 Steel tubular breasting dolphin*

of vertical piles to absorb the berthing forces. The 36.6m piles varied in outside diameter from 800mm at the top to 1300mm at the bottom, the latter being closed by a full plate on which 15 roller cutters were mounted. The piles were pitched through a reinforced concrete template placed on the sea bed and then drilled down by rotating them by means of a hydraulically powered rotary table operated from a jack-up platform. The cuttings were washed up the annular space between the outside of the pile and the rock and this space was afterwards grouted with a sand-cement mix.

Broadhead<sup>(8.3)</sup> described a pulling test made on a mooring dolphin pile to confirm that the lateral resistance of the weak rocks below the sea bed would not be exceeded at the working load. The test pile had a bottom diameter of 1300mm and the pull was applied at a point 24m above the sea bed. The load-deflection curve obtained at a measuring point 22.86m above the sea bed is shown in [Figure 8.9](#) and is compared with the theoretical deflection curve assuming fixity at sea-bed level or support

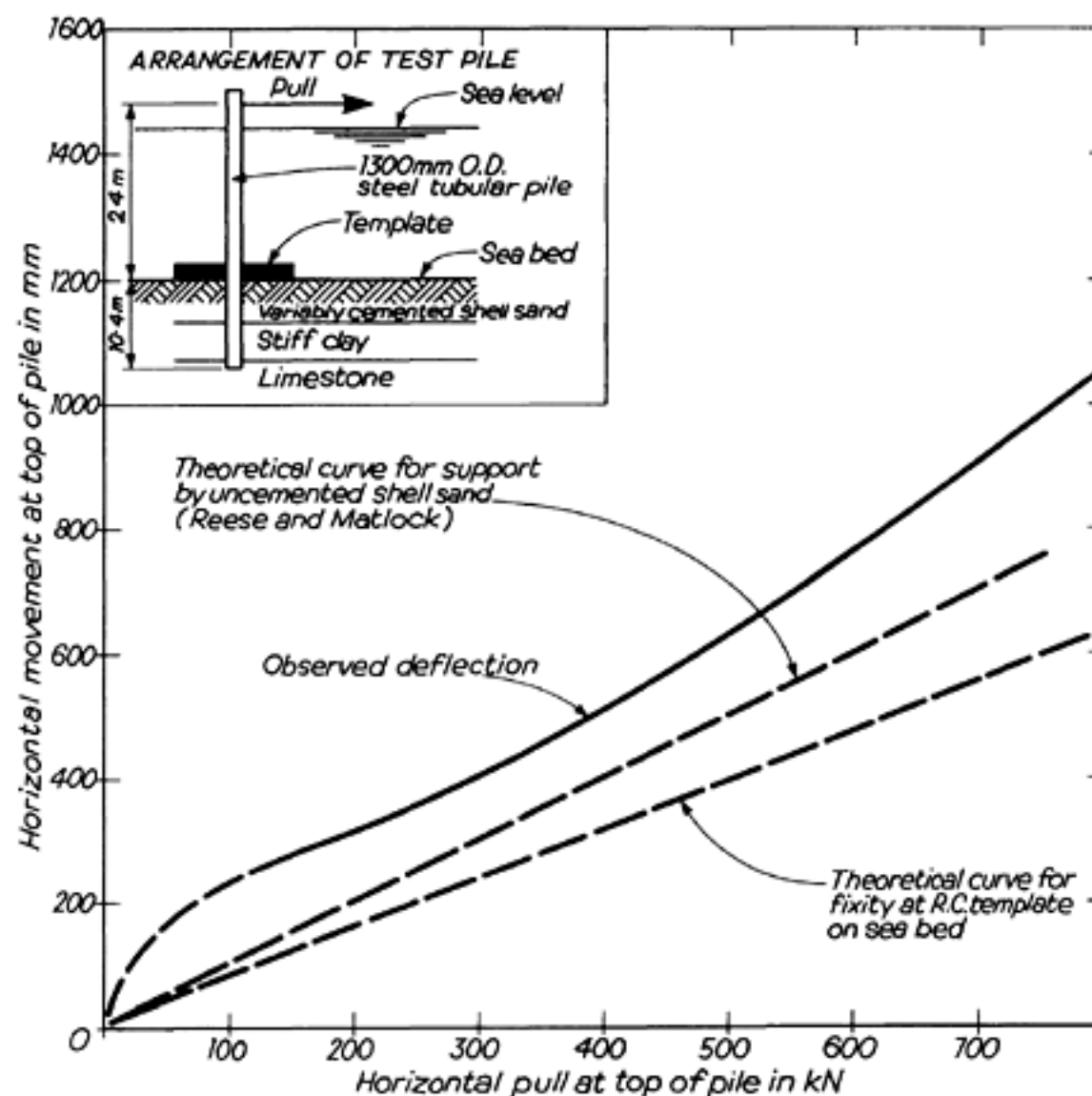


Fig. 8.9 Load-deflection curve for 1300mm O.D. steel tubular pile due to horizontal load at head of pile (after Broadhead<sup>(8.3)</sup>)

from an uncemented shell sand below sea bed, using the elastic analysis of Reese and Matlock (see [6.3.4](#)).

### 8.1.2 Mooring forces on piles

Mooring structures are not required to carry any pull from ropes during the operation of berthing ships other than a restraining longitudinal movement at the final stages of the berthing operation.

When the ship is fully moored four ropes are attached to bollards or bitts fixed to the jetty structure, or mounted on independent mooring dolphins in positions such as those shown in [Figure 8.7](#). Using this type of layout the ship is restrained from excessive ranging against the fenders, and also from moving away from the berth under the influence of offshore waves or currents. The load on any individual rope due to winds or currents acting on the ship or to checking the way of a ship during berthing cannot be calculated with any accuracy. It depends on the tensioning of the rope and its angle to the berthing line.

The wind and current forces on the ship can be calculated using the equations given below for calculating the current force on a pile (equation 8.10) or the wind force on a pile (equation 8.17).

Mooring dolphins should be designed to be as rigid as possible. This is to restrict the ranging of ships which is exaggerated by the lifting and sagging of the mooring ropes. Independent mooring dolphins can take the form of pile groups set back from the berthing line as shown in [Figure 8.7](#), or placed beyond the ends of the berthing head. Piles in mooring dolphins can be raked in two directions to resist longitudinal and transverse pulls ([Figure 8.10](#)). Where rock is present at or at a short distance below the sea bed, anchorages are required to withstand the uplift on tension piles as described in [Section 6.2.4](#).

### 8.1.3 Wave forces on piles

Jetties are normally sited in sheltered waters or in locations selected as not being subject to severe storm waves.

Consequently the forces on piles due to wave action are considerably less severe than those caused by the impact from berthing or the pull from mooring ropes. Also, berthing operations are not expected to take place when heavy wave action is occurring. Therefore it is the usual practice to disregard wave forces on piles forming the berthing head of a jetty and any associated independent

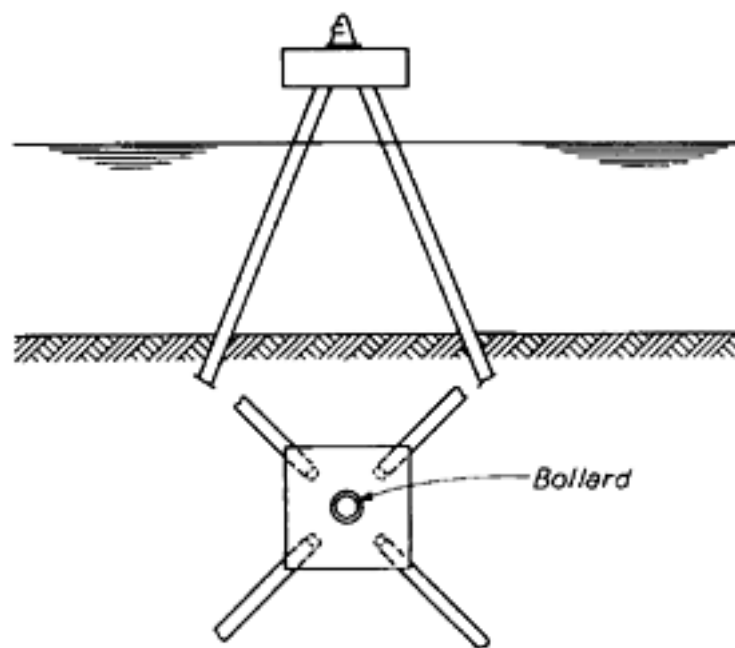


Fig. 8.10 Mooring dolphin with piles raked in two directions

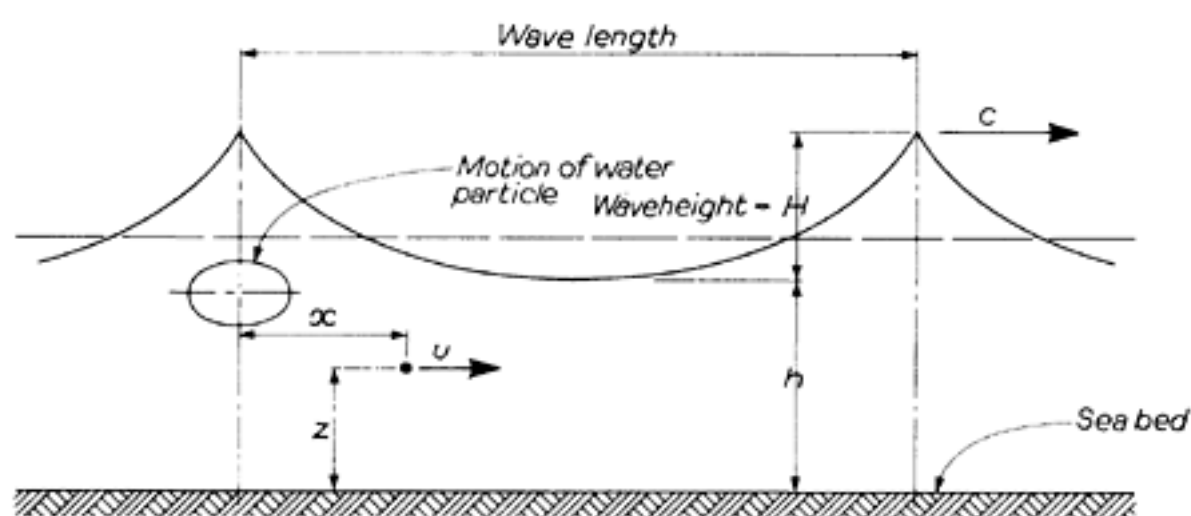


Fig. 8.11 Shape of breaking wave

dolphin structures where these are sited in sheltered waters. However, in the case of island berthing structures for large vessels, which are sited in deep and relatively unsheltered waters, the wave forces may represent a significant proportion of the total force required to be calculated. Also, piles supporting the approach trestle to a jetty are not required to withstand berthing impact forces. Thus wave forces, even in fairly sheltered waters, when combined with wind pressures on the superstructure and current drag on the piles, may produce substantial loading transverse to the axis of the trestle.

A simple approach to the calculation of wave forces on fixed structures is to assume that the maximum wave force can be expressed as the equivalent static force caused by a solitary wave of the shape shown in [Figure 8.11](#). This shape is representative of a breaking wave. An oscillatory wave has a different shape but the factors given in [Figure 8.12](#) and [Table 8.1](#) for use with equations 8.8 and 8.9 are applicable only to breaking wave conditions. Drag and inertial forces are exerted on the structure by the water particles which move in an elliptical path as shown. From the work of Wiegel *et al.* ([8.4](#)), Reid and Bretschneider([8.5](#)), Dailey and Stephen([8.6](#)), and Bretschneider([8.7](#)), it is possible to calculate the water particle velocity  $u$  at any point having co-ordinates  $x$  horizontally from the wave crest and  $z$  vertically above the sea-bed. The water particle velocity can be related to the velocity of advance of the wave crest (the wave celerity  $c$ ) and expressed in terms of  $(u/c)^2$  and  $\frac{1}{g} \times \frac{du}{dt}$  for various ratios of  $x$  and  $z$  to the height  $h$  of the trough of the wave above the sea bed.

The solitary-wave theory is limited in its application to a range of conditions defined by the ratio of the wave period to the water depth. Because the equations given below are applicable only to breaking wave conditions they represent the maximum force which can be applied to a structure. Breaking wave conditions are unlikely to occur in deep water berths for large tankers, and these conditions are likely to be found only in fairly shallow water on exposed jetty sites, for example along the line of the approach structure from the shore to a deep-water berth. However, as noted by Newmark([8.8](#)) the solitary-wave theory is often applied to situations beyond its strict range of validity for want of a better theory. For deep-water structures the solitary-wave theory gives *over-conservative* values of wave force. However, the following equations based on this theory together with the dimensionless graphs are simple and easy to use. It is suggested that the equations are used for all parts of a deep-water berthing-head

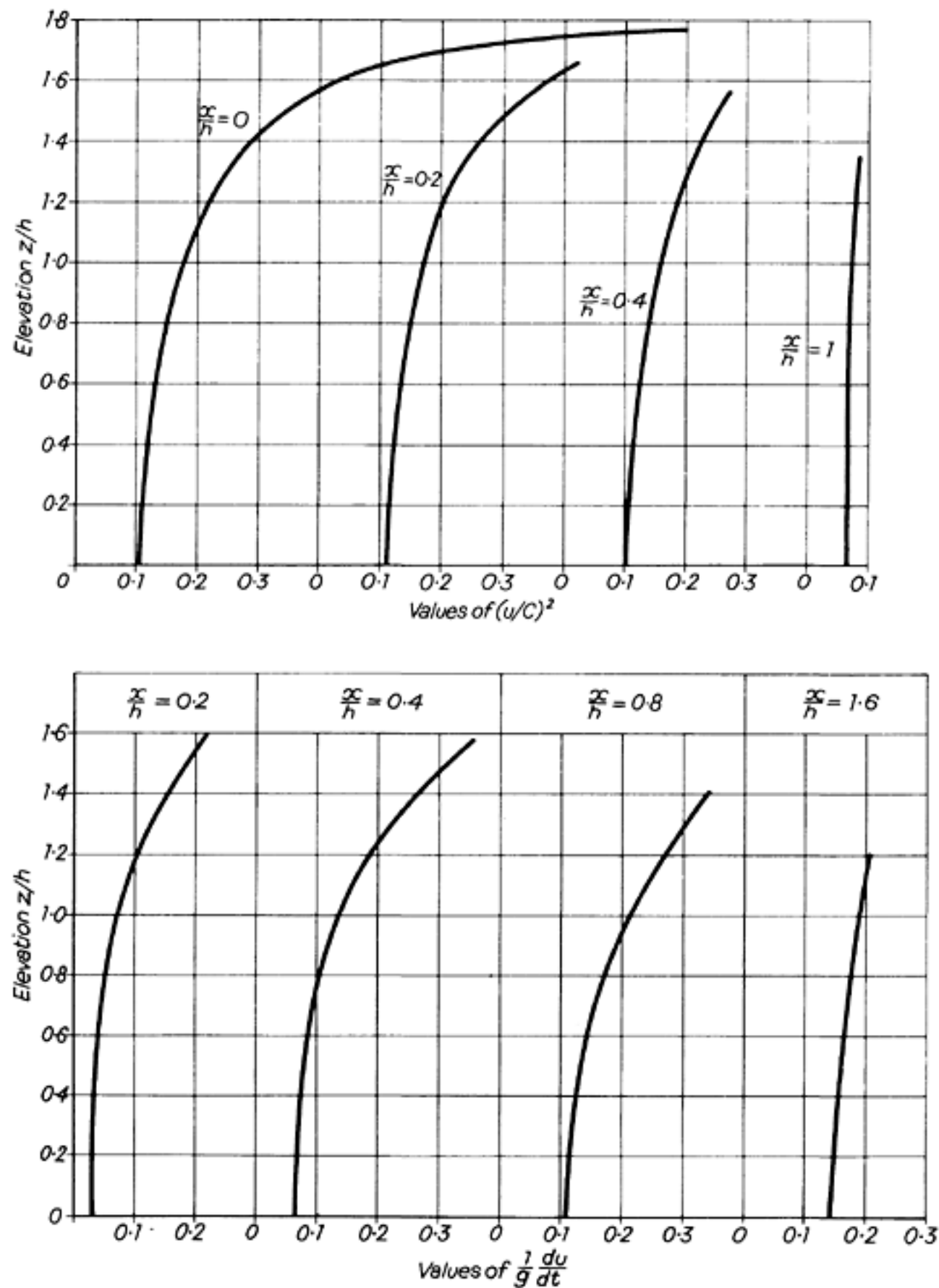


Fig. 8.12 Design curves for calculating velocity and acceleration of water particles in breaking wave

structure and for the shallow-water approach whenever it is necessary to calculate wave forces. If these forces together with current drag, wind forces and berthing impact forces do not produce excessive bending stresses on the piles then the calculations need not be further refined. It must be kept in mind that the cross-sectional area of a pile may be governed by considerations of corrosion and driving stress rather than the stress resulting from environmental forces. Where the wave forces calculated by the solitary-wave theory are a significant factor in the design of the piles more detailed calculations should be made taking into account the relationship between wave height, water depth and wave period. Methods of general application can be found in the publications of the US Army Coastal Engineering Research Centre<sup>(8.9)</sup>. In general wave theories, the wave force on a fixed structure is taken as the sum of the drag and inertial forces exerted by the wave. These are expressed by the commonly-used Morison equation<sup>(8.10)</sup>

$$f = f_D + f_I = C_D \frac{wu^2}{2g} + C_M \frac{w \pi D}{g} \frac{du}{dt} \quad \dots(8.7)$$

where  $f$ ,  $f_D$  and  $f_I$  are the wave force, drag force and inertial force, respectively, per unit area of object in the path of the

wave,  $C_D$  is a drag coefficient,  $w$  is the density of water,  $g$  is the gravitational acceleration,  $u$  is the horizontal particle velocity of water,  $C_M$  is a mass coefficient,  $D$  is the diameter of the cylindrical object, and  $du/dt$  is the horizontal acceleration of a water particle.

Newmark(8.8) reduced equation 8.7 to a simple expression given in lb-ft-sec units. By taking the weight of sea water as 64lb/ft<sup>3</sup> and the gravitational acceleration as 32.2ft/sec the equation becomes

$$f = f_D + f_I = \left[ 50C_D h \left( \frac{u}{c} \right)^2 + 50C_M D \cdot \frac{1}{g} \cdot \frac{du}{dt} \right] \text{lb/ft}^2 \quad \dots(8.8)$$

In SI units, equation 8.8 becomes

$$f = \left[ 7.8C_D h \left( \frac{u}{c} \right)^2 + 8C_M D \cdot \frac{1}{g} \cdot \frac{du}{dt} \right] \text{kN/m}^2 \quad \dots (8.9)$$

Values of  $\left( \frac{u}{c} \right)^2$  and  $\frac{1}{g} \cdot \frac{du}{dt}$  for different positions relative to the location of the wave crest are shown in [Figure 8.12](#) and [Table](#)

[8.1](#). This table also lists the average values of  $\left( \frac{u}{c} \right)^2$  and  $\frac{1}{g} \cdot \frac{du}{dt}$ , together with the heights to the centroid of the two components. The wave forces and moments applied to each increment of height of pile projecting above the scoured sea bed up to wave crest level, and on any underwater bracing or jacket members, are integrated to obtain the total horizontal force on the pile or group of piles and also the overturning moment about the point of fixity below the sea bed. The drag coefficient  $C_D$  is related to the Reynold's number which in turn varies with the water particle velocity and the diameter of the submerged object. For the practical design of jetty structures  $C_D$  can be taken as ranging from 0.5 to 1.0 for cylindrical members and up to 2.0 for rectangular, H, or I sections. Because of the many uncertain factors involved, Newmark (8.8) recommends a value of 0.5 for the drag coefficient  $C_D$  of cylindrical members, and 1.5 to 2.0 for the mass coefficient  $C_M$ . He also recommends that shielding effects produced by closely-spaced piles or bracing members should be disregarded when calculating wave forces. Barnacle growth on piles and bracings should be taken into account by allowing an appropriate increase in diameter. It has been reported(8.11) that marine growths more than 200mm in thickness have occurred around steel piles of the North Sea gas production platforms after about eight years of exposure. The growths extend down to sea bed where the water depths were about 25m. If drag forces due to marine growths are excessive, provision can be made for the members to be cleaned periodically by divers.

### 8.1.4 Current forces on piles

The velocities and directions of currents (or tidal streams) affecting the structure are obtained by on-site measurements which should include the determination of the variation in current velocity between the water surface and the sea bed. A curve is plotted relating the velocity to the depth and the current drag force is calculated for each increment of height of the pile above the sea bed. Any scour below the sea bed should be provided for.

Table 8.1 Surface elevations, velocities, and accelerations for solitary breaking wave

Distance from crest $x/h$	Surface elevation $z_s/h$	$\left( \frac{u}{c} \right)^2$					$\frac{1}{g} \left( \frac{du}{dt} \right)$				
		At surface	At $z=h$	At bottom	Average value	Height to centroid	At surface	At $z=h$	At bottom	Average value	Height to centroid
0	1.78	1.000	0.176	0.109	0.226	1.19	0	0	0	0	
0.2	1.67	0.430	0.170	0.106	0.181	1.03	0.242	0.073	0.031	0.081	1.14
0.4	1.57	0.276	0.156	0.099	0.150	0.92	0.347	0.137	0.060	0.133	1.02
0.6	1.48	0.201	0.133	0.092	0.123	0.83	0.380	0.184	0.087	0.164	0.93
0.8	1.41	0.138	0.106	0.078	0.097	0.80	0.357	0.214	0.110	0.180	0.88
1.0	1.35	0.092	0.082	0.070	0.077	0.70	0.321	0.225	0.127	0.186	0.78
1.2	1.29	0.062	0.063	0.058	0.061	0.65	0.280	0.225	0.140	0.187	0.73
1.4	1.25	0.041	0.046	0.048	0.047	0.61	0.243	0.209	0.146	0.182	0.68

1.6	1.21	0.029	0.032	0.038	0.035	0.59	0.209	0.192	0.148	0.173	0.65
1.8	1.18	0.020	0.023	0.029	0.027	0.56	0.174	0.171	0.145	0.159	0.62
2.2	1.13	0.009	0.011	0.018	0.014	0.50	0.122	0.128	0.130	0.130	0.57
2.6	1.08	0.004	0.005	0.009	0.007	0.50	0.088	0.091	0.109	0.102	0.53
3.0	1.05	0.002	0.002	0.004	0.003	0.50	0.065	0.067	0.084	0.078	0.51
3.4	1.03	0.001	0.001	0.002	0.002	0.50	0.049	0.049	0.062	0.058	0.50
5.0	1.01	0.000	0.000	0.000	0.000	0.50	0.012	0.012	0.017	0.016	0.50

Table 8.2 Values of drag coefficient  $C_D$  (after Evans and Adamchak<sup>(8.12)</sup>)

Type of member	Length/width ratio		
	1	5	$\infty$
Flat plate (perpendicular to flow)	1.16	1.20	1.90
Prism (axis perpendicular to flow) square	—	—	2.00
2:1 (long side parallel to flow)	—	—	1.50
Cylinder (axis perpendicular to flow)			
Reynold's number= $10^5$	0.63	0.74	1.20
Reynold's number= $5 \times 10^5$	—	0.35	0.33

Current forces are calculated from the equation

$$F_D = 0.5C_D\rho V^2A \quad \dots(8.10)$$

where  $F_D$  is the total drag force per unit length,  $C_D$  is a drag coefficient,  $\rho$  is the mass per unit volume of water,  $V$  is the current velocity and  $A$  is the projected area per unit length. Values of the coefficient  $C_D$  used to calculate the current drag forces are shown in [Table 8.2](#). The values for cylindrical members are related to the Reynold's number of the member. For water at normal temperatures the Reynold's number is given by

$$R_e = VD \times 10^5 \text{ in sec/ft}^{-2} \text{ units} \quad \dots(8.11)$$

or

$$R_e = 9.3VD \times 10^5 \text{ in sec/m}^{-2} \text{ units} \quad \dots(8.11a)$$

where  $D$  is the diameter of the member. If piles or other submerged members are placed in closely spaced groups, shielding of current forces in the lee of the leading member will occur. Shielding can be allowed for by modifying the drag coefficient. Values of the shielding coefficient have been established by Chappelaar<sup>(8.13)</sup>. Where currents are associated with waves it may be necessary to add the current velocity vectorially to the water-particle velocity  $u$  to arrive at the total force on a member. Also, the possibility of an increase in the effective diameter of a submerged member due to barnacle growth must be considered. Having calculated the current force on a pile it is necessary to check that oscillation will not take place as a result of vortex shedding induced by the current flow. This oscillation occurs transversely to the direction of current flow when the frequency of shedding pairs of vortices coincides with the natural frequency of the pile. The standard formula for calculating the natural frequency is

$$N = \frac{K}{L^2} \sqrt{\frac{EI}{M}} \quad \dots(8.12)$$

where  $N$  is the natural frequency,  $K$  is a constant,  $L$  is the pile length,  $E$  is the elastic modulus,  $I$  is the moment of inertia, and  $M$  is the effective mass per unit length of the pile.  $K$  is equal to 0.56, 2.45 and 3.56 respectively for cantilevered, propped and fully-fixed piles. The elastic modulus is expressed in units of force. In the case of a cylindrical pile the effective mass  $M$  is equal to the mass of the material of the pile plus the mass of the water displaced by the pile. Where hollow tubular piles are filled with water the mass of the enclosed water must be added to the mass of the material. In the case of a steel tubular pile with a relatively thin wall the effective mass is approximately equal to the mass of steel plus twice the mass of the displaced water. Oscillation can take place in line with the direction of flow when the frequency of shedding single vortices coincides with the natural frequency. Cross-flow motion can be expected when

$$V/ND = 5 \quad \dots(8.13)$$

and in-line motion when

$$V/ND = 2.5 \quad \dots(8.14)$$

where  $D$  is the diameter of the pile and  $V$  is the average velocity of flow within a depth of 12m from water surface. The most severe conditions can be expected before the piles are braced together or tied into the deck structure.

Very severe oscillations were experienced during the construction of the Immingham Oil Terminal([8.14](#), [8.15](#)). At this site in the Humber Estuary, piles were driven through water with a mean depth of 23m and where ebb currents reach a mean velocity of 2.6m/s (5 knots). The piles were helically-welded steel tubes with outside diameters of 610mm and 762mm and a wall thickness of 12.7mm. Before the piles could be braced together they developed a cross-flow motion which at times had an amplitude of  $\pm 1.2$ m. Many of the piles broke off at or above the sea bed. A completed dolphin consisting of

a cap block with a mass of 700 tonnes supported by 17 piles swayed with a frequency of 90 cycles per minute and an amplitude of  $\pm 6\text{mm}$ .

Extensive research into the motion was sponsored by the Construction Industry Research and Information Association<sup>(8.16)</sup>. Sainsbury and King<sup>(8.14)</sup> put forward tentative design criteria stating that the maximum current velocity which can be withstood by an unbraced pile structure is given by

$$V_{max} = 2.2 \sqrt{\frac{EID^2}{ML^4}} \quad \dots(8.15)$$

and the maximum velocity before breakage of a cantilevered pile occurs by cross-flow vibration is given by

$$V_{max} = 1.9 \sqrt{\frac{EID^2}{ML^4}} \quad \dots(8.16)$$

If the observed current velocity at the site exceeds either maxima in equations 8.15 and 8.16, then either the  $EI$  value must be increased or special measures must be taken to brace the piles to increase their natural frequency.

Moored ships can transmit forces due to current drag onto the piles supporting the mooring bollards. The current drag on the ship is calculated from equation 8.10.

### 8.1.5 Wind forces on piles

Wind forces exerted directly on piles in a jetty structure are likely to be small in relation to the quite substantial wind forces transmitted to the piles from deck beams, cranes, conveyors, stacked containers, sheds and pipe trunkways. In a jetty approach the combined wind and wave forces which usually act perpendicularly to the axis of the approach can cause large overturning moments on the pile bents, particularly when the wind forces are acting on pipe trunkways or conveyor structures placed at a high elevation, say at a location with a high tidal range. Wind forces on moored ships also require consideration, and allowance should be made where necessary for the accretion of ice on structures.

Wind forces can be calculated from equation 8.10 by taking the mass of air as  $1.29\text{g/l}$  or this equation can be conveniently expressed in Imperial units as

$$F = 0.00256 V^2 C_D A \quad \dots(8.17)$$

where  $F$  is the wind force in pounds,  $V$  is the sustained wind velocity in m.p.h. at the elevation of the portion of the structure under consideration,  $C_D$  is a drag coefficient, and  $A$  is the projected area of the object in square feet (including an allowance for ice accretion).

The values of the drag coefficient for use with equations 8.10 and 8.17 are as shown in [Table 8.2](#) and shielding coefficients [\(8.13\)](#) can be applied for closely-spaced members. Wind velocities can be corrected for height by means of the equation

$$V_2 = V_1 \left( \frac{H_2}{H_1} \right)^{1/7} \quad \dots(8.18)$$

where  $H_2$  and  $H_1$  are the two elevations concerned. It should be noted that wind velocities based on short-duration gusts may be over conservative when considering wind forces on large ships.

### 8.1.6 Forces on piles from floating ice

Forces on piles from floating ice are somewhat similar in character to the impact forces from berthing ships in that the ice is transported by winds or currents and is brought to rest by the piles. However it is usual to express the force in terms of an empirical static formula. The American Petroleum Institute<sup>(8.17)</sup> gives the equation

$$F = C f_c A \quad \dots(8.19)$$

where  $F$  is the total force due to the ice striking an object,  $C$  is a coefficient,  $f_c$  is the compressive strength of the ice, and  $A$  is

the area struck by the ice.

The compressive strength of ice varies with the temperature, salt content and rate of application of the load, but strengths in the range of 1.4 to 3.4N/mm<sup>2</sup> are quoted. The coefficient  $C$  depends on the shapes of the ice floe and the object struck, and on the speed of striking. The range quoted is from 0.3 to 0.7([8.17](#)).

The Institute of Applied Hydraulics and Coastal Engineering Laboratory of Denmark has considered

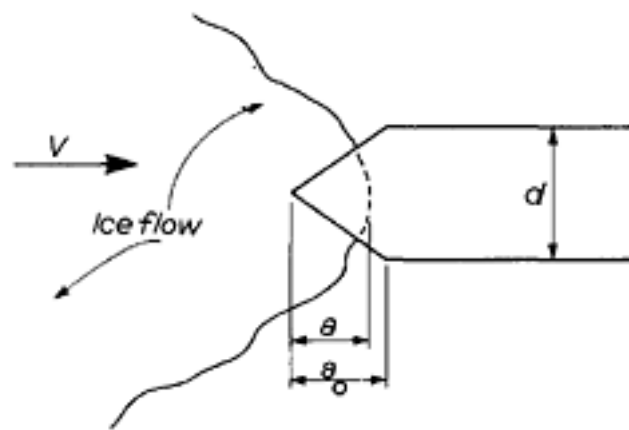


Fig. 8.13 Penetration by fixed object of moving ice floe

the problem in dynamic terms. Its equation as stated by Tryde<sup>(8.18)</sup> for the maximum force is in a form similar to equation 8.19, namely

$$F_{max} = N_c c t d \quad \dots(8.20)$$

where  $F_{max}$  is the maximum force,  $N_c$  is a dimensionless rupture factor,  $c$  is the shearing strength of the ice,  $t$  is the thickness of the ice, and  $d$  is the width of an object struck by the ice (Figure 8.13).

Tryde does not give values for  $N_c$ , but if the shearing strength of ice is taken as one-half of the compressive strength then by comparison with equation 8.19,  $N_c$  ranges from 0.6 to 1.4.

The actual force is related to the velocity of impact thus.

$$\text{Actual force } F_a = N_c c t \left( \frac{a}{a_0} \right) d \quad \dots(8.21)$$

$$\text{where } \left( \frac{a}{a_0} \right)^2 = \frac{V^3 \gamma_c}{g N_c} \times \frac{A}{d a_0} \quad \dots(8.22)$$

in which  $\gamma_c$  is the unit weight of ice,  $V$  is the velocity of approach of the ice floe,  $A$  is the area of the ice floe, and  $a/a_0$  is a reduction factor related to the penetration of the floe by an object, as shown in Figure 8.13.

An ice sheet can also cause pressure on a piled structure as a result of the current drag on the ice surrounding the structure. Changes in water level can cause uplift or down-drag on the piles. The maximum force is limited by the buckling of the ice sheet, which depends on the thickness and compressive strength of the ice. It is usual to base design pressures on local experience obtained by measurements on dams and other fixed structures. Evans and Adamchak<sup>(8.12)</sup> have quoted thicknesses of the North Polar ice cap of 2.7m in the summer and 4m in the winter, with occasional pressure ridges 27 to 30m below the surface.

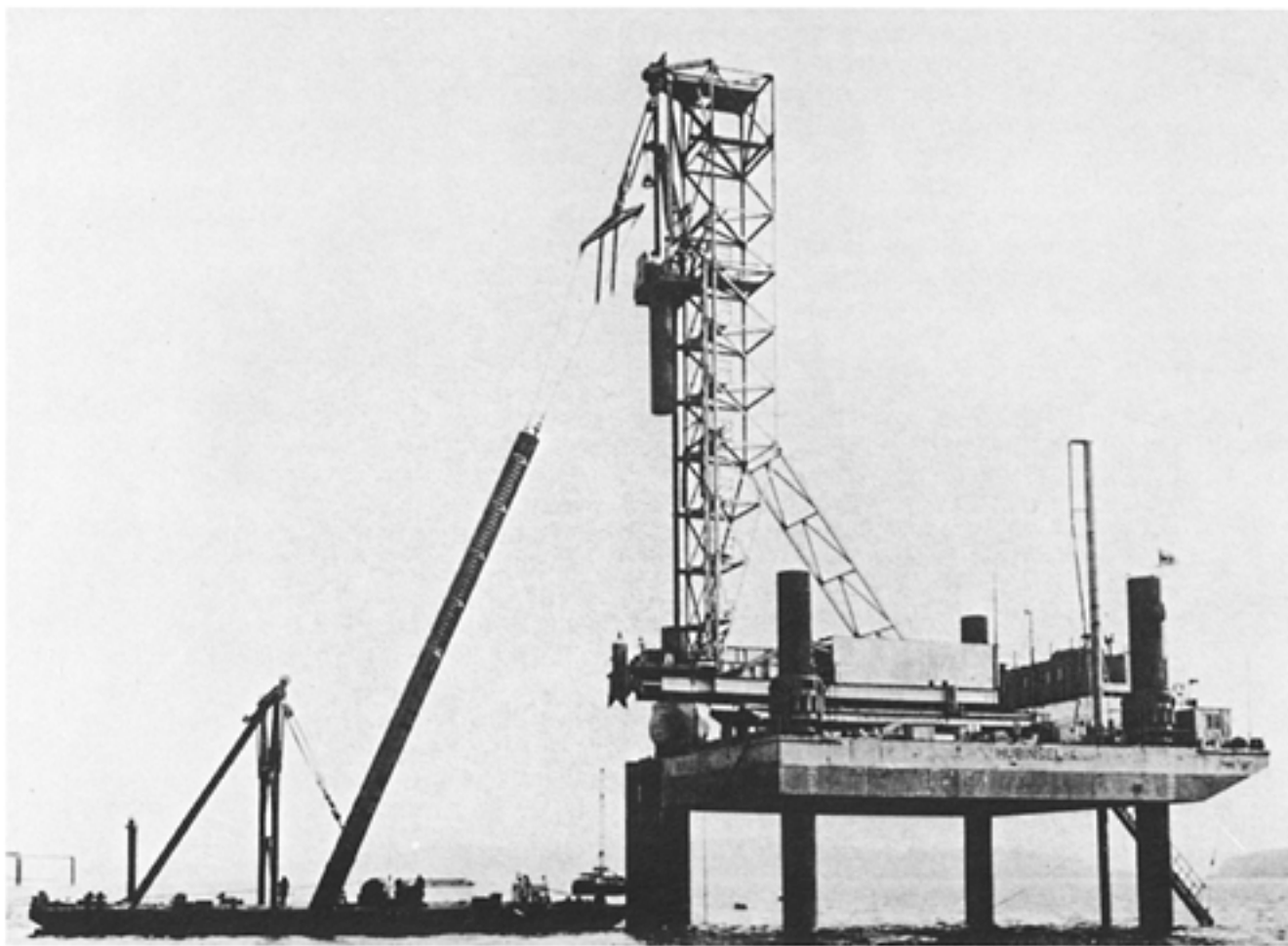
The development of offshore petroleum production facilities in the Arctic regions has stimulated research on the effects of ice on fixed structures. Reference should be made to the proceedings of the various international conferences on offshore technology for the present state-of-the-art.

### 8.1.7 Materials for piles in jetties and dolphins

For jetties serving vessels of light to moderate displacement tonnage and of shallow draught, timber is the ideal material for fender piles. It is light and resilient and easy to replace. As already noted the face of a timber fender pile can be protected by a renewable timber rubbing strip. The type of timber used for fender piles is governed by considerations of the attack by organisms present in the sea water. Suitable types of timber are described in Chapter 10.

For jetties and berthing structures in deep water serving large vessels, either steel or prestressed concrete tubular piles can be used. Steel piles have the advantage that they can withstand rough handling while being loaded onto barges and lifted into the leaders of the floating piling frame or jack-up platform (Figure 8.14). They can withstand hard driving to attain the penetration depths necessary to achieve the required uplift and lateral resistance. However, they require expensive cleaning and coating treatment above the soil line, supplemented by cathodic protection to enable them to resist corrosion in sea water. The types of steel suitable for piling in marine structure are discussed in Section 2.2.6.

Prestressed concrete piles also possess considerable resilience, but repair is a difficult problem if they are subjected to accidental heavy impact damage. Prestressed concrete piles are suitable for approach structures and for jetty heads protected by independent berthing structures. Problems of sea-water attack on steel and concrete structures are discussed in [Chapter 10](#).



*Fig. 8.14 Lifting steel tubular pile from delivery barge for pitching into guide frame. Piles were drilled down by grabbing in conjunction with casing oscillator*

## 8.2 Fixed offshore platforms

A fixed offshore platform for petroleum gas production, for a lighthouse, or for some other navigation structure, when supported by piles usually takes the form of a tubular jacket which is transported to the site on a barge or on flotation tanks. The jacket is lifted into a vertical position and sunk onto the sea bed. Piles are pitched through the vertical or inclined tubular legs or guides on the structure and driven down to the required penetration ([Figure 8.15](#)). Other types of platform structure include semi-buoyant structures tied down to sea-bed anchorages (tension leg platforms) and large-diameter floating caissons towed to the site and sunk onto the sea beds with or without support by piling. Piles may be needed to support the caisson or to restrain it against uplift when used for oil storage in shallow water. Because of their location in deep water exposed to severe wave action, the forces acting on fixed platform structures are different in character from those on jetties in relatively shallow and sheltered waters. Whereas in berthing structures the dominant forces are those caused by the berthing of ships, the offshore platform is served only by small vessels and the environmental forces resulting from waves, winds and currents have a dominating influence on design. In very deep water, the environmental forces can account for three-quarters of the total load on a main supporting member.

The economics in the design and construction of offshore platforms for petroleum and gas production are viewed from a standpoint very different from that applied to jetty design. In the latter case the main requirements are low capital cost, ease of maintenance and a long life. The time required for construction is not usually a critical factor in design. However only a limited life is required from oil and gas production platforms but assurance of stability in the most severe exposure conditions is of vital importance, and rapidity of installation at sea is essential. This is because of the limited periods during which the state of the sea will permit the operation of large floating cranes and other constructional

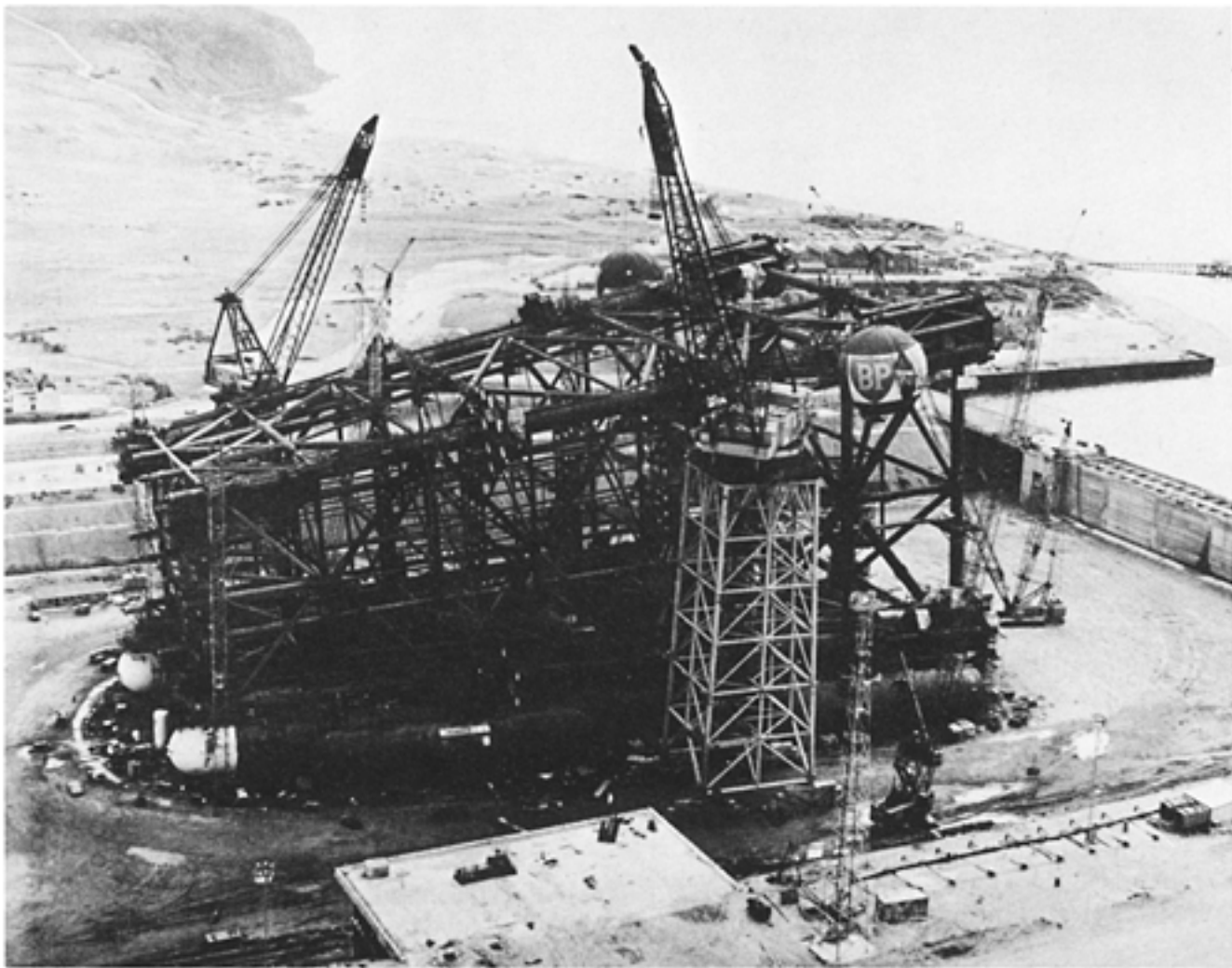


Fig. 8.15 Fabrication of platform for Forties Field (North Sea) of British Petroleum Co., showing guides for clusters of piles around each leg

Table 8.3 Minimum safety factors for various loading conditions

Loading condition	Minimum safety factor
1. Design environmental conditions with appropriate drilling loads	1.5
2. Operating environmental conditions during drilling operations	2.0
3. Design environmental conditions with appropriate producing loads	1.5
4. Operating environmental conditions during producing operations	2.0
5. Design environmental conditions with minimum loads	1.5

plant. The assembly of constructional plant can cost tens of thousands of pounds a day, and the risks of leaving a partly-constructed platform to the mercy of winter gales cannot be contemplated, quite apart from the costs of any delay in the production of gas or petroleum.

Fixed platforms must be designed to be stable under the most severe combination of predicted maximum wave wind and current forces, and to allow drilling or other production operations to proceed safely in certain specified storm conditions. Certifying authorities usually demand a specific safety factor for a 100-year wave combined with the corresponding wind force and maximum current velocity, referred to as the *design* environmental conditions. The maximum forces due to operations on the platform such as drilling are combined with specified wind and sea conditions, and are known as the *operating* environ-

mental conditions. The American Petroleum Institute<sup>(8.17)</sup> requires the safety factors on the ultimate bearing capacity of piled foundations not to be less than the minima given in [Table 8.3](#).

The continuing world-wide search for petroleum deposits offshore has led to the design of platforms at locations of ever-increasing depths of water and increasingly hostile environments. Design and construction methods are continually changing. Because of this information on design and construction practices is not included in this chapter. The reader is referred to design and construction recommendations in the current publications of the American Petroleum Institute<sup>(8.17,8.19)</sup> and the UK Department of Energy<sup>(6.6)</sup>. Construction methods have been described in detail by Gerwick<sup>(8.20)</sup>.

### 8.3 Pile installations for marine structures

Where marine structures are connected to the shore, as in the case of a jetty head with a trestle approach, the piles may be driven either as an ‘end-on’ operation with the piling equipment mounted on girders cantilevering from the completed pile bents, or as an operation from a floating or jack-up barge. In tidal waters there is usually sufficient water depth to float a barge with a draft of to ~~1m~~ to a location close inshore. However this can be inconvenient where tidal flats or saltings cover a long depth of the approach or where it is unsafe to ground the barge on the sea bed at low water.

Where the ‘end-on’ method is used the spacing between pile bents is limited by the ability of the girders to cantilever when carrying the weight of the piling frame, hammer and suspended pile. Loading can be minimized by utilizing the buoyancy of tubular piles with permanently or temporarily closed ends, or by using trestle guides of the types shown in [Figures 3.7](#) and [3.8](#) in conjunction with a pile-mounted hammer and a crane barge for lifting and pitching the piles.

Piling barges for deep-water locations range in length from about 60 to 120m with a width of one-third to one-half of the length and an overall depth of 1/12 to 1/15 of the length. Adequate depth is necessary to provide sufficient strength for towing the barge to the site from a distant location, and to give sufficient freeboard for safe operation when moored at the work site. These barges are normally self-contained with accommodation for the barge and rig crew.

Jack-up barges operate most efficiently when provided with mechanically adjustable pile guides installed either by cantilevering from the side of the barge as shown in [Figure 3.8](#) or spanning a ‘moonpool’ inset in the barge hull.

If possible, piles should be driven to their full design penetration without the need to weld-on additional pile lengths, to drive insert piles, or to clean out the soil plug or drill below the initial refusal level of an open-ended tubular pile. Gerwick<sup>(8.20)</sup> gave an example of times required for welding add-on lengths of 1.37m OD tubular piles; they varied from ~~31~~ hours for 25mm wall thickness to ~~10~~ 10 hours for 64mm thickness. Such delays cause increased driving resistance due to ‘take-up’ (*i.e.* the increase of skin friction). However, there are many situations where piles cannot be driven to their full penetration without the need for lengthening or for ‘drilling-and-driving’ techniques.

Cleaning out the soil plug is an effective way of reducing the driving resistance, thus obtaining deep penetration, because of the elimination of base resistance. It is particularly advantageous for obtaining deep penetration into granular soils, say to develop uplift resistance, to avoid excessive settlement due to vibration effects, or to reach rockhead. This is because the base resistance in a granular soil represents the major proportion of the total resistance to the driving of the pile. Removal of the soil plug is not particularly effective for piles penetrating deeply into clays where the base resistance is only a very small proportion of the total resistance. Drilling out the soil within the pile does not reduce the external skin friction of the surrounding clay.

Sullavan and Ehlers<sup>(8.21)</sup> have described the dangers of uncontrolled jetting below the toe level of tubular piles. This can have a weakening effect both on skin friction and end resistance. They recommend controlled rotary drilling with bentonite mud to stabilize the hole. In clays, the diameter of the hole drilled below the pile toe level should not be greater than 75% of the outside diameter of the pile and at least 150mm less than the pile diameter. They recommend drilling to about 50% of the pile diameter in sands. To obtain the full design base resistance the pilot hole should be stopped 3 to 4.5m above the design penetration. For vertical piles the maximum depth of pilot hole should not exceed 12 to 15m above the design penetration, or about 7.5m with raking piles. Centralizers are required to keep the drilling pipes in line with the pile axis. A Calweld drill was used to clean out the soil from within the 2 000mm and 2 200mm outside diameter steel tubular piles used for the breasting and mooring dolphins of The British Petroleum Company’s tanker terminal in the Firth of Forth. To install the vertical piles a rotary table and guide frame were mounted on top of the pile as shown in [Figure 8.16](#). A full-face drilling bit was used and the cuttings were removed by air-water reverse circulation up the 200mm drilling pipe. The drill bit was maintained in correct alignment by centralizers and a

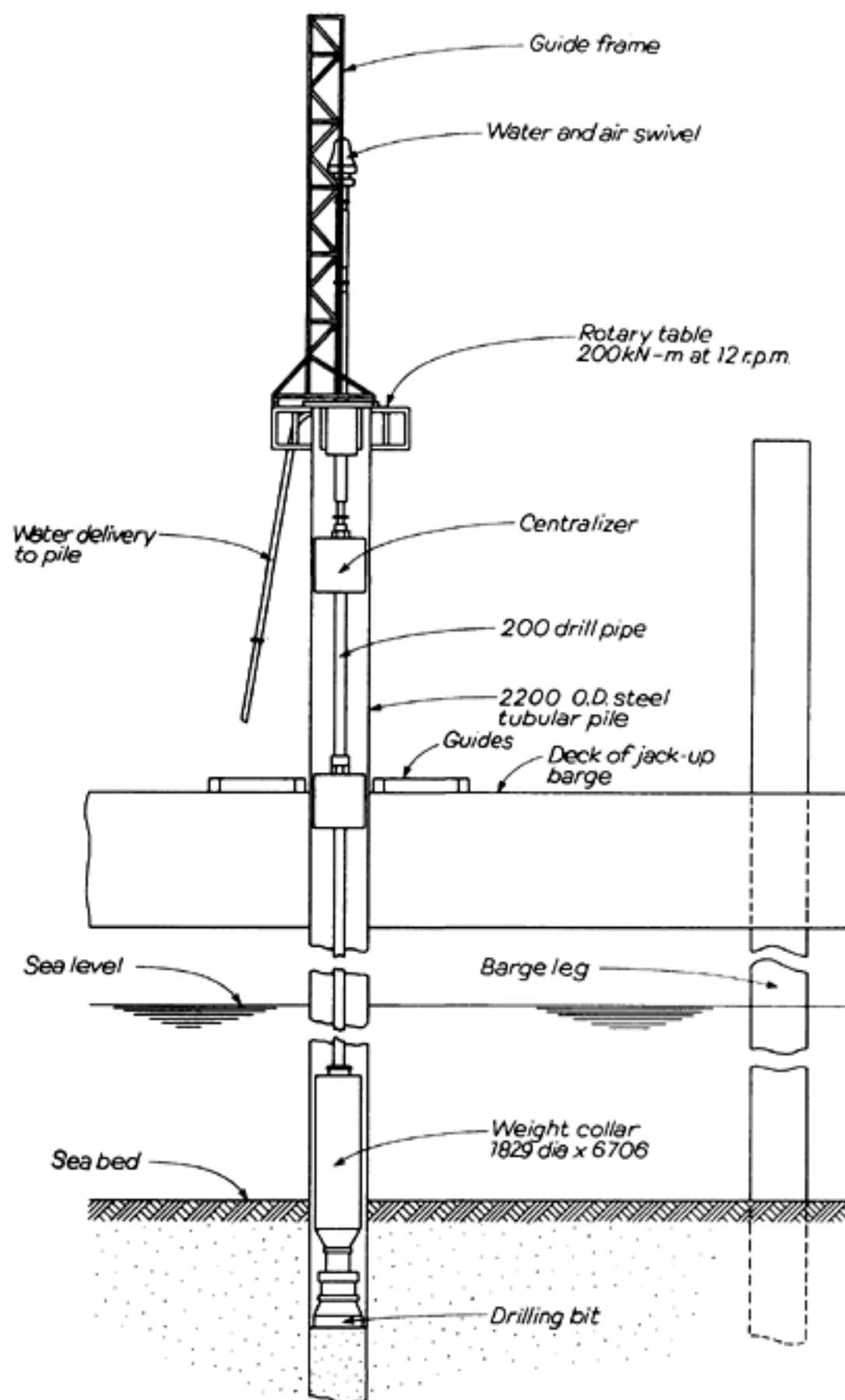


Fig. 8.16 Drilling soil from within 2200mm O.D. piles using reverse-circulation drill (Forth Tanker Terminal, The British Petroleum Co.)

heavy collar was provided to maintain pressure on the bit. Drilling was continued into rockhead, and followed by the separate operation of drilling-in a 560mm steel tubular dead anchor to a depth of 15m into the rock to provide an uplift resistance of 7.4MN. It is evident from [Figure 8.16](#) that the numbers of heavy components required to be assembled for drilling out the soil from within and below large diameter piles are considerably greater than those required for the simple operation of driving the pile by hammer. Space is limited on the deck of a construction barge, and where floating vessels are used the heavy equipment may need to be secured to the deck by bolting or chain tackle. Hence the successive operations of driving the pile to refusal, removing the hammer, assembling the drilling gear, then drilling and removing the equipment can be very protracted. Therefore, if 'drill and drive' operations are required the aim should be to restrict the drilling phase to only one operation.

Insert piles can be used where piles driven to their full design penetration fail to attain a satisfactory resistance, or where 'drilling and driving' techniques are unable to achieve the required penetration.

Where insert piles are used, or where single piles are driven within the tubular guides of a jacket, the transfer of load from the insert pile to the main pile, and from the main pile to the leg, is made by welded joints at the pile heads or by grouting the annular space between the members. Both methods can be used together. The grout is prevented from flowing out from between the bottom of the jacket leg and the pile by means of cement in bags packed into the space by a diver, or by providing packers

(wiper seals) on the legs. The design of the grout bond from the pile to the jacket or between piles is described in 6.2.5. The need for terminating an insert pile at the head of the exterior pile or jacket guide can be avoided, in the case of large-diameter sections by driving the insert pile with a 'slim-line' underwater hydraulic hammer.

Instead of relying on the bond stress between pile and grout, mechanical keying devices of the type described in 6.2.5 can be used. They may be essential to transfer the load from the legs of deep-water platforms to large-diameter piles where the large thickness of the annulus is of some significance concerning the development of sufficient bond strength between grout and steel. The shrinkage of a grout rich in cement can be quite significant within an annulus that is, say, 75 to 100mm thick, and it has a weakening effect on the grout bond. In these conditions the development of an allowable bond stress even in the lower range recommended by the American Petroleum Institute may be impossible to achieve, and shear keys on both pile and sleeve are necessary to provide the means of transferring the load through a grouted annulus. Shear keys on the inner surface of a raking sleeve may prevent the pile from being lowered through the sleeve but they are unlikely to cause an obstruction when used in a vertical sleeve and pile.

The alternative to adopting insert piles or 'drilling and driving' techniques to mobilize compressive or uplift resistance in stiff to hard clays, is to provide an enlarged base to the piles. This can be achieved by using a rotary under-reaming tool operating below the toe of an open-ended steel tubular pile. The enlarged base provides both increased resistance to compressive loads and a positive anchorage against uplift. The uncertainty concerning the ability of available hammers to drive straight-sided piles to a deep penetration is avoided, and the time required to operate an under-reaming tool, to clean out the soil, to place the reinforcement and to fill the base with concrete, is not much greater than that required to drive one or more insert piles. However, there can be difficult problems when the arms of the expanding cutter fail to retract.

When open-end piles are driven into deep granular soil deposits the driving resistance may be very low for the reasons described in [Section 4.3.3](#). As a result, calculations of resistance to axial com

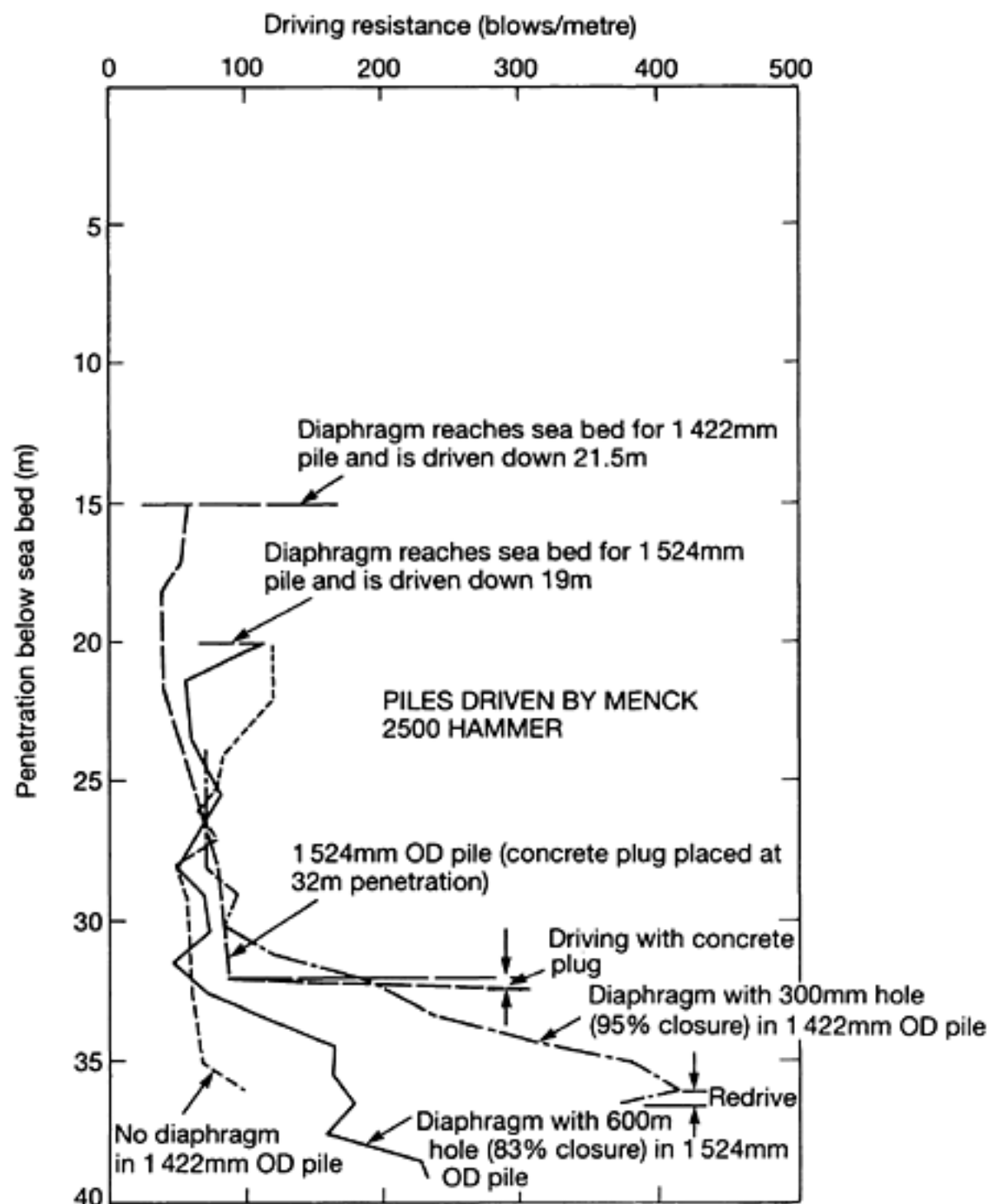


Fig. 8.17 The effects of different methods of plugging steel tubular piles driven with open ends, Hadera coal unloading terminal

pression loads based on dynamic testing are correspondingly low, indicating very deep penetration of the pile to achieve the required resistance. These penetrations are often much greater than those required for fixity against lateral loading. Although base resistance to axial loading can be achieved by grouting beneath the pile toe as described in [Section 3.3.6](#), the operations of cleaning-out the pile and grouting are slow and relatively costly. An alternative method of developing base resistance of open-end piles which has been used on a number of marine projects is to weld a steel plate diaphragm across the interior of the pile. The minimum depth above the pile toe for locating the diaphragm is the penetration below sea bed required for fixity against lateral loading. However a further penetration is necessary to compact the soil within the plug and to develop the necessary base resistance. It is not possible to achieve a resistance equivalent to a solid-end pile but the penetration depths are much shorter than those required for an open-end pile.

The diaphragm method was used for the piling at the Hadera coal unloading terminal near Haifa([8.22](#)). Open-end piles 1424 and 1524mm OD were proposed but initial trial driving showed that very deep penetrations, as much as 70m below sea bed in calcareous sands, would be needed to develop the required axial resistance. The blow count diagram in [Figure 8.17](#) showed quite low resistance at 36m below sea bed. Another trial pile was driven to 32m, cleaned-out, and plugged at the toe with concrete. An acceptable driving resistance of about 300 blows per metre was obtained by driving the plugged pile but it was appreciated that the plugging operations would be costly and would seriously delay com

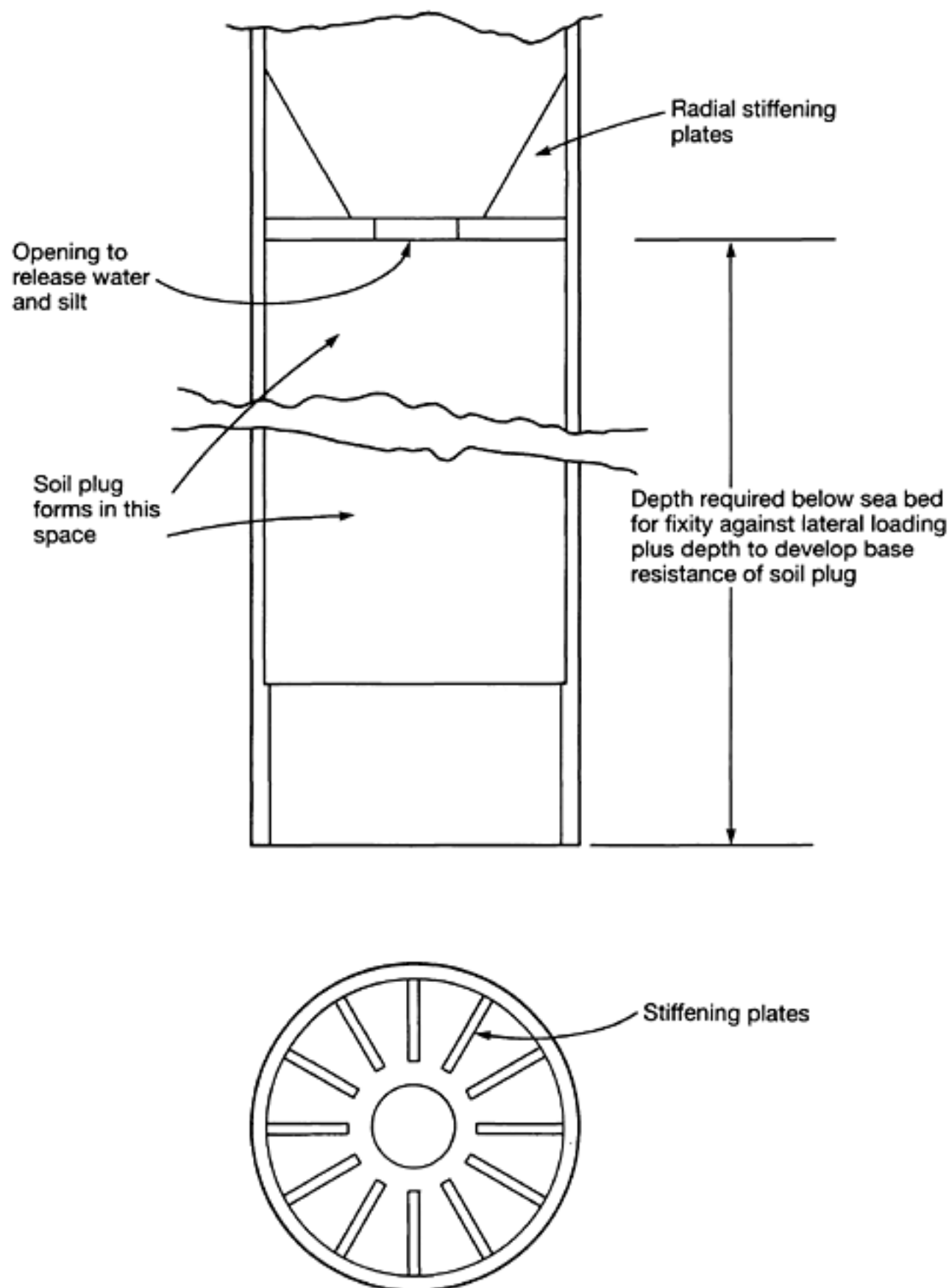


Fig. 8.18 Internal diaphragm for tubular steel pile

pletion of the project. Trials were then made of the diaphragm method. A diaphragm with a 600m hole giving 83% closure of the cross-section was inserted 20m above the toe. This increased the driving resistance at 39m below sea bed and another trial with a 300m hole (95% closure) gave a higher resistance at 37m ([Figure 8.17](#)).

The diaphragm method is ineffective if a very deep penetration is required because the long plug cannot compress sufficiently to mobilize the end-bearing resistance of the diaphragm and settlements at the working load would be excessive. It is also ineffective in clays or where clays are overlying the granular bearing stratum. A hole is necessary in the diaphragm for release of water pressure in the soil plug and to allow expulsion of silt. Stresses on the underside of the diaphragm are high during driving and radial stiffeners are needed ([Figure 8.18](#)). The pile wall below the diaphragm must be sufficiently thick to prevent bursting by circumferential stresses induced by compression of the soil in the plug.

## 8.4 References

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## 8.5 Worked examples

### Example 8.1

A breasting dolphin is constructed by linking at the head four 350×350mm reinforced concrete piles of the same design as used in Example 6.5. Find the kinetic energy which can be absorbed by the pile group for an impact at a point 8m above the

sea bed. The maximum energy absorption value is to be taken as the figure which stresses the piles to their yield point. The piles are driven to a total penetration depth of 9.0m in the same soil conditions as described in Example 6.5. The piles can be considered as fixed at the surface of the stiff clay stratum, and the ultimate resistance moment of each pile at the yield point is 125kN m. Therefore from equation 8.4, work done in deflecting piles to yield point

$$= \frac{4 \times 125^2 (8 + 2.5)}{6 \times 26 \times 10^6 \times 0.0833 \times 0.35^4} = 3.37 \text{kJ (kN m)}.$$

### Example 8.2

A steel tubular pile having an outside diameter of 1300mm and a wall thickness of 30mm forms part of a pile group in a breasting dolphin. The pile is fabricated from high-tensile alloy steel to BS 4360 Grade 55c. The piles are driven into a stiff over-consolidated clay ( $c_u = 150\text{kN/m}^2$ ). Calculate the maximum cyclic force which can be applied to the pile at a point 26m above the sea bed at the stage when the failure in the soil occurs at sea-bed level, the deflection of the pile head at this point, and the corresponding energy absorption value of the pile.

Steel to Grade 55c should have a minimum yield strength of  $417\text{N/mm}^2$  and an elastic modulus of  $2 \times 10^5\text{MN/m}^2$ .

$$\text{Moment of inertia of pile} = \pi(1.30^4 - 1.24^4)/64 = 0.024\text{m}^4.$$

$$\text{Moment of resistance of pile at yield point} = \frac{417 \times 0.024}{0.65} = 15.4\text{MN m}.$$

The first step is to establish the  $p$ - $y$  curves. In equations 6.66 and 6.67, the submerged density of the soil is  $1.2\text{Mg/m}^3$ , and a value of 0.25 can be taken for the factor  $J$ .

*At sea-bed level*

$$\text{Critical depth } x_r = \frac{6 \times 1.3}{\frac{1.2 \times 9.81 \times 1.3}{150} + 0.25} = 22.1\text{m}$$

$$N_c = 3 + 0 + 0 = 3.$$

$$p_u = 3 \times 150 \times 1.3 = 585\text{kN per m depth}.$$

For cyclically applied loading take,

$$p_b = 0.72p_u = 0.72 \times 585 = 421\text{kN per m depth}.$$

In the absence of laboratory compression tests, the appropriate value of  $\epsilon_c$  in equation 6.64 can be taken as 0.01, and the  $p$ - $y$  curves will be derived in the same manner as for a normally consolidated clay. Therefore,

$$y_c = 2.5 \times 0.01 \times 1.3 = 0.0325\text{m} = 32.5\text{mm}.$$

The deflection corresponding to  $p_b$  is  $3y_c = 3 \times 32.5 = 97\text{mm}$

Other points of the  $p$ - $y$  curve are calculated from equation 6.68. Thus for  $y = 15\text{mm}$

$$p = 0.5 \times 585 \times \sqrt[3]{\frac{15}{32.5}} = 226\text{kN per m depth}.$$

Similarly for  $y = 25\text{mm}$ ,  $p = 268\text{kN per m depth}$

$$y = 50\text{mm}, p = 338\text{kN per m depth}$$

$$y = 75\text{mm}, p = 386\text{kN per m depth}$$

Beyond the critical point at  $3y_c$ , the  $p$ - $y$  curve decreases linearly from  $p_b = 0.72p_u$  to zero at  $x/x_r = 0$

The  $p$ - $y$  curve at sea-bed level for the six points established above is shown in [Figure 8.19a](#).

*At 0.5m below sea bed*

$$N_c = 3 + \frac{1.2 \times 9.81 \times 0.5}{150} + \frac{0.25 \times 0.5}{1.3} = 3.13$$

$$p_u = 3.13 \times 150 \times 1.3 = 610 \text{ kN per m depth.}$$

$$p_b = 0.72 \times 610 = 439 \text{ kN/m at } y = 97 \text{ mm.}$$

For  $y = 15 \text{ mm}$ ,  $p = 610 \times 0.5 \sqrt[3]{\frac{1.5}{32.5}} = 236 \text{ kN}$  per m depth. Similarly

$$y = 25 \text{ mm, } p = 280 \text{ kN per m depth;}$$

$$y = 50 \text{ mm, } p = 352 \text{ kN per m depth;}$$

$$y = 75 \text{ mm, } p = 403 \text{ kN per m depth.}$$

The  $p$ - $y$  curve falls linearly at  $15y_c = 487\text{mm}$  to a value of  $p = 0.72 \times 610 \times 0.5/22.1 = 10\text{kN/m}$

The  $p$ - $y$  curve for  $x = 0.5\text{m}$  is also plotted in [Figure 8.19a](#) and the curves for values of  $x$  of 1.0, 1.5, 2.0 and 2.5m below sea bed, established in a similar manner, are also shown on this figure.

The value of  $p = 421\text{kN/m}$  represents the pressure at which yielding of the soil at the sea bed occurs. Therefore,

$$\text{bending moment at sea-bed level } M_t = 26 \times 0.421 = 10.9\text{MN m.}$$

The deflections at various points below the sea bed are obtained from [Figures 6.40a](#) and [6.40b](#), taking as a first trial

$R = 3.78$ , corresponding to a  $k$  value from equation 6.8 of about  $24\text{MN/m}^2$ . Required penetration depth is

$3.5 \times 3.78 = 13.2$ , say 14m. Then

$$Z_{\max} = \frac{14}{3.78} = 3.7$$

From equation 6.60,  $M_A = 10.9\text{MNm}$

From equation 6.62,  $M_B = 0.421 \times 3.78 \times M_h = 1.6M_h\text{MNm}$

From equation 6.61,  $y_A = \frac{10.9 \times 3.78^2 \times 1000}{2 \times 10^5 \times 0.024} y_m = 32.4y_m\text{mm}$

From equation 6.63,  $y_B = \frac{0.421 \times 3.78^3 \times 1000}{2 \times 10^5 \times 0.024} y_h = 4.7y_h\text{mm}$

$x$ (m)	$Z = \frac{x}{R}$	$y_m$	$y_A = 32.4y_m$ (mm)	$y_h$	$y_B = 4.7y_h$ (mm)	$y_A + y_B = y$ (mm)
0	0	+1.0	+32.4	+1.40	+6.6	+39.0
0.5	0.13	+0.78	+25.3	+1.32	+6.2	+31.5
1.0	0.26	+0.63	+20.4	+1.15	+5.4	+25.8
1.5	0.40	+0.50	+16.2	+1.00	+4.7	+20.9
2.0	0.53	+0.40	+13.0	+0.90	+4.2	+17.2
2.5	0.66	+0.32	+10.4	+0.80	+3.8	+14.2

The above values of  $y$  are referred to the  $p$ - $y$  curves to obtain the corresponding values of  $p$  and hence to obtain  $E_s$  from equation 6.80, as tabulated below.

$x$ (m)	$y$ (mm)	$p$ (kN/m)	$p' = \frac{p}{1.3}$ (kN/m <sup>2</sup> )	$E_s = -p'/y$ (kN/m <sup>2</sup> /m)
0	39.0	320	246	6.3
0.5	31.5	310	238	7.6
1.0	25.8	295	227	8.8
1.5	20.9	290	223	10.7
2.0	17.2	285	219	12.7
2.5	14.2	280	215	15.1

The values of  $E_s$  are plotted against depth in [Figure 8.19b](#), from which an average constant value of  $E_s$  of  $8 \times 10^3\text{kN/m}^2/\text{m}$  is obtained. From equation 6.8,

$$R (\text{obtained}) = \sqrt[4]{\frac{2 \times 10^5 \times 0.024}{8}} = 4.9$$

This value of  $R$  (obtained) is plotted against  $R$  (tried) in [Figure 8.19c](#), from which a second trial value of  $R$  of 6.5 is taken.

This higher value requires a deeper penetration of the pile, *i.e.*  $L > 3.5 \times 6.5 = 22.75$ ; say 23m. Thus  $Z_{max} = 23/6.5 = 3.5$ , and from equation 6.61,

$$y_A = \frac{10.9 \times 6.5^2 \times 1\,000}{2 \times 10^5 \times 0.024} y_m = 95.9 y_m \text{ mm.}$$

From equation 6.63,

$$y_B = \frac{0.421 \times 6.5^3 \times 1000}{2 \times 10^5 \times 0.024} y_h = 24.1 y_h \text{ mm.}$$

From Figures 6.40a and 6.40b the computed deflections are as tabulated below.

$x$ (m)	$Z = \frac{x}{R}$	$y_m$	$y_A = 95.9 y_m$ (mm)	$y_h$	$p' = \frac{p}{1.3}$ (kN/m <sup>2</sup> )	$y_B = 24.1 y_h$ (mm)
0	0	+1.00	+95.9	+1.45	+34.9	130.8
0.5	0.08	+0.85	+81.5	+1.37	+33.0	114.5
1.0	1.15	+0.75	+71.9	+1.30	+31.3	103.2
1.5	0.23	+0.65	+62.3	+1.20	+28.9	91.2
2.0	0.31	+0.57	+54.7	+1.11	+26.7	81.4
2.5	0.38	+0.52	+49.9	+1.05	+25.3	75.2

From the  $p$ - $y$  curve

$x$ (m)	$y$ (mm)	$p$ (kN/m)	$y_A + y_B$ (mm)	$E_s = -p'/y$ (kN/m <sup>2</sup> /m)
0	130.8	385	296	2.3
0.5	114.5	420	323	2.8
1.0	103.2	455	350	3.4
1.5	91.2	470	362	4.0
2.0	81.4	470	362	4.4
2.5	75.2	470	362	4.8

From Figure 8.19b, the second trial value of  $E_s = 3.3 \times 10^3 \text{ kN/m}^2$ , and

$$R(\text{obtained}) = \sqrt[4]{\frac{2 \times 10^5 \times 0.024}{3.3}} = 6.2.$$

This is sufficiently close to the equality line for 6.5 to be accepted as the final value of  $R$  (see Figure 8.19c).

The deflection of the pile head at the loading for the critical value of  $H = 421 \text{ kN}$  for soil rupture is the sum of the following deflections (a) to (c).

(a) Deflection of pile considered as cantilever fixed at sea bed

$$= \frac{0.421 \times 26^3 \times 1000}{3 \times 2 \times 10^5 \times 0.024} = 514 \text{ mm.}$$

(b) Deflection of pile at sea bed due to soil compression (from table above)=130.8mm.

(c) Deflection of pile head due to slope of pile below sea bed.

This can be obtained from the difference of the deflections at the sea bed and 1.0m below the sea bed. From the above table the deflection at 1m below sea bed=103.2mm. Therefore, slope below sea bed =  $130.8 - 103.2 = 27.6 \text{ mm}$  in 1m. Thus, deflection at pile head =  $26 \times 27.6 = 718 \text{ mm}$

Total deflection at pile head =  $514 + 131 + 718 = 1363 \text{ mm}$

It is necessary to check the bending moments at and below the sea bed to ensure that the resistance moment of the pile section is not exceeded. From Figures 6.40a and 6.40b, for  $Z_{max} = 23/6.5 = 3.5$ ,

$x$ (m)	$Z = \frac{x}{R}$	$M_m$	$M_A = 10.9M_m$ (MNm)	$M_h$	$M = M_A + M_B$ (MNm)	$M_B = 0.421 \times 6.5M_h$ $= 2.74M_h$ (MNm)
0	0	+1.00	+10.9	0	0	+10.9
0.5	0.08	+0.98	+10.7	+0.10	+0.3	+11.0
1.0	1.15	+0.97	+10.6	+0.15	+0.4	+11.0
1.5	0.23	+0.95	+10.4	+0.20	+0.5	+10.9
2.0	0.31	+0.94	+10.2	+0.27	+0.7	+10.9
4.0	0.62	+0.85	+9.3	+0.40	+1.1	+10.4
8.0	1.23	+0.55	+6.0	+0.45	+1.2	+7.2

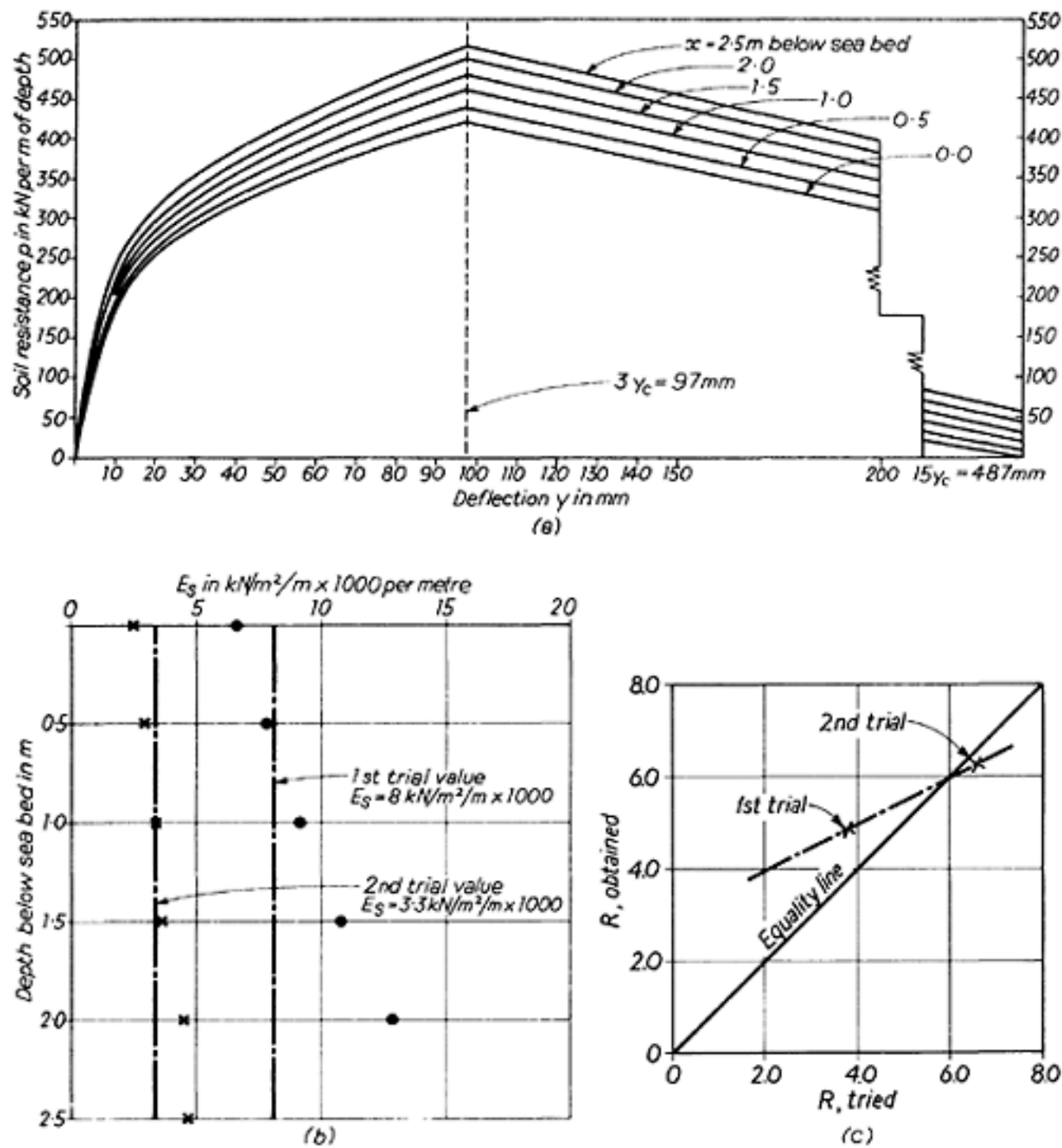


Fig. 8.19

The maximum bending moment of 11.0 MNm provides a safety factor of  $15.4/11.0 = 1.4$  against yielding of the steel. From equation 8.3, the kinetic energy absorption value of the pile for horizontal movement at the stage of soil rupture at sea-bed level

$$= \frac{1}{2} \times 421 \times 1363/1000 = 287 \text{ kJ (kN/m)}.$$

In a similar manner to that set out above, it is possible to obtain pile head deflections and bending moments for various stages of horizontal loading up to the stage of yielding of the steel and hence to draw curves of deflection and energy absorption against horizontal load. The horizontal load for soil rupture can be checked from Figure 6.30, in which  $e/B = 26/1.3 = 20$  and  $M_u/c_u B^2 = 11.0 \times 1000/(150 \times 1.3^2) = 43$ . This gives  $H_u/c_u B^2 = 1.7$ . Therefore,

$$H_u = 1.7 \times 150 \times 1.3^2 = 431 \text{ kN}.$$

This agrees closely with the calculated value of 421 kN.

The deflection of the pile at sea-bed level caused by a lateral force of 421 kN applied at the sea bed can be calculated using Randolph's curves (Section 6.3.8).

Effective Young's modulus of equivalent solid section pile

$$= E'_p = \frac{4 \times 2 \times 10^5 \times 0.024}{\pi \times 0.65^4} = 34.2 \times 10^3 \text{ MN/m}^2$$

An average constant soil modulus of  $3.3 \text{ MN/m}^2$  was used to calculate pile deflections and bending moments from the

Davisson and Gill curves. For undrained loading take Poisson's ratio  $\nu_u=0.5$ .

$$\text{Shear modulus} = G_c = \frac{3.3}{2(1 + 0.5)} = 1.1 \text{ MN/m}^2$$

$$G_s = 1.1(1 + 0.75 \times 0.5) = 1.5 \text{ MN/m}^2$$

$$\text{Critical length} = l_c = 2 \times 0.65 \left( \frac{34.2 \times 10^3}{1.5} \right)^{2/7} = 22.9 \text{ m}$$

$$\text{Homogeneity factor} = 1$$

In [Figure 6.54a](#),

$$\frac{y r_0 G_c \left( \frac{E_p'}{G_c} \right)^{1/7}}{H_0} = \frac{y \times 0.65 \times 1.5 \left( \frac{34.2 \times 10^3}{1.5} \right)^{1/7}}{0.421}$$

$$= 9.7y$$

At 0.5m below sea bed

$$z/l_c = 0.5/22.9 = 0.02 \text{ m}$$

$$\text{giving } 9.7y = 0.26, y = \frac{0.26 \times 10^3}{9.7} = 27 \text{ mm}$$

Using the Poulos method,

$$K_R = \frac{2 \times 10^5 \times 0.024}{3.3 \times 23^4} = 5.2 \times 10^{-3}$$

From [Figure 6.51a](#), for  $L/D = 23/1.3 = 17.7$ ,  $I_H = 5.5$

$$y_0 = \frac{5.5 \times 0.421 \times 10^3}{3.3 \times 23} = 30 \text{ mm}$$

### Example 8.3

A cross-section of an approach trestle giving roadway access to a cargo jetty is shown in [Figure 8.20](#). The trestle is sited at right-angles to the direction of maximum current velocity and travel of storm waves. The distribution of current velocity with depth is shown on the cross-section. The deck slab and other components of the superstructure impose a total horizontal wind force of 25kN on each pile

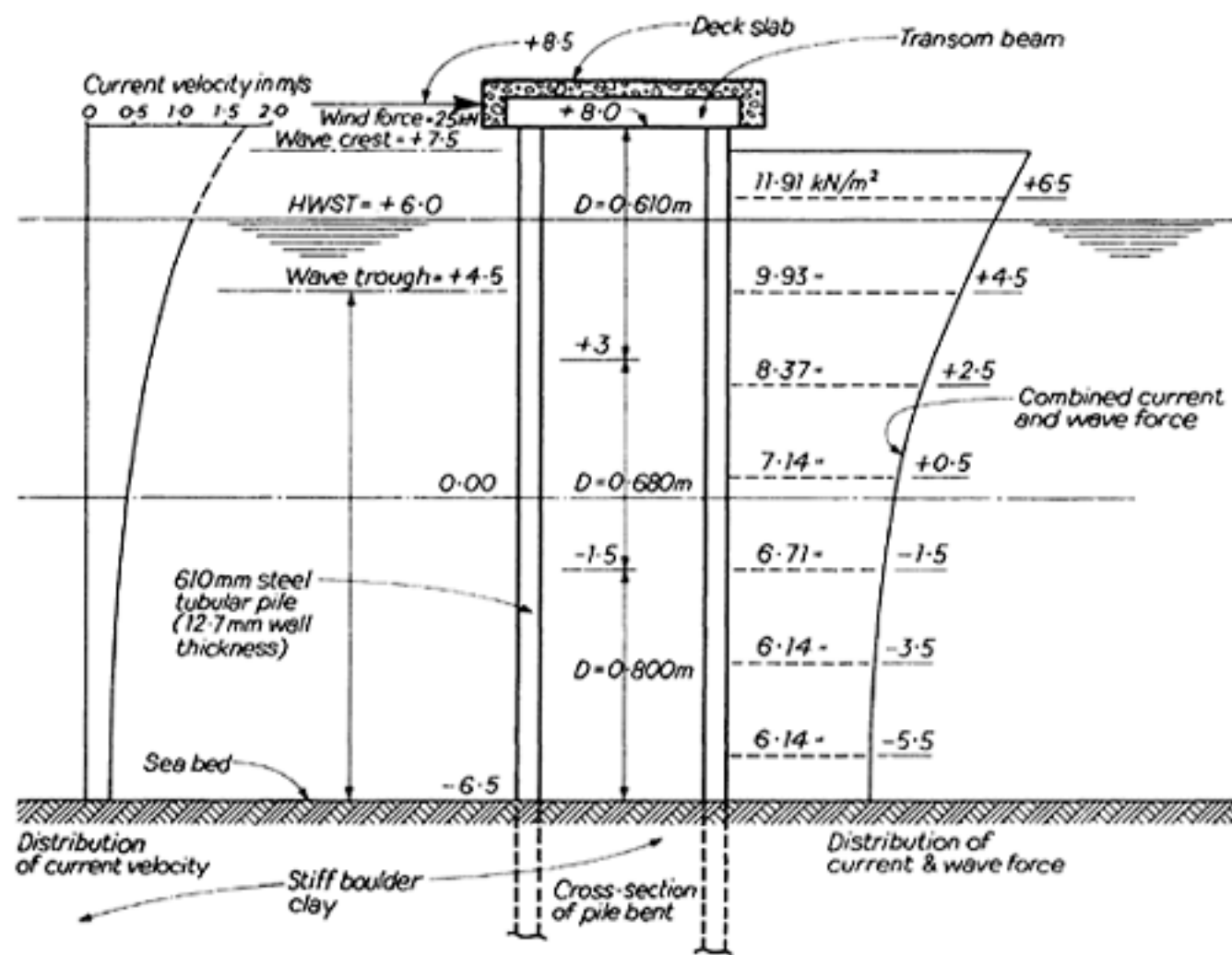


Fig. 8.20

bent. Storm waves have a maximum height from crest to trough of 3m. Determine the distribution of current and wave forces on the pile bent and calculate the bending moments on the piles produced by these forces. The maximum horizontal force on the piles will be due to the combined current and wave action at HWST (+6.0m). At this stage of the tide the storm wave crest will be at +7.5m. The underside of the transom beam is at +8.0m and therefore the wind force on the exposed length of pile from +7.5 to +8.0m will be relatively small and can be neglected. It is convenient to divide the length of the pile into 2m elements. Allowance is made for barnacle growth on the piles. Thus,

- From +7.5m to +3.0m: no increase of diameter (i.e.  $D=0.61\text{m}$ ).
- From +3.0m to -1.5m: increase of 70mm ( $D=0.68\text{m}$ ).
- From -1.5m to sea bed: increase of 190mm ( $D=0.80\text{m}$ ).

A drag force coefficient of 0.5 is used to calculate the current and wave drag forces, and an inertia coefficient of 2.0 is used to calculate the wave inertia forces. Thus in equation 8.10,

$$F_D = 0.5 \times 0.5 \times \rho \times V^2 \times A = 0.25V^2AkN \text{ (for } \rho = 1Mg/m^3\text{)}$$

In equation 8.9,

$$f = 7.8 \times 0.5 \times 11 (u/c)^2 + 8 \times 2 \times D \left( \frac{1}{g} \cdot \frac{du}{dt} \right) = 42.9 (u/c)^2 + 16D \left( \frac{1}{g} \cdot \frac{du}{dt} \right).$$

The calculated wave and current forces are shown in [Table 8.4](#) and [Figure 8.20](#). The bending moments shown in [Table 8.4](#) have been calculated on the assumption of virtual fixity of the pile at a point 1.5m below the sea bed in the stiff boulder clay. Scour would not be expected around the piles in this type of soil. From [Table 8.4](#), the combined wave and current forces produce a maximum bending moment at the point of fixity of 690.57kN m. Bending moment due to wind force on deck slab

$$= \frac{1}{2} \times 25 \times (15.0 + 1.5) = 206.25kN \text{ m}$$
  
**Total bending moment** 
$$= 896.82kN \text{ m/pile.}$$

**Moment of inertia of pile section** 
$$= \pi(0.6100^4 - 0.5846^4)/64 = 1.063 \times 10^{-3}m^4.$$

**Extreme fibre stress on pile** 
$$= \frac{896.82 \times 0.305}{1.063 \times 10^{-3} \times 10^3} = 257MN/m^2.$$

The direct stress resulting from the dead load of the deck slab and self weight of the pile is added to the bending stress calculated above. It is also necessary to calculate the susceptibility of the pile to current-induced oscillations. Assuming the pile to be filled with fresh water, the effective mass is approximately equal to the mass of metal plus twice the mass of the displaced water. Therefore

$$M = 187 + (2 \times \frac{1}{4}\pi \times 0.61^2 \times 1000) = 771.5kg/m.$$

Table 8.4 Calculations for current and wave forces in Example 8.3

Elevation (m) (CD)	Average height above sea bed (m)	Pile diameter (m)	Current		$\frac{z}{h}$	Wave drag		Wave inertia		Total force (kN/ m <sup>2</sup> )	Horizontal load on element (kN)	Bending moment (kNm)
			Velocity (m/s)	Force (kN/ m <sup>2</sup> )		( $u/C$ ) <sup>2</sup>	Force (kN/ m <sup>2</sup> )	$\frac{1}{g} \frac{du}{dt}$	Force (kN/ m <sup>2</sup> )			
+7.5—+5.5	13	0.610	1.2	0.36	1.18	0.21	9.01	0.26	2.54	11.91	14.53	×14.5=210.69
+5.5—+3.5	11	0.610	0.8	0.16	1.00	0.18	7.72	0.21	2.05	9.93	12.15	×12.5=151.87
+3.5—+1.5	9	0.680	0.6	0.09	0.82	0.15	6.43	0.17	1.85	8.37	11.38	×10.5=119.49
+1.5— 0.5	7	0.680	0.4	0.04	0.64	0.13	5.58	0.14	1.52	7.14	9.71	×8.5= 82.53

[illegible]

Taking the pile length  $L$  to wave crest level and ignoring the mass of the pile above this level, when the pile is in an unsupported condition cantilevering from the sea bed,

$$N = \frac{0.56}{14^2} \sqrt{\frac{200 \times 10^9 \times 1.063 \times 10^{-3}}{771.5}} = 1.50 H_z.$$

Average velocity of flow in upper 12m from surface = 0.6m/s. Thus

$$\frac{V}{ND} = \frac{0.6}{1.50 \times 0.61} = 0.65.$$

Therefore cross-flow or in-line oscillations should not take place.

### Example 8.4

A steel tubular pile having an outside diameter of 863.6mm and a wall thickness of 25.4mm forms part of the piled supports for a jacket-type offshore platform structure. The loading conditions transferred from the base of the jacket to the pile consist of horizontal load of 420kN at scoured sea-bed level, a bending moment of -180kNm and an axial load of 6.3MN. The pile is driven to a depth of 10m into a medium-dense sand (and).  $\gamma_{sub} = 1.3 \text{ Mg/m}^3$  Calculate  $\phi = 35^\circ$  the maximum deflection, maximum bending moment, and maximum working stress on the pile. Moment of inertia of pile section = 307 300cm<sup>4</sup>. Elastic modulus of steel =  $2 \times 10^5 \text{ MN/m}^2$

The first step is to draw the  $p$ - $y$  curves over the critical range of depth. Depths of 0.5, 1.0, 1.5 and 2.0m will be considered.

Calculating the ultimate soil resistance above the critical depths from equation 6.71, at  $x = 0.5\text{m}$ ,  $\alpha = \frac{1}{2} \times 35 = 17.5^\circ$   
 $\beta = 45 + (\frac{1}{2} \times 35) = 62.5^\circ$   $K_0 = 0.4$   $K_A = \tan^2 [45 - (\frac{1}{2} \times 35)] = 0.271$ ,  $B = 0.864\text{m}$  and  $x = 0.5\text{m}$ . Therefore,

$$\begin{aligned} p_{cr} &= 1.3 \times 9.81 \times 0.5 \left[ \frac{0.4 \times 0.5 \times \tan 35^\circ \sin 62.5^\circ}{\tan 27.5^\circ \cos 17.5^\circ} + \frac{\tan 62.5^\circ}{\tan 27.5^\circ} (0.864 \right. \\ &\quad \left. + 0.5 \tan 62.5^\circ \tan 17.5^\circ) + 0.4 \times 0.5 \tan 62.5^\circ (\tan 35^\circ \sin 62.5^\circ - \tan 17.5^\circ) \right. \\ &\quad \left. - (0.271 \times 0.864) \right] \\ &= 28.3 \text{ kN m depth of pile.} \end{aligned}$$

Similarly for  $x = 1.0\text{m}$ ,  $p_{cr} = 75.5 \text{ kN m}$ .

$$x = 1.5\text{m}, p_{cr} = 142 \text{ kN m.}$$

$$x = 2.0\text{m}, p_{cr} = 227 \text{ kN m.}$$

$$x = 4.0\text{m}, p_{cr} = 757 \text{ kN m.}$$

$$x = 8.0\text{m}, p_{cr} = 2726 \text{ kN m.}$$

Calculating the ultimate soil resistance below the critical depth from equation 6.72, at 3m,  $p_{cd}$

$$= 0.271 \times 0.864 \times 1.3 \times 9.81 \times 3 (\tan^8 62.5^\circ - 1) + (0.4 + 0.864 \times 1.3 \times 9.81 \times 3 \times \tan 35^\circ \tan^4 62.5^\circ) = 1778 \text{ kN m depth of pile}$$

The values of  $p_{cd}$  and  $p_{cr}$  are plotted against depth in [Figure 8.21a](#), which shows that over the range of depth relevant to the calculations only the values of  $p_{cr}$  are applicable to the  $p$ - $y$  curves.

*Drawing the  $p$ - $y$  curves at 0.5m*

$$\text{Point } y_u = 3 \times 0.864 / 80 = 0.0324 \text{ m.}$$

$$\text{Point } y_m = 0.864 / 60 = 0.0144 \text{ m.}$$

$$\text{For } x/B = 0.5 / 0.864 = 0.579.$$

From [Figure 6.46a](#), coefficient  $A'$  for cyclic load=1.0.

From [Figure 6.46b](#), coefficient  $B'$  for cyclic load=0.7.

From equation 6.73,  $p_u = 1.0 \times 28.3 = 28\text{kN/m}$ .

From equation 6.74,  $p_m = 0.7 \times 28.3 = 20\text{kN/m}$ .

From equation 6.76, 
$$m = \frac{28 - 20}{0.0324 - 0.0144} = 444.$$

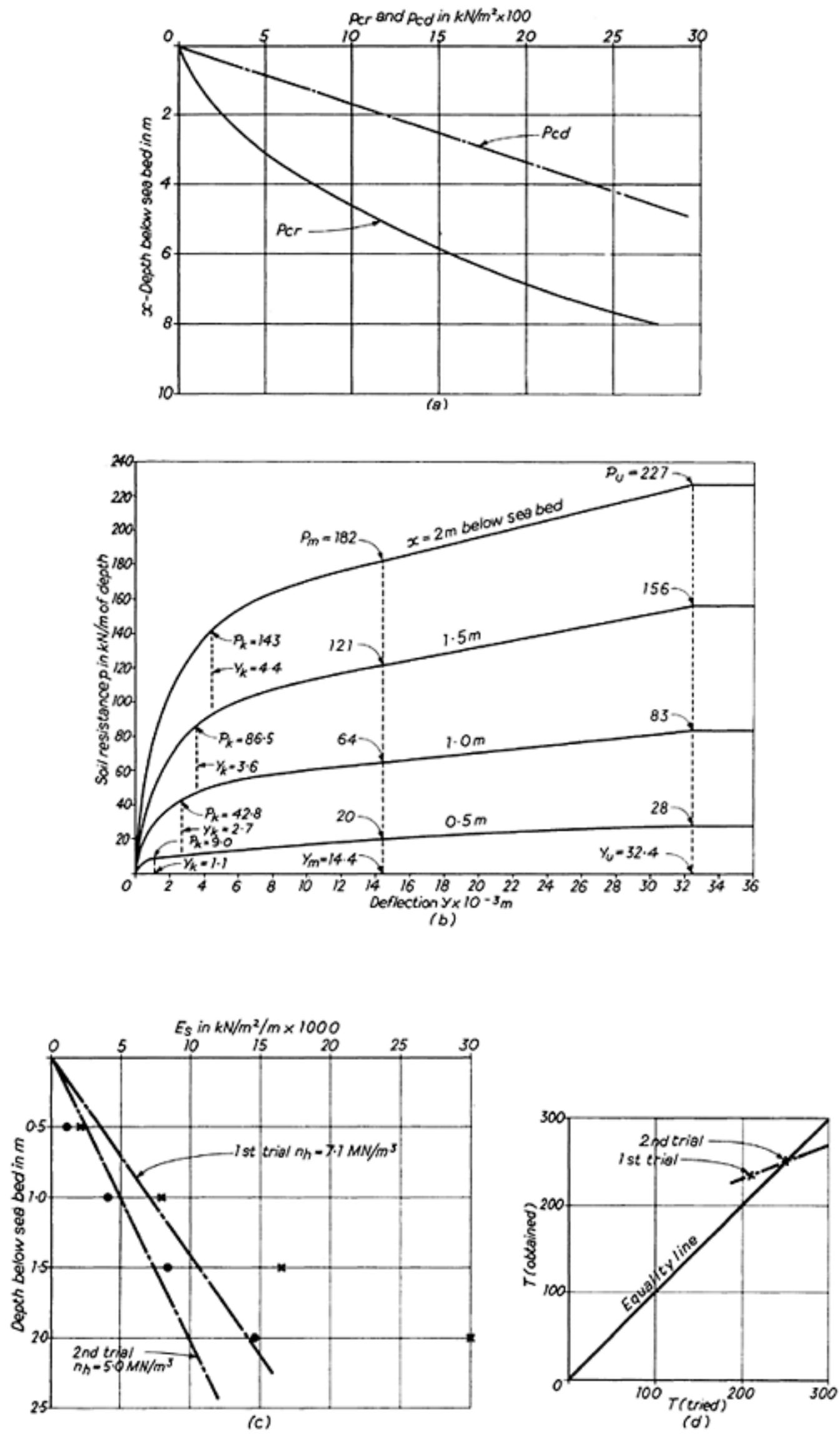


Fig. 8.21

From equation 6.77,  $n = \frac{20}{444 \times 0.0144} = 3.1$  and thus  $1/n = 0.32$ .

From equation 6.78,  $C = \frac{20}{0.0144^{0.32}} = 78$ .

The value of  $n_h$  according to Reese is used in equation 6.79 (see [Figure 6.20](#)) giving

$$y_k = \left( \frac{78}{16.3 \times 10^3 \times 0.5} \right)^{3.1/2.1} = 1.1 \times 10^{-3} \text{m}.$$

For  $y = 2\text{mm}$ , equation 6.75 gives

$$p = 78 \times (0.002)^{0.32} = 11 \text{kN/m}.$$

Similarly for  $y = 4\text{mm}$ ,  $p = 13 \text{kN/m}$ ;

$$y = 6\text{mm}, p = 15 \text{kN/m};$$

$$y = 10\text{mm}, p = 18 \text{kN/m}.$$

and  $p_k = 78 \times (0.001)^{0.32} = 9 \text{kN/m}$ .

The  $p$ - $y$  curves for depths of 1.0, 1.5 and 2.0m are calculated in a similar manner and the full set of curves is shown in [Figure 8.21b](#).

*First trial to obtain T*

Using the value of Reese (*et al.*) of  $16.3 \text{MN/m}^3$  for  $n_h$ , from equation 6.9,

$$T = \sqrt[5]{\frac{20 \times 307\,300}{16.3 \times 10^{-6}}} = 207 \text{cm} = 2.07 \text{m}.$$

From equation 6.64,

$$\frac{HT^3}{EI} = \frac{420 \times 207^3}{20 \times 10^3 \times 307\,300} = 0.61 \text{cm} = 6.1 \text{mm}.$$

$$\frac{Mt}{HT} = -\frac{180}{420 \times 2.07} = -0.207.$$

The pile length  $L$  of 10m is greater than  $4T = 4 \times 2.07 = 8.28 \text{m}$ . Tabulating the deflections from [Figure 6.41](#) and equation 6.64, the corresponding pressures and soil moduli obtained from [Figure 8.21b](#) are as follows.

$x \text{ (m)}$	$Z = \frac{x}{T}$	$C_y$ ( <a href="#">Figure 6.41</a> )	$y \text{ mm}$ $= C_y \times 6.1$	$p \text{ (kN per m depth)}$	$p' = \frac{p}{0.864}$ ( $\text{kN/m}^2$ )	$E_s = \frac{p'}{y}$ ( $\text{kN/m}^2/\text{m} \times 1000$ )
0.5	0.24	1.80	11.0	19	22	2.0
1.0	0.48	1.50	9.2	63	73	7.9
1.5	0.72	1.20	7.3	104	120	16.4
2.0	0.97	0.94	5.7	152	176	30.9

The values of  $E_s$  are plotted in [Figure 8.21c](#), from which  $E_s/x = n_h = 7.1 \text{MN/m}^3$ . From equation 6.9 the corresponding value of  $T$  is

$$\sqrt[5]{\frac{20 \times 307300}{7.1 \times 10^{-6}}} = 244\text{cm} = 2.44\text{m}.$$

This value is plotted in [Figure 8.21](#)d. It is fairly close to the equality line and a value of  $T$  of 260cm is taken for a second trial, for which  $HT^3/EI = 1.20\text{cm}$  and  $M_t/HT = -0.165$ . The tabulated deflections, pressures and soil moduli are then

$x$ (m)	$Z = \frac{x}{T}$	$C_y$	$y$ (mm)	$p$ (kN/m)	$p'$ (kN/m <sup>2</sup> )	$E_s$ (kN/m <sup>2</sup> /m)
0.5	0.19	1.97	23.6	23	27	1.1
1.0	0.38	1.68	20.2	71	82	4.1
1.5	0.58	1.40	16.8	125	145	8.6
2.0	0.77	1.17	14.0	181	209	14.9

Plotting the  $E_s$  values in [Figure 8.21c](#), a new value of  $n_h = 5\text{MN/m}^3$  is obtained for which  $T$  becomes

$$\sqrt[5]{\frac{20 \times 307\,300}{5 \times 10^{-6}}} = 262\text{cm} = 2.62\text{m}.$$

This is very close to the value assumed for the second trial ([Figure 8.21d](#)) and a further trial is unnecessary. The bending moments on the pile are obtained from the coefficients  $A_m$  and  $B_m$  (Figures [6.37c](#) and [6.38c](#)) used in equation 6.52, for which

$$HT = 420 \times 2.60 = 1\,092\text{kNm} \text{ and } M_t = -180\text{kNm}, \text{ and } Z_{max} = 10/2.60 = 3.8$$

The tabulated values of the bending moments are as follows.

$x$ (m)	$z = \frac{x}{T}$	$A_m$	$B_m$	$A_m HT$ (kNm)	$B_m M_t$ (kNm)	$M_{A+M_B}$ (kNm)
0	0	0	+1.0	0	-180	-180
0.5	0.19	0.17	+0.99	+185	-178	+7
1.0	0.38	0.36	+0.98	+393	-176	+217
1.5	0.58	0.50	+0.97	+546	-175	+371
2.0	0.77	0.62	+0.92	+677	-166	+511
2.5	0.96	0.70	+0.88	+764	-158	+606
3.5	1.35	0.76	+0.71	+830	-128	+702
4.5	1.73	0.70	+0.53	+764	-95	+669

From this table the maximum bending moment on the pile is seen to be 702kN m. Then maximum bending stress on extremes fibres

$$\begin{aligned} &= \frac{702 \times 1\,000 \times 1\,000 \times 431.8}{307\,300 \times 10^4} = 99\text{N/mm}^2 \\ \text{Axial stress} &= \frac{6.3}{\frac{1}{4} \times \pi(0.864^2 - 0.813^2)} = \frac{94\text{N/mm}^2}{\text{Total stress} = 193\text{N/mm}^2} \end{aligned}$$

A high-tensile steel to BS 4360 Grade 50 B would be expected to have a minimum yield strength of 355N/mm<sup>2</sup>, and therefore the combined working stress from bending and axial loading is within safe limits.

## CHAPTER 9

### Miscellaneous piling problems

#### 9.1 Piling for machinery foundations

##### 9.1.1 General principles

The foundations of machinery installations have the combined function of transmitting the dead loading from the machinery to the supporting soil, and of absorbing or transmitting to the soil in an attenuated form the vibrations caused by reciprocating or rotating machinery. In the case of reciprocating machinery or equipment such as forging hammers or presses, the dynamic loads transmitted to the soil take the form of thrusts in a vertical horizontal or inclined direction. Rotating machinery creates a torque on the shaft, resulting in lateral loads or moments applied to the foundation block. Dynamic loading from hammers or presses, or from low-speed reciprocating engines has a comparatively low frequency of application, but the vibrations resulting from out-of-balance components in high-speed rotating machinery can have a high frequency.

The higher the frequency of dynamic loading, the less is the amplitude which can be permitted before damage to the machinery occurs, or before damage to nearby structures, and noise and discomfort to people in the vicinity, becomes intolerable. When the frequency of vibration of a machine and its foundations approaches the natural frequency of the supporting soil, resonance occurs and the resulting increased amplitude may result in damage to the plant and excessive settlement of the soil. The latter is particularly liable to happen when the vibrations are transmitted to loose or medium-dense granular soils.

When the mass of the machine and its foundations and vibration characteristics of the soils are known, it is possible to calculate the resonant frequency of the combined machine-foundation-soil system. In order to avoid resonance, the frequency of the applied dynamic loading should ideally not exceed 50% of the resonant frequency for most impact hammers or reciprocating machinery. In the case of high-speed rotating machinery it is probable that the applied frequency will be higher than the resonant frequency of the machine-foundation-soil system. For this condition the aim should be to ensure that the applied frequency is at least 1.5 times the resonant frequency. The need for the wide divergency is to allow for the starting-up and shutting-down periods when the frequency of the machine passes through the resonant stage. If the applied frequency is too close to the resonant frequency the stage of resonance at the acceleration or slowing down of the machine might be too protracted.

When designing shallow foundations for machinery, vibrations which might cause damage or nuisance to the surroundings can be absorbed or attenuated by increasing the mass of the foundation block. There are old-established 'rules-of-thumb' which require the ratio of the mass of the foundation to the mass of the machine to be in the range of 1:1 to 4:1 depending on the type of machine. The resulting required mass of the foundation may be excessive for loose or weak soils leading to excessive settlement, even under static loading conditions, and necessitating the provision of a piled foundation. Also, it may be necessary to employ piles on sites where the water table is at a depth of less than one-half of the width of the block below the underside of the base or even within a depth of twice the width of the block. This is because water transmits amplitudes of vibration almost undamped over long distances, which might result in damaging effects over a wide area surrounding the installation. Similarly, piles may be desirable if a rigid stratum of rock or strongly cemented soil exists within a depth of  $1\frac{1}{2}$  times the block width. Such a stratum reflects energy waves and magnifies their amplitude of vibration.

Generally, the effect of providing a piled foundation to a reciprocating or rotating machine is to increase the natural frequency of the installation in the vertical, rocking, pitching, and also possibly longitudinal, modes. This is because of the behaviour of the mass of soil enclosed by the pile group

acting with the pile cap and the piles themselves. The soil mass may be relatively small where the piles act in end bearing, or large in the case of friction piles. The natural frequency may be decreased in the lateral and yawing modes of vibration because of the low resistance of piles to lateral loads at shallow depths.

To ensure that the ratio of the frequency of the disturbing moment or disturbing force applied by the machinery to the natural frequency of the machine-foundation-soil system is either greater or less than the required value, it is necessary to calculate the natural frequency of the system. This is a complex matter, particularly for piled foundations, and is beyond the scope of the present book. The reader is referred to the publications of Barkan<sup>(9.1)</sup>, Hsieh<sup>(9.2)</sup>, Whitman and Richart<sup>(9.3)</sup> and Richart *et al.*<sup>(9.4)</sup> for general guidance. Irish and Walker<sup>(9.5)</sup> have established design curves relating the natural frequency of piles of various types to their effective length, both for the vertical and the rocking modes of vibration.

### 9.1.2 Pile design for static machinery loading

Piles and pile groups carrying static loads from machinery should be designed by the methods described in Chapters 4 and 5. Particular attention should be paid to the avoidance of excessive differential settlement of the pile cap; the differential movement should not exceed 8mm. The centre of gravity of the machine combined with the pile cap and supporting piles should be located as nearly as possible on a vertical line through the centroid of the pile group, and the eccentricity of the combined masses should not be greater than 5% of the length of the side of the pile group. If possible the centre of gravity of the machine and soil mass should be below the top of the pile cap.

### 9.1.3 Pile design for dynamic loading from machinery

Very little systematic research combined with full-scale testing has been undertaken to study the behaviour of piles under dynamic loading. In the few instances where full-scale tests have been made they were related to particular projects and no useful comparative results have been obtained. Generally it can be stated that the effect of applying dynamic loads to piles in cohesive soils is to reduce their skin friction and end-bearing value, *i.e.* to reduce their ultimate carrying capacity, and the effect in granular soils is to reduce their skin friction but to increase their end-bearing resistance at the expense of increased settlement under working load.

The reduction in the skin friction and end-bearing resistance of piles in cohesive soils is the result of a reduction in the shearing strength of these soils under cyclic loading. The amount of reduction for an infinite number of load repetitions depends on the ratio of the applied stress to the ultimate stress of the soil. It is the usual practice to double the safety factor on the combined skin friction and end bearing to allow for the dynamic application of load (see 6.2.2).

The torque of rotating machinery can cause lateral loading on the supporting piles. The deflection under lateral loading can be calculated by the methods described in Chapter 6. To allow for dynamic loading the deflections calculated for the equivalent static load should be doubled.

The type of pile, whether driven, driven-and-cast-in-place, or bored-and-cast-in-place, is unlikely to have any significant effect on the behaviour of piles installed wholly in cohesive soils. It is possible that the lateral movements of piles with driven pre-formed shafts (*e.g.* precast concrete or steel H-piles) will be greater than those of cast-in-place piles, because of the formation of an enlarged hole around the upper part of the shaft (see Figure 4.5).

The skin-frictional resistance of a pile to static compressive loading in a granular soil is relatively low. This resistance is reduced still further when the pile is subjected to vibratory loading, and it is advisable to ignore all frictional resistance on piles carrying high-frequency vibrating loads. If such piles are terminated in loose to medium-dense soils there will be continuing settlement to a degree which is unacceptable for most machinery installations. It is therefore necessary to drive piles to a dense or very dense granular soil stratum and even then the settlements may be significant, particularly when high end-bearing pressures are adopted. This is due to the progressive attrition of the soil grains at their points of contact. The continuing degradation of the soil particles results in the slow but continuous settlement of the piles. If possible, piles carrying vibrating machinery should be driven completely through a granular soil stratum for termination on bedrock or within a stiff clay.

A problem of piled foundations for machinery sensitive to small differential settlements was experienced at John Brown Company's shipyard at Clydebank. At this site 18.6m of loose to medium-dense silty sand were overlying stiff glacial till. Gear cutting machinery comprising large hobbing, shaving and grinding machines had to operate to an accuracy of 0.009mm and each machine was installed in a separate enclosure under conditions of constant temperature and humidity. It was essential to avoid any appreciable settlement of the machines due to vibrations caused by their own motion, or transmitted from elsewhere

on the shipyard and the adjacent main road. It was expected that settlements of raft or piled foundations terminated in the medium-dense sand (with a standard penetration test N-value of 20 to 30 blows per 300mm) would be excessive and a type of pile had to be selected which could be driven through the deep sand layers to reach the glacial till. The possibility of the compaction of the sand due to driving a number of piles in a closely-spaced group was considered and this led to the choice of a small-displacement pile in the form of a Larssen BP2 box-section driven with an open end. It would have been possible to use water jetting to assist the penetration of these piles, but all the piles were driven by a double-acting hammer into the glacial till without recourse to jetting. The building surrounding the plant was carried by driven-and-cast-in-place piles (Franki piles) terminated at a penetration of about 4.6m into the sand stratum.

A few years later the printing presses of the International Publishing Corporation were constructed in very similar soil conditions on the bank of the River Clyde upstream of Clydebank. At this site steel H-piles were driven through the silty sand into glacial till to support the machinery foundations.

## 9.2 Piling for underpinning

### 9.2.1 Requirements for underpinning

Underpinning of existing foundations may be required for the following purposes.

1. As a remedial measure to arrest the settlement of a structure.
2. As a precautionary measure carried out in advance to prevent the excessive settlement of a structure when deep excavations are to be undertaken close to its foundations.
3. As a strengthening measure to enable existing foundations to carry increased loading, or to replace the deteriorating fabric of a foundation.

An example of the use of piling as a remedial measure is shown in [Figure 9.1a](#). The column has settled exclusively due to the consolidation of the soft clay beneath its base. Piles are installed on each side of the base and the load transferred to the pile heads by needle beams inserted below the base.

A typical use of piles as a precautionary underpinning measure is shown in [Figure 9.1b](#). A deep basement is to be constructed close to an existing building on shallow strip foundations. Underpinning of the foundation adjacent to the basement is required since yielding of the ground surface as a result of the relief of lateral pressure due to the excavations would cause excessive settlement. Rows of piles are installed close to the wall foundation inside and outside the building and the loads are transferred to them by a system of longitudinal and transverse beams. The external row of piles also serves as a support for the horizontal sheeting members used to retain the face of the excavation.

Piling as a strengthening measure is shown in [Figure 9.1c](#). Pits are excavated beneath the existing foundation and piles are jacked down to a bearing on a hard incompressible stratum. Underpinning of the foundations may be required where the existing piles have deteriorated due to attack by aggressive substances in the soil or ground water. New piles can be installed in holes drilled through the cap or raft ([Figure 9.1d](#)). The new pile heads are bonded to the reinforcement of the existing substructure.

Piling has a somewhat limited application to underpinning work. This is because it is always necessary to excavate pits below the existing substructure to place supporting beams or pads. In a high proportion of the cases where remedial or strengthening works are required a suitable bearing stratum exists at no great depth and it is cheaper to take the pits down to this stratum and to backfill the void with mass concrete rather than to install piles in conditions with a low headroom and a restricted working area. Also a considerable force may be required to jack down an underpinning pile even though the soil is removed from the piling tube at each stage of jacking. There may be insufficient mass in the existing structure to provide the required reaction to this jacking force. It is not usually feasible to employ jacked piles beneath two-storey or three-storey buildings of load-bearing wall construction, since the usual mass concrete strip foundation and brick footing walls have insufficient bending strength to withstand the loading that results from jacking down a pile, even though a spreader beam is used between the ram of the jack and the foundation.

### 9.2.2 Piling methods in underpinning work

Bored piles are suitable for underpinning light structures and they are installed outside the periphery of the existing foundations as shown in [Figure 9.1a](#). Tripod rigs ([Figure 3.34](#)) are suitable for installing the piles since they can be operated inside buildings in conditions of low headroom. However, it is

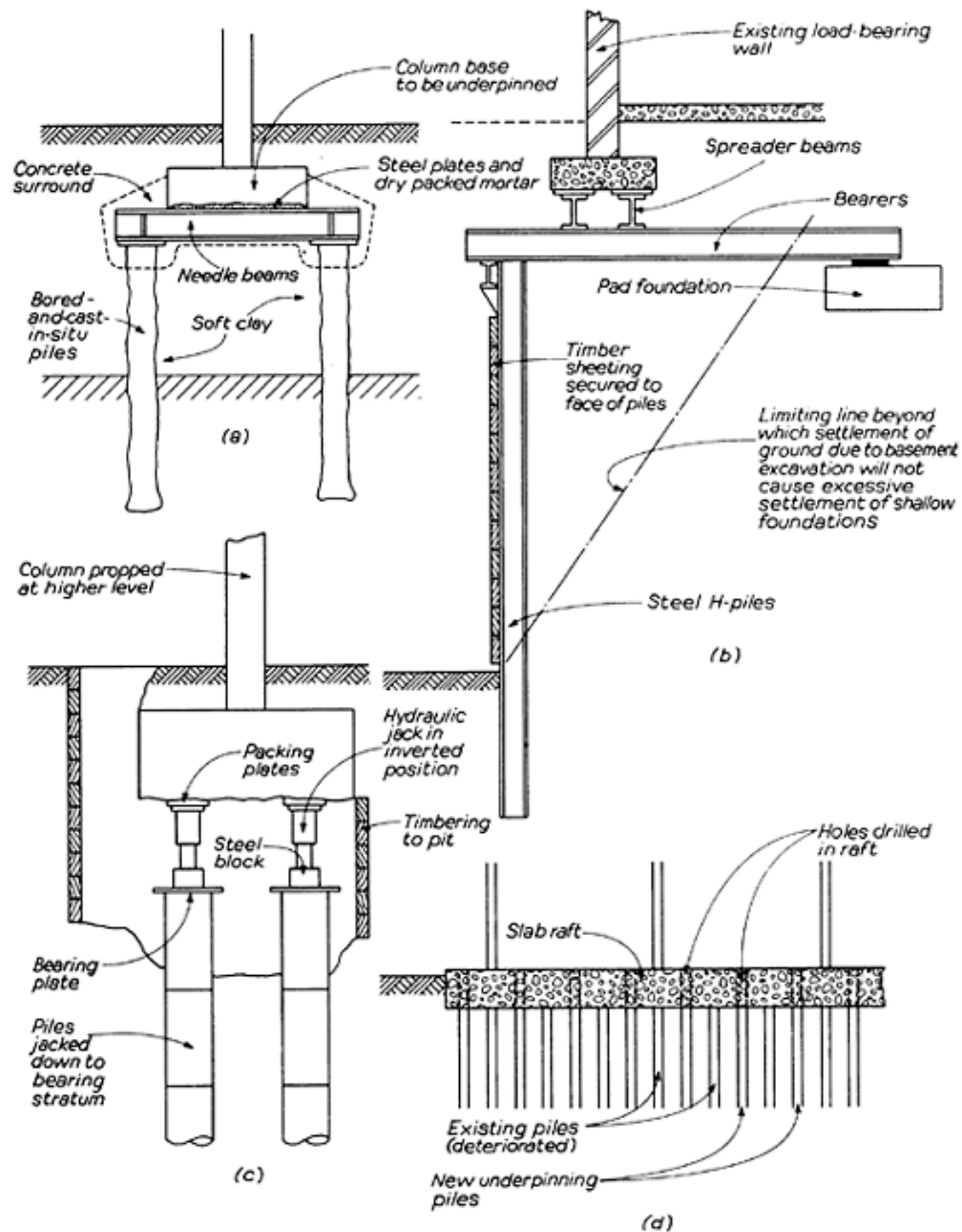


Fig. 9.1 Use of piles in underpinning

- (a) Remedial measures to support column base
- (b) Precautionary measures in underpinning strip foundation adjacent to deep excavation
- (c) Jacked piles to strengthen column base
- (d) Drilled piles to replace existing piles beneath raft slab

desirable, for reasons of economy, to install piles outside a building as far as possible. This can be done at the corners of buildings as shown in [Figure 9.2](#). Precast reinforced concrete sections or steel H-piles can be concreted or grouted into the pile boreholes in cases where it is desired to transfer the loading to underpinning piles as quickly as possible after installing them.

Light structures can be underpinned from a single row of bored piles located outside the building. After concreting the piles, cantilever brackets are cast onto their heads as shown in [Figure 9.3a](#). The bending resistance of a small-diameter pile is relatively low, and therefore the form of construction is limited to strip foundations of light buildings or to lightly-loaded columns. Heavier structures can be underpinned by pairs of piles located outside the building but carrying a cantilevered bracket as shown in [Figure 9.3b](#). This system can cause difficulties in pile design. The compression pile is required to carry heavy loading and there may be problems in achieving the required resistance to uplift in skin friction on the tension pile.

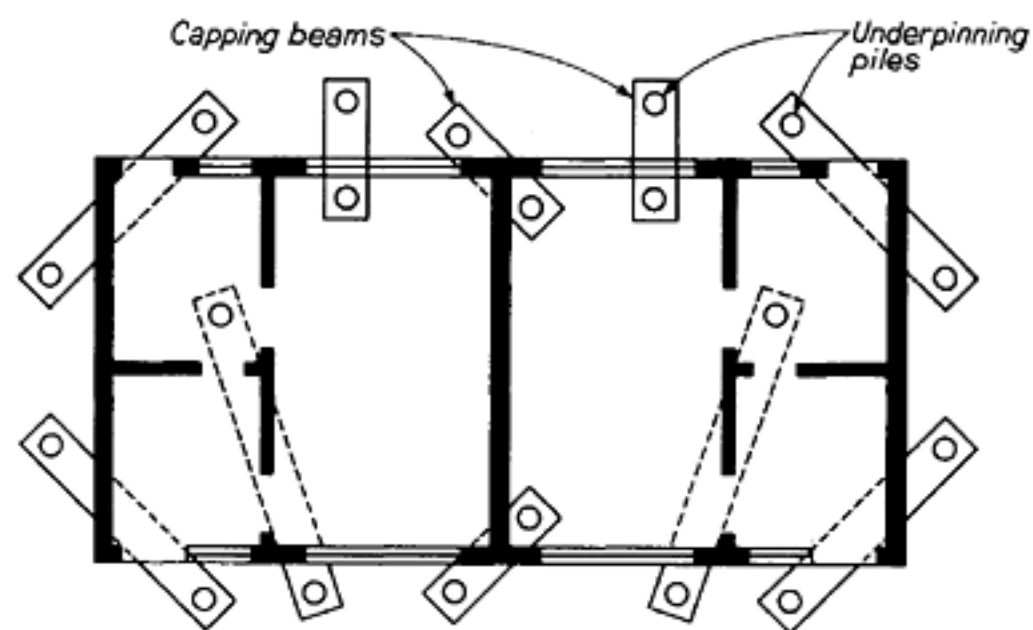


Fig. 9.2 Layout of underpinning piles for light structures

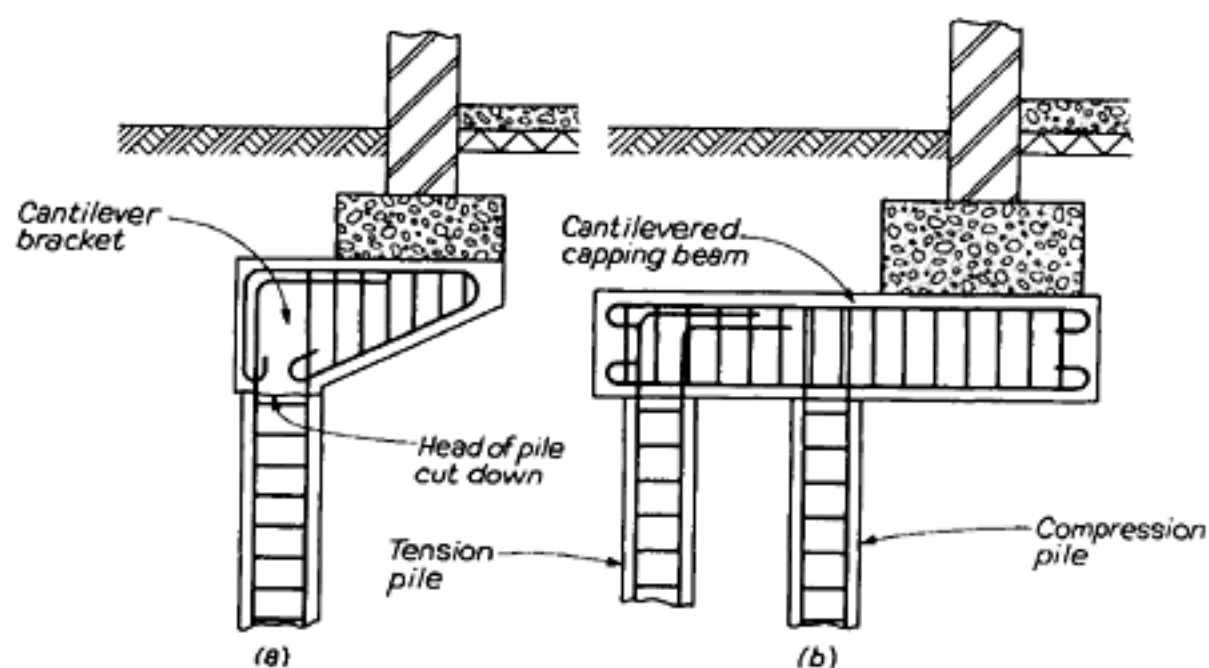


Fig. 9.3 Cantilevered brackets for supporting light structures

- (a) From single piles
- (b) From pairs of piles

The Fondedile piling system employing the 'Palo Radice' (root pile) is suitable as a means of underpinning structures undergoing settlement or for strengthening existing foundations to enable them to carry heavier loads. Small-diameter holes lined temporarily with casing tubes are drilled through the existing foundations or through both a load-bearing wall and its foundation ([Figure 9.4](#)). A rich sand-cement mortar is then pumped down a tremie pipe to fill the borehole and any cavities in the existing structure. Reinforcement is provided in the form of a single bar for small-diameter (100mm) piles or a cage or tube for the larger-diameter (250–300mm) piles. The casing is extracted with the assistance of compressed air to push the grout into the cavities and against the walls of the drilled holes.

The usual method of application is to install the piles in relatively large numbers at differing angles of rake so as to provide resistance to lateral loads and overturning moments as well as to support loads in a vertical direction. The rotary drilling machines used by the Fondedile Co. require a working space of only  $2 \times 1.5\text{m}$  and a headroom of 1.8m, and the drilling is performed with relatively little vibration. Thus the piles can be installed in confined surroundings and very close to the foundations to be underpinned ([Figure 9.5](#)). Other systems employ minipiles of the type described in [Section 2.6](#) such as small diameter steel tubes which are drilled down or set in drilled holes. Grout is pumped down the tubes which remain in place as permanent reinforcement. A detailed account of the Fondedile system and underpinning with other types of small diameter piles with numerous case histories is given by Lizzi([9.6](#)).

Bored piles installed by continuous flight auger as described in 2.4.2 are also suitable for strengthening and underpinning work since the rotary drilling methods and continuous support given to the soil result in little vibration and a negligible loss of ground in properly controlled operations.

Heavily loaded foundations can be underpinned by jacking piles down to the bearing stratum using the dead load of the existing foundations and superstructure as the reaction to the jacking operation.

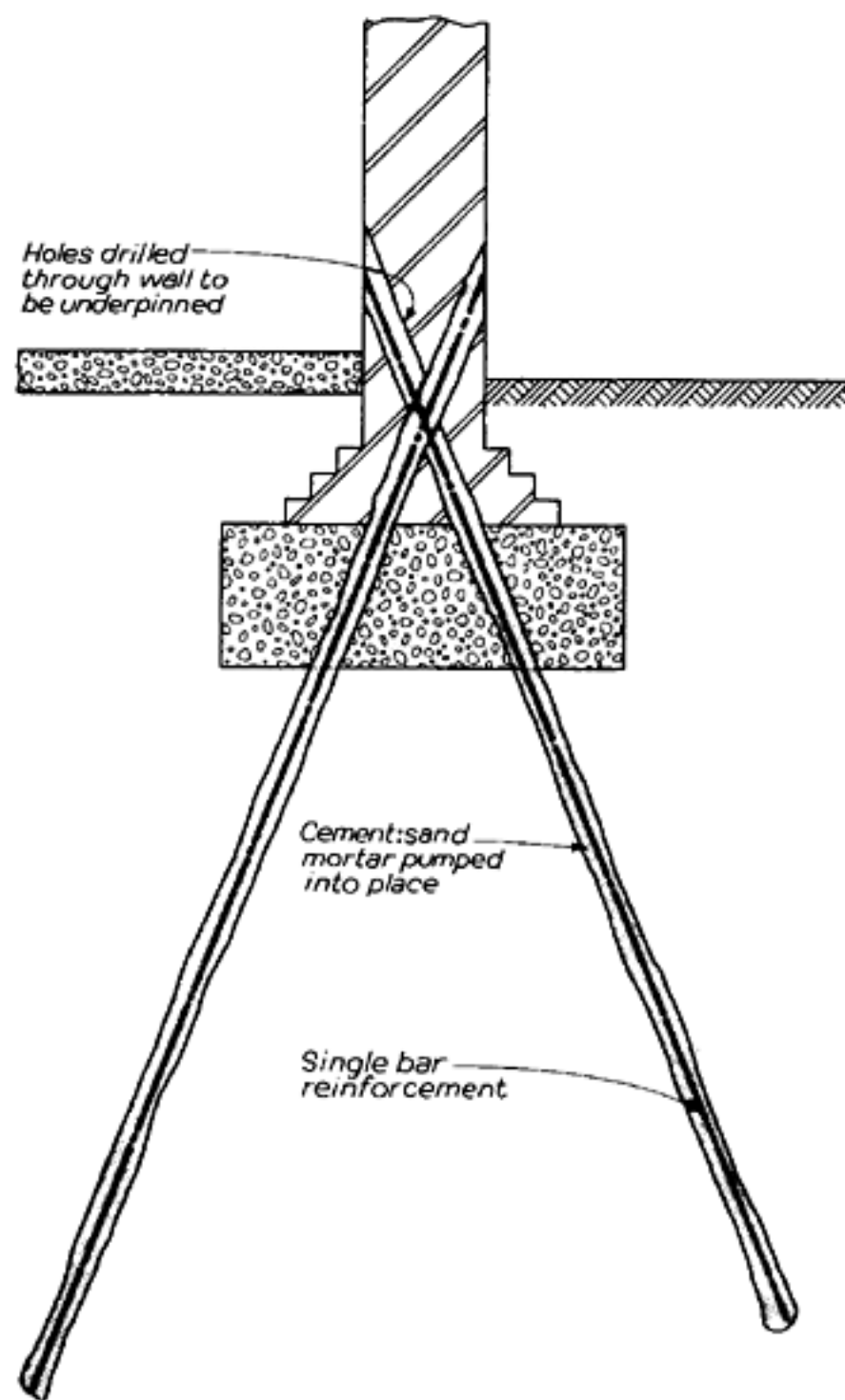
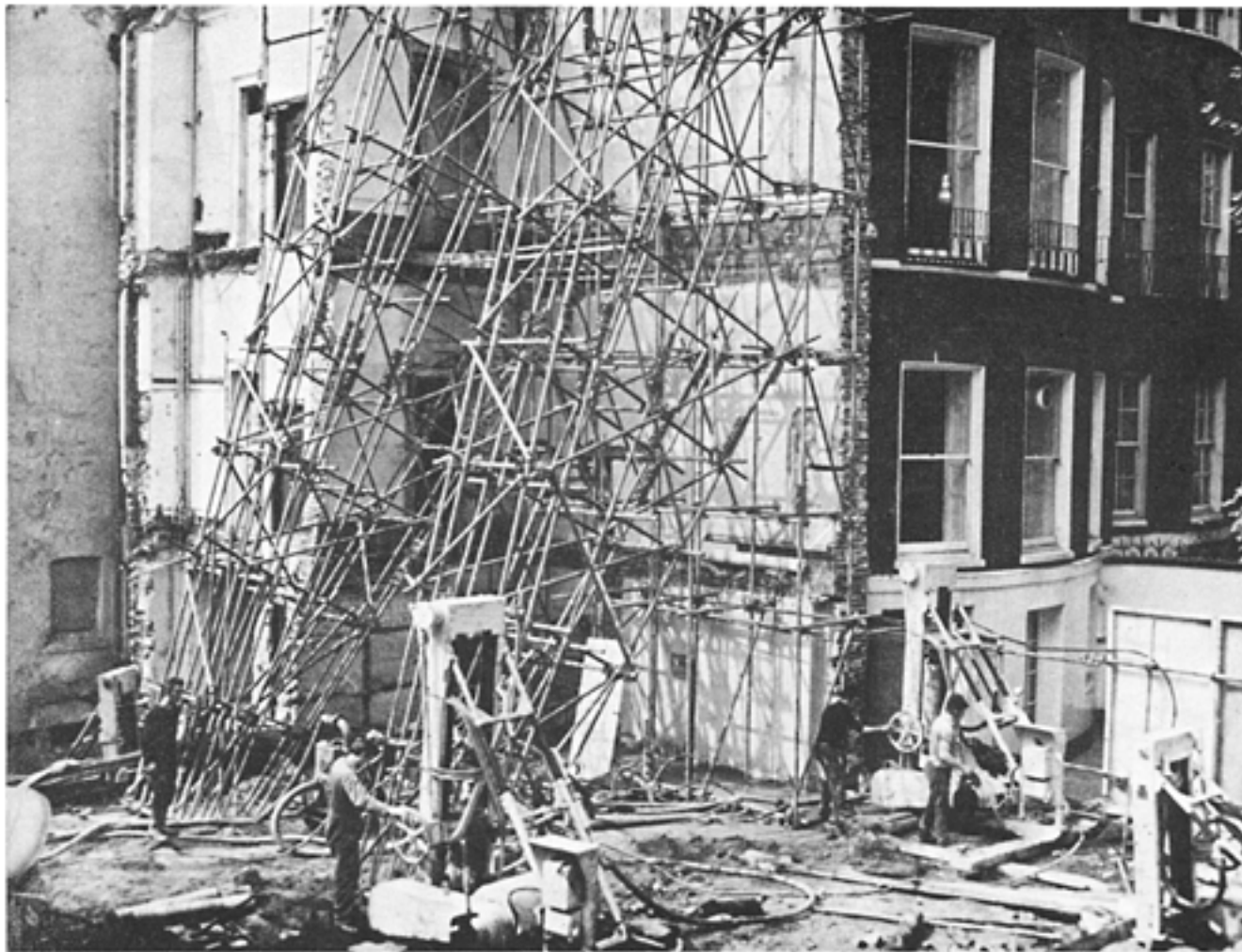


Fig. 9.4 Underpinning with Fondedile piles (*Pali Radice*)

The Franki Miga pile consists of 300mm square by 760mm long precast concrete sections with a central 50mm hole. After excavating a pit beneath the foundation, a hole is dug in the floor of the pit to receive the pointed bottom unit of the pile. A hydraulic jack is placed on top of this unit and is surmounted by short lengths of steel plate or beam sections to spread the load onto the underside of the existing foundation. The bottom unit is then jacked down until it is flush with the bottom of the pit. The jacks and packers are removed and another precast concrete section added, which is in turn jacked down ([Figure 9.6](#)). Additional units are now added and jacked down until the bearing stratum is reached or until the resistance of the pile as measured by the pressure gauge on the jack indicates that the desired 'preload' has been attained. The elements are next bonded together by inserting short steel bars into the longitudinal central hole and grouting them with cement. On the completion of the jacking operation short lengths of steel beam or rail sections are driven hard into the space between the pile head and the foundation, or between the pile head and the spreader beams. The jack is then removed and the head of the pile, packers and spreaders encased solidly in concrete.

It is essential to maintain the load on the jack until the packing is completed. This is to avoid any rebound of the pile head, and subsequent settlement when the load from the structure is transferred to the piles.

The 'Pretest' piling system developed in the USA by Spencer White and Prends([9.7](#)) employs 1.2m lengths of open-ended steel tube, which are jacked down using the foundation to be underpinned as the reaction in the manner illustrated in [Figure 9.1c](#). A pit is excavated beneath the foundation to provide a working headroom of about 1.8m. A hydraulic jack having a capacity of 400kN is used to force down the tube. The jack has a ram diameter of 114mm and a stroke of 280mm. After jacking down each tube, the jack and bearing plate are taken away and drilling tools are used to remove the soil from within the tube. An auger is used in clays or damp granular soils, and an 'orange peel' grab is used in granular soils below the water table. The drilling is advanced by manipulating the tools by



*Fig. 9.5 Fondedile piling rig working close to foundations to be underpinned*

hand, using rods 1.2m long. Cautiously operated water jetting can also be used to lift granular soils from the piling tube. After completing 'mucking-out', another tube is coupled-on, the jack is put back into position and the string of tubes forced down, followed again by soil removal. The ram of the hydraulic jack is screwed and it carries a collar which is tightened against the cylinder after each stage of raising the ram. This is a safeguard against a sudden drop of the ram caused by a loss of oil pressure in the cylinder.

After the underpinning pile has been jacked down to the bearing stratum, or the desired value of preload has been attained, a pair of hydraulic jacks is inserted between the head of the pile, and a bearing plate packed up to the underside of the existing foundation. The thrust on the rams of these jacks is adjusted to apply a load of 1.5 times the working load onto the pile.

When downward movement of the pile has ceased a short length of steel H-section with end-bearing plates is wedged tightly into the space between the jacks ([Figure 9.7](#)). The latter can then be removed and used for the same procedure on the adjoining piles.

Where piles are installed in rows or closely-spaced groups by preloading or 'pretesting' methods, the operation of jacking an individual pile relieves some of the load on the adjacent piles which have already been installed and wedged-up. It then becomes necessary to replace the jacks and re-load these piles, after which the inserted struts are re-wedged. Alternatively, all the pre-testing jacks can remain in position until the last pile in the group or row is jacked down. Then all the loads on the jacks are balanced, the struts installed and the jacks removed. The final operation is to encase the struts and pile heads in concrete well rammed-up to the underside of the existing foundation.

Where jacked piles are installed either by the Franki 'Miga' system or the 'Pretest' system, care must be taken to restrain the existing foundation, or the rows of jacks and struts, from moving horizontally



*Fig. 9.6 Jacking down Franki Miga pile*

due to lateral or eccentric thrusts. Raking shores to the superstructure, strutting of the existing foundation to the walls of the underpinning pit, or bracings between jacks and pile heads can be used to restrain lateral movement.

The existing columns or walls of the structure to be underpinned can be used to provide the reaction to jacking if they are sufficiently massive. Niches are cut into the faces of the structure and concrete corbels or brackets are cast into these pockets to form the bearing members for the jacks, as shown in [Figure 9.8](#). The pairs of jacks must, of course, be operated simultaneously to avoid applying eccentric loading to the existing structure.

Where H-section piles are used to provide underpinning combined with lateral support to a deep excavation, as shown in [Figure 9.1b](#), they can be installed by placing them in holes previously drilled by mechanical auger or, in stable ground, by tripod rigs. If the drill holes are given continuous support by a bentonite slurry or by casing there should be a negligible loss of ground around the borehole and the installation of the piles is effected with little noise or damaging vibration. Where it is necessary to use tripod rigs in unstable granular soils, there is a risk of loss of ground around the boreholes as

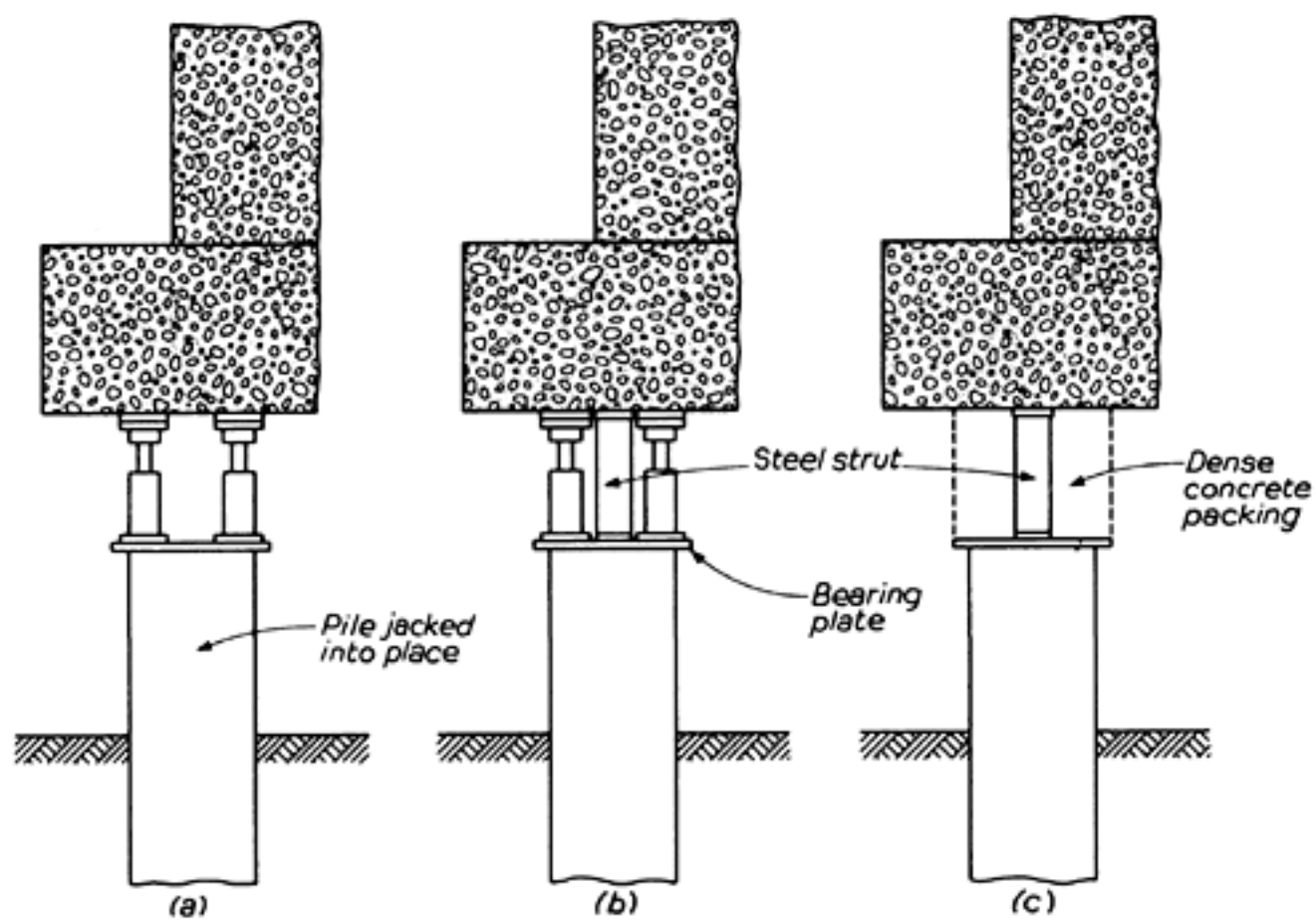


Fig. 9.7 The 'Pretest' underpinning system

- (a) Jacking down underpinning pile
- (b) Insertion of steel strut
- (c) Steel strut wedged into place before encasement in concrete

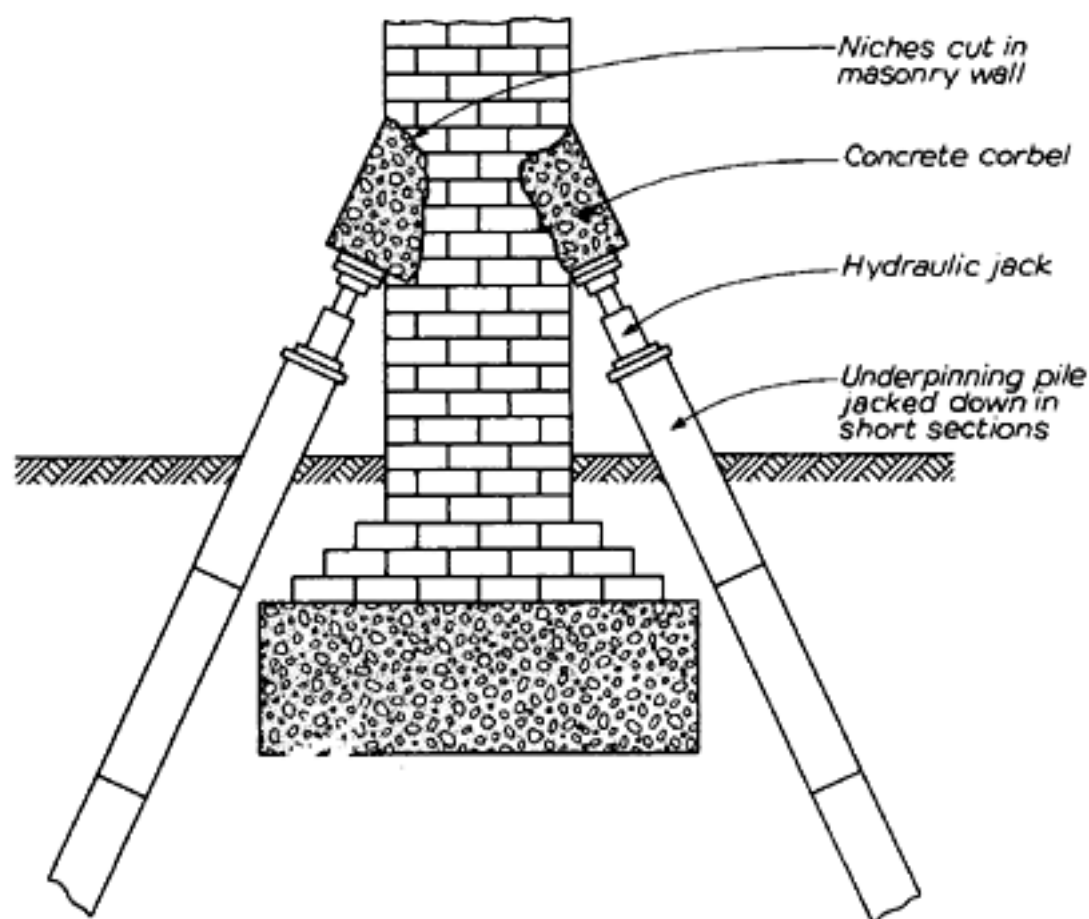


Fig. 9.8 Underpinning load-bearing wall by jacking piles from corbels

described in [Section 3.3.5](#). If there is such a risk it will be necessary to shore the building temporarily with supports bearing on the ground outside the zone of subsidence. Below the level planned for the base of the excavation the space between the pile and the borehole is filled with a weak sand-cement mortar. This provides the required passive resistance to lateral loads on the piles and allows the latter to be removed if permitted by the planned sequence of underpinning and construction of the permanent work.

Bored piles used in the combined role of underpinning and lateral support to the sides of the excavation can be arranged in a single row ([Figure 9.9a](#)) or a double row of abutting piles, or in a single row of interlocking piles ([Figure 9.9b](#)). When they are abutting the system is known as 'contiguous piling', and when interlocking as 'secant piling'. Contiguous piles are cheaper to install, but because it is impossible to drill the holes in a truly vertical direction or to maintain a constant shaft

diameter there are always gaps present between adjacent piles. Below the water table, sand and silt can bleed through these gaps causing a considerable loss of ground, and the installation of the piles in a double staggered row cannot

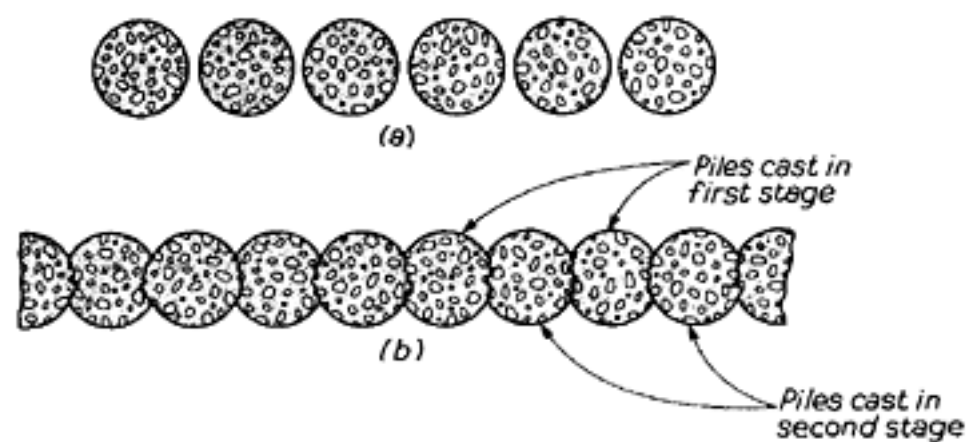


Fig. 9.9 Bored piles used for combined underpinning and lateral support

(a) Contiguous piles

(b) Secant piles

prevent this happening. Contiguous piles are best suited to underpinning and to support excavations in firm to stiff clays or damp granular soils above the water table.

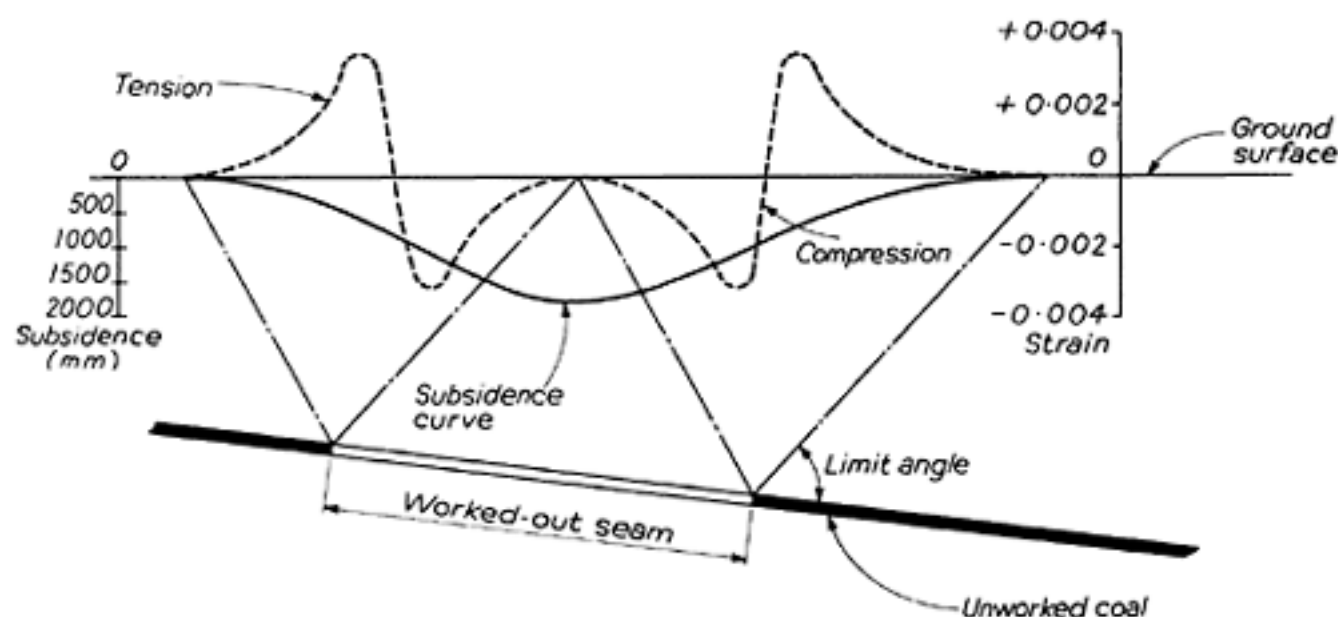
Where the excavation is to be performed in water-bearing granular soils, any loss of ground can be avoided by adopting secant piling (Figure 9.9b). Alternate piles are first installed by conventional drilling and casting concrete *in situ*. The soil in the space between the pile shafts is then drilled out and a chisel is used to cut a groove in the wall of the pile on each side of the drill hole. A weak bentonite-cement concrete can be used in the commencing row of piles to facilitate the grooving operation. Concrete is next placed to fill the drill hole, thus forming the interlocked and virtually watertight wall.

Longitudinal reinforcement is provided in the piles to the extent necessary to carry vertical loading, eccentric loads from the underpinning bearers, and lateral loading from earth and hydrostatic pressure.

### 9.3 Piling in mining subsidence areas

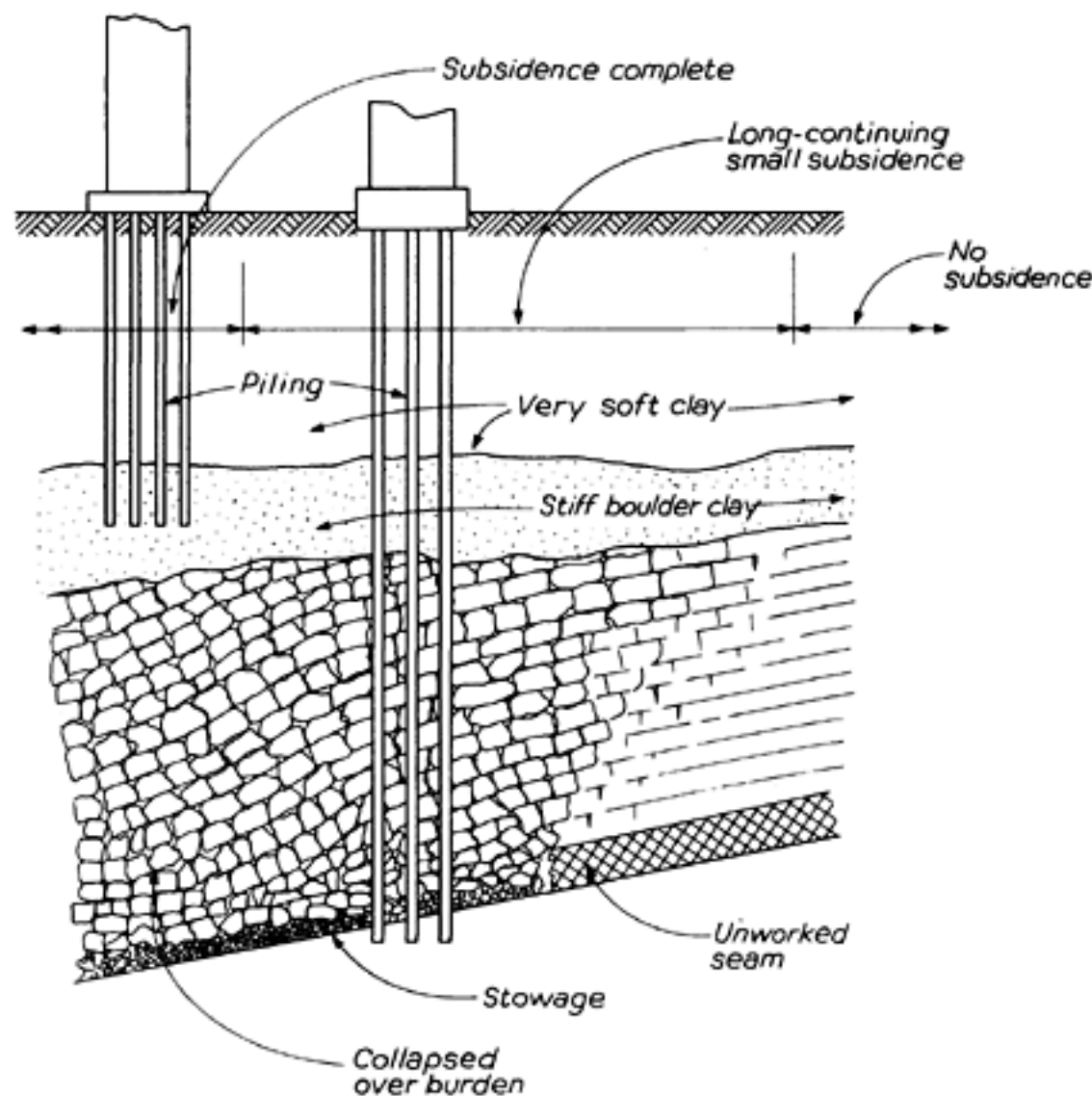
The form in which subsidence takes place after extracting minerals by underground mining depends on the particular technique used in the mining operations. In Great Britain the problems of subsidence mainly occur in coal-mining areas where the modern practice is to extract the coal by 'longwall' methods. Using this technique the entire coal seam is removed from a continuously advancing face. The roof of the workings behind the face is supported by multiple rows of hydraulically-operated props. As the face moves forward the props in the rear are systematically lowered to allow the roof of the workings to sink down onto heaps of mine dirt or 'stowage'. The overlying rock strata and over-burden soil follow the downward movement of the roof and the consequent subsidence of the ground surface is in the form of a wave which advances parallel to and at approximately the same rate as the advancing coal face. The subsidence is accompanied by very substantial horizontal strains of the ground surface, these strains being tensile at the crest of the wave and compressive at the trough and thus taking the form shown in Figure 9.10. The magnitude of the strain can range from as much as plus or minus 0.8% of the overburden thickness above shallow workings to 0.2% over deep seams.

The horizontal ground movements make it virtually impossible to use piled foundations in areas where longwall mining is proposed, or is currently being practised. The horizontal shearing forces accompanying the strains are so high that it is quite uneconomical to attempt to resist them by heavily strengthening the pile shafts. However, in areas where subsidence following longwall mining has virtually ceased it is possible to use piled foundations if it is recognized that some residual movement will take place



*Fig. 9.10 Profile of ground subsidence over longwall mine workings*

as the collapsed strata slowly reach final equilibrium. In these cases it is desirable to terminate piles in a soil layer overlying rockhead, as shown on the left-hand side of [Figure 9.11](#). The soil acts as a cushion, preventing any concentration of load on the broken rock strata. Long-term movements may be substantial near the boundary of the worked-out seam. If the workings are shallow, piles may be taken down through the collapsed overburden to intact rock layers below the coal seam as shown on the right-hand side of [Figure 9.11](#). Bored-and-cast-in-place piles are used for this purpose, but it is essential to isolate the shaft of the pile from the overburden above the coal seam in order to avoid heavy compressive loading caused by downdrag from the collapsing strata. This isolation is achieved by placing the concrete within a shell formed from stiffened light-gauge steel sheeting, the sheeting terminating at the base of the coal seam. Below this level the concrete can be cast against the surface



*Fig. 9.11 Piling through collapsed ground over longwall mine workings*

of the stable strata to form a 'rock socket', as shown in [Figure 9.12](#). The space between the shell and the wall of the drill hole through the overburden can be filled with bentonite slurry, soft bitumen or loosely placed rock fragments. A minimum clearance of 150mm should be provided to accommodate minor lateral movements as the rock strata adjust themselves to their equilibrium position. A bentonite or bitumen infill to seal the annulus completely would be used if there is a risk of emission of natural gas from the coal seam.

In the nineteenth century and the early years of the present century coal and other minerals were extracted using mining techniques variously known as 'pillar and stall', 'bord and pillar', and 'stoop and room'. A main heading or road was driven from the shaft to follow the coal seam to the planned boundary of the workings. Transverse galleries were then driven from the main roadway to form a rectangular, triangular, or lozenge-shaped pattern of galleries separated by pillars of unworked coal. These pillars served to support the overlying rock strata until the general area had been mined. The pillars were then either left intact or were wholly or partially removed as the coal extraction operations retreated towards the shaft. Where the pillars were wholly removed the pattern of subsidence followed that of longwall mining ([Figure 9.10](#)). Where they were left in place they remained, and still remain, in an unpredictable state of stability which has resulted in complex problems concerning building over abandoned mine workings, problems which are still encountered in the towns and cities of Britain to the present day([9.8](#)).

The instability of coal pillars may be due to the slow decay of the coal, to changes in the ground-water regime in flood workings, to increased loading on the ground surface, to an increase in the load transferred to pillars due to the collapse of neighbouring areas, or to longwall mining in deeper coal seams. If massive rock strata such as the thick sandstones of the Coal Measures are overlying the partly-worked

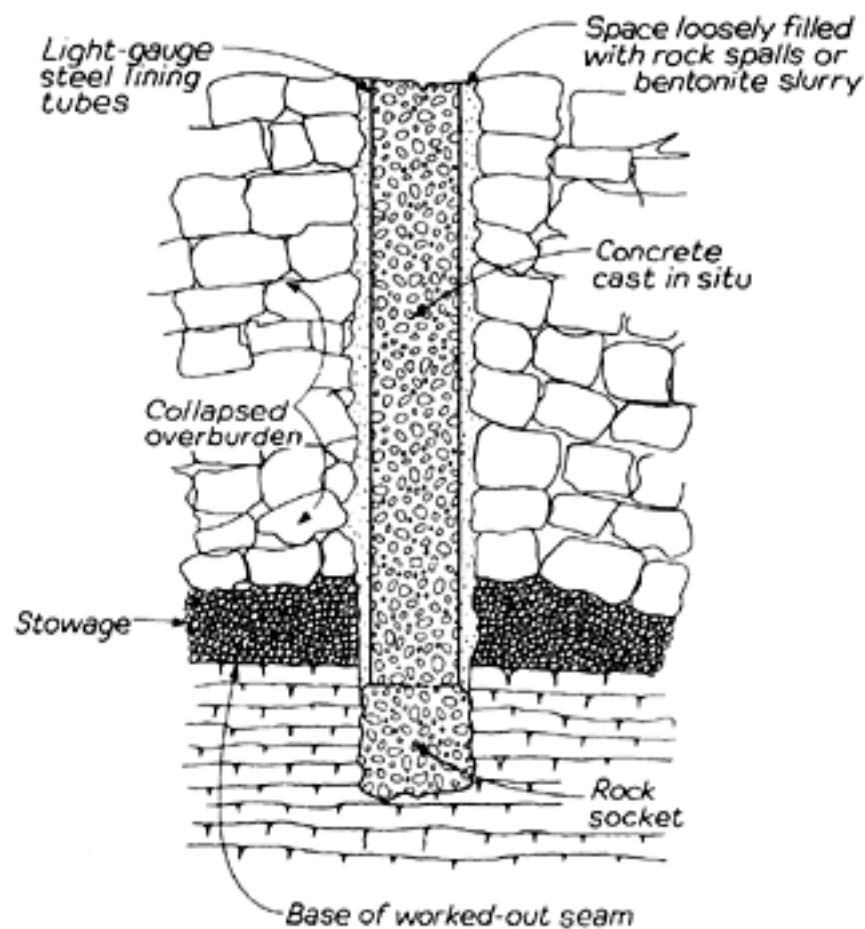


Fig. 9.12 Isolating shafts of bored piles from surrounding collapsed ground

seam they may form a bridge over the cavities such that the collapse of the weak strata forming the roof of the working will not extend above the base of the massive rock stratum (Figure 9.13a). Provided that the coal pillars themselves do not decay, the workings may remain in a stable condition for centuries and it will be quite satisfactory to construct piled foundations overlying them. Again it is desirable to terminate the piles in a soil layer to avoid any concentration of loading at the rockhead.

Where massive rock strata are not present and the overburden consists only of weak and thinly-bedded shales, mudstones and sandstone bands overlain by soil, a collapse of the roof will eventually work its way up to the ground surface to form a chimney-like cavity known as a 'crownhole' (Figure 9.13b).

Piling should be avoided above these unstable, or potentially unstable, areas, but if the workings lie at a fairly shallow depth, it is possible to install bored-and-cast-in-place piles completely through the overburden, terminating them in a stable stratum below the coal seam as shown in Figure 9.13b. The pile shaft must be isolated from the soils and rocks of the overburden in the manner illustrated in Figure 9.12. Any collapse of the strata over pillar and stall workings usually takes place in a vertical direction with little lateral movement, but nevertheless a generous space (a minimum of 150mm) should

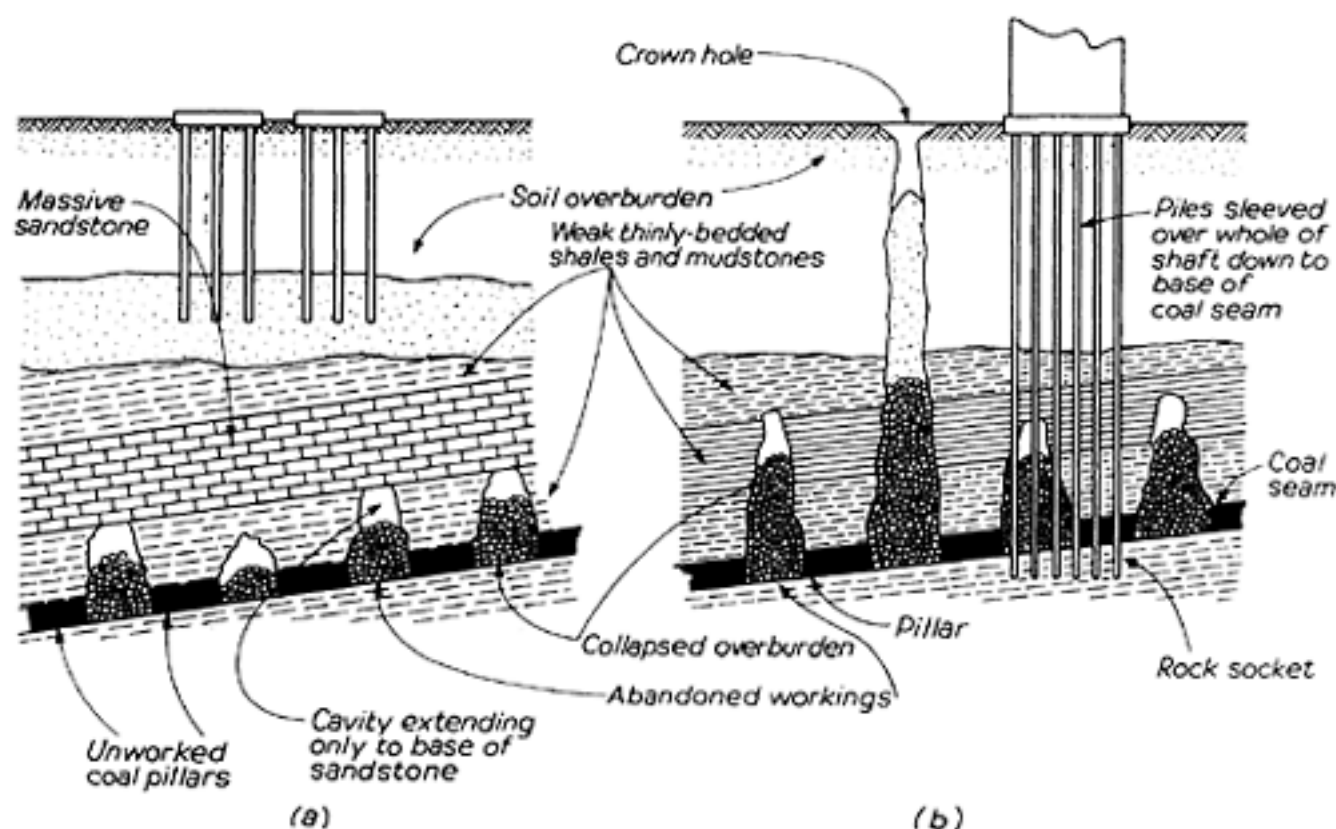


Fig. 9.13 Piling in areas of abandoned 'pillar and stall' mine workings

(a) Where massive rock forms stable roof over workings

*(b) Where roof over workings is weak and unstable*

be allowed between the pile shaft and the walls of the drill hole. As noted above it is necessary to seal the annulus around the piles to prevent gas seepage to the surface. Large-diameter piles are preferable to small sections because of their higher resistance to lateral loading that may be due to local distortions of the rock strata. The large-diameter drill holes also serve as a means of access for inspecting any cavities and to enable geologists to judge the stability conditions of the overburden. When inspecting the geological conditions in the shafts full precautions should be taken against the collapse of the walls or base of the drill hole and against the presence of explosive or asphyxiating gases (see [11.3.3](#)). The advice of an experienced mining engineer should be sought before exploring abandoned workings from the pile borehole.

## 9.4 Piling in frozen ground

### 9.4.1 General effects

In most parts of the UK the depth of penetration of frost into the ground does not exceed 600mm, and consequently frozen soil conditions are not detrimental to piled foundations. However, in countries lying in the northern latitudes with continental-type climates the penetration of frost below the surface gives rise to considerable problems in piling work. In the southern regions of Canada and in Norway the frost penetrates to depths of 1.2 to 2.1m. In far-northern latitudes the ground is underlain by great depths of permanently frozen soil known as 'permafrost'. About 49% of the land mass of the former USSR is a permafrost region, which generally lies north of latitude 50°. The depth of permafrost extends to 1.5km in some areas. Permafrost regions are also widespread in Northern Canada, Alaska and Greenland.

In areas where frost penetration is limited to a deep surface layer overlying non-frozen soil, the effect on pile foundations is to cause uplift forces on the pile shaft and on the pile caps and ground beams. These effects occur in frost-susceptible soils, *i. e.* soils which exhibit marked swelling when they become frozen. Frost-susceptible soils include silts, clays, and sand-silt-clay mixtures. Swelling of these soils occurs when water in the pores migrates into layers or lenses and becomes frozen. The increase in volume when the ice lenses form results in a heave of the ground surface. When these soils are frozen onto the shafts of piles, or onto the sides of pile caps and ground beams, the uplift forces tend to lift the foundations. The soil heave also causes uplift forces to develop on the undersides of the pile caps and ground beams. The uplift forces on the sides of the sub-structure are referred to as 'adfreezing' forces and measures to prevent the upward movement of piled foundations are described in the next section of this chapter.

The foundation problems presented by permafrost are much more severe, because of the extreme conditions of instability of this material within the depths affected by piling work. The permanently frozen ground is overlain by an 'active layer' that is subject to seasonal freezing and thawing. In the winter adfreezing occurs on foundations sited within frost-susceptible soils in the active layer. In summer there is rapid and massive collapse of thawing ice lenses in the active zone. Severe freeze-thaw conditions in highly frost-susceptible soils can result in the formation of dome-shaped ice-caverns as much as 6m high above the permafrost. The thickness of the active layer is not constant, but varies with cyclic changes in the climate of the region, with changes in the cover of vegetation such as mosses and lichens, and with the effects of buildings and roads constructed over the permafrost. The laws governing the physical, chemical and mechanical properties of frozen soil have been reviewed by Anderson and Morgenstern([9.9](#)).

Tsytoich([9.10](#)) has described three modes of formation of permafrost: these are when water-bearing soils are frozen through; when ice and snow are buried; and when ice is formed in layers in the soil. The latter formation, referred to by Tsytoich as 'recurrent vein ice', frequently occupies some 50% of the top 20m of the soil in the northern parts of the former USSR and is responsible for the severe foundation problems in the region. The recurrent vein ice can contain layers of unfrozen water within the permanently frozen soil and foundation pressure applied to such ground can result in substantial settlement. Because of the variation in thickness of the active layer the upper zone of the permafrost can undergo considerable changes such as major heaving, the collapse of ice caverns and the migration of unfrozen water.

### 9.4.2 The effects of adfreezing on piled foundations

Penner and Irwin([9.11](#)) measured the uplift forces caused by adfreezing on 89mm steel pipe anchored into unfrozen soil. The measurements were made in the Leda clay of Ontario in a region where a deep penetration of frost occurs below the ground surface. The Leda clay consists of a 70% clay fraction and a 30% silt fraction. The formation of ice lenses in the soil caused a surface heave of 75 to 100mm where the frost penetrated to a depth of 1.2m. The adfreezing force on the steel pipe was 96kN/m<sup>2</sup>.

Further measurements in the Leda clay have been reported by Penner and Gold<sup>(9.12)</sup>. When the frost penetrated to a depth of 1.09m, causing a surface heave of 100mm the measured peak adfreezing forces on anchored columns were as follows.

Steel:	113kN/m <sup>2</sup>
Concrete:	134kN/m <sup>2</sup>
Timber:	86kN/m <sup>2</sup>

Penner and Irwin<sup>(9.11)</sup> quote similar measurements by Kinoshita and Ono<sup>(9.13)</sup> as follows.

Iron pipe:	204kN/m <sup>2</sup>
Vinyl pipe:	193kN/m <sup>2</sup>
Concrete pipe:	134kN/m <sup>2</sup>
Epoxy-resin coated concrete pipe:	600kN/m <sup>2</sup>

Dalmatov<sup>(9.14)</sup> (as quoted by Penner and Irwin) has expressed adfreezing forces by the equation

$$T = \mu h_a (c - 0.5bt_s) \quad \dots(9.1)$$

where  $T$  is the total upward force due to frost heave,  $\mu$  is the perimeter of the foundation in contact with the soil in centimetres,  $h_a$  is the thickness of the frozen zone in centimetres,  $t_s$  is the surface temperature (°C), and  $b$  and  $c$  are constants

determined experimentally.

Dalmatov's values of  $b$  and  $c$  for timber piles in a silty sandy clay were 0.1 and 0.4 respectively. The measured forces on the steel pipe in Leda clay when expressed by Dalmatov's equation gave the same values of  $b$  and  $c$  but Penner and Irwin<sup>(9.11)</sup> regard this as somewhat coincidental.

Measurements of the magnitude of frost heave forces on steel plates restrained from uplift in Leda clay were made by Penner<sup>(9.15)</sup>, the plates being placed at various depths in the frozen layer. When the surface heave was 55mm the maximum swelling pressures of 54 to 91kN/m<sup>2</sup> occurred at the base of the frozen soil layer.

Adfreezing forces on the shafts of piles and on the sides of pile caps and ground beams can be eliminated or greatly reduced by removing frost-susceptible soils from around these substructures to a depth equal to the maximum penetration of frost predicted. These soils are replaced by suitable non-susceptible material such as clean sandy gravel or crushed and graded rock. Open gravels should not be used since ground water movements at periods of thawing might wash fine soil particles into the voids in the gravel, leading to the formation of a silty gravel susceptible to frost heave.

Uplift forces on the undersides of pile caps and ground beams can be reduced by interposing a layer of compressible material between the substructure and the soil. Cellular cardboard can be used for this purpose as shown in [Figure 7.13](#). Skaven-Haug<sup>(9.16)</sup> has described the use of blocks of compressed peat around basement retaining walls as a frost-retarding and filter layer to prevent the migration of fine soil particles into non-susceptible coarse gravels. If this expedient is adopted the risk of formation of methane by biological decay of the peat must be considered.

### 9.4.3 Piling in permafrost regions

Piled foundations are generally employed where structures in permafrost regions are sited in areas of frost-susceptible soils. Shallow foundations cannot normally be used because of the massive volume changes which take place in the active layer under the influence of seasonal freezing and thawing.

The general principle to be adopted when designing piled foundations is to anchor the piles securely into a zone of stable permafrost or into non-susceptible material such as well-drained sandy gravel or relatively intact bedrock. Where the piles are anchored into the permafrost layers their stability must be maintained by conserving as far as possible the natural regime which existed before construction was commenced in the area. Thus buildings must be supported well clear of the ground ([Figure 9.14](#)) to allow winds at sub-zero temperatures to remove the heat from beneath the buildings, and so prevent thawing of the active layer in the winter season.

The depth to which piles should be taken into the permafrost depends on the state of stability of this zone. Consideration must be given to the recurrence of cyclic changes in the upper layers, to the presence of layers of unfrozen water, and to the pre-treatment which can be given to the permafrost by thawing, compaction of the soil, and re-freezing.

Compressive loads on the piles are carried almost entirely by adfreezing forces on the pile shaft. Little end-bearing resistance is offered by the frozen ground due to the re-packing and re-crystallization of ice under pressure and the migration of

unfrozen water. Uplift forces on the piles must be allowed for. These occur as a result of adfreezing in the active layer in the winter season.

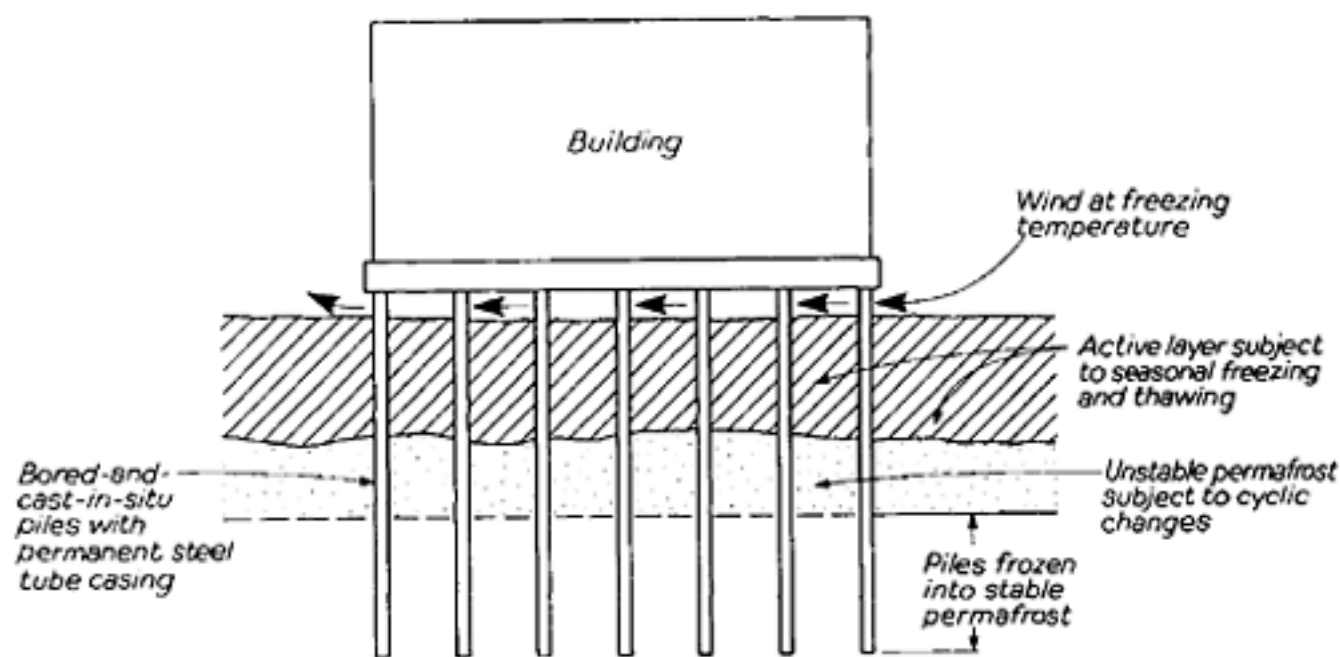


Fig. 9.14 Piling into permafrost

Piles should not be driven into permafrost since this will cause splitting of the frozen ground, thus allowing thawing waters to penetrate deeply into the cracks, and so upsetting the stable regime. Drilled-and-cast-in-place piles are suitable but the concrete must not be placed in direct contact with the frozen ground. North American practice is to use powered rotary augers to drill into the permafrost to the required depth. A permanent steel casing is then placed in the drill hole and filled with concrete. The heat of hydration thaws the surrounding ground and as the concrete cools the freezing of the melt water bonds the pile permanently to the permafrost. Water can be poured into the annulus between the casing and the permafrost to ensure full bonding. In Russia a form of reverse-circulation drilling with steam or gas burners is used to thaw the soil and flush the cuttings to the surface before freezing-in the permanent steel casing. Holes should be provided in the casing to drain off the water before the concrete is placed. If this is not done freezing of the water inside the casing may cause bursting of the steel.

## 9.5 Piled foundations for bridges on land

### 9.5.1 Selection of pile type

Bridge construction for developed countries is subject to many constraints concerned with access to sites and environmental conditions. These have an important influence on the selection of a suitable pile type and equipment for installation. In undeveloped territories the constraints are fewer and selection of suitable pile types is influenced mainly by the ground conditions.

When constructing new main highways it is desirable to complete under- and over-bridges at an early stage in the overall construction programme in order to facilitate the operation of earthmoving and paving equipment along the length of the highway without the need for detours or the use of existing public highways by construction equipment. Hence access to bridges will be difficult at this early stage and it may be impossible to route the piling equipment and material deliveries along the cleared highway alignment without interfering with the early earthmoving operations.

In the case of small bridges, such as those carrying minor roads over or beneath the main highway, it is desirable to use light and easily transportable equipment to install a number of small- or medium-diameter piles rather than a few large-diameter piles requiring heavy equipment. Suitable types are precast concrete or steel sections which have the advantage over bored piles of the facility to drive them on the rake thus providing efficient resistance to lateral forces, which are an important consideration in most bridge structures. Only small angles of rake are feasible with bored piles (3.4.5) and it is usually preferable to provide only vertical bored piles suitably reinforced to resist horizontal loads and bending moments.

Some of the most difficult access problems are involved with bridges in deep cuttings where the bridge is constructed in an isolated excavation in advance of the main earthmoving operations. It is possible to install the piling for piers of bridges with spill-through abutments at the toe of the cutting, and in the median strip from plant operating from ground level before bulk excavation is commenced for the bridge. This operation is similar to that adopted for 'top-down' basement construction in building work (5.9). It may also be possible to excavate the cutting to a temporary steep slope to enable piles to

be driven at the toe of a cutting using trestle guides ([Figure 3.8](#)), the piles being pitched by a crane standing at the crest of the cutting. However, such operations involve a risk of instability of the slope due to surcharge load, and, in the case of clay slopes, to excess pore pressures caused by soil displacement.

Bridge construction, or reconstruction, in urban areas involves piling in severely restricted sites with the likely imposition of noise abatement regulations. Driven types of pile have the advantages of speed and simplicity. Compliance with noise regulations may be possible by adopting a bottom driven type (2.2.4 and 3.2) in conjunction with sound absorbent screens surrounding the piling equipment. If possible pile caps should be located above ground-water level in order to avoid pumping from excavations which could cause loss of ground or settlement of adjacent buildings due to general drawdown of the ground-water table.

Piling over or beneath railways involves special difficulties. The presence of overhead electrification cables will probably rule out any form of bored or driven pile requiring the use of equipment with a tall mast or leaders. The railway authority will insist on piling operations being limited to restricted periods of track possession by the contractor if there is any risk of equipment or materials falling on to the track. Soil disturbance by large-displacement driven piles may cause heave or misalignment of the rails. If it is at all possible the design of the bridge should avoid the need for piling the foundations. As noted above, many of the constraints described in the preceding paragraphs do not apply to bridges in undeveloped territories. However, conditions of access to remote bridge sites should be investigated. Equipment should be capable of being transported over poor roads and across weak bridges of limited width.

### 9.5.2 Imposed loads on bridge piling

The various types of loading imposed on bridge foundations have been reviewed by Hambly<sup>(9.17)</sup> in a wide-ranging report published by the Building Research Establishment. The types of loading reviewed are:

- Dead and live loads on superstructure
- Dead load of superstructure
- Earth pressure (including surcharge pressure) on abutments
- Creep and shrinkage of superstructure
- Temperature variations in superstructure
- Traffic impact and braking forces on bridge deck (longitudinal and transverse)
- Wind and earthquake forces on superstructure
- Impact from vehicle collisions, locomotives and rail wagons
- Construction loads including falsework.

In UK practice the loading requirements are specified in British Standard 5400.

Dead and live load combinations should be considered in relation to permissible differential settlements between piers or between piers and abutments in longitudinal and transverse directions. Permissible settlements are often poorly defined or not defined at all by bridge designers. Hambly<sup>(9.17)</sup> states that a relative rotation of up to 1 in 800 is permitted by some designers for continuous deck bridges. Others limit the rotation to 1 in 4000, which is equivalent to a differential settlement of only 5mm between the supports of a 20m span bridge. This would be difficult to ensure for bridges with longer spans even if supported by piles taken down to a competent bearing stratum. The 1 in 800 criterion is frequently adopted for bridges with simply supported spans, but larger rotations may be needed in special conditions such as bridges in mining subsidence areas. The distribution of live load when assessing total and differential settlement is usually a matter of judgement. Full live load on the whole or part of the spans should be allowed for calculating immediate settlements but the contribution of live load to consolidation settlement may be small in relation to that from the dead loading. [Figure 9.15](#) shows the loading on a typical pier foundation for the 4km-long elevated section of the Jeddah-Mecca Expressway designed by Dar al-Handasah, consulting engineers. The piers support the 36m continuous spans of the three-lane carriageway. It will be noted that the predominant horizontal force on the piers was in a longitudinal direction, the resulting bending moments increasing the loads on the outer piles of the 8-pile group by about 25% above the combined vertical dead and live loads. It was possible to carry the horizontal forces and bending moments by 770mm diameter bored and cast-in-place base-grouted piles of the type described in [Section 3.3.7](#) using the Bauer process.

Horizontal earth and surcharge pressures on bridge abutments and wing walls are resisted much more efficiently by raking piles than by vertical piles. However, rakers provide a high degree of rigidity to the foundation in a horizontal direction which may require designing for at-rest earth pressures rather

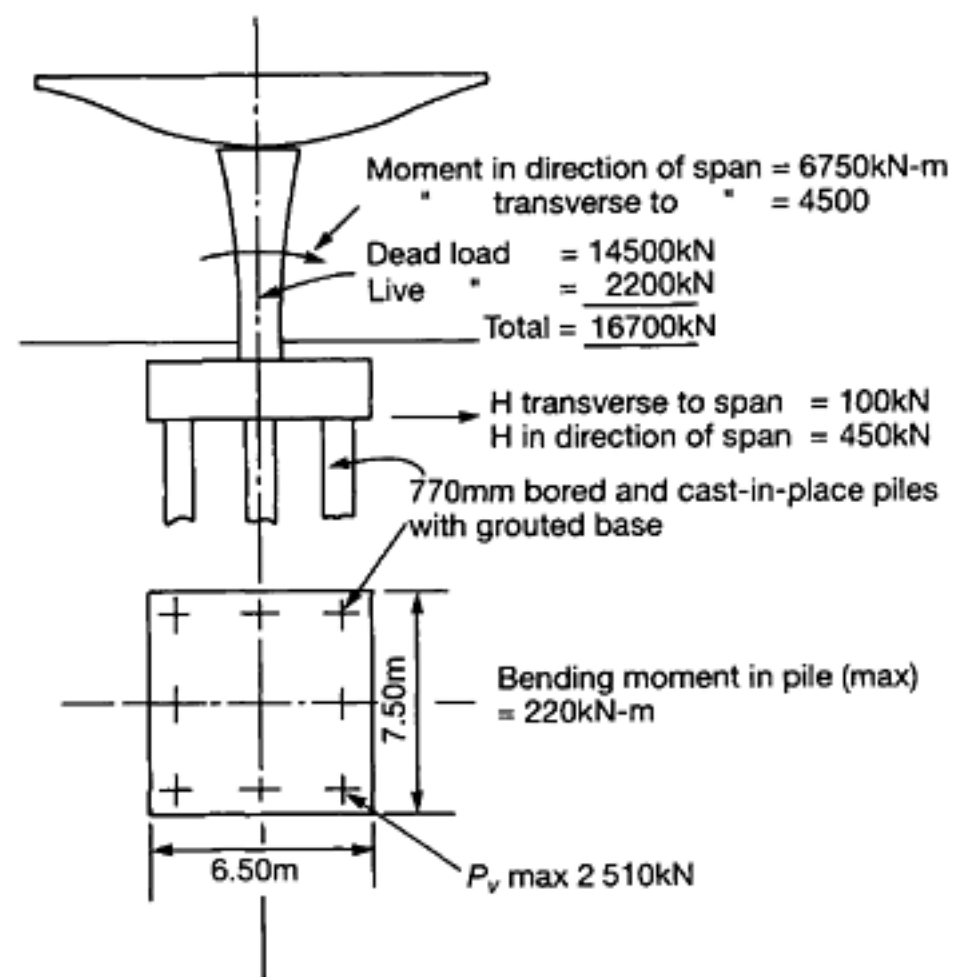


Fig. 9.15 Vertical and horizontal loads on viaduct piers of Jeddah-Mecca Expressway

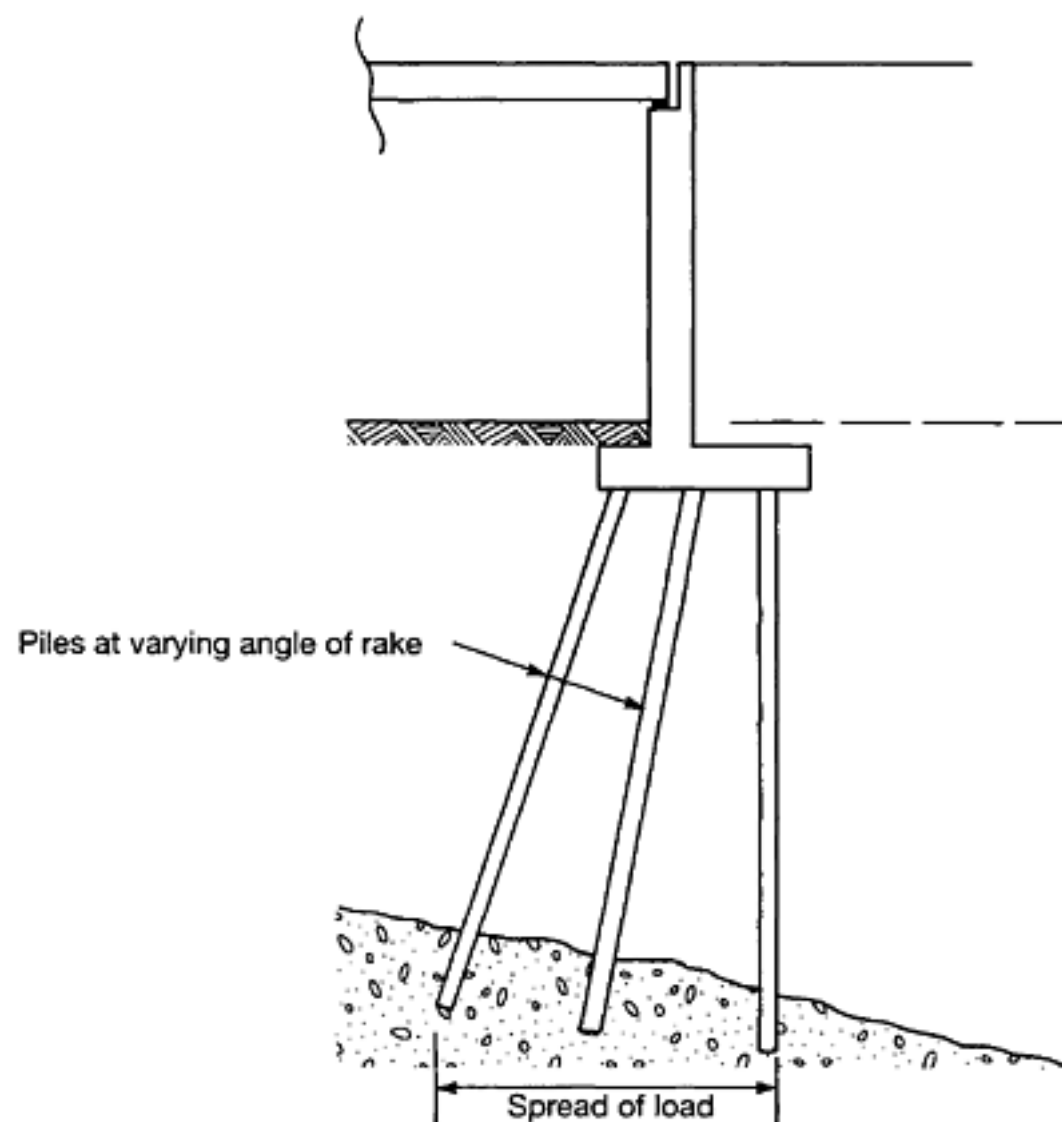


Fig. 9.16 Bridge abutment supported by raking piles

than the lower active pressures which depend on permitted yielding of the retaining structure. At-rest pressures are likely to be operating when the top of the abutment is strutted by the bridge deck as well as being restrained at the toe by the rows of raking piles.

The simple and computer-based methods of determining individual pile loads in groups of vertical and raking piles carrying a combination of vertical and lateral loads were described in [Section 6.5](#). Hambly<sup>(9.17)</sup> has pointed out the desirability of varying the angle of rake in order to avoid concentration of load on the bearing stratum ([Figure 9.16](#)).

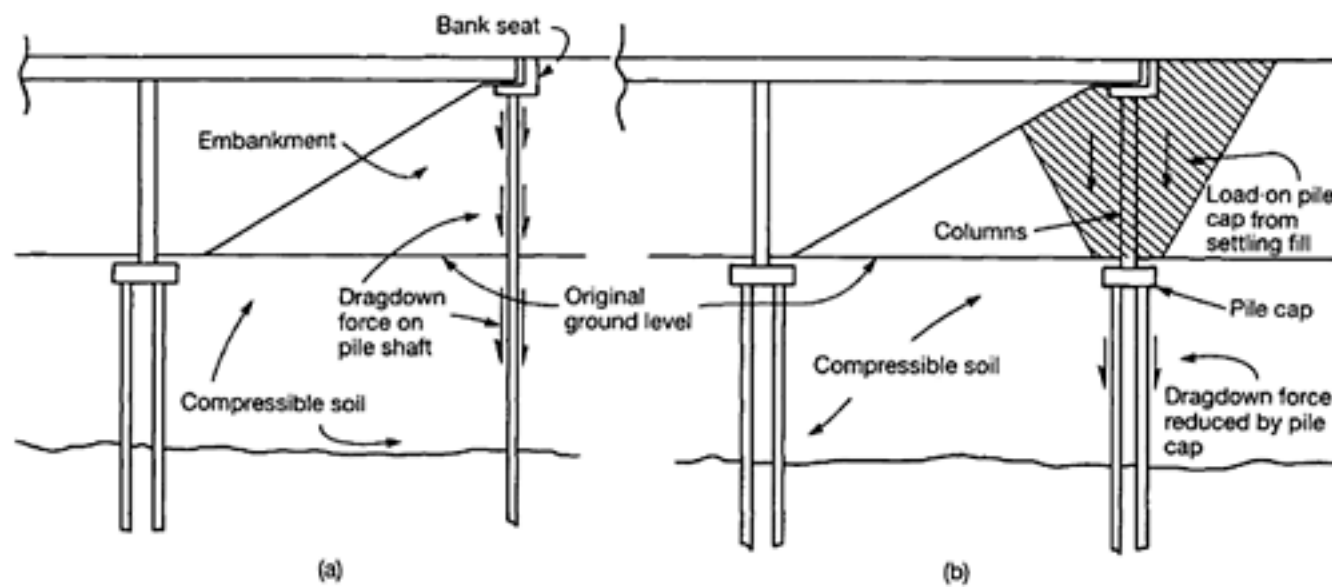


Fig. 9.17 Piling for bridges with spill-through abutments

(a) Bank seat carried by piles driven from completed embankment

(b) Bank seat carried by columns with pile cap at original ground level

In the case of bridges with spill-through abutments and embanked approaches the piles supporting the bank seats are best installed from the surface of the completed embankment (Figure 9.17a). In this way the dragdown forces from the settling embankment and any underlying compressible soils are carried only by vertical piles. The dragdown force can be minimized by using slender sections in high strength materials. If the piles are constructed at ground level with the bank seat supported on columns erected on a pile cap, the latter will act as a 'hard-spot' attracting load from the embankment fill (Figure 9.17b). The distribution of dragdown and lateral forces on bank seat and abutment piles will vary depending on their distance from the crest of the embankment and their location beneath the abutment pile cap. Taking the case of piles driven through soft clay to a bearing in a stiff but compressible clay (Figure 9.18a), the piles nearest to the crest of the embankment carry high negative skin friction both from the embankment fill and from the consolidating clay, while the piles nearer to the embankment toe carry considerably less negative skin friction. Because the bearing stratum is compressible the pile toes nearer to the embankment crest will yield more than those nearer to the toe and hence there is a tendency to a backward tilting of the pile group. This tilting movement is opposed by the forward movement of the fill and forward strain in the soft clay, and thus high bending moments are set up in the piles.

In the case of the piles supporting the bridge abutment (Figure 9.18b), they are prevented from backward tilting due to the incompressible stratum beneath the pile toes, but are subjected to lateral pressures due to strains set up by the embankment loading. The combined effects of these pressures and the earth pressure on the back of the abutment causes forward tilting of the pile group and raking piles

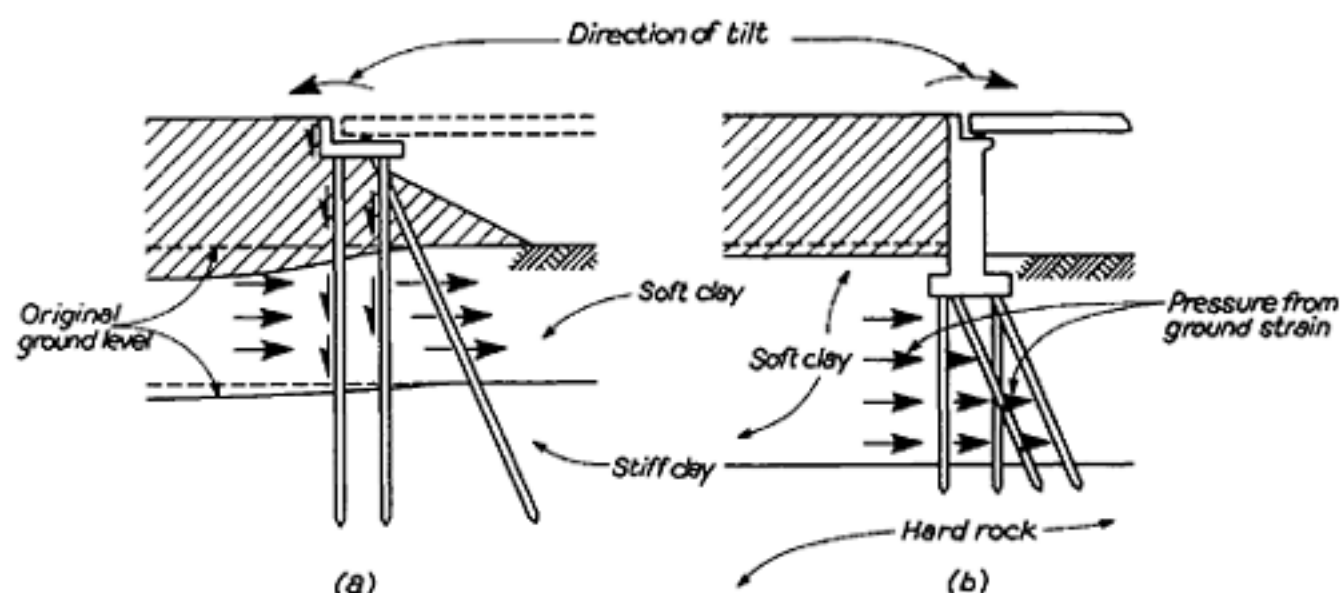


Fig. 9.18 Surcharge loading on pile group supporting bridge abutment

(a) Piles bearing on compressible stratum

(b) Piles bearing on incompressible stratum

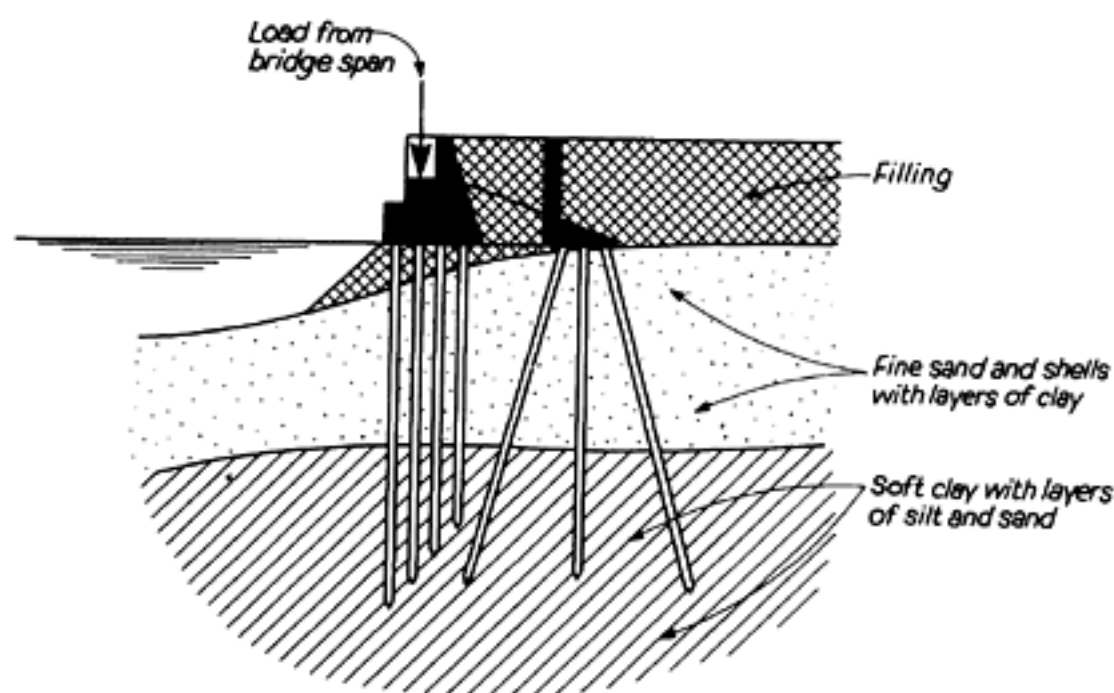


Fig. 9.19 Surcharge on pile group supporting bridge abutment in Denmark (after Bjerrum *et al.* (9.19))

are required to restrain this movement. Stermac *et al.* (9.18) observed a backward tilt of several centimetres in pile-supported abutments of bridges in Ontario. The piles were driven through soft clay to end bearing on glacial till or bedrock. Repeated repositioning of roller bearings supporting the bridge girders was necessary in some of the abutments. Bjerrum *et al.* (9.19) recorded the movements of a bridge abutment in Denmark where the supporting piles were terminated in a soft clay (Figure 9.19). Initially the abutment tilted backwards due to negative skin friction on the rear piles. Subsequently, when the arch span of the bridge was placed the front piles became more heavily loaded and the abutment tilted forwards. The net forward movement was 160mm.

wex (9.20) measured the bending moments and negative skin friction on a free-standing vertical pile driven close to the toe of an embankment placed on a soft peaty clay in Somerset. The head of the pile was restrained from lateral movement by a jack. Thus the pile acted as a beam fixed at the lower end in gravel and marl (Figure 9.20) and freely supported at the top. Readings were commenced 61 days after placing the embankment and by day 404 the negative skin friction on the pile was 120kN and the maximum bending moment was 30kNm. The observed drag-down forces and bending moments are shown in Figure 9.20. It was estimated that lateral pressures of up to 29kN/m<sup>2</sup> would have developed on the pile from the time of constructing the embankment.

De Beer and Wallays (9.21) have established a method of calculating the lateral pressure on vertical piles due to unsymmetrical surcharge loading. The surcharge is represented by a fictitious fill of height  $H_f$  with a sloping front face, as shown for three arrangements of piles and embankment loading in Figures 9.21a to 9.21c. The height  $H_f$  is given by

$$H_f = H \times \frac{\gamma}{1.8} \quad \dots(9.2)$$

where  $\gamma$  is the density of the fill in tonne/m<sup>3</sup>.

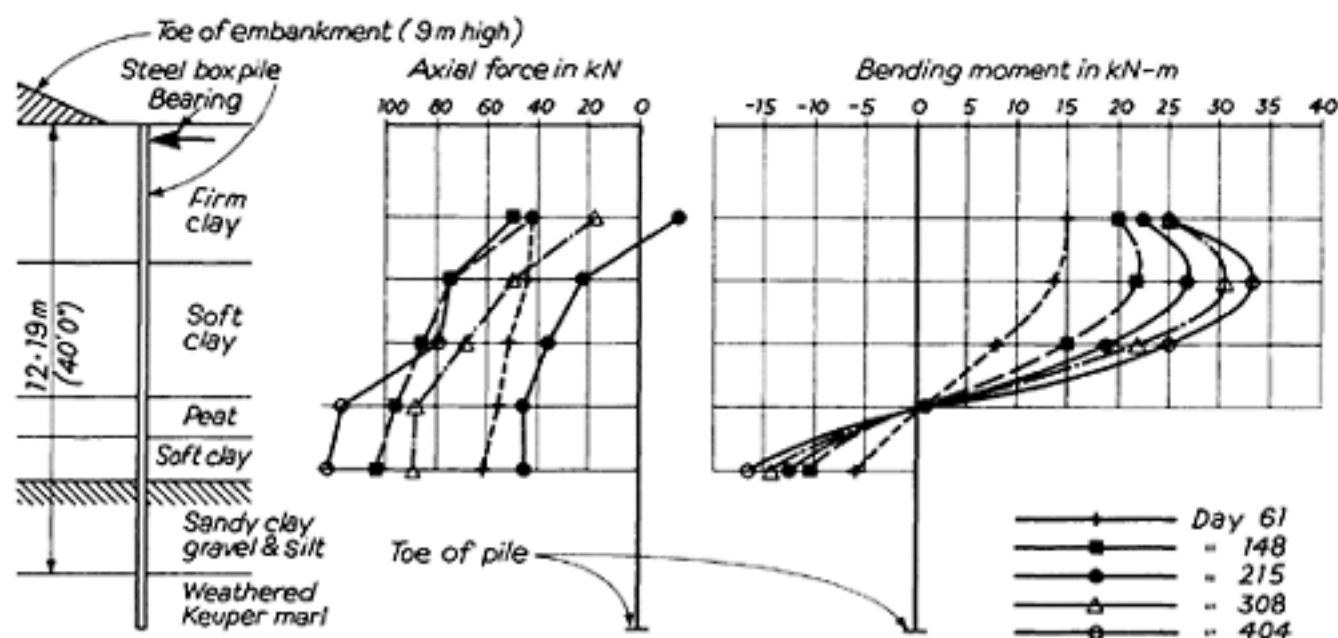


Fig. 9.20 Axial loading and bending moments on test pile due to surcharge loading from

*embankment (after Wex*[9.20](#)*)*

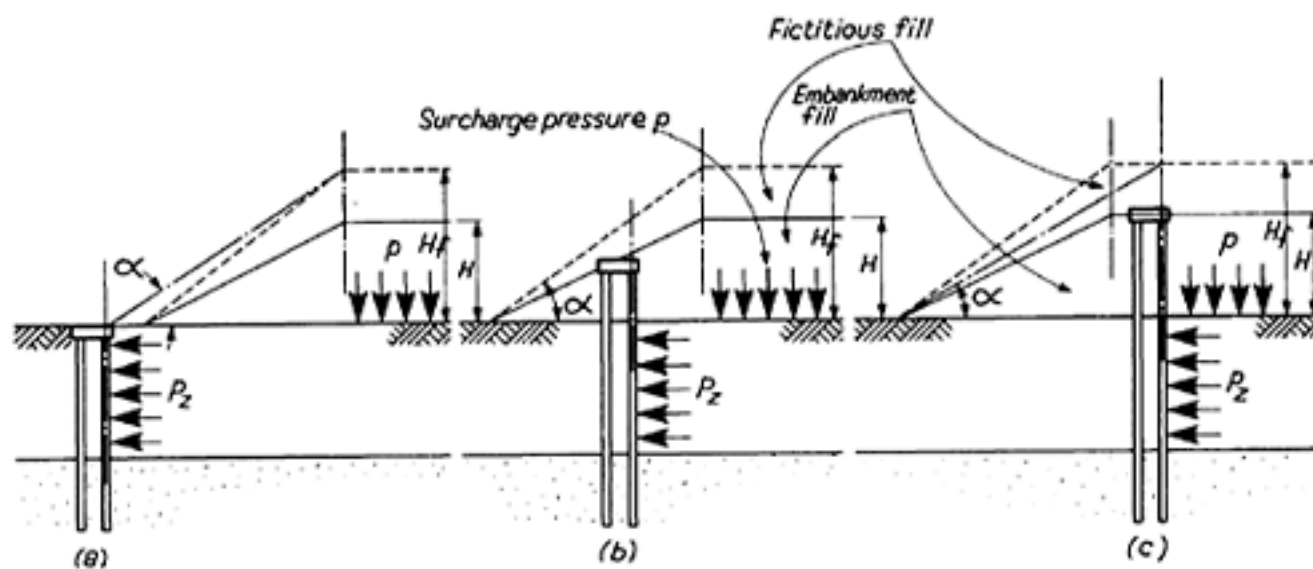


Fig. 9.21 Calculation of lateral pressure on vertical piles due to unsymmetrical surcharge loading (after De Beer and Wallays<sup>(9.21)</sup>)

The fictitious fill is assumed to slope at an angle  $\alpha$  which is drawn by one of the methods shown in Figures 9.21a to 9.21c, depending on the location of the surcharge loading in relation to the piles.

The lateral pressure on the piles is then given by

$$p_z = fp \quad \dots(9.3)$$

where  $f$  is a reduction factor given by

$$f = \frac{\alpha - \frac{1}{2}\phi}{90^\circ - \frac{1}{2}\phi} \quad \dots(9.4)$$

$p$  is the surcharge pressure, and  $\phi$  is the effective angle of shearing resistance of the soil applying pressure to the pile.

It should be noted that when  $\alpha \leq \frac{1}{2}\phi$  the lateral pressure becomes negligible. De Beer and Wallays point out that the method is very approximate. It should not be used to obtain the variation in bending moments along the pile shaft but only to obtain the maximum moment. They also make the important point that the calculation method cannot be used if the safety factor for conditions of overall stability of the surcharge load is less than 1.6. For lower safety factors the method of Brinch Hansen (6.19) is recommended for obtaining the ultimate lateral resistance of the piles and hence their contribution to the restraint of the surcharge fill against slipping.

Driving piles within or close to the toe of clay slopes can result in the development of excess pore pressure which may cause slipping of the slope. Massarsch and Broms<sup>(9.22)</sup> have developed a method of predicting the excess pore pressures induced by the soil displacement.

It is very difficult to avoid relative settlement between a piled bridge abutment and the fill material forming an embanked approach behind the abutment. Settlement of the fill often occurs even when well-compacted granular material is used. Relative settlement can be large where the embankment is placed on a compressible clay. The concept of allowing piles to yield under load was adopted by Reid and Buchanan<sup>(9.23)</sup> for the purpose of reducing the relative settlement of a piled bridge abutment and the approach embankment which was founded on soft compressible clay. The arrangement of piles is shown in Figure 9.22. The piles beneath the embankment close to the abutment were at close-spacing and were designed to carry the whole of the embankment load with a safety factor of 2. After the first four rows the spacing was increased to a 3 to 4m grid and the piles were made successively shorter so that they would yield under a progressively increasing proportion of the embankment load. The piles had circular caps 1.1m to 1.5m diameter. Loading from the embankment was distributed to the pile heads by a flexible membrane consisting of two layers of Terram plastics fabric reinforced with Paraweb strapping.

## 9.6 Piled foundations for over-water bridges

### 9.6.1 Selection of pile type

Because of the desirability of avoiding different types of piling on the same bridge project the piling used for piers constructed in over-water locations will usually dictate the type to be used for the

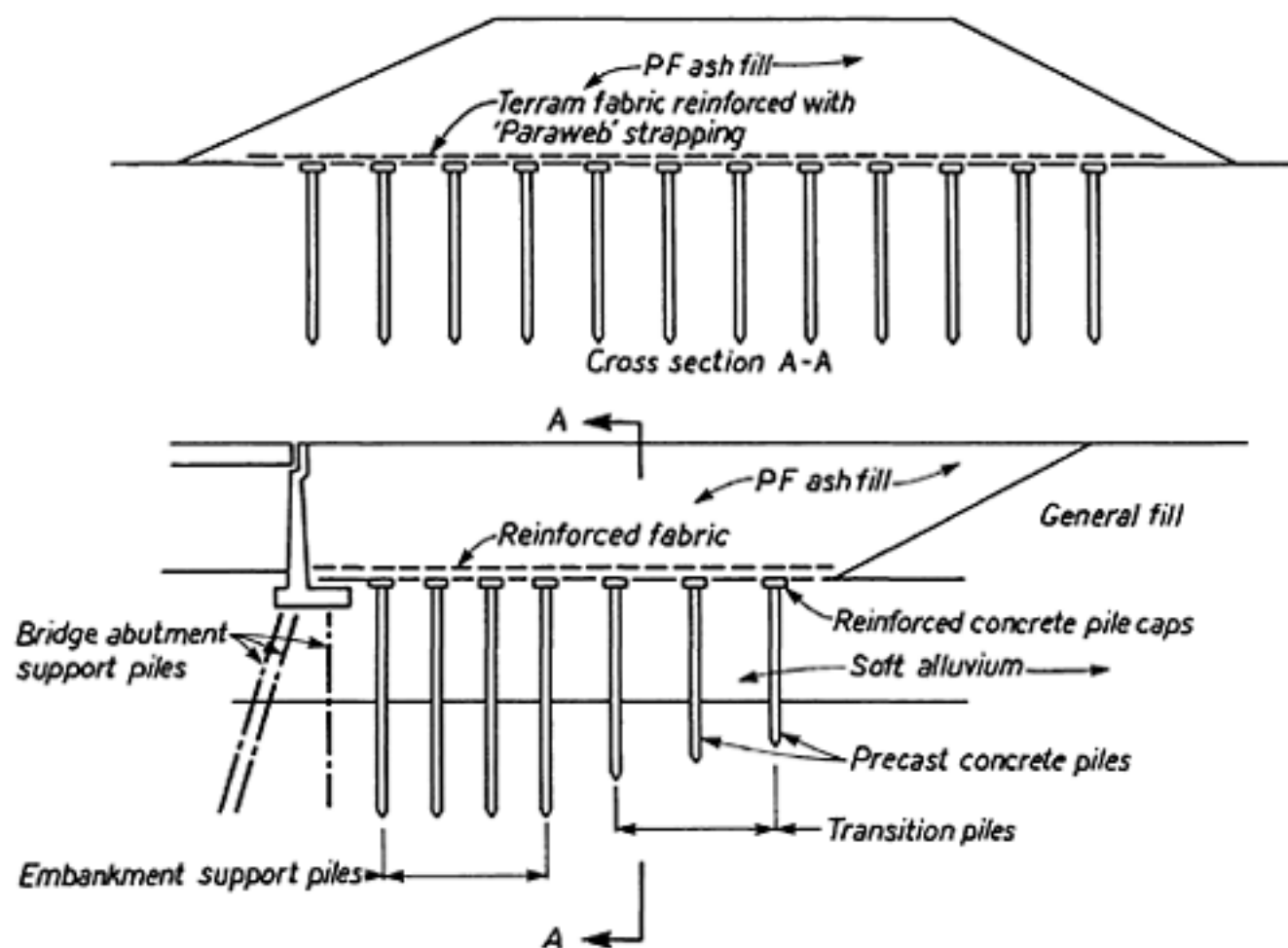


Fig. 9.22 Arrangement of settlement reducing piles beneath bridge approach embankment (after Reid and Buchanan<sup>(9.23)</sup>)

abutments. Driven piles are the favoured type for over-water piers. The installation of bored piles is limited to work carried out either in a pumped-out cofferdam, or in a permanent casing driven below river bed. In fast-flowing rivers the casing will have to be taken down to a sufficient depth below the river bed to obtain fixity against overturning in conditions of bed scour. Tubular steel piles or precast concrete piles of cylindrical section are preferred to H-sections in order to minimize current drag and eddies causing bed scour. The need for raked piles for efficient resistance of lateral forces again favours a driven type of pile. Where precast prestressed cylindrical piles are used in deep-water locations or for deep penetrations below bed level there can be problems with handling long heavy piles. Also, forming joints to extend partly driven piles can cause difficulties and delays.

Attrition by soil particles of the exterior surface of piles at the sea or river bed can be a factor influencing the material of the pile and its wall thickness. This is more likely to be a problem where the bed level is constant or changing over a limited range rather than rivers where seasonal floods cause wide variations in bed contours.

A notable example of precast concrete piling for bridge works is the over-water sections of the 25km causeway between Saudi Arabia and Bahrain Island<sup>(9.24)</sup>. The bridge sections of the causeway form a total length of 12.5km and were constructed in water depths ranging from 5 to 12m. A single 3.50m OD  $\times$  0.35m wall thickness precast concrete cylinder supports the 50m span box girder carrying the two-lane carriageway of the dual carriageway bridge (Figure 9.23). The cylinders were cast vertically in short sections at the shore-based casting yard. The sections were then formed into complete piles by longitudinal prestressing and transported to the bridge locations by a 1000 tonne crane barge. The crane lifted the piles into the guides of the twin drilling rigs mounted on a jack-up platform (Figure 9.24). Reverse-circulation drilling using a Wirth rotary table was used to take the piles down to the required penetration below sea bed where the base was plugged with concrete.

### 9.6.2 Imposed loads on piers of over-water bridges

In addition to the loadings listed in Section 9.5.2 the piles of over-water bridges are required to withstand lateral forces from current drag and wave action, pressure from floating flood debris or ice, and impact from vessels straying from the designated navigation channels.

Current drag and wave forces can be calculated using the methods described in Sections 8.1.3 and 8.1.4. The profile of the current velocity with depth varying from a maximum at the water surface to a minimum at bed level must be considered in relation to the bending moments on piles in deep fast flowing rivers. Current-induced oscillation can also be a problem in these conditions. It is also necessary

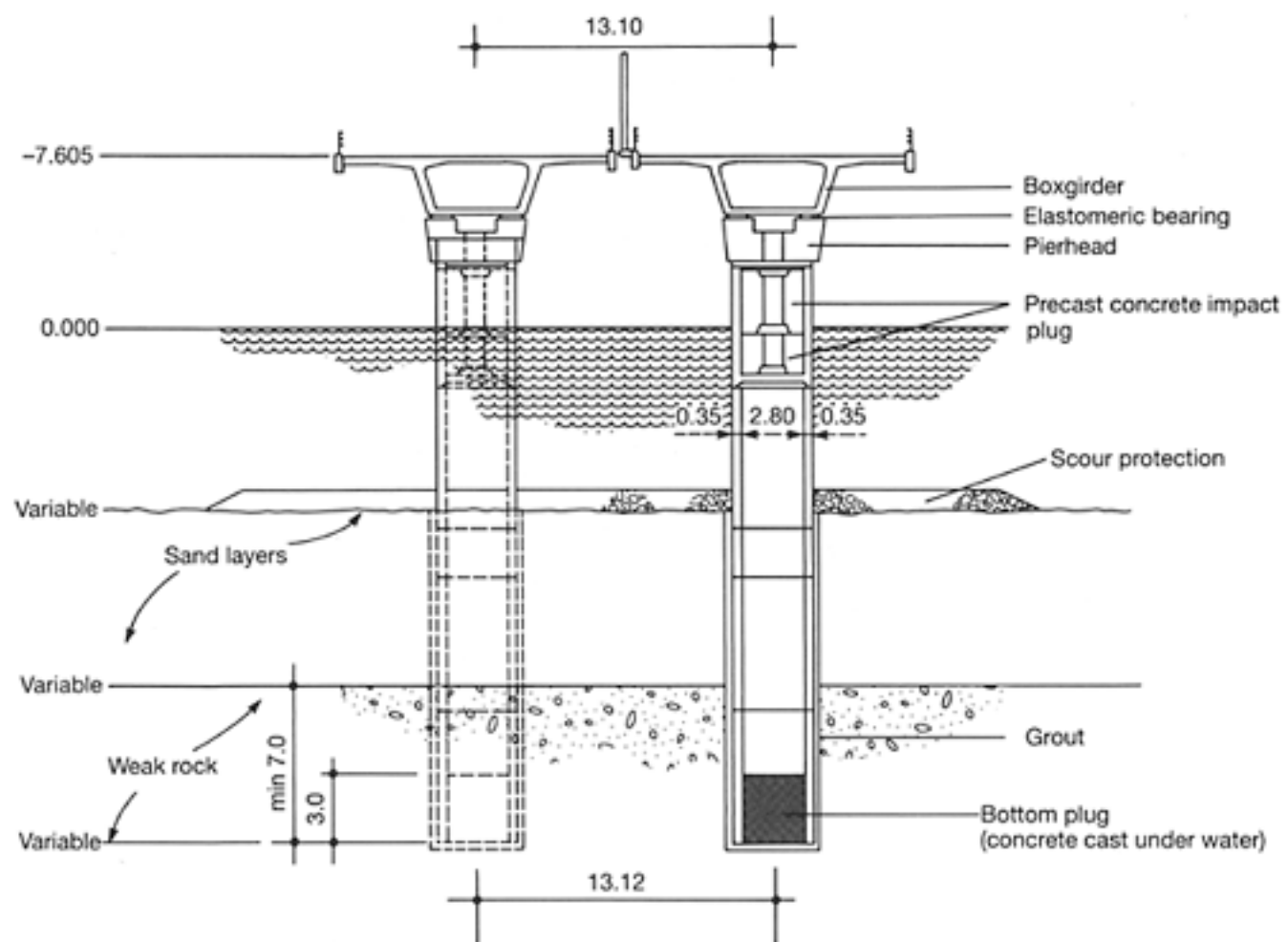


Fig. 9.23 Cylinder pile foundations for the Saudi Arabia-Bahrain Causeway (after Beestra et al. (9.24) Courtesy of Ballast Nedam)



Fig. 9.24 Drilling equipment for sinking 3.50m OD precast concrete cylinder piles, Saudi Arabia-Bahrain Causeway

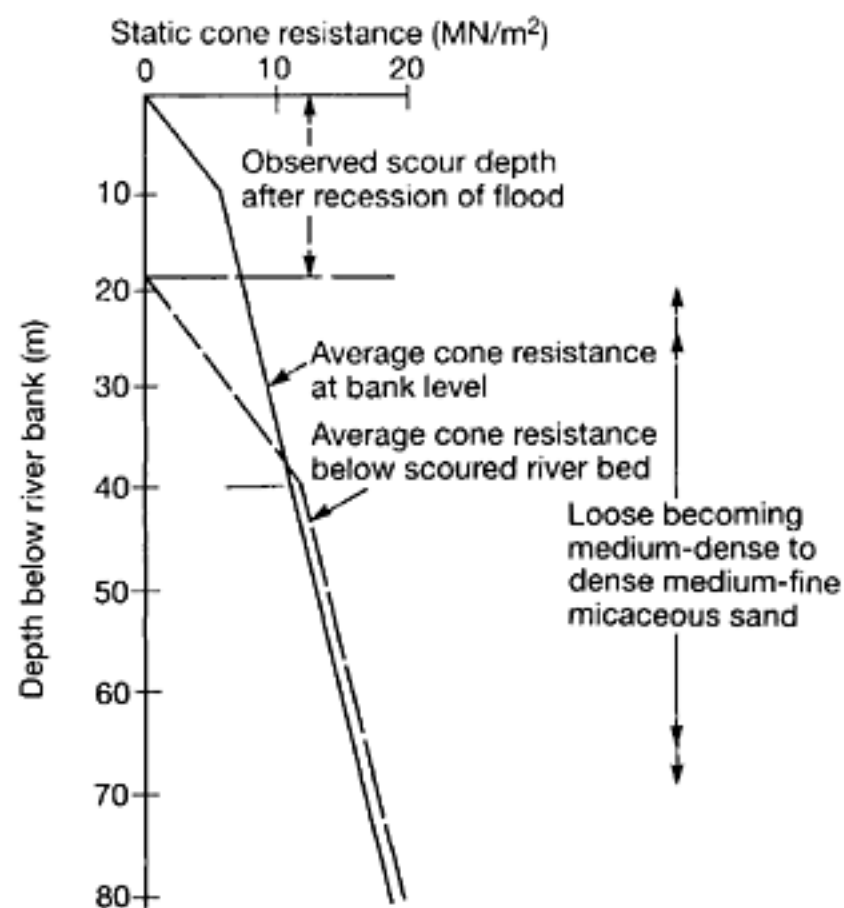


Fig. 9.25 Reduction in cone resistance due to unloading by river bed scour, Jamuna River, Bangladesh (after Broug<sup>(4.37)</sup>)

to calculate the lateral deflections at pile head level because these can induce bending of the bridge superstructure in the horizontal plane.

The depth of *scour* below river bed around piles at times of peak flood must be estimated for the purpose of calculating bending moments due to current drag forces and wave action on piles. The scour consists of three components, (a) general scour from changes in bed levels across the width of the channel, (b) formation of troughs in 'sand waves' which move downstream with the passage of the flood, and (c) local scour around the piles. An extreme example of the influence of bed scour on bridge foundations is given by the design studies for a proposed multi-span bridge crossing the Jamuna river in Bangladesh<sup>(9.25)</sup>. At the selected site near Sirajgang the 15km wide flood plain is proposed to be reduced to 4.8km width by training works. Scour depths up to 25m below bank level have been measured in the unrestricted waterway during seasonal floods. It was estimated that the scour depths could be increased by 10m around the piles in the constricted channel. In addition to the loss of skin friction support to the proposed 2.5m OD tubular steel piles, the effect of relief of overburden pressure on the coefficient of lateral pressure at the pile/soil interface had to be considered. The skin friction and end-bearing resistance of the piles was calculated from the results of static cone penetration resistance made at low-water season river-bed levels. The reduction in cone resistance with depth due to the unloading of the river bed was calculated by the method developed by Broug (see [Section 4.3.7](#) and reference 4.37). It was possible to obtain some confirmation of the validity of this approach by observations of the  $q_c$ /depth profile on the river banks and in a deep scour hole about 3km downstream of the

crossing site ([Figure 9.25](#)).

*Impact by ships* can be a severe problem in the design of bridge support piles in situations where impact cannot be absorbed by massive structures such as caissons or piers constructed inside cofferdams. It is difficult to achieve an economical solution to the problem particularly at deep water locations. The incidence of random collisions between ships straying from the navigable channel and bridge piers has not decreased since the introduction of shipborne radar. In fact it may have increased because of the false sense of security given by such equipment.

Three possible methods of protecting piled foundations may be considered. In shallow water not subject to major bed changes and with a small range between high and low water the pile group can be surrounded by an *artificial island* protected against erosion by rockfill. [Figure 9.26](#) shows a cross-section of one of four islands protecting the piers of the Penang Island Bridge<sup>(9.26)</sup>. This form of protection has the added advantage of preventing local scour around the foundations. The island must be large enough to prevent impact between the overhanging bows of a ship and the bridge pier or pile if the vessel should ride up the slope of the island when drifting out of control in a fast-flowing river.

*Piles can be strengthened* against buckling under direct impact by increasing the wall thickness and a group of piles can be given lateral restraint by a diaphragm connecting them at some point between the cap and bed level. The cylinder piles of the Bahrain Causeway bridge were strengthened by the

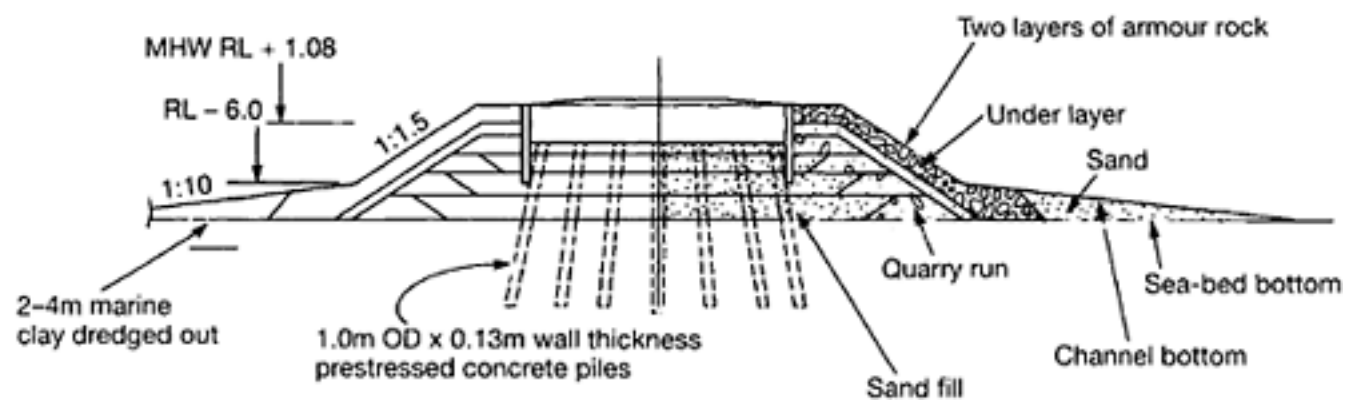


Fig. 9.26 Artificial islands protecting the piers of the Penang Island Bridge, Malaysia (after Chin Fung Kee and McCabe<sup>(9.26)</sup>)

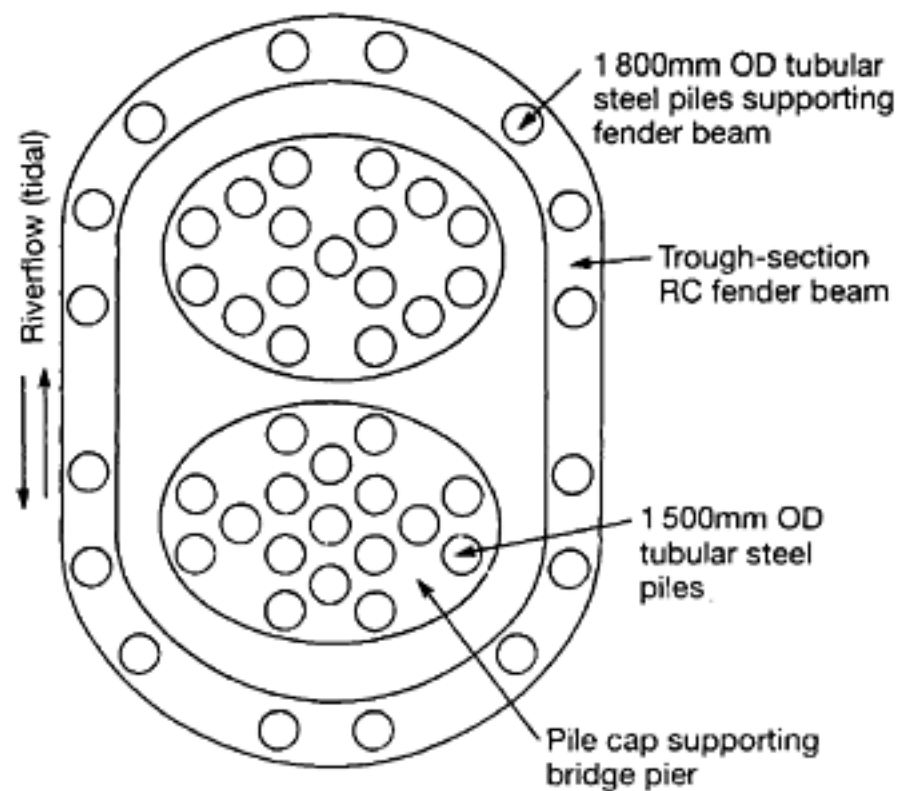


Fig. 9.27 Fender beam and piles protecting the river piers of the Sungei Perak Bridge, Malaysia (after Stanley<sup>(9.27)</sup>)

insertion of precast concrete elements to increase the thickness over the zone of possible impact ([Figure 9.23](#)).

*Fender piles* installed independently of the piers can be installed in deep-water locations. Piles are required to protect the sides of the piers as well as the ends in case of impact at an angle to the axis of the pier. The arrangement of fender piles capped by a massive reinforced concrete ring beam to protect the piers of the Sungei Perak Bridge in Malaysia are shown in [Figure 9.27](#). The ring beam was constructed by placing precast concrete trough sections on the piles, sealing the joints between the sections and placing the reinforcement and concrete infill in dry conditions. The loading on fender piles is calculated in the same way as fender piles for berthing structures ([Section 8.1.1](#)).

### 9.6.3 Pile caps for over-water bridges

It can be advantageous to locate pile caps at or below low river or low tide level. It avoids floating debris building-up between piles, and ensures that if collision by vessel does occur the impact will be on a massive part of the substructure instead of directly on a pile. Also a vessel is likely to sheer off at the first impact with a pile cap whereas it might become trapped when colliding with a group of piles. A pile cap at or below water level is preferable, aesthetically, to one exposed at low water. However high-level pile caps are economical for a bridge requiring a high navigation clearance, but such an arrangement would have to be restricted to approach spans in water too shallow to be navigable by vessels which could demolish piles supporting a high-level deck bridging the navigation channel.

Pile caps partly submerged or wholly below water level can be constructed within sheet pile cofferdams ([Figure 9.28a](#)). The sheet piles can be cut-off at low water to give protection against scour. Alternatively, if a heavy lifting barge is available a precast concrete cap in the form of an open-topped box can be lowered on to collars welded to the heads of the piles and prevented from floating by clamps. The annulus between the pile wall and the opening in the box can be sealed by quick-setting concrete or



The latter method was used for fifteen of the piers carrying the 3.3km-long bridges between the islands of Sjaelland and Falster in Denmark<sup>(9.28)</sup>. A group of forty-nine 700mm tubular steel piles in two concentric rings supported the deep-water piers in the navigable channel. The first operation was to form a level bed by dredging with protection against scour by rockfill. Then the piles were driven leaving their heads projecting about 6m above the prepared bed ([Figure 9.29](#)). During these operations a precast reinforced concrete conical base unit weighing 440 tonne was being fabricated on shore. It was taken out to the pier site and lowered over the projecting pile heads on to three pinning piles. Concrete was then pumped into place under water to about mid-height of the base unit. The next operation was to lower a temporary circular steel cofferdam on to the top of the base unit to which it was locked by stressed rods. The cofferdam consisted of an assembly of steel plate rings 10 to 11m in diameter and 3m deep. The joints between the rings were sealed by rubber sheeting. This was followed by dewatering the cofferdam and constructing the pier, after which the cofferdam was flooded and lifted off the base unit by floating crane for transport to the next pier location.

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## 9.8 Worked example

### Example 9.1

An embankment 9m high consisting of fill having a density of  $2.1\text{Mg/m}^3$  is placed with its toe 1.5m from a row of vertical piles supporting a bridge pier. The piles are driven through 8m of soft to firm clay into a stratum of stiff clay. Calculate the lateral pressure on the piles within the stratum of soft to firm clay which has drained shearing strength parameters of  $c' = 0$  and  $\phi' = 28^\circ$

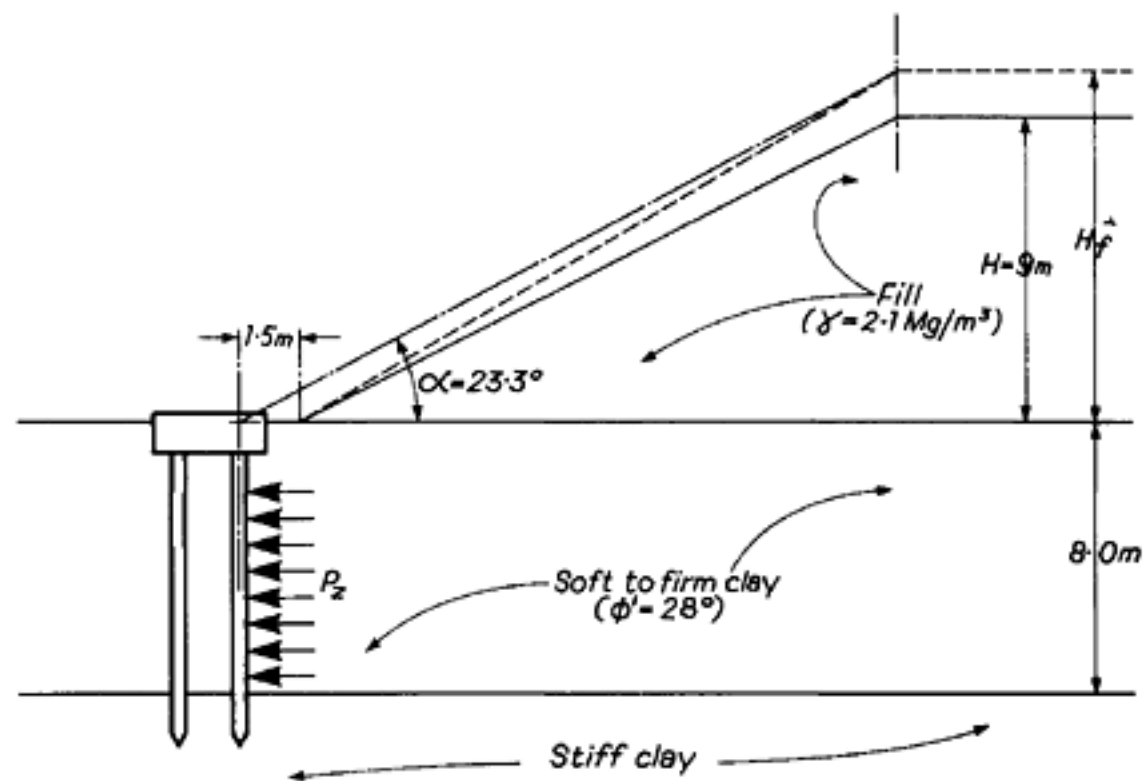


Fig. 9.30

From equation 9.2,  $H_f = 9 \times 2.1 / 1.8 = 10.5\text{m}$ . From Figure 9.30,  $\alpha = 23.3^\circ$ . Therefore from equation 9.4,

$$f = \frac{23.3^\circ - (28^\circ \times \frac{1}{2})}{90^\circ - (28^\circ \times \frac{1}{2})} = 0.12.$$

Surcharge pressure from embankment  $p = 2.1 \times 9 \times 9.81 = 185\text{kN/m}^2$ .

From equation 9.3, lateral pressure on piles  $= 0.12 \times 185 = 22\text{kN/m}^2$ .

# CHAPTER 10

## The durability of piled foundations

### 10.1 General

In all situations consideration must be given to the possibility of the deterioration of piled foundations due to aggressive substances in soils, in rocks, in ground waters, in the sea, and in river waters. Piles in river or marine structures are also exposed to potentially aggressive conditions in the atmosphere and they may be subjected to abrasion from shifting sand or shingle, or damage from floating ice or driftwood.

In considering schemes for protecting piles against deterioration due to these influences, the main requirement is for detailed information at the site investigation stage on the environmental conditions. In particular, adequate information is required on the range of fluctuation of river or sea levels and of the ground-water table. In the latter case, the highest levels are required when considering the likely severity of sulphate attack on concrete piles or the corrosion of steel piles, and the lowest possible levels are of considerable importance in relation to the decay of timber piles. The possibility of major changes in ground-water levels due to, say, drainage schemes, irrigation, or the impoundment of water must be considered.

In normal soil conditions it is usually sufficient to limit chemical analyses of soil or ground-water samples to the determination of pH-values, sulphate content and chloride content. Where the sulphate content exceeds 0.24% in soils it is advisable to determine the water-soluble sulphate content, expressing this in grammes of  $\text{SO}_4$  per litre of water extracted (10.1) (see [Table 10.1](#)).

Where fill material is present, particularly household refuse or industrial wastes, full chemical analyses are required to identify potentially aggressive substances. Attention to the need to obtain representative samples for these analyses is important. Preferably the samples should be obtained from trial pits in which zones of contaminated material can be identified by their appearance, supplemented by on-site testing to determine the pH-value.

Bacterial action can be an influence in the corrosion of steel piles. Samples of soil and ground water should be obtained in sterilized containers, which are then sealed for transportation to the bacteriological laboratory for later analyses. Where steel piles are used for foundations in disturbed soils or fill material on land, an electrical resistivity survey is helpful in assessing the risk of corrosion and in the design of schemes for cathodic protection (see [10.4.2](#)).

Investigations for marine or river structures should include a survey of possible sources of pollution which might encourage bacteriological corrosion, such as contaminated tidal mud flats, discharges of untreated sewage or industrial effluents, dumping grounds for industrial or household refuse, and floating rubbish discharged from ships or harbour structures. The pattern of sea or river currents should be studied and water samples taken at various stages of spring and neap tides, or at dry-weather and flood discharge stages in rivers. Particular attention should be paid to sampling water from currents originating at the areas of contamination previously identified. Chemical and bacteriological analyses should be made on the full range of samples to assess the daily or seasonal variation in potentially aggressive substances. Other items for study include the presence and activity of organisms such as weeds and barnacles, and molluscan or crustacean borers (see [10.2.2](#)).

### 10.2 Durability and protection of timber piles

#### 10.2.1 Timber piles in land structures

Timber piles permanently below ground-water level have an indefinite life. There are numerous examples of stumps of timber piles that are more than 2 000 years old being found in excavations below the water

Table 10.1 Classification of concrete in soils and ground-waters affecting concrete in piles and recommended precautions

Class	Concentration of sulphate and magnesium					Groundwater conditions
	In soil or fill			In ground water (g/l)		
	By acid extraction	By 2:1 water extract (g/l)				
	%SO <sub>3</sub>	SO <sub>4</sub>	Mg	SO <sub>4</sub>	Mg	
1	<0.24	<1.2	—	<0.4	—	Mobile Dry or static
2	If greater than 0.24 classify on basis of 2:1 extract	1.2–2.3	—	0.4–1.4		Mobile Dry or static
3		2.3–3.7		1.4–3.0		Mobile Dry or static
4		3.7–6.7	<1.2	3.0–6.0	<1.0	Mobile Static
		3.7–6.7	>1.2	3.0–6.0	>1.0	Mobile Static
5		>6.7	<1.2	>6.0	<1.0	
		>6.7	>1.2	>6.0	>1.0	

*Types of cement*

- A Portland cement to BS 12
- B Portland blastfurnace cement to BS 146
- C High slag blastfurnace cement to BS 4246
- D Combination of Portland cement to BS 12 and blastfurnace slag to BS 6699
- E Portland pfa cements to BS 6588
- F Combination of Portland cement to BS 12 and pfa to BS 3892: Part 1
- G Pozzolanic pfa-cement to BS 6610:1991
- H Sulphate-resisting Portland cement to BS 4027
- I High-slag blastfurnace cement to BS 4246 containing not less than 74% slag by mass of nucleus
- J Combinations of Portland cements to BS 12 and blastfurnace slag to BS 6699 containing not less than 70% slag and not more than 90% slag by mass of slag plus cement
- K Portland pfa cements to BS 6588 containing not less than 26% pfa by mass of nucleus
- L Combinations of Portland cements to BS 12 and pfa to BS 3892: Part 1 containing not less than 25% pfa and not more than 40% pfa by mass of pfa plus cement

table. While timber does not decay from fungal attack if the moisture content is kept below 20% it is impossible to maintain it in this dry condition when buried in the ground above water level. Hence damp timber which does not have natural durability is subject to decay by fungal attack, resulting in its complete disintegration. [Figure 10.1](#) shows an example of the decay of timber piles above the water table. [Figure 10.1a](#) shows the cavities left by the complete decay of the timber. The timber capping beams have also decayed, allowing the stone lintels to sink down onto the ground surface. [Figure 10.1b](#) is a view down a cavity which is partly filled by soil debris and fragments of decayed timber. The piles were driven into clay fill in the early 19th century. Preservative treatment can, however, give a useful life to timber piles in the zone above ground-water level. If treatment is applied to properly air-seasoned wood at the correct moisture content for the impregnation of the preservative, a life of several decades may be achieved.

Potter<sup>(10.2)</sup> has classified various grades of durability in terms of their approximate life when in contact with the ground ([Table 10.2](#)). Types of softwoods and tropical hardwoods suitable for timber piles are listed in [Section 2.2.1](#) ([Table 2.1](#)). Their durability is classified by Potter as

Precast concrete piles and pile shells (a)			Bored and cast in-place piles with shaft diameters equal to or greater than 600mm (b)			Bored and cast in-place piles with shaft diameters less than 600mm. Concrete filling to thin metal shells less than 140mm diameter (c)		
Cement type and min. cement content (kg/m)	Max. water/ cement ratio		Cement type and min. cement content (kg/m)	Max. water/ cement ratio		Cement type and min. cement content (kg/m)	Max. water/ cement ratio	
A-L	300	0.60	A-L	320	0.60	A-G	370	0.50
						—	340	0.55
						I-L	340	0.55
A-L	300	0.60	A-L	320	0.60	I-L	340	0.55
A-L	300	0.60	A-L	320	0.60	—	370	0.50
						I-L	380	0.50
A-L	300	0.60	A-L	320	0.60	A-G	370	0.55
						—	340	0.55
						I-L	340	0.55
A-G	330	0.50	A-G	340	0.55	—	370	0.50
—	290	0.55	—	340	0.55	I-L	380	0.50
I-L	310	0.55	I-L	360	0.55	A-G	370	0.50
I-L	310	0.55	I-L	360	0.55	—	340	0.55
						I-L	340	0.55
—	330	0.50	—	380	0.50	—	410	0.45
I-L	350	0.55	I-L	400	0.50	I-L	430	0.45
I-L	350	0.50	I-L	400	0.50	—	370	0.50
						I-L	380	0.50
—	360	0.45	—	340	0.55	—	410	0.45
I-L	380	0.45	I-L	360	0.50			
I-L	380	0.45	I-L	360	0.50	—	410	0.45
						I-L	380	0.50
As for Class 4 with surface protection.						As for Class 4 with surface protection. Thin shells must be durable.		

*Notes*

(1) The minimum cement contents recommended above are suitable for

(a) Precast concrete with low workability (12–25mm slump)

(b) Cast-in-place concrete in pile boreholes with high workability (75mm min slump)

(c) Cast-in-place concrete in pile boreholes and shells (100mm min slump).

(2) The above mixes are not suitable for placing under water by tremie pipe for which the minimum cement content should be 400kg/m<sup>2</sup> and slump of 150mm or more.

(3) The recommended mixes relate to concrete with a nominal maximum aggregate size of 20mm. The cement contents shown in the table can be reduced by using a suitable plastizer in the mix.

Douglas fir

—moderately durable

Pitch pine

—durable

Larch

—moderately durable

Douglas fir-larch

—moderately durable

Western red cedar

—durable

Greenheart

—very durable

Jarrah

—very durable

Opepe

—very durable

Teak

—very durable

Precautions against fungal attack must be commenced at the time that the timber is felled. It should be carted away from the forest as quickly as possible and then stacked on firm, well-drained and elevated ground from which all surface soils which might harbour organisms have been stripped. The timber should be stacked clear of the ground with spaces between the baulks to encourage the circulation of air and the drying of the timber to the moisture content suitable for the application of the preservative treatment.

Suitable methods of preserving timber for piling work involve pressure impregnation with creosote,



*(a) Cavities left by complete decay of piles and timber capping sills*



*(b) View down cavity left in clay after complete decay of timber pile*

*Fig. 10.1 Decay of timber piles above ground-water level (Crown copyright reserved)*

or a solution of copper sulphate and potassium dichromate (Celcure) following the recommendations of British Standards 1282 and 5268 and the publications of the Timber Research and Development Association. These standards specify the 'loadings' of the preservative solution which are adequate for British and other temperate climates. In the USA the specifications of the American Wood Preservers' Association are followed. Biological deterioration including termite attack is much more severe in

Table 10.2 Durability classification of the heartwood of untreated timbers

Grade of durability	Approximate life in ground contact (years)
Very durable	More than 25
Durable	15 to 25
Moderately durable	10 to 15
Non-durable	5 to 10
Perishable	Less than 10

tropical countries and the loadings or the selection of resistance species for these conditions should be specified in consultation with a specialist authority in the country under consideration.

Softwoods such as Scots pine and southern pine can be impregnated completely with preservative solutions, but the harder woods, including Douglas fir, can only be treated to a limited depth after the timber has first been incised and then subjected to a long-sustained application of pressure. There can be a useful advantage in using round timber for piles, in which the outer zone of sapwood can be thoroughly impregnated to a depth which will resist fungal decay over a long period of years. For example, the sapwood of Scots pine or Baltic redwood can be treated to a depth of 75mm, whereas if squared timbers are used much of the sapwood is cut away to expose the less absorptive heartwood which cannot be impregnated properly. Because the interior of the harder timbers remains untreated, careful attention should be paid to bolt holes. When these are drilled after the main impregnation treatment, preservative should be poured into the holes. Incisions made by lifting hooks, dogs or slings should be painted with the solution.

Similar attention should be given to the end grain after trimming the tip to receive the shoe or preparing the butt for the driving cap or ring ([Figure 2.2](#)). The exposed end grain should be given two heavy coats of the preservative.

Some hardwoods, for example ekki, greenheart, jarrah, okan and opepe, can be used without preservative treatment, but in these cases it is essential to specify that no sapwood is left on the prepared timber. It is difficult to distinguish between sapwood and heartwood in greenheart and either expert advice should be sought to ensure exclusion of the former or a preservative should be used to treat the sapwood as a precautionary measure. Timber used for piling is normally required to have large cross-sectional dimensions making it impracticable to remove the sapwood. BS 8004 strongly recommends using round logs when the preservative-treated sapwood provides a deep uniformly treated protective zone around the pile. The adoption of preservative treatment by using creosote or some other solution does not give indefinite life to the timber above ground-water level, and it may be preferable to adopt a form of composite pile having a concrete upper section and timber below the water line, as shown in [Figure 2.1a](#).

10.2.2 Timber piles in river and marine structures

The most destructive agency which can occur in piles immersed in brackish or saline waters in estuaries or in the sea is attack by molluscan or crustacean borers. These organisms burrow into the timber, forming networks of holes that eventually result in the complete destruction of the piles. Timber jetties in tropical waters have been destroyed in this way in a matter of months.

The Building Research Establishment([10.3](#)) lists the types of marine boring organisms as follows.

Molluscan borers	{ <i>Teredo</i> ('shipworm') <i>Bankia</i> <i>Martesia</i> (in tropical waters only) <i>Xylophaga dorsalis</i>
Crustacean borers	{ <i>Limnoria</i> ('gribble' or 'sealouse') <i>Chelura</i> <i>Sphaeroma</i>

The young mulluscan borers enter the timber through minute holes in the surface or through incisions. They then grow to a considerable size (*bankia* can grow to a diameter of 25mm and to nearly 2m long) and destroy the wood as they grow ([Figure 10.2b](#)). The crustaceans work on the surface of the

timber, forming a network of branching and interlacing holes ([Figure 10.2b](#)). Their activity depends on factors such as the salinity, temperature, pollution level, dissolved oxygen content and current velocity of the water. A salinity of more than 15 parts per 1000 (the normal salinity of sea water is between 30 and 35 parts per 1000) is necessary for the survival of most species of borer, but *sphaeroma* have been found in nearly fresh tropical waters in South America, South Africa, India, Ceylon, New Zealand and Australia.

Chellis([10.4](#)) states that *teredo* and *limnoria* do not attack in current velocities higher than 0.7m/s (1.4 knots) and 0.9m/s (1.8 knots) respectively. Although activity from some species may be marked in tropical waters, borers have been found above the Arctic Circle. They show cyclic activity rising to a peak in some years, and not infrequently dying away completely. Conversely, previously trouble-free areas can become infested with borers brought in by ships or driftwood.

It was stated in the twenty-first report of the Sea Action Committee of the Institution of Civil Engineers([10.5](#)) that no species of timber is absolutely free from borer attack, but certain species are highly resistant and in many conditions of exposure they may be considered to have practical immunity. The report lists the more-resistant species as greenheart, pynkadou, turpentine, totara and jarrah. These types would be expected to have a life of many years in tropical waters. In commenting on the suitability of various types of preservative, the report concludes that ordinary coal-tar creosote is the most satisfactory, and states that in British waters any timber which is *efficiently* impregnated with creosote should be practically immune to borer attack.

A comparative study was made by the Sea Action Committee, of the relative effectiveness as preservatives of creosote, Celcure (copper sulphate-potassium dichromate), and creosote with the addition of copper napthenate. The latter material was tried as there was evidence that some copper salts were poisonous to borers. The treated specimens consisted of Douglas fir and they were exposed at Singapore, where the borer attack was mainly by *martesia* with some *teredo*, and at Colombo.

The observations of the relative severity of attack after ~~10~~ 10 years of exposure were as follows.

Treatment	Singapore	Colombo
Coal-tar cresote to BS 144	Badly damaged	Slightly to badly damaged
5% Celcure	Very badly damaged	Very badly to badly damaged
Coal-tar creosote with 1 % of copper napthenate	Very badly to badly damaged	Slightly to badly damaged
Coal-tar creosote with 5% of copper napthenate	Very badly damaged	Very badly to badly damaged

It was concluded from the experiments that creosote gave marginally the best treatment and that the addition of copper napthenate gave no advantage.

*Limnoria tripunctata* are tolerant to creosote but the species can be effectively controlled by the addition of copper pentachlorophenate to the creosote.

The Building Research Establishment([10.3](#)) lists the following timbers as having heartwood resistant to borer attack. Those marked with an asterisk are believed to be the best for marine work.

- Afrormosia (*Pericopsis elata*)
- African padauk (*Pterocarpus soyauxii*)
- Andaman padauk (*Pterocarpus dalbergioides*)
- \*Basralocus (Angelique) (*Dicorynia guianensis*)
- \*Belian (*Eusideroxylyn zwageri*)
- \*Brush box (*Tristania conferta*)
- \*Ekki (*Lophira alata*)
- \* Greenheart (*Ocotea rodiaei*)
- Iroko (*Chlorophora excelsa*)
- Ironbark (*Eucalyptus siderophloia*)
- Jarrah (*Eucalyptus marginata*)
- Kapur (*Dryobalanops spp*)
- \*Manbarklak (*Eschweilera longipes*)
- Muninga (*Pterocarpus angolensis*)
- \*Okan (*Cyclicadiscus gabunensis*)
- \*Opepe (*Nauclea diderrichii*)
- \*Pyinkado (*Xylia xylocarpa*)
- \*Red louro (*Ocotea rubra*)

\* Southern blue gum (*Eucalyptus globulus*)

Teak (*Tectona grandis*)

\* Turpentine (*Syncarpia laurifolia*).

The sapwood of these timbers is liable to be attacked by borers, and if it is impossible to ensure the removal of all sapwood the timber should be treated with creosote as a precautionary measure. Greenheart fenders in Milford Haven were attacked in the sapwood by *teredo*, causing about 10mm of damage in 5 years.

The methods of preparing, air-seasoning, and preserving timber against borer attack are the same as those described for fungal decay in [Section 10.2.1](#) above. However, great care is necessary to avoid making incisions through which borers can enter the untreated wood in the interior of the pile. The timber should be handled by slings rather than hooks or dogs after creosoting, and purpose-made devices should be used to give pressure impregnation of the bolt holes after drilling.

Chellis([10.4](#)) describes the following other methods of protecting timber piles against attack by borers.

1. Tipping stone around the piles (this protects only the length covered by the stones).
2. Sleeving the timber with galvanized iron, copper or aluminium sheeting.
3. Encasing the piles.
4. Jacketing the piles with precast concrete tubes and filling the space between the timber and the tubes with cement grout.
5. Coating the piles with cement-sand mortar, applied with a spray gun (*e.g.* the ‘Shotcrete’ or ‘Guniting’ processes).

In tropical and sub-tropical countries timber piles can be destroyed by termites above the waterline unless a resistance species is used, or they are given the usual preservative treatment. Also the end grain at the heads of piles is particularly susceptible to attack by fungi or beetles when in a damp condition. The pile heads can be protected by heavy coats of hot-applied creosote followed by capping with metal sheeting, bituminous felt, or glass-fibre set in coal-tar pitch.

Some species of wood corrode iron fastenings by the secretion of organic acids. Either non-ferrous fastenings should be used or steel components should be heavily coated with tar or sheathed in plastics. Stainless steel fastenings can be used if the type of steel is resistant to corrosion by sea water.

The abrasion of timber piles by shingle on the sea bed has been mentioned. While protection by metal sleeving can be adopted, non-ferrous metal is expensive and it may be preferable to use sacrificial timber strapped around the main bearing piles, or to accept the cost of periodical renewal.



(a) Attack by *Teredo*

Fig. 10.2 Attack on timber piles by marine borers (Crown copyright reproduced with permission of BRE, Princes Risborough Laboratory)



*Fig. 10.2 (b) Attack by limnoria (Crown copyright reproduced with permission of BRE, Princes Risborough Laboratory)*

## 10.3 Durability and protection of concrete piles

### 10.3.1 Concrete piles in land structures

Properly mixed concrete compacted to a dense impermeable mass is one of the most permanent of all constructional materials, and gives little cause for concern about its long-term durability in a non-aggressive environment. However, concrete can be attacked by sulphates and sulphuric acid occurring naturally in soils, by corrosive chemicals which may be present in industrial waste in fill materials, and by organic acids and carbon dioxide present in ground water as a result of decaying vegetable matter<sup>(10.7)</sup>. Attack by sulphates is a disruptive process whereas the action of organic acids or dissolved carbon dioxide is one of leaching. Attack by sulphuric acid combines features of both processes.

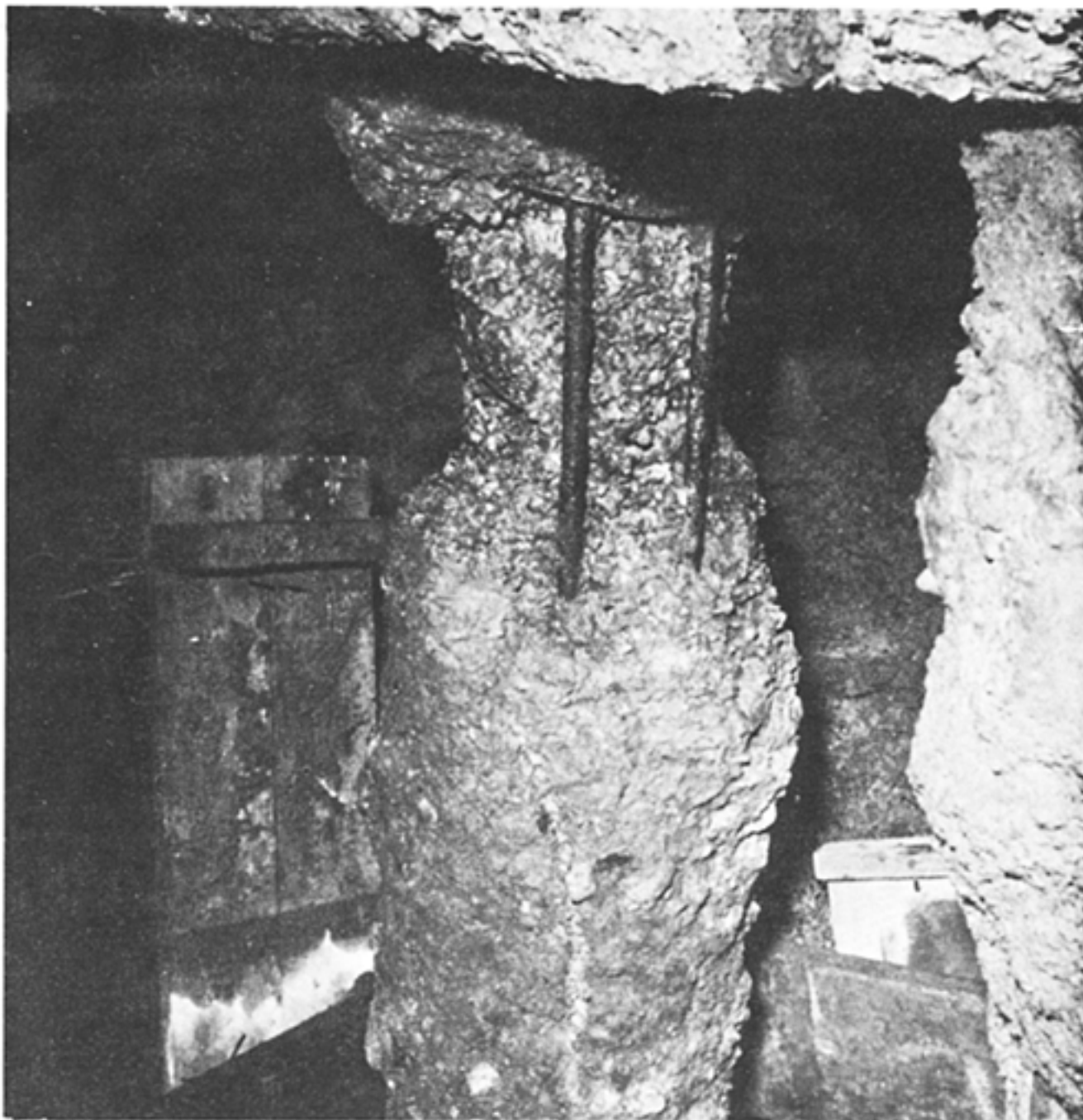
The naturally-occurring sulphates in soils are those of calcium, magnesium, sodium and potassium. The basic mechanism of attack by sulphates in the ground is a reaction with hydrated calcium aluminate in the cement paste to form calcium sulphoaluminate. The reaction is accompanied by an increase in molecular volume of the minerals, resulting in the expansion and finally the disintegration of the hardened concrete. Other reactions can also occur, and in the case of magnesium sulphate, which is one of the most aggressive of the naturally occurring sulphates, the magnesium ions attack the silicate minerals in the cement in addition to the sulphate reaction. Ammonium sulphate, which attacks Portland cement very severely, does not occur naturally. However, it is used as a fertilizer and may enter the ground in quite significant concentrations, particularly in storage areas on farms or in the factories producing the fertilizer. Ammonium sulphate is also a by-product of coal-gas production and it can be found on sites of abandoned gasworks. Because calcium sulphate is relatively insoluble in water, it cannot be present in sufficiently high concentrations to cause severe attack. However, other soluble sulphates can exist in concentrations that are much higher than that possible with calcium sulphate. This is particularly the case where there is a fluctuating water table or flow of ground water across a sloping site. The flow of ground water brings fresh sulphates to continue and accelerate the chemical reaction. High concentrations of sulphates can occur within the root mass of well-grown trees and hedgerows due to the movement and subsequent evaporation of sulphate-bearing ground water drawn from the surrounding ground by root-action. The severity of attack by soluble sulphates must be assessed by determining the soluble sulphate content and the proportions of the various cations present in an aqueous extract of the soil. These determinations must be made in all cases where the concentration of sulphate in a soil sample exceeds 0.5%.

Free sulphuric acid may be formed in natural soil or ground water as a result of the oxidation of pyrites in some peats, or in ironstone or alum shales. Sulphuric acid can also be present in industrial waste materials which have been contaminated by leakages from copper and zinc smelting works, and from dyeing processes. The acid has an effect on the cement in hardened concrete that is similar to that of sulphate attack. [Figure 10.3](#) shows the disintegration of the concrete in the shaft of a bored-and-cast-in-place pile caused by the seepage of sulphuric acid into porous fill material.

In the UK, sulphates occurring naturally in soils are generally confined to the Keuper Marl, and to the Lias, London, Oxford, Kimmeridge and Weald clays. They are also found in glacial drift associated with these formations, and they can exist in high concentration in some peats and in the root zone of well-grown trees and hedgerows due to migration of sulphate-bearing water from surrounding areas towards the roots. Sulphates may be present in the form of gypsum plaster in brick rubble fill.

The sulphate content of the ground water gives the best indication of the likely severity of sulphate attack, particularly that resulting from soluble sulphates. Where the water samples are taken from boreholes care should be taken to ensure that the sample is not diluted by water added to assist the drilling. If possible the ground water should be sampled after a long period of dry weather. Ground water flow across a sloping site through sulphate bearing ground results in the highest concentration on the downhill side of the site and the flow may continue into permeable soil deposits which are not naturally sulphate-bearing. An account of the distribution of sulphates in various ground conditions in Great Britain is given by Bessey and Lea <sup>(10.6)</sup>. Methods of analysis to determine the sulphate content and pH value of soils and ground waters are set out in BS 1377 and Building Research Establishment Current Paper 2/79. The usefulness of these recommended test methods have been reviewed critically by Eglinton<sup>(10.7)</sup>.

A dense, well-compacted concrete provides the best protection against the attack by sulphates on concrete piles, pile caps and ground beams. The low permeability of dense concrete prevents or greatly restricts the entry of the sulphates into the pore spaces of the concrete. For this reason high-strength precast concrete piles are the most favourable type to use. However, for the reasons explained in [Chapter 2](#), precast concrete piles are not suitable for all site conditions and the mixes used for the alternatives of bored-and-cast-in-place or driven-and-cast-in-place piles must be designed to achieve the required degree of impermeability and resistance to aggressive action.



*Fig. 10.3 Disintegration of concrete in bored-and-cast-in-place pile due to attack by sulphuric acid leaking into fill from industrial processes*

In British practice recommendations for the types of cement and the mix proportions are given in Building Research Establishment Digest 363(10.1). As in the case of previous BRE recommendations the five classes of severity of attack have been retained but an innovation is the ‘promotion’ or ‘relegation’ of a component from one class to another in accordance with the conditions of exposure, the mobility of the ground water table and other environmental conditions. The recommended mixes are suitable for precast concrete piles and pile shells but in some cases the workability of the concrete is too low for placing in bored piles or in the pre-formed shells of small diameter driven piles. Table 10.1 is an adaptation of BRE Digest 363 Table 1 to a form suitable for concrete in piles. It is important to use this table in conjunction with the Digest in order to note the various comments and qualifications to the recommendations.

The mixes suitable for concrete in pile caps, ground beams and blinding concrete depend on the size, shape and amount of reinforcement of the members which govern the workability requirements. Reference should be made to the Digest Table 1 for the recommended mixes for the particular conditions of exposure, and dimensions of the structural members.

The recommendation for a reduction by one class for sulphate conditions in Clauses 2, 3 and 4 if the ground water is static has been followed in compiling Table 10.1. However it would be unwise to assume a static ground water table at a shallow depth for cast-in-place concrete piles where the concrete may be weaker than in the body of the pile due to accumulation of laitance. Weak concrete used as a blinding layer beneath pile caps is also vulnerable to sulphate attack when the resulting expansion of the blinding concrete could lift the cap. The recommendation for placing a membrane between floors and fill, or hardcore containing sulphates, should be considered for the undersides of slender pile capping beams or shallow pile caps.

The use of high-alumina cement (BS 915) or supersulphated cement (BS 4248) for high sulphate concentrations is referred to in the Digest. The latter cement is attacked by ammonium sulphate to which high-alumina cement alone is resistant. Also, there is some experience to indicate that supersulphated cement has less-resistant properties to attack by magnesium sulphate than those of sulphate-resisting cement to BS 4350. In American practice, Type I cement to ASTM C150–71 is similar as regards properties to ordinary Portland cement, and the Type V classification is similar to sulphate resisting cement. Neither high-alumina cement nor supersulphated cement is favoured for piling work. In any case, approval of the use in structural concrete of the former type has at present been withdrawn from codes of practice in Britain and in some other countries. Structural concrete is deemed to include all concrete in foundations. The withdrawal of approval has been due to the property of the cement to ‘chemical conversion’<sup>(10.7)</sup> which results in a serious loss of strength. While this reduction of strength may not be critical in the case of foundations subjected to relatively low levels of stress, the conversion is accompanied by a marked reduction in the sulphate-resisting properties of the cement. Conversion is particularly liable to take place in warm and damp conditions. These may occur in piles above water level in marine structures, and in large-diameter bored-and-cast-in-place piles where the heat of hydration of the cement is dissipated only slowly. The use of this cement also causes serious practical difficulties in placing the concrete in pile shafts due to its rapid setting. Supersulphated cement is costly and difficult or impossible to obtain in many countries including the UK. It has a low heat of hydration and is therefore rather slow to harden. This makes it unfavourable for use in precast concrete piles because of the long period required between casting and driving. Special care is required when using this cement in cold weather. The recommendation in [Table 10.1](#) for using Portland cements incorporating ground blastfurnace slag or pozzolanas such as pfa will be noted.

The use of high-alumina or supersulphated cement as a precaution against sulphate attack is not required by the German specification DIN 1045. Instead, reliance is placed on the resistance of dense imperme

Table 10.3 Classification of aggressiveness of liquids of mainly natural composition according to DIN 4030

Line	Examination	Degree of aggressiveness		
		Slight	Severe	Very severe
1	pH value	6.5–5.5	5.5–4.5	below 4.5
2	Lime-dissolving carbonic acid (CO <sub>2</sub> ) in mg/l determined by marble test according to Heyer	15–30	30–60	over 60
3	Ammonium (NH <sub>4</sub> ) in mg/l	15–30	30–60	over 60
4	Magnesium (Mg) in mg/l	100–300	300–1 500	over 1 500
5	Sulphate (SO <sub>4</sub> ) in mg/l	200–600	600–3000	over 3 000

*Notes*  
The maximum aggressiveness is applicable to the evaluation of water even if it is only achieved by one of the values in Lines 1 to 5. If two or more values lie in the upper quarter of a range (in the lower quarter for the pH) the aggressiveness is increased by one stage. This increase does not apply to sea water.  
Greater aggression occurs at higher temperatures or if concrete is subject to abrasion by swift-flowing or agitated water. The aggressiveness declines in low temperatures or if water is present only in small volumes and is quiescent, *e.g.* in soils having low permeability ( $K < 5\text{--}10\text{m/s}$ ).

Table 10.4 Precautions in concrete mixes for degrees of aggressiveness as recommended in DIN 1045

Degree of aggressiveness to DIN 4030	Precautions recommended in DIN 1045
Slight	Permeability to DIN 1048 $e_{max} \leq 60\text{mm}$ Water-cement ratio $\leq 0.60$
Severe	Permeability to DIN 1048 $e_{max} \leq 30\text{mm}$ Water-cement ratio $\leq 0.50$
Very Severe	Use protective coating on concrete
From 400mg/l of SO <sub>4</sub> in water From 3 000mg/kg of SO <sub>4</sub> in soil	Use sulphate-resisting cement to DIN 1164

able concrete made with a low water-cement ratio, but sulphate-resisting cement is specified with the higher concentrations of sulphates. The specification DIN 4030 recognizes three classifications of aggressiveness according to the chemical analyses of the soil and ground waters. These are shown in [Table 10.3](#) and the requirements of DIN 1045 for concrete to resist these degrees of aggressiveness are shown in [Table 10.4](#).

Coatings of tar or bitumen on the surface of precast concrete piles do not give adequate protection against sulphate attack since they are readily stripped off by abrasion as the piles are driven down in all but the softer soils. Protection can be given to the pile surface by metal sheathing, or glass-fibre wrapping impregnated with bitumen, but the latter is likely to be torn when piles are driven into gravelly or stony soils.

Pile caps and ground beams can be protected on the underside by a layer of heavy gauge polyethylene sheeting laid on a sand carpet or on blinding concrete. The vertical sides can be protected after removing the formwork by applying hot bitumen spray coats, bituminous paint, trowelled-on mastic asphalt or adhesive plastics sheeting.

The leaching of concrete exposed to flowing river or ground water containing organic acids or dissolved carbon dioxide was mentioned at the beginning of this section. The recommendations for concrete exposed to acid attack as determined by the pH value of the soil or ground water are covered by Table 2 of BRE Digest 363. Organic acids are present in run-off water from moorlands, and in ground water in peaty and lignitic soils.

There is extensive experience on the behaviour of concrete in peaty soils and ground waters in Germany and it will be noted from [Table 10.3](#) and [10.4](#) that, as in the case of sulphate attack, reliance is placed on the use of dense impermeable normal Portland cement concrete to resist attack in acidic conditions rather than the use of special cements. Areas of peaty soils and ground waters are extensive in Holland, where they adopt the precautionary measures specified in DIN 1045.

On a site where there was severe contamination with acid industrial waste the author advised a protective scheme for the foundation piling consisting of West's precast concrete shell piles ([Figure 2.30](#)) coated externally with bitumen over the portion of the shaft within the fill. As a safeguard against partial stripping of the bitumen by bricks and concrete in the fill, the concrete shells were regarded as sacrificial. The main loadbearing element consisted of a PVC sleeve (weight 800gm/m<sup>3</sup>) lowered down the shells after completion of driving onto a concrete plug in the lower part of the pile. The PVC sleeve was then filled with concrete. A flexible PVC membrane (Bituthene sheeting) was provided beneath the pile caps. This was lapped and bonded to the PVC pile sleeve.

### 10.3.2 Concrete piles in marine structures

Precautions against the aggressive action by sea water on concrete need only be considered in respect of precast concrete piles. Cast-in-situ concrete is used only as a hearting to steel tubes or cylindrical precast concrete shell piles, where the tube or shell acts as the protective element. A rich concrete, well-compacted to form a dense impermeable mass, is highly resistant to aggressive action and, provided a cover of at least 50mm is given to all reinforcing steel, precast concrete piles should have satisfactory durability over the normal service life of the structures they support.

When the disintegration of reinforced concrete in sea water does occur it is usually most severe in the 'splash zone' and is the result of porous or cracked concrete caused by faulty design or poor construction. Evaporation of the sea water in the porous or cracked zone is followed by the crystallization of the salts and the resulting expansive action causes spalling of the concrete and the consequent exposure of the reinforcing steel to corrosion by air and water. The expansive reaction that occurs when corrosion products are formed on the steel accelerates the disintegration of the concrete. Freezing of sea water in porous or cracked concrete can cause similar spalling. However, where concrete piles are wholly immersed in sea water there is no degradation of properly-made and well-compacted concrete.

In an extensive review of literature and the inspection of structures which had been in the sea for 70 years Browne and Domone([10.8](#)) found no disintegration in permanently immersed reinforced concrete structures even though severe damage had occurred in the splash zone. They concluded that corrosion of the steel cannot occur with permanent immersion because the chloride present is restricted to a uniform low level and the availability of oxygen is low.

Although sea water typically has a sulphate content of about 230 parts per 100000, the presence of sodium chloride has an inhibiting or retarding effect on the expansion caused by its reaction with ordinary Portland cement. The latter material is, therefore, quite satisfactory for the manufacture of precast concrete piles for marine conditions but to avoid disintegration in the splash zone the concrete should have a minimum cement content of 360kg/m<sup>3</sup> and a maximum water-cement ratio of 0.45 by weight. Brown and Domone([10.8](#)) state that Type II cement to the American specification ASTM C-150-71

is advantageous for concrete structures in a marine environment. The German specification DIN 1045 requires concrete in sea water to resist the 'severe' condition in [Table 10.4](#). Air entrainment of concrete as a safeguard against frost attack on piles above the water line is unnecessary if the water-cement ratio is less than 0.45.

The concrete in precast piles should be moist cured for 7 days after the removal of the formwork. Great care should be taken in handling the piles to avoid the formation of transverse cracks which would expose the steel to corrosion in the splash zone. Coatings on precast concrete piles to protect them against deterioration in the splash zone are of little value since they are soon removed by the erosive action of waves, and by abrasion from floating debris or ice.

## 10.4 Durability and protection of steel piles

### 10.4.1 Steel piles for land structures

Corrosion of iron or steel in the electrolyte provided by water or moist soil is an electro-chemical phenomenon in which some areas of the metal surface act as anodes and other areas act as cathodes. Pitting occurs in anodic areas, with rust as the corrosion product in cathodic areas. Air and water are normally essential to sustain corrosion but bacterial corrosion can take place in the absence of oxygen, *i.e.* in anaerobic conditions. Anaerobic corrosion is caused by the action of sulphate-reducing bacteria which thrive below the sea or river bed in polluted waters, particularly in relatively impermeable silts and clays.

An exhaustive investigation of the corrosion rates of steel sheet piles and bearing piles in soils was made by Romanoff<sup>(10.9)</sup> on behalf of the US National Bureau of Standards. Steel piles which had been in the ground for periods of between 7 and 40 years were examined. The soil types ranged from permeable sands to relatively impervious clays. Soil resistivities ranged between 300 and 50200 ohm-cm and pH values between 2.3 and 8.6. Romanoff concluded from observations of the condition of the piles that where they were driven into *undisturbed* natural soil, the type and amount of corrosion was so small that it would not affect significantly the strength or useful life of the piling to support structures. Some localized pitting corrosion and loss of mill-scale were seen on steel surfaces but the loss of metal was considered to have a negligible effect on the serviceability of the piles. Corrosion had occurred in some instances where piles had been driven through fill above the water table, or in the zone extending 0.6m above and below the water table.

Romanoff pointed out that undisturbed natural soils are so deficient in oxygen they will not sustain the process of corrosion. Romanoff also found that determinations of soil resistivity and pH-value had no relevance to the incidence of corrosion in the undisturbed soil conditions covered by the Bureau of Standards research. He did not encounter any cases of anaerobic corrosion by sulphate-reducing bacteria but the possibility of their occurrence should not be overlooked at the site investigation stage. Undisturbed samples of the soil should be sealed in their containers and submitted for bacteriological examination.

In a later study Romanoff<sup>(10.10)</sup> examined steel sheet piles which had been driven through fill material. Inspections were made at thirteen locations where piles had been installed for periods of between 11 and 30 years. With only one exception the piles showed only shallow attack on the metal with some localized pitting corrosion. The single exception was at a site where sheet piles had been driven through 6m of clinker filling. Severe attack on the metal and pitting up to 6mm deep had occurred over large areas. However, it was pointed out that these piles were continuing to give useful service 23 years after they had been driven. Romanoff concluded that the relatively small amount of corrosion over the portion of the pile in fill or in undisturbed soil above the water table is the result of the formation of a galvanic corrosion cell between the upper part of the pile above the water table and the lower permanently immersed part. The upper portion is small in volume compared with the lower portion and it acts as a cathode, while the lower part in soil deficient in oxygen is the anode. Because of the much greater mass of steel in the anodic portion only a small proportion is sacrificed in protecting the cathodic part.

Similar corrosion rates for piling in land structures have been recorded by Morley<sup>(10.11)</sup>. The British Steel Corporation investigations showed corrosion losses below the soil line varying from nothing to 0.03mm per year with a mean of 0.01mm per year. No precautions are required for such low rates of loss of thickness. Where piles in land structures are extended above ground, mild steel thickness losses of 0.2mm per year were measured over a 10-year period in a marine environment. Morley considered that a more usual figure for the UK would be less than 0.1mm per year. Paint treatment would be a suitable precautionary measure for the exposed steel provided that it is accessible for maintenance.

Morley<sup>(10.11)</sup> reported a corrosion rate of 0.05mm per year for steel piling immersed in fresh water except at the waterline in canals where the rate was as high as 0.34mm per year. This locally higher corrosion zone may be due to abrasion by floating debris or to cell action between parts of the structure in different conditions of oxygen availability. The British Steel Corporation recommend that at the waterline in non-tidal fresh water conditions corrosion protection should consist of isocyanate cured pitch epoxy paint applied to a depth of 1m above and below the waterline to a dry film thickness of 400µm. The paint must be applied over blast-cleaned steel. A cheaper treatment giving a reduced performance would be standard coal tar epoxy paint to a dry film thickness of 250µm also on blast-cleaned steel. Paint coatings are not satisfactory for protection against bacterial corrosion. Any pinholes in the coating or areas removed by abrasion serve as points of attack by the organisms. Cathodic protection (see [10.4.2](#)) is effective but higher current densities are required than those needed to combat normal corrosion in aerobic conditions.

Where steel piles are buried in fill or disturbed natural soil, the thickness of metal in a bearing pile should be such that the steel section will not be overstressed due to wastage of the metal by corrosion over the period of useful life of the structure. Taking the highest figure of 0.08mm per year in the range established by the US Bureau of Standards for disturbed ground, a steel H-pile with web and flange thicknesses of 15.5mm exposed to the soil on both sides will lose 50% of its thickness in a period of 48 years although there may be localized areas of deeper pitting. In contaminated ground where corrosion of steel may be higher than normal, some protection over the length of pile above and to a depth of 0.6m below the water table can be given by two coats of coal-tar-pitch paint applied cold to a metal surface, which has first been cleaned by wire brushing to remove dirt, rust and loose mill-scale. A coating of this type is relatively inexpensive and elaborate cleaning of the metal is not justified since the paint serves only to retard corrosion by a few years.

Other protective measures in contaminated disturbed ground include jacketting the pile with concrete or filling the shafts of hollow piles with concrete capable of carrying the full load. Where the water table is shallow the pile cap can be extended down to a depth of 0.6m below water level to protect the steel of the piles.

#### 10.4.2 Steel piles for marine structures

Steel piles supporting jetties, offshore platforms, and other river or marine structures must be considered for protection against corrosion in five separate zones. These are as follows.

1. *Atmospheric zone*: exposed to the damp conditions of the atmosphere above highest water levels or to airborne spray.
2. *Splash zone*: above mean high water level and exposed to waves and spray and wash from ships.
3. *Intertidal zone*: between mean high and mean low water levels.
4. *Continuous immersion zone*: below lowest water level.
5. *Underground zone*: below the soil line.

Morley and Bruce<sup>(10.12)</sup> made an extensive survey of the extent of corrosion on steel piling in marine structures at various sites in the UK, Cyprus and the United Arab Emirates. Average and probable maximum corrosion loss rates for the five zones are shown diagrammatically in [Figure 10.4](#). They point out that design thicknesses to allow for corrosion loss, and methods of protection should take into account the variation in corrosion over these zones. The presence of marine growth also has a considerable influence on protective measures. There is no growth within the atmospheric and splash zones, but in the intertidal and continuously immersed zones heavy growths of barnacles and weeds can develop, which damage the paint treatment and prevent its renewal. However, the growth can shield the steel from exposure to the oxygen and in this way reduce the rate of corrosion.

As in the case of canals, Morley and Bruce found a tendency to a local increase in corrosion at low water level. This was thought to be caused by abrasion from fenders and floating debris, and loss of marine growth by abrasion and wash from ships, particularly those with bow thrust propellers. Evidence for bacterial corrosion was sought in the survey, but it was concluded that although bacterial *activity* occurs to some extent on most marine structures, the residual thickness measurements showed that this activity did not cause a significant amount of corrosion damage.

In discussing protective measures Morley and Bruce noted that it is improbable that a life of paint coatings from application to first maintenance will exceed 12 to 15 years, and hence the cost of painting should be balanced against the alternative of increasing steel thickness, or the use of high tensile steel at mild steel stresses. The latter method enables an additional corrosion loss of 35% to be obtained without loss of loadbearing capacity at an additional steel cost of about 8%. It should also be noted

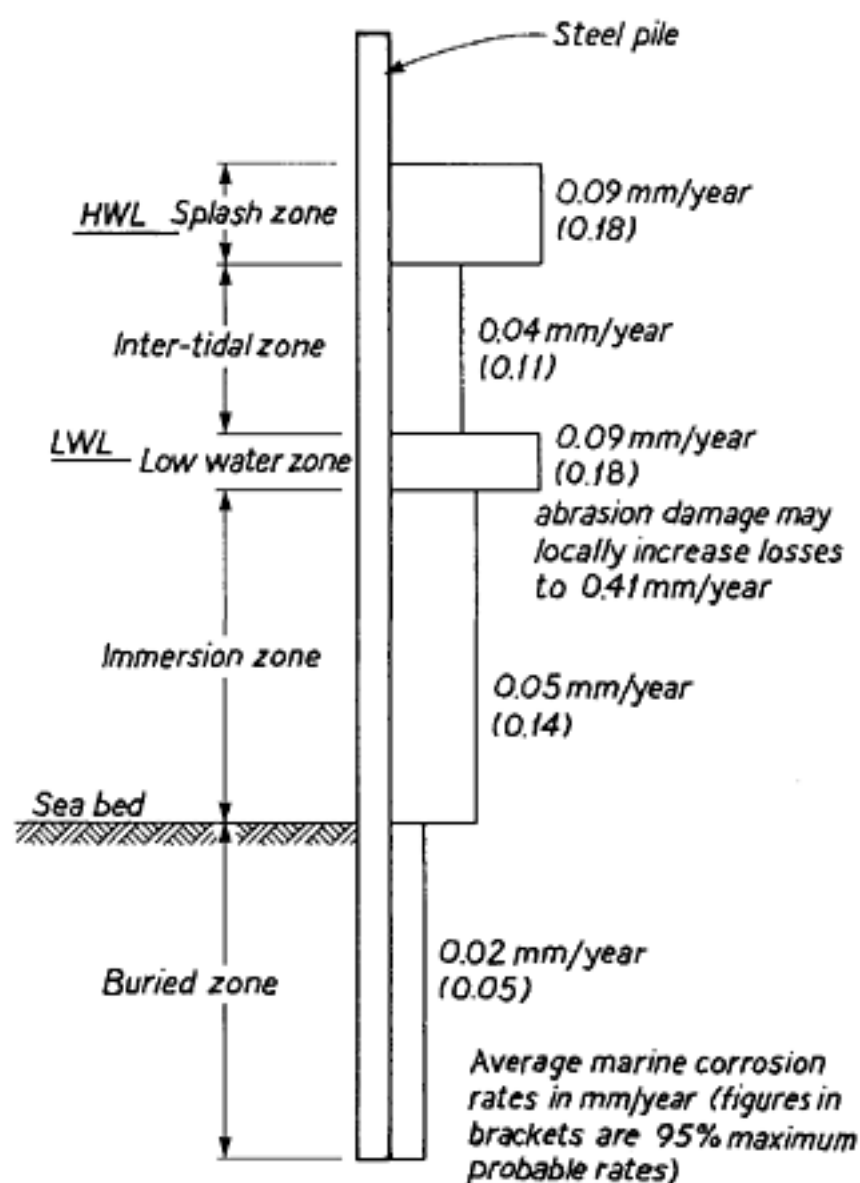


Fig. 10.4 Loss of thickness by corrosion for steel piles in seawater (after Morley and Bruce<sup>(10.12)</sup>)

that steel thicknesses may be determined by the stresses caused during driving (see Table 2.10) giving a reserve available for the lower stresses under service conditions. Also, maximum stresses for working conditions in marine structures may be at or near the soil line where corrosion losses are at the minimum rate.

The British Steel Corporation<sup>(10.11)</sup> recommend the following protective measures for marine structures:

Atmospheric zone and splash zone	Coal tar epoxide paint to 250µm film thickness or preferably isocyanate cured coal tar epoxide paint to 400µm film thickness to give estimated 10 year life
Intertidal zone	Bare steel to nominal or increased thickness to allow for corrosion loss (because of uncertainty in driving depths, it may be necessary to extend the paint treatment from the splash zone into the intertidal zone)
Continuously immersed zone	Bare steel or cathodic protection
Underground zone	No protection necessary

Piles forming the main supporting structures in important jetties or in offshore platforms exposed to a marine environment require elaborate and relatively expensive treatment to ensure a long life. The steel in the atmospheric zone is protected by paint and the first essential is to obtain thorough cleaning of the metal. This is achieved by the application of sand or grit blasting to obtain a white metal or near white metal condition. Hedborg<sup>(10.13)</sup> recommends the application of a zinc silicate priming coat to the cleaned surface to a dry film thickness of 50 to 75µm followed by top coats of vinyl or epoxy paints. Epoxy coal-tar paints are usually employed for the latter and they are applied in three or more coats to obtain an overall film thickness (including the primer) of 175µm.

The most severe conditions of corrosion are experienced in the splash zone where Hedborg<sup>(10.13)</sup> quotes corrosion rates of 0.13 to 0.25mm per year in the Canal Zone of the USA and the Hawaiian Islands, and a rate of 0.88mm per year which has been observed on a platform at Cook Inlet, Alaska. Paint coatings used in this zone comprise a zinc silicate primer followed by three or more coats of epoxy coal-tar paint to obtain an overall dry film thickness of 400µm. However such coatings have a life of only a few years, after which wastage takes place on the exposed steel at the rates quoted above. Cathodic

protection is ineffective in the splash zone and the thickness of metal should be such as to ensure that wastage due to corrosion will not curtail the design life of the structure. This is achieved over the length in the splash zone, either by increasing the thickness of the steel, or by providing cover plates of steel to the same specification as the piles, bracings, or jacket members or of a corrosion-resistant material such as monel metal. Above the splash zone continual maintenance by periodic cleaning and painting is needed on exposed steelwork.

Below the splash zone the bare or painted steel is protected by cathodic means. Cathodic protection utilizes the characteristic electrochemical potential possessed by all metals. The metals which are higher in the electromotive series act as anodes to the metals lower in the series which form the cathodes. Thus if a steel structure is connected electrically to a zinc anode the current escapes to the soil or water through the anode and the structure thus forms the cathode, so preventing the escape of metallic ions from the structure. The two methods of cathodic protection used in marine structures are the sacrificial anode system and the power-supplied (or impressed-current) system. In the former large masses of metal such as magnesium, aluminium or zinc, which are higher in the electromotive series than steel, are used as the anodes. In the power-supplied system the anodes are non-wasting and consist of graphite, lead-silver or other noble metals. They supply direct current from a generator or transformer rectifier to the structure acting as the cathode. The anodes are not welded directly to the piles as they would become detached when driving through guides or jacket members. Instead they are electrically bonded to the jacket or to bracing members.

The sacrificial anode system is generally preferred for marine structures since it does not require the use of cables which are liable to be damaged by vessels or objects dropped or lowered into the water from the structures. However, Hedborg<sup>(10.13)</sup> points out that divers are required to replace sacrificial anodes which are designed to have, for example, a 10-year life. In depths of water of up to 60m the anodes can be replaced by divers at a reasonable cost, but in deep-water platform structures the diving costs increase steeply with increasing depths. Sacrificial systems can be designed to be replaced without using divers, and the life of the anodes can be extended by reducing the area of steel requiring protection, *i.e.* by painting the steel. While a long life cannot be achieved by painting, marine growth will replace the paint and hence maintain the protection. The choice between sacrificial anode systems, with or without a coated structure, and power-supplied systems is a matter of economics, taking into account the capital costs of installation, the current consumption, the costs of maintenance and the intended life of the structure.

Where hollow steel piles are plugged at their base with concrete or impervious soil it is sometimes the practice to pump out the sea water and replace it with fresh water containing a corrosion-inhibiting compound. The addition of sodium nitrite and sodium carbonate to form a 2% solution can be used for this purpose. However, the need for this has been questioned because an empty or sea water filled pile contains little oxygen which is quickly used up in the early corrosion process, leaving none to maintain the corrosion.

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## CHAPTER 11

### Site investigations, piling contracts, pile testing

The importance of a thorough site investigation as an essential preliminary to piling operations cannot be over-emphasized. Because of the present uncertainty concerning the relationships between the simple parameters of shearing resistance and density of soils and the ultimate carrying capacity of piles the assessment of allowable loads is still largely a matter of experienced judgement. Therefore accurate and detailed descriptions of soil and rock strata and an adequate programme of field and laboratory tests are necessary if the engineer is to exercise his judgement in the most favourable conditions. By correlating experience on the behaviour of piles under load with detailed information on the ground conditions, engineers can accumulate a mass of experience which, when compared with developing theories on pile bearing capacity, will eventually lead to the establishment of completely reliable design rules for predicting ultimate resistance and deformation under working load.

Detailed descriptions of the ground conditions are also essential if the piling contractor is to select the most appropriate equipment for pile installation, while giving prior warning of possible difficulties when driving or drilling through obstructions in the ground.

The engineer must have assurance that the piles have been correctly designed and installed in a sound manner without defects which might impair their bearing capacity. To this end piling contracts must define clearly the responsibilities of the various parties, and the installation of piles must be controlled at all stages of the operations. It will have become evident from the earlier chapters of this book that load testing cannot be dispensed with as a means of checking that the correct assumptions have been made in design and that the deflections under the working load conform, within tolerable limits, to those predicted. Load testing is also one of the most effective means of checking that the piles have been soundly constructed.

The various aspects of site investigations, piling contracts and specifications, control of installation, load testing and other forms of test are discussed in the following sections of this chapter.

#### 11.1 Site investigations

##### 11.1.1 Planning the investigation

At the time when a site investigation is planned it is not always certain that piled foundations will be necessary. Therefore the programme for the site work should follow the usual pattern for a foundation investigation with boreholes that are sufficient in number to give proper coverage of the site both laterally and in depth. If it becomes evident from the initial boreholes that piling is required, or is an economical alternative to the use of shallow spread foundations, then special attention should be given to ascertaining the level and characteristics of a suitable stratum in which the piles can take their bearing. Where loaded areas are large in extent, thus requiring piles to be arranged in large groups rather than in isolated small clusters, the borings should be drilled to a depth of 1.5 times the width of the group below the intended *base level* of the piles, or 1.5 times the width of the equivalent raft below the base of the raft ([Figure 11.1](#)). This depth of exploration is necessary to obtain information on the compressibility of the soil or rock strata with depth, thus enabling calculations to be made of the settlement of the pile groups in the manner described in 5.2 and 5.3. If the piles can be founded on a strong and relatively incompressible rock formation the drilling need not be taken deeper than a few metres below rockhead to check that there are no layers or lenses of weak weathered rock which might impair the base resistance

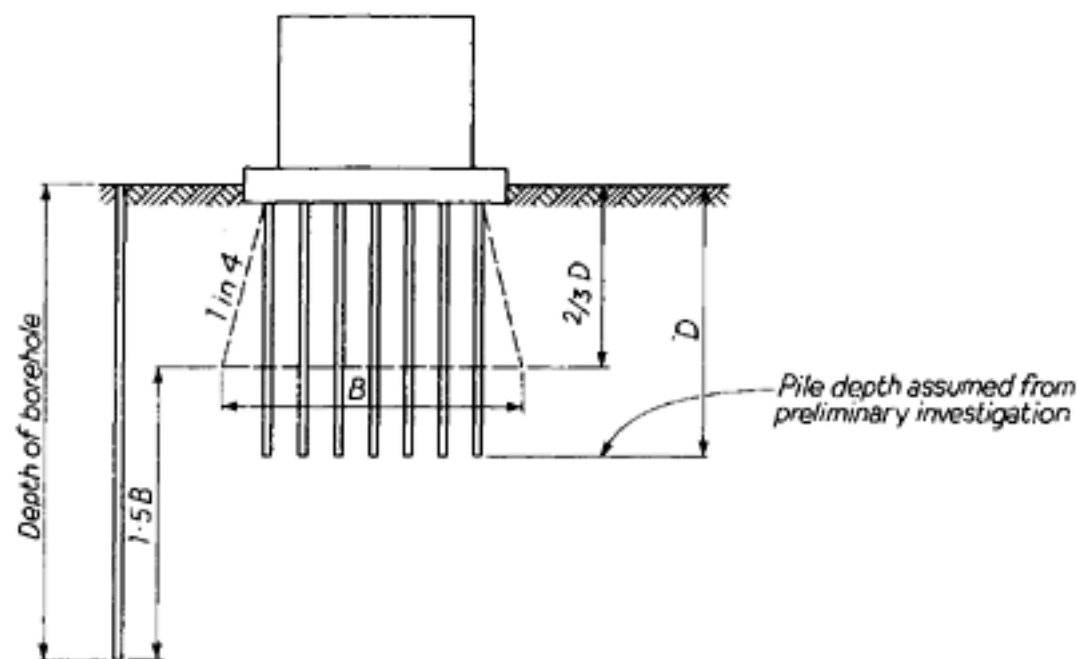


Fig. 11.1 Required depth of boreholes for pile groups in compressible soils

of individual piles. However, before permission is given for the drilling depth to be curtailed in this manner there must be reliable geological evidence that the bearing stratum is not underlain by weak compressible rocks which might deform under pressures transmitted from heavily loaded pile groups, and that large boulders have not been mistaken for bedrock. Particular care is necessary in interpreting borehole information where the site is underlain by weathered rocks or by alternating strong and weak rock formations dipping across the site. Without an adequate number of *cored* boreholes and their interpretation by a geologist, wrong assumptions may be made concerning the required penetration depth of end-bearing piles. Two typical cases of misinterpretation are shown in [Figure 11.2](#).

Where piles are end bearing on a rock formation it may be desirable, for economic reasons, to obtain a detailed profile of the interface between the bearing stratum and the overburden, so enabling reliable predictions to be made of the required pile lengths over the site. Cased shell and auger borings followed by rotary core drilling to prove the rock conditions can be costly when drilled in large numbers at the close spacing required to establish a detailed profile. Geophysical exploration by seismic refraction on land and by continuous seismic profiling at sea are economical methods of establishing bedrock profiles over large site areas. However the success of these indirect methods depends on there being a sufficient contrast in seismic velocity between the rock stratum (or other strong relatively incompressible material) and the overburden. Also there must be adequate correlation with the control boreholes.

Geophysical methods are not usually economical for small site areas, but where the overburden is soft or loose, either uncased wash probings or continuous dynamic cone penetration tests (see [11.1.4](#)) are cheap and reliable methods of interpolating between widely-spaced cable percussion boreholes.

Information on ground-water conditions is vital to the successful installation of driven-and-cast-in-place and bored-and-cast-in-place piles. The problems of installing these pile types in water-bearing soils and rocks are discussed in 3.4.7 to 3.4.8. Whenever possible standpipes or piezometers should be installed in selected boreholes for long-term observations of the fluctuation in ground-water levels. Simple forms of in situ permeability tests are described in 11.1.4.

Trial pits are often a useful adjunct to borehole exploration for a piling project. Shallow trial pits are excavated in filled ground to locate obstructions to piling such as buried timber or blocks of concrete. Deep trial pits may be required for the direct inspection of a rock formation by a geologist, or to conduct plate bearing tests to determine the modulus of deformation of the ground at the intended pile base level (see [4.7](#) and [5.5](#)). It may be more convenient and economical to make these tests at the preliminary test piling stage.

It is not the intention to describe site investigation techniques in detail in the following section of this chapter. Details of drilling and sampling methods, geophysical surveying, and the various forms of in-situ tests are given in BS 5930. In Germany, site investigation practice is regulated by DIN 4021 and 4022. Detailed information on soil testing procedure is given in BS 1377 (Methods of testing soils).

### 11.1.2 Boring in soil

Cable percussion borings give the most reliable information for piling work. Operation of the boring tools from the winch rope gives a good indication of the state of compaction of the soil strata. If the casing is allowed to follow down with the boring and drilling, and water to aid drilling is used sparingly,

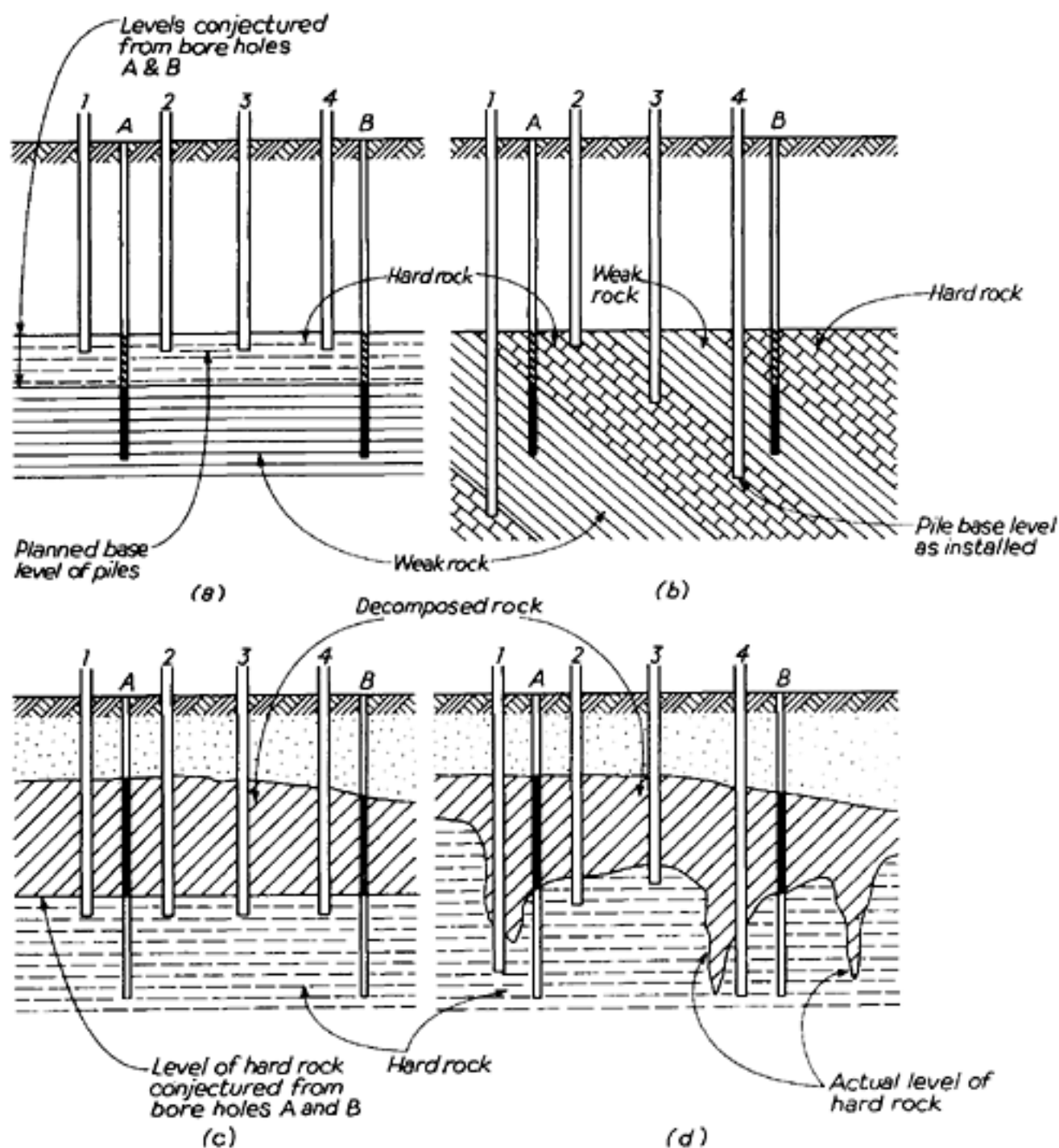


Fig. 11.2 Misinterpretation of borehole information

- (a) Horizontal stratification interpreted by interpolation between boreholes A and B. Piles 1 to 4 planned to have uniform base level
- (b) Actual stratification revealed by drilling boreholes for piles 1, 2, 3 and 4, showing base levels required by dipping strata
- (c) Uniform level of interface between decomposed rock and hard rock interpreted by interpolation between boreholes A and B. Piles 1 to 4 planned to have uniform base level
- (d) Actual profile of hard rock surface

reliable information can be obtained on ground-water conditions. Such information cannot be obtained from wash borings or by drilling in uncased holes supported by bentonite slurry. Borings by continuous flight auger are satisfactory provided that there is a hollow drill stem down which sample tubes can be driven below the bottom of the boring and measurements of the ground-water level obtained.

Information on the size of boulders is essential for a proper assessment of the difficulties of driving piles past these obstructions or of drilling through boulder deposits for bored piles. Cable percussion or flight auger drilling cannot penetrate large and hard boulders, and it is the usual practice to bring a rotary drill over the hole to core through the boulder, so obtaining information on its size and hardness. To avoid delays and standing time of the two types of drilling equipment it is more economical to continue rotary drilling past the boulder down through the remaining soil overburden to bedrock while the percussion rig is moved to an adjacent hole. It is usually possible to obtain information on the soil conditions below the level of the boulder from these adjacent boreholes.

In UK practice 'undisturbed' samples of cohesive soils are obtained from cable percussion boreholes by means of 100mm open-drive thick-wall sample tubes. Recent developments in site investigation techniques include the use of thin-wall sample tubes pushed into the soil. It has been found possible to sample very stiff to hard clays or even weak rocks and sands with this equipment. The samples are of high quality with very little disturbance of the fabric of the material.

### 11.1.3 Drilling in rock

Weak rocks can be drilled by percussion equipment, but this technique is useful only to determine the level of the interface between the rock formation and the soil overburden. Little useful information

is given on the characteristics of the rock layers because they are reduced to a gritty slurry by the drilling tools. No information can be obtained on the structure of the rock mass but some indication of the strength of weak rocks can be obtained from standard penetration tests (see [11.1.4](#)). Percussion boring can obtain reliable information from rocks which have been weathered to a stiff or hard clayey consistency such as weathered chalk, marl or shale. Open drive tubes, or preferably the thin-wall pushed-in tubes, can be used in these weathered rocks to obtain undisturbed samples for laboratory testing.

The most reliable information on the strength and compressibility of rocks is obtained by rotary core drilling, supplemented as necessary by in-situ tests. The core diameter must be large enough to ensure complete or virtually complete recovery of weak or heavily jointed rocks. Drilling to recover large-diameter cores, say up to the ZF size (165mm core diameter) can be expensive, but the costs are amply repaid if claims by contractors for the extra costs of installing piles in 'unforeseen' rock conditions can be avoided. Also, by a careful inspection and testing of the cores to assess the effects of the joint pattern on deformability, and to observe the thickness of any pockets or layers of weathered material, the required depth of the rock socket (see [4.7.3](#)) can be reliably determined. It must be remembered that drilling for piles in rock by chiselling and baling or by the operation of a rotary rock bucket ([Figure 3.28](#)) will form a weak slurry at the base of the pile borehole which may make it impossible to ascertain the depth to a sound stratum for end-bearing piles, whereas if there has been full recovery of the cores from an adequate number of boreholes together with sufficient testing of core specimens the required base level of the piles can be determined in advance of the piling operations.

#### 11.1.4 In-situ and laboratory testing in soils and rocks

*Vane tests* to determine the undrained shearing strength of soft silts and clays have little application to piling operations. Skin friction in these soils contributes only a small proportion of the total pile resistance and it is of no great significance if laboratory tests for shearing strength on conventional 'undisturbed' soil samples indicate shearing values that are somewhat lower than the true in-situ strengths. The lateral resistance of piles is particularly sensitive to the shearing strength of clays at shallow depths, and if the calculation methods can be refined to a greater degree of certainty than exists at present then the vane test may have a useful application.

The most useful all-round test for piling investigations is the *standard penetration test*, which in clays, silts and sands is performed with an open-ended tube (SPT) and in gravels and weak rocks is made by plugging the standard tube with a cone end, when the test is sometimes known as the *dynamic cone penetration test* (CPT). The blow counts (blows per 300mm of penetration) for the SPT and CPT have been correlated with the angle of shearing resistance of cohesionless soils ([Figure 4.1a](#)) by Peck, Hanson and Thornburn([4.19](#)). Terzaghi and Peck([11.1](#)) have given the following approximate correlation with the consistency of cohesive soils.

<i>N</i> value (blows/300mm)	Consistency	Approx. unconfined compressive strength kN/m <sup>2</sup>
Below 2	Very soft	Below 25
2 to 4	Soft	25–50
4–8	Medium	50–100
8–15	Stiff	100–200
15–30	Very stiff	200–400
Over 30	Hard	Over 400

Stroud([5.7](#)) has established a relationship between the standard penetration test and the undrained shear strength of stiff over-consolidated clays as shown in [Figure 5.15](#). The cone-ended standard penetration test (CPT) can also be made in weak rocks and hard clays. Useful correlations have been established between the blow-count values of stiff to hard clays and the modulus of deformation (see [Figure 5.15](#)). The test should also be made if percussion borings are carried down below rockhead.

The standard penetration test is liable to give erroneous results if the drilling operations cause loosening of the soil below the base of the borehole. This can occur if the borehole is not kept filled with water up to ground level, or above ground level, to overcome the head of ground water causing 'blowing' of a granular soil. Careful manipulation of the 'shell' or baler is also necessary to avoid loosening the soil by sucking or surging it through the clack valve on the baler. It is particularly necessary to avoid misinterpretation of SPT data on piling investigations since denser conditions than indicated by the test may make it impossible to drive piles to the required penetration level.

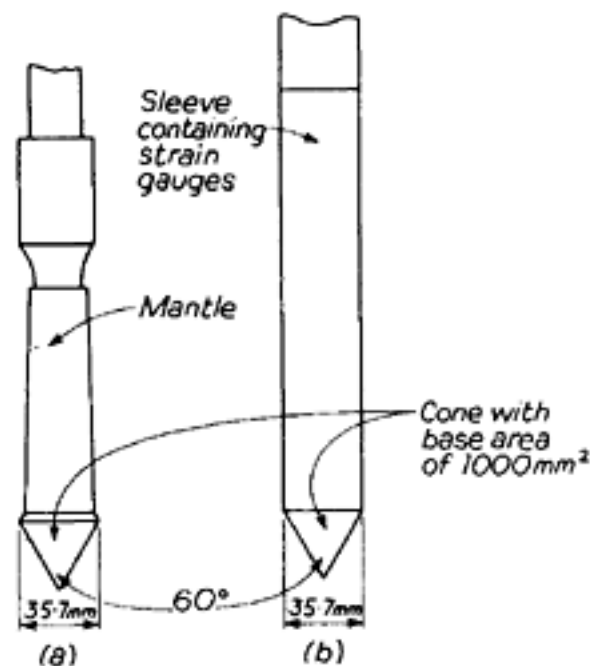


Fig. 11.3 Types of cones for static cone penetration tests

(a) Mechanical cone (Dutch Government Soil Mechanics Laboratory)

(b) Electric cone (Fugro N.V.)

The standard penetration test cannot be performed satisfactorily at deep-sea locations, say for example in site investigations for piled foundations for offshore oil production platforms. This is because the hammer is operated at the surface and the inertia of 100m or more of drill rods from the hammer to the SPT sampler would make it impossible to achieve anything like the standard blow as performed at normal drilling depths on land investigations. Underwater hammers operating in air in sealed containers are available for deep water site investigations.

The application of the static cone penetration test to the design of individual piles is described in 4.3.7 and to the design of pile groups in 5.3. Because of the experience gained in these applications the test is particularly useful for piling investigations. A correlation has been established between the static cone resistance and the angle of shearing resistance of cohesionless soils (see [Figure 4.14](#)). It also gives information on the resistance to the driving of piles over the full depth to the design penetration level. The standard mechanical cone as developed by the Dutch Government Soil Mechanics Laboratory at Delft is shown in [Figure 11.3a](#). This cone is generally used in conjunction with mechanically-operated penetration devices which measure the thrust on the cone and on the sleeve separately by means of hydraulic cylinders mounted on the machines. The electric cone developed by Fugro N.V. shown in [Figure 11.3b](#) has electrical-resistance strain gauges mounted behind the cone and inside the sleeve, giving continuous readings of penetration resistance by means of electrical signals recorded on punched tape and on a pen recorder at the surface. The type shown in [Figure 11.3b](#) can be used in deep-sea site investigations. The cone, together with rods and a drive unit, are lowered together to the bottom of the borehole and take their reaction from the drill string through latches bearing against the core barrel. The signals from the electrical strain gauges are transmitted by cable to the recording unit on the drilling vessel. In a comparison of the penetration resistance readings reported by Joustra([11.2](#)) the cone resistance by the electric cone was 3.3% less than that of the mechanical cone in sands in Amsterdam.

The *continuous dynamic cone penetration test* is a useful and much neglected method of logging the stratification of layered soils such as interbedded sands, silts and clays. The Borros penetrometer employs a 63kg hammer impacting on a 50.5mm cone at a rate of 20 blows per minute. The number of blows required for a penetration of 100mm is denoted as  $n$ . The torque on the cone is measured to provide an additional means of interpreting the data. There is very little published information in the UK on correlation between  $n$  values and the SPT N-value or  $q_c$  values from static cone penetration tests. Cearns and

McKenzie([11.3](#)) have published relationships between  $n$  and the SPT N for sands, gravels and chalk as shown in [Figure 11.4](#). Approximate determinations of the deformation modulus of soil and rocks can be made by expanding a cylindrical rubber membrane against the walls of the borehole and measuring the increase in diameter of the cylinder over an increasing range of cell pressures. Apparatus developed for this purpose includes the *Menard 'pressuremeter'*([6.26](#)) the *Camkometer*([11.4](#)) and the *dilatometer*([11.5](#)). The Menard pressuremeter is operated in boreholes ranging in diameter from 34mm to 80mm. Three independent rubber cells are mounted, one above the other, in the probe to apply a uniform pressure to the soil. The central cell is filled with water pressurised pneumatically, and the upper and lower cells are expanded by air. The Camkometer is a self-boring pressuremeter with a rotating drilling bit mounted at the base of the probe. The cell is expanded by compressed air and the radial deformation is measured by electrical

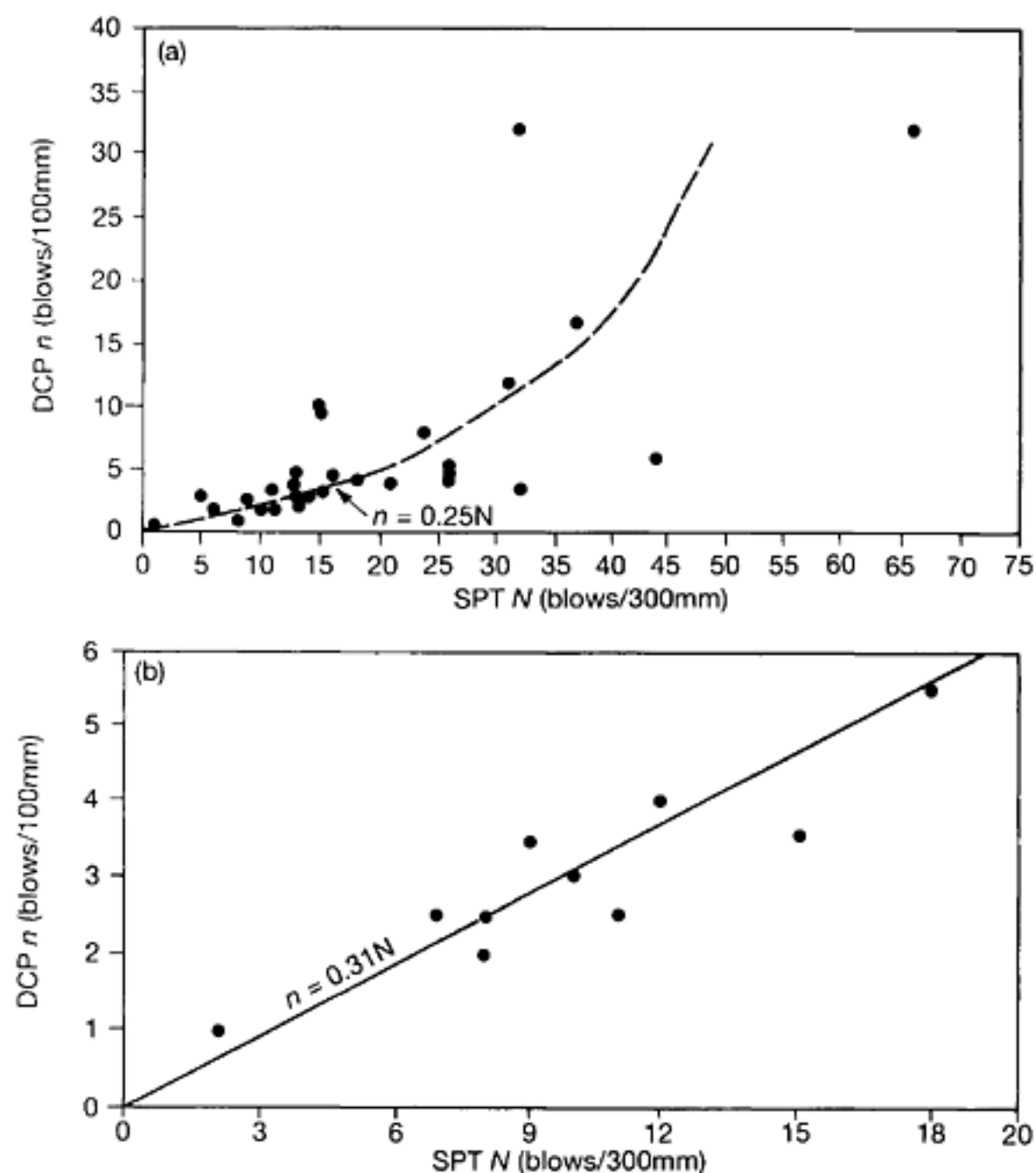


Fig. 11.4 Relationship between Dynamic Cone Penetration Test  $n$  and Standard Penetration Test  $N$  (after Cearns and McKenzie<sup>(11.3)</sup>)

(a) in sands and gravels and  
(b) in chalk

gauges attached to feelers within the probe. The device is particularly useful in a very soft or loose soil where disturbance of the soil around a pre-drilled hole could be detrimental to accurate measurements. The *Marchetti dilatometer*<sup>(11.5)</sup> is a spade-shaped device which is pushed or hammered into the soil. A load-cell is mounted on the vertical face of the spade and pushed against the soil or rock.

As noted in 6.3.7, the pressuremeter has useful applications to determinations of the ultimate resistance to lateral loads on piles and the calculation of deflections for a given load. Because the pressuremeter only shears a soil or rock (there is no compression of the elastic soil or rock) the slope of the pressure volume change curve in [Figure 6.49a](#) gives the shear modulus  $G$ . This can be converted to the Young's modulus from the equations,

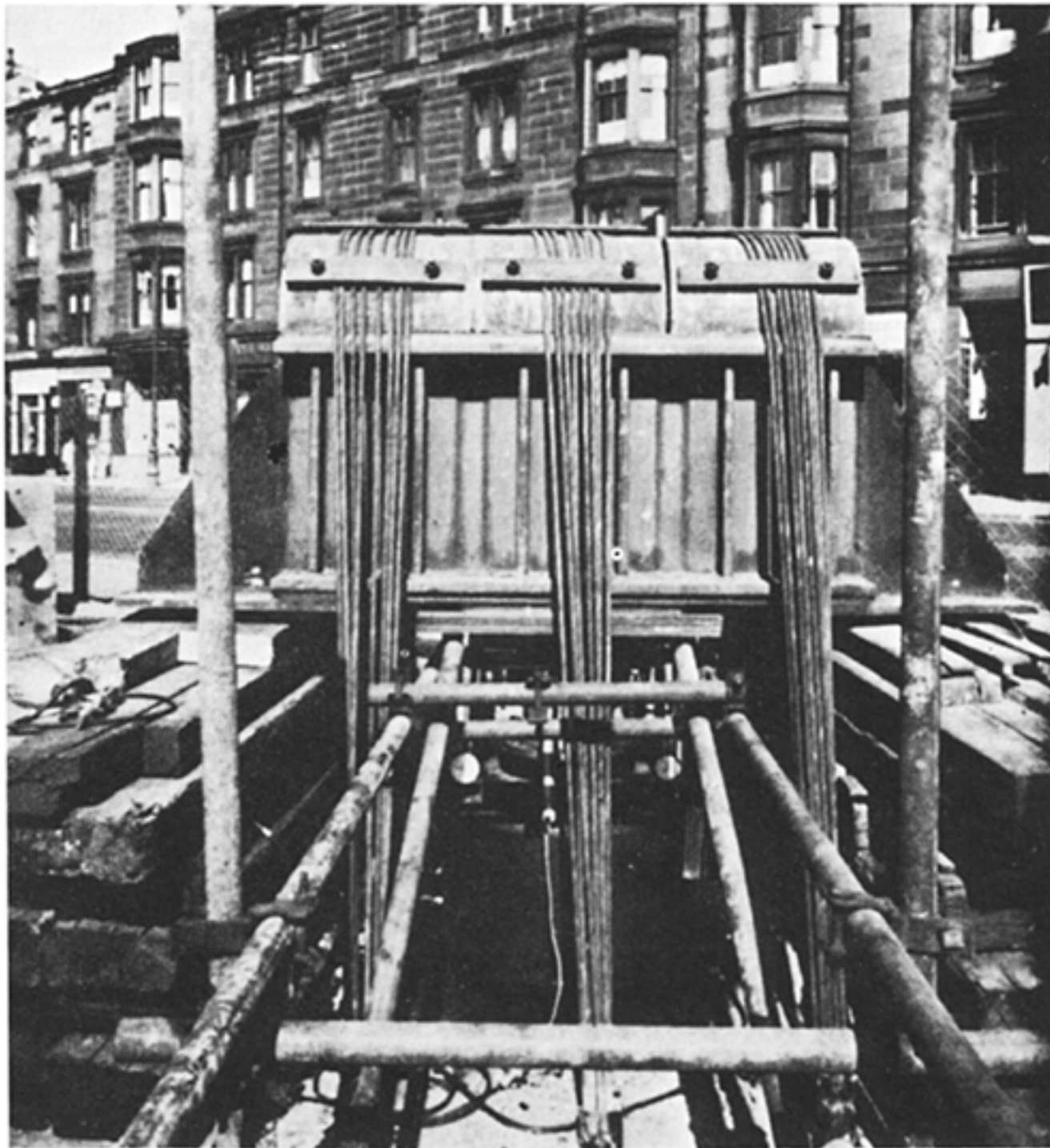
$$\text{Undrained modulus } E_u = 2G(1 + \nu_u) \quad \dots(11.1)$$

$$\text{Drained modulus } E' = 2G(1 + \nu') \quad \dots(11.2)$$

where  $\nu_u$  and  $\nu'$  are the values of the Poisson's ratio for undrained and drained conditions respectively. In clays  $\nu$  is generally taken as 0.5, and for drained conditions in clays and granular soils it is typically between 0.1 and 0.3. Menard took a value of 0.33 to obtain the deformation modulus  $E_m$ . When using the pressuremeter to obtain  $E$  values for pile group settlements using

the methods described in [Chapter 5](#), it is necessary to take into account the drainage conditions in the period of loading. *Plate bearing tests* can be used to obtain both the ultimate resistance and deformation characteristics of soils and rocks. When used for piling investigations these tests are generally made at an appreciable depth below the ground surface, and

rather than adopting costly methods of excavating and timbering pits down to the required level it is usually more economical to drill holes 1 to 1.5m in diameter by power auger or grabbing rig. The holes are lined with casing and the soil at the base is carefully trimmed by hand and the plate accurately levelled on a bed of cement or plaster of Paris<sup>(11.6)</sup>. The deformation of the soil or rock below the test plate can be measured at various depths by lowering a probe down a tube inserted in a drill hole beneath the centre of the plate. This device<sup>(11.7)</sup> is helpful in obtaining the modulus of deformation of layered soils and rocks. The load is transmitted to the plate through



*Fig. 11.5 Jacking arrangements for plate loading tests at base of 1.2m borehole. Reaction is provided by cables anchored by grouting into rock*

a tubular or box-section strut and is applied by a hydraulic jack bearing against a reaction girder as described for pile loading tests (see [11.4.1](#)). The reaction girder, anchorages and jacking arrangements for a loading test on a 1.2m diameter plate are shown in [Figure 11.5](#).

Small-diameter plate loading tests can be made using a 143mm plate in a 150mm borehole but it is, of course, impossible to trim the bottom of the hole or to ensure even bedding of the plate. However, these tests can be useful means of obtaining the ultimate resistance of stiff to hard stony soils([11.8](#)) or weak rocks([11.9](#)). They do not give reliable values of the deformation modulus.

Simple forms of *in situ permeability test* can give useful information for assessing problems of placing concrete in bored-and-cast-in-place piles in water-bearing ground. The *falling-head test* consists of filling the borehole with water and measuring the time required for the level to drop over a prescribed distance. In the *constant-head test* water is poured or pumped into the borehole and the quantity required to maintain the head at a constant level above standing ground-water level is recorded. The procedure for obtaining the coefficient of permeability is described in BS 5930. Pumping-in tests made through packers in a borehole, or pumping-out tests with observations of the surrounding drawdown are too elaborate for most piling investigations. Sufficient information to evaluate ground-water problems can often be obtained by baling the borehole dry and observing the rate at which the water rises to its standing level. Such simple tests provide useful background information for contractors tendering for bored piling work and it is false economy to neglect them.

It was noted in [Section 11.1.2](#) that high-quality undisturbed samples of soil can be obtained by means of pushed-in samples in thin-walled tubes. Equipment has also been developed for measuring very small strains in samples undergoing triaxial compression tests. Thus reliable Young's modulus values can be

obtained from samples of soil or weak rock without the need for employing special in-situ testing equipment or making plate-bearing tests in deep boreholes. Small-strain Young's modulus values are useful for calculating consolidation settlements in the mass of ground beneath pile groups rather than the base settlement of individual piles. They are also applicable to the determination of the deflection of laterally loaded piles. The adoption of improved sampling and triaxial compression testing techniques will result in undisturbed shear strength values higher than those obtained in earlier practice, particularly in very stiff to hard clays. These higher values may require modification of correlations established between the shear strength of clays and skin friction and end-bearing resistance of piles.

Laboratory tests on rock cores should include the determination of the unconfined compression strength of the material, either directly in the laboratory or indirectly in the field or laboratory by means of point load strength tests. Young's modulus values of rock cores can be obtained by triaxial compression testing using the transducer equipment for small strain measurements.

## 11.2 Piling contracts and specifications

### 11.2.1 Contract procedure

In Britain it is the usual practice for the piling works for foundations on land to be executed as a sub-contract to the main general contract. Foundation piling rarely forms a high proportion of the total cost of a project on land, and the administrative arrangements and preliminary works required for carrying out the piling as a separate advance contract are unjustified in most cases. Works which are required in advance of a piling contract include such items as fencing, levelling and grading, the construction of site roads, the erection of site offices, and the supply of electrical power and water. On some large contracts it is the practice to let a separate contract for these advance works, in which case it may be feasible also to let a separate contract for the piled foundations. Piling contractors prefer to do the work in this way rather than as sub-contractors. The responsibilities of each party are defined more clearly and, from the piling contractor's point of view, a main contractor has the advantage that he is paid retention money after the usual period following the completion of his work. Where a piling contractor works as a sub-contractor it is usual for part of his retention money to be withheld until the main contractor's retention is released, which may be several years after the completion of the piling work.

Piling normally forms a high proportion of the cost of marine construction and it is usually undertaken directly by the main contractor. Any sub-contracting is limited to specialist services such as grouting or the construction of anchorages to tension piles, and to the supply of prefabricated components.

Piling carried out as sub-contract work may be done by a nominated contractor, *i.e.* a specialist piling contractor, who has submitted a tender to the Engineer in advance of the main contract and is then selected for nomination in the main contract documents. Alternatively, the main contractor can, after appointment, invite tenders from selected specialist contractors, or from an open tender list.

The Federation of Civil Engineering Contractors has prepared a form of sub-contract which can be used with the addition of a suitable clause to cover piling work. On building contracts under the control of an architect the contract conditions are laid down in the Standard Form of Building Contract prepared by the Royal Institution of British Architects. In a similar manner the National Federation of Building Trades Employers has prepared standard forms of contract for nominated or non-nominated subcontractors. Model procedures for piling contracts and specifications are given in a publication of the Institution of Civil Engineers([11.10](#)).

In the following comments on the responsibilities for piling work it must be understood that the Engineer is not a party to the contract but acts as the agent for his employer and accepts responsibility on his behalf. The Architect acts in a similar way for the Employer in the RIBA form of contract. The term 'Contractor' refers to the piling contractor whether he is the main contractor or a nominated or non-nominated sub-contractor.

The Institution of Civil Engineers' publication describes four basic types of contractual arrangement under which piling may be undertaken. These are:

- (a) Civil Engineering works with an Engineer responsible to an Employer for design and supervision.
- (b) Building works with an Architect responsible to Employer for design and supervision and advised by an Engineer.
- (c) Building or civil engineering works with a Contractor responsible to an Employer for design and construction.
- (d) Building works with an Architect responsible to an Employer for design and supervision but having no engineering advisor.

The need for the Engineer to define his status and responsibilities is discussed in relation to procedures (b) and (c). Irrespective of whether piling work is executed as a main contract or by sub-contract, in British practice tenders are invited by one of three principal methods. These are as follows.

- |           |   |
|-----------|---|
| Method 1: | The Engineer is responsible for deciding the type or alternative type of pile, the working loads, and the allowable settlement under test load. The Engineer specifies the material to be used, the working stresses, fabrication methods and penetration depths. Tenders are invited on the basis of a detailed specification and drawings, which should be accompanied by a site investigation report, and a site plan showing existing surface levels, proposed regrading levels, and the operating levels for the piling rigs.  |
| Method 2: | The Engineer invites tenders for one or an alternative system of piling from specialist contractors. The invitation to tender is accompanied by a pile layout showing individual pile loads or column and wall loadings, and by a detailed specification including such items as materials, working stresses, performance under load test and other criteria of acceptability. The Contractor decides on the required type (or alternative type) of pile, the diameter and the penetration depth for the specified working loads, and bases his tender on his own estimates of performance. Site information as described for Method 1 should also be supplied. |
| Method 3: | The Engineer supplies a drawing to the tendering Contractor showing the wall and column layout of the structure together with the loadings; the site information as described for Method 1 is also supplied. No specification is issued and the Contractor is expected to submit a brief specification with his tender, and to guarantee the successful performance of the piles.   |

Method 1 has the advantage that the responsibility of each party is clearly defined. The Contractor has the responsibility only of selecting the most efficient type of plant to do the job and to install the piles in a sound manner complying with the specification. The method has the disadvantage that the knowledge and experience of the Contractor may not be fully utilized, since the Engineer may not always select the most suitable pile for the job. In exercising his responsibility for deciding on the pile diameters and penetration depths, the Engineer may instruct the Contractor to install preliminary test piles before making final decisions on the dimensions of the working piles.

Method 2 provides the widest choice of piling systems and utilizes the experience of the Contractor to the fullest extent, but greater care is needed in defining responsibility. In particular, the Engineer must specify precisely his requirements for performance under loading tests, both on preliminary and working piles. While the Contractor is responsible for selecting the type, diameter and penetration depth of the piles, he should be requested to submit his calculations for these selections for the approval of the Engineer. The statement concerning working loads on columns, walls or individual piles should make it clear as to whether or not the loads have been factored in compliance with the Eurocode or other structural codes of practice.

Method 3 is unsatisfactory in most respects. It is usually stated in the tender invitation or it is implied that the Contractor assumes responsibility for all aspects of the work. The Contractor must decide whether or not load testing is required and the criteria for successful performance under test. The method can work satisfactorily if the Engineer invites tenders only from those firms who have the necessary experience, and can be relied on to act in the best interests of the employer. However, problems can arise when, because of unforeseen variations in the ground conditions, the Contractor is obliged to increase substantially the penetration depth, or to increase the number of piles or even to abandon a particular system. These problems inevitably lead to claims by the Contractor, and the Engineer may find it difficult to recommend the employer to accept them since it reflects on his ability in selecting the Contractor. Inevitably much bitterness and recrimination follow. The method is also open to abuse when inexperienced contractors approach the employer directly to be put on the tender list. Again the Engineer may find it difficult to refuse tenders from these firms since it might be implied that he was unable to control the work in a proper manner.

Other items of responsibility which must be defined in the conditions of contract are as follows.

*The site investigation* is undertaken by the Engineer before inviting tenders for the piling. The Engineer should include all relevant details in the site investigation report in the tender documents.

*The facilities* provided by the main contractor, or to be included in the piling contract, should be stated. These include such items as access roads, hardstandings for piling plant, storage areas, fencing, watching, lighting, and the supply of electrical power and water. Hardstandings (working platforms) for large piling plant may need to be of substantial construction and the Engineer should state the form in which they will be provided including the level of the platform in relation to the pile commencing surface and cut-off level.

*Underground services and obstructions* can be a contentious item. It is normally the Engineer's responsibility to locate all known buried services and other obstructions to pile installation. It is unfair to the Contractor for the Engineer to disclaim all responsibility for the accuracy of the location plan, and to expect the Contractor to accept the consequences of damage to services. On the other hand the Engineer has the right to expect that the Contractor will not push on blindly with the piling work with complete disregard for the safety of the operatives or the consequences of damage. The Contractor is expected to keep a close watch on the conditions as the piling progresses and make his own enquiries as to the likely presence of further underground obstructions. The consequential damages can be very severe if, for example, a water main is broken which floods the running lines of an underground railway. Hence the clause in the conditions of contract covering underground obstructions needs to be carefully worded to be fair to the interests of all parties.

### 11.2.2 Piling specifications

In the interests of brevity the specifications should quote the British or other standard specifications listed in Appendix A when dealing with matters of materials, fabrication, and methods of test. BS 8004 can be quoted for working stresses on the various types of pile (see [Chapter 2](#)) but the clauses in this British Standard dealing with pile installation are not written with sufficient precision for use directly as specification clauses. It is not the intention in this chapter to give model clauses for piling work. These can be drafted with the guidance of the Institution of Civil Engineers' model specifications([11.10](#)), BS 8004 or other codes, but some matters which require particular attention are listed below.

*Setting out.* The responsibility for setting out is clear if the piling contractor is the main contractor. The Engineer has no responsibility in the matter but if he is prudent he should check the positions of the piles from time to time, since if these are inaccurately placed the remedial work can be very costly. Problems arise when a piling sub-contractor does his setting-out from a main contractor's grid-lines. If these are inaccurate or if, as sometimes happens, the numbering is obscured (or level pegs are confused with line pegs) then there can be major errors in pile positions and the main contractor may decline to accept responsibility for the cost of the replacement piling. If the specification does not define the responsibility for setting out, the piling sub-contractor must have a clear understanding with the main contractor on this matter.

*Ground heave.* In the case of the Method 1 type of contract the Engineer, in specifying the type and principal dimensions of the pile, must accept responsibility for the effects of ground heave, as described in 5.7. However, if the contract is of the Method 2 category the matter is not so clear, and piling contractors are reluctant to accept responsibility for ground heave, either for remedial work to risen piles, or for repairing damage to surrounding structures. In the author's opinion, the contractor for a Method 2 type of contract should have full responsibility, since he decides on the type and dimensions of the pile and he should have the necessary experience to decide whether or not ground heave will be detrimental. If the Contractor considers that there will be detrimental effects he must provide in his tender for pre-boring or other suitable precautionary measures, or he should decline to tender if his system cannot be used without damaging effects. If the Method 3 form of contract is adopted the Engineer must place full responsibility for ground heave effects onto the Contractor.

*Loss of ground due to boring.* The consequences of a loss of ground while boring for piles were described in 5.7. The responsibilities for these are similar to those for ground heave.

*Noise and vibration.* The Contractor is responsible for selecting the plant for installing piles and is therefore responsible for the effects of noise and vibration (see [3.1.7](#)). If the local authority has regulations limiting noise emissions these should be stated in the specification (or conditions of contract).

*Piling programme.* If the Engineer wishes to install the piles for the various foundations in a particular sequence to suit the main construction programme he should state the sequence in the specification, since it may not be the most economical one for the piling contractor to follow.

*'Set' of driven piles.* This should not be stated in precise terms in specifications for driven, or driven-and-cast-in-situ piles. The 'set' for a particular site and working load cannot be established until preliminary piles have been driven and the driving records checked against the ground conditions assumed in design. The set may have to be modified as a result of loading tests.

*Tolerances.* Tolerances in plan position, vertical deviation from the required rake, and deviation in level of the pile head, should be specified. Suitable values for tolerances are given in 3.4.6 and 3.4.11.

*Piling records.* The Engineer should specify the form in which he requires the Contractor to submit records (see [11.3](#)).

*Cutting down pile heads.* The specification should define whether it is the main contractor's or the piling contractor's responsibility to remove excess lengths of pile projecting above the nominal cut-off level. The responsibility for cutting away concrete to expose reinforcement, and trimming and preparing the heads of steel piles should also be stated.

*Method of measurement.* The method of measuring pile lengths as installed should be defined clearly in the specification. It is the usual practice to bill the quantities on the basis of the length from pile head or cut-off level to toe level. The *total* length is billed and the number of piles on which the total quantity is based should be stated. An item is billed for credits to be given by the Contractor if the measured length is less than the length billed for tender purposes. Another item is billed for payment to be made to the Contractor if the total measured length exceeds the billed length. It should be clearly stated that payment will be made only on the total measured length as installed and that credit will be demanded from the Contractor only if this total is less than the billed total, or extra payment to the Contractor will be authorized only if the measured total length exceeds the billed total. That is to say, credits or extras will not be reckoned on the basis of measured lengths of *individual* piles.

*Removal of spoil.* The respective responsibilities for the removal of spoil from bored piles, the removal of cut-off lengths of pile, trimming off laitance and ground raised by ground heave, and the disposal of used bentonite slurry, should be covered in the specification.

## 11.3 Control of pile installation

### 11.3.1 Driven piles

Control of driven-pile operations commences with the inspection and testing of the prefabricated piles before they are driven. Thus timber piles should be inspected for quality, straightness, and the application of preservative. The operations of casting precast concrete piles on site or in the factory should be inspected regularly and cubes or cylinders of the concrete should be made daily for compression testing at the appropriate age. Materials used for concrete production should be tested for compliance with the relevant standards. In the case of steel piles, tests should be made for dimensional tolerances and full documentation of the quality of the steel in the form of manufacturers' test certificates should be supplied with each consignment. Welding tests should be made for piles fabricated in the factory or on site. Full radiographic inspection of welds may be necessary only for marine piles, where the exposure conditions are severe (see [10.4](#)). The coating treatments should be checked for film thickness, continuity and adhesion. The Institution of Civil Engineers' specification([11.10](#)) draws attention to the need for degaussing steel piles by generating a counter-active magnetic field when welding is in progress. Degaussing may be needed to counter magnetization of the pile heads caused by driving. This can be detrimental to the quality of welds made for pile extensions.

A suitable form for recording the details of each pile driven is shown in [Table 11.1](#). While it is essential for the toe level and final set of every pile to be recorded every item shown in the table need not necessarily be filled in, although there are advantages in filling in all the items, including the pile driving log over the full depth for every pile driven. If, for example, piles are to be driven to end bearing on a hard stratum it may be sufficient to record the sets in blows for each 25mm of penetration after the pile has reached the hard stratum. On the other hand, where piles are supported in skin friction, say in a stratum of firm to stiff clay, or in a granular soil overlain by weak soils, it is essential to record for every pile the level at which the bearing stratum is encountered and hence to check that the required length of shaft to be supported by skin friction is obtained. For this purpose, when using metric units the blows required for each 500mm or each 250mm of penetration, must be recorded over the full depth of driving of each pile, until the final metre or so when the sets are recorded in blows for each 25mm. Sometimes final sets are recorded as penetration depths for 10 to 25 blows on the hammer. The advantage of recording the full driving log for piles of every category is that if troubles arise, such as pile breakage, the records of each pile can be scrutinized, and any which show peculiarities can be singled out for special examination or testing.

Eurocode 7, Section 7.20, lists the items required to be recorded for each pile driven. These are much the same as the items shown in [Table 11.1](#) with the addition of a requirement to record the power takeoff of a vibrating hammer, if used to drive the piles. Records are required to be kept for a period of 5 years after completing the works.

At the preliminary piling stage the driving records are compared with the site investigation data and with the results of loading tests, and suitable criteria regarding final sets for terminating driving are established. If the methods of [Chapter 4](#) have been used for calculating the penetration depth of friction

Table 11.1

<b>DAILY PILE RECORD FOR DRIVEN PILES</b> PILE RECORDS TO BE SUBMITTED TO OFFICE DAILY A SEPARATE SHEET TO BE USED FOR EACH PILE												
BLOCK NUMBER				DRAWING NUMBER								
SUB CONTRACTOR				PILE TYPE								
<b>1.GENERAL</b>	PILE REF. NO.			PILE DIA.			LEVEL OF TOE					
	GROUND LEVEL			CUT OFF LEVEL			CONCRETED LEVEL					
<b>2.DRIVING</b>	DATE STARTED			DATE COMPLETED			AIR TEMPERATURE					
	ERROR IN POSITION ON PLAN			ERROR IN PLUMB			DEPTH DRIVEN					
<b>3. OBSTRUCTIONS</b>	TYPE			DEPTH ENCOUNTERED			PENETRATION TIME					
<b>4.*STEEL</b> MAIN LINKS OR HELIX	NO. OF BARS			DIAMETER			LENGTH					
	CENTRES OF BARS/PITCH			DIAMETER			COVER TO ALL STEEL					
<b>5.CONCRETE</b> CAST-IN-SITU WORK ONLY	DATE STARTED			TYPE OF CEMENT			CONCRETE TEMPERATURE					
	MIX			SLUMP			SUPPLIER					
<b>6.*HAMMER</b>	TYPE		MASS	CAST-IN-SITU CONCRETE PILES ONLY ESTIMATED QUANTITY OF CONCRETE IN BULB								
<b>7.FINAL SET</b>	NO. OF BLOWS			DROP OF HAMMER			MOVEMENT OF PILE					
<b>8.PRECAST CONCRETE PILES ONLY</b>	COMPONENTS OF PILE			<b>A</b>	<b>B</b>	<b>C</b>	<b>D</b>					
	LENGTH OF COMPONENT											
	REF. NO./DATE CAST											
	O.D. LEVEL OF JOINT											
<b>9.PILE DRIVING LOG</b>  OTHER REMARKS MAY BE RECORDED ON REVERSE SIDE OF THIS SHEET.	DEPTH DRIVEN	NO. OF BLOWS	HAMMER DROP	DEPTH DRIVEN	NO. OF BLOWS	HAMMER DROP	DEPTH DRIVEN	NO. OF BLOWS	HAMMER DROP	DEPTH DRIVEN	NO. OF BLOWS	HAMMER DROP
<b>NOTE</b> * IF THERE ARE NO CHANGES TO BE RECORDED ITEMS 4 & 6 NEED BE COMPLETED FOR THE FIRST PILE ONLY IN EACH BLOCK.				SIGNED _____ CONTRACT SITE ENGINEER								



cubes or cylinders taken daily for compression tests. The quantity of concrete placed in the shaft of each pile should be recorded as a check against the possible collapse of the soil during the withdrawal of the tube. By checking the level of the concrete as each batch is placed an indication is given of possible ‘necking’ of the shaft. The volume of concrete in an enlarged base should be recorded as a check on the design assumptions for the diameter of base. A check should be made on the level of the reinforcing cage after withdrawing the drive tube on every pile driven. This is a safeguard against the concrete being lifted with the tube.

Thin shell piles should be inspected before placing the concrete by shining a light down the hole. This reveals any torn or buckled shells.

### 11.3.3 Bored-and-cast-in-place piles

The record form for bored-and-cast-in-place piles is shown in [Table 11.2](#). Recording of the soil strata in the pile boreholes is an essential check on the site investigation data. The items shown to be recorded in Section 7.10 of Eurocode 7 are similar to those listed in [Table 11.2](#), with the addition of items to cover continuous flight auger piles. These are: the pitch of the screw, the penetration per revolution, torque of drilling motor, and pumping pressure of grout or concrete.

If the boreholes are free of water, the conditions at the base of small-diameter piles in dry boreholes can be checked by shining a light down to the bottom before placing the concrete. In the case of large-diameter piles the base of all piles should be inspected from a safety cage of the type shown in [Figure 11.7](#) or by lowering the inspector in an approved chair or harness (BS 2380). The safety precautions should follow the procedure described in British Standard 5573 (Safety precautions in the construction of large diameter boreholes for piling and other purposes). The procedures and problems in placing concrete in pile boreholes are described in 3.4.6 and 3.4.7. An essential factor in controlling these operations is the maintenance of records of the quantity of concrete placed in each pile, and preferably in addition, the level of the concrete in the shaft as each batch is placed. Tests to control the quality of materials and mixing of concrete are, of course, required.

## 11.4 Load testing of piles

### 11.4.1 Compression tests

Two principal types of test are used for compressive loading on piles. The first of these is the constant rate of penetration (CRP) test developed by the Building Research Establishment ([11.11](#)), in which the compressive force is progressively increased to cause the pile to penetrate the soil at a constant rate until failure occurs. The second type of test is the maintained load (ML) test in which the load is increased in stages to some multiple, say 1.5 times or twice the working load with the time-settlement curve recorded at each stage of loading and unloading. The ML test may also be taken to failure by progressively increasing the load in stages.

The CRP method is essentially a test to determine the ultimate load on a pile and is therefore applied only to preliminary test piles or research-type investigations. The method has the advantage of speed in execution and because there is no time for consolidation or creep settlement of the ground the load-settlement curve is easy to interpret. BS 8004 states that penetration rates of 0.75mm/min are suitable for friction piles in clay and 1.55mm/min for piles end bearing in a granular soil. The CRP test is not suitable for checking the compliance with the specification requirements for the maximum settlement at given stages of loading. There is also the difficulty of pricing tenders for this form of load testing since the failure load on the pile is not known with any certainty until the test is made.

The ML test is best suited for contract work, particularly for proof loading tests on working piles. It is also suitable for use where empirical methods are employed to predict the ultimate load from measurement of residual deflections after returning the test load to zero at four or five stages up to the maximum (see below). The load at each stage is held until the rate of settlement has decreased to less than 0.25mm/hour and is still decreasing.

The section of the Institution of Civil Engineers’ specification dealing with loading tests defines the *specified working load* (SWL) as the specified load on the head of a pile as shown on the Engineer’s drawings or in the particular specification or in provided schedules. This is differentiated from the *Design Verification Load* (DVL) which is defined as ‘a load which will be substituted for the specified working load for the purpose of a test and which may be applied to an isolated or singly loaded pile at the time of testing the given conditions of the Site’. The DVL takes into account special conditions which may not apply to all piles on the site such as negative skin friction, or variations in pile head casting level.

Table 11.2

<b>DAILY PILE RECORD FOR LARGE-AND SMALL-DIAMETER BORED PILES</b>						
PILE RECORDS TO BE SUBMITTED TO OFFICE DAILY A SEPARATE SHEET TO BE USED FOR EACH PILE						
BLOCK NUMBER			DRAWING NUMBER / /			
<b>1. General</b>	PILE REF. NO.		PILE DIA.		LEVEL OF BASE	
			UNDERREAM DIA.			
	GROUND LEVEL		CUT OFF LEVEL		CONCRETED LEVEL	
<b>2. Drilling</b>	DATE STARTED		DATE COMPLETED		AIR TEMP	
	ERROR IN POSITION ON PLAN		ERROR IN PLUMB		DEPTH BORED	
<b>3. Obstructions</b> Natural Unnatural	TYPE		DEPTH ENCOUNTERED		PENETRATION TIME	
	TYPE		DEPTH ENCOUNTERED		PENETRATION TIME	
<b>4. *Steel</b> main steel links or helix	NO. OF BARS		DIAMETER		LENGTH	
	CENTRES OF BARS/PITCH		DIAMETER		COVER TO ALL STEEL	
<b>5. Concrete</b>	DATE STARTED		DATE COMPLETED		CONCRETE TEMP.	QUANTITY ACTUAL: THEORETICAL:
	MIX		SLUMP		SUPPLIER	
<b>6. Borehole</b> log and rock excavation	DEPTH OF SOIL	DESCRIPTION OF SOIL	DEPTH OF ROCK	DESCRIPTION OF ROCK	DEPTH OF ROCK AUGERED	DEPTH OF ROCK CHISELLED
<b>7. *Casing</b>	DEPTH OF TEMPORARY CASING		DEPTH OF PERMANENT CASING		REASON FOR USE OF PERMANENT CASING	
<b>8. *Water</b>	DEPTH ENCOUNTERED		DETAILS OF STRONG FLOW		DETAILS OF REMEDIAL MEASURES	
	DEPTH TO STRONG FLOW					
<p>Note : * If there are no changes to be recorded, items 3, 4, 7 and 8 need be completed for the <i>first pile only</i> in each block.</p> <p>Remarks</p> <p style="text-align: center;">SIGNED CONTRACT SITE ENGINEER</p>						

Table 11.3 Loading sequence for proof load test to 100% DVL plus 50% SWL

Load	Minimum time of holding load
25% DVL	30 minutes
50% DVL	30 minutes
75% DVL	30 minutes
100% DVL	1 hour
75% DVL	10 minutes
50% DVL	10 minutes
25% DVL	10 minutes
0	1 hour
100% DVL	6 hours
100%DVL+25% SWL	1 hour
100% DVL+50% SWL	6 hours
100% DVL+25% SWL	10 minutes
100% DVL	10 minutes
75% DVL	10 minutes
50% DVL	10 minutes
25% DVL	10 minutes
0	1 hour

The ICE recommends that a proof load test should normally be the sum of the DVL plus 50% of the SWL applied in the sequence shown in [Table 11.3](#).

Eurocode 7, Section 7.5.2.1, requires the load-testing procedure to be in accordance with the ISSMFE Subcommittee on Field and Laboratory Testing recommended procedure ‘Axial Pile Loading Test, Suggested Method’ as published in the *ASTM Geotechnical Testing Journal*, June 1985, pp. 79–90.

A similar system can be devised for a test to twice the working load. If it is desired to obtain the ultimate load on a preliminary test pile it is useful to adopt the ML method for up to twice the working load, and then to continue loading to failure at a constant rate of penetration. A further modification of the ML test consists of returning the load to zero after each increment. This form of test is necessary if the net settlement curve is used as the basis of defining the failure load (see [11.4.2](#)). It is essential to maintain a constant rate of load application as specified in BS 8004. It has been found that the slower the rate the smaller is the ultimate failure load. Hence the need for standardization both for ML and CRP testing. CRP and ML tests use the same type of loading arrangements and pile preparation. A square cap is cast onto the head of a concrete pile with its underside clear of the ground surface. Steel piles are trimmed square to their axis and a steel plate is welded to the head, stiffened as necessary by gussets. Suitable loading arrangements for applying the load to the pile by a hydraulic jack using as the reaction, either kentledge, tension piles or cable anchors are shown in [Figures 11.8, 11.9, and 11.10](#) respectively. The clearances between the pile and the reaction support systems are noted in each case. These are necessary to avoid the induced horizontal pressures from the supports having an appreciable effect on the skin friction and base load of the test pile. It is uneconomical to space the supports so widely apart that all effects are eliminated, and if necessary the contribution of these surcharge effects should be calculated and allowed for in the interpretation of the test results.

Where piles are installed through fill or soft clay these materials give positive support in skin friction to the test pile, whereas they may add to the working load in negative skin friction on the permanent piles. It may therefore be desirable to sleeve the pile through these layers. This can be done by using a double sleeve arrangement in the manner shown in [Figure 11.11](#).

Alternatively the outer casing can be withdrawn, after filling the annular space between it and the steel tube encasing the test pile with a bentonite slurry.

It is inadvisable to test raking piles by a reaction from kentledge or tension piles since the horizontal component of the jacking force cannot be satisfactorily restrained by the jacking system. Cable anchors inclined in the same direction as the raking piles can be used but it is preferable to determine the ultimate or allowable loads on raking piles by installing special vertical piles for loading tests.

The combined weight of the kentledge and reaction girders, or the calculated resistance capacity of tension piles or cables, must be greater than the maximum jacking force required. In the case of kentledge loading the combined weight should be about 20% greater than this force. Cable anchorages or tension piles should have an ample safety factor against uplift. The former can be tested by stressing the anchors



*Fig. 11.7 Safety cage used for inspection of pile boreholes*

after grouting them in. If there is any doubt about the uplift capacity of tension piles a test should be made to check the design assumptions. Increased capacity of tension piles in clays can be obtained by under-reaming them (see [6.2](#)).

The reaction girders and load-spreading members should be so arranged that eccentric loads caused by any lateral movement of the pile head will not cause dangerous sidesway, or buckling of the girders. Connections should be bolted so that they will not become dislodged if there is a sudden rebound of load due to the failure of the pile shaft or of the jack. Similarly the kentledge stack should not be arranged in such a way that it may topple over.

Restraint by a pair of anchors from a single pile to each end of the reaction girder is not a good practice as it can cause dangerous sidesway of a deep girder. The piles or anchor cables should be

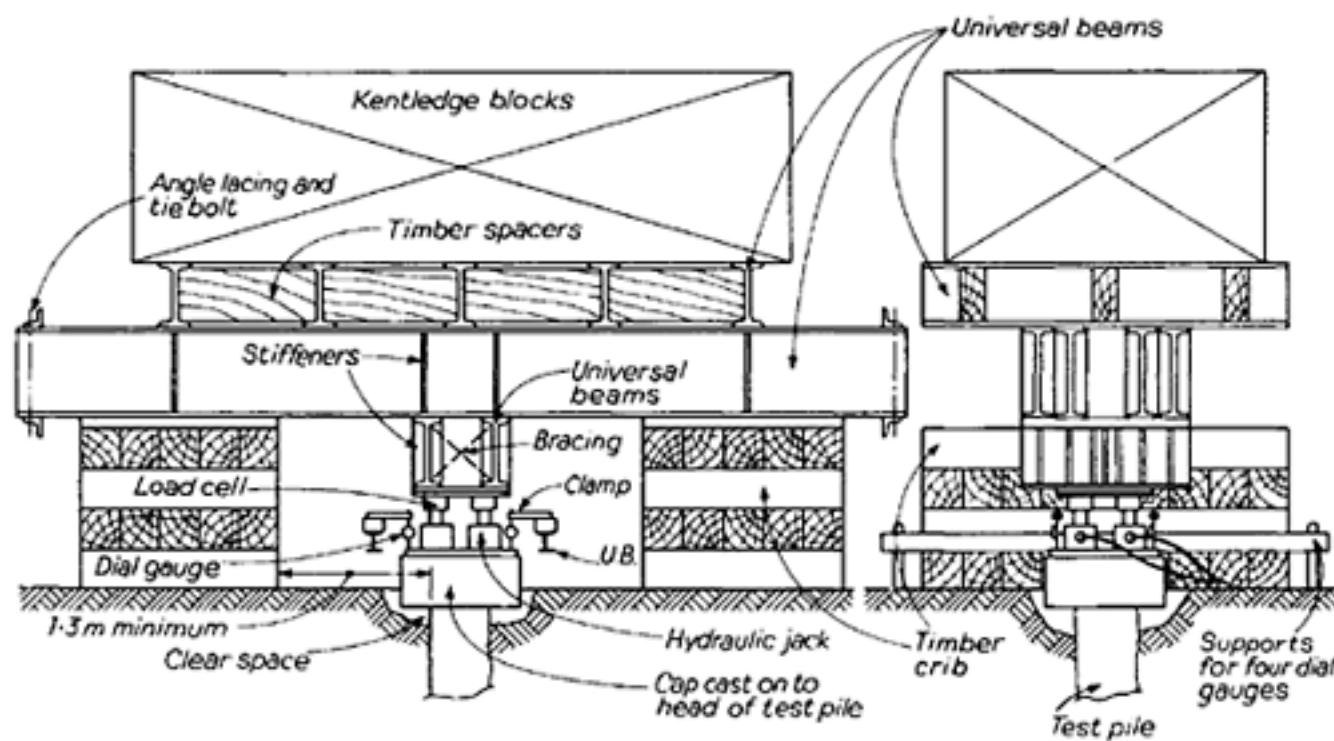


Fig. 11.8 Testing rig for compressive test on pile using kentledge for reaction

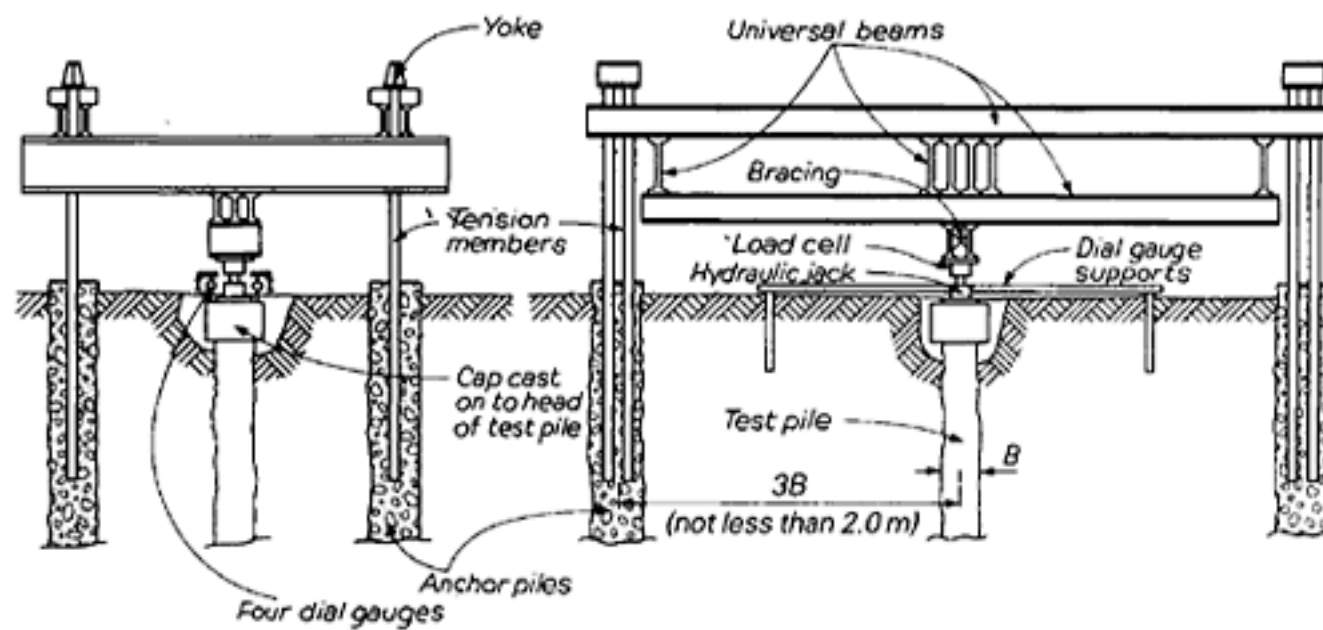


Fig. 11.9 Testing rig for compressive test on pile using tension piles for reaction

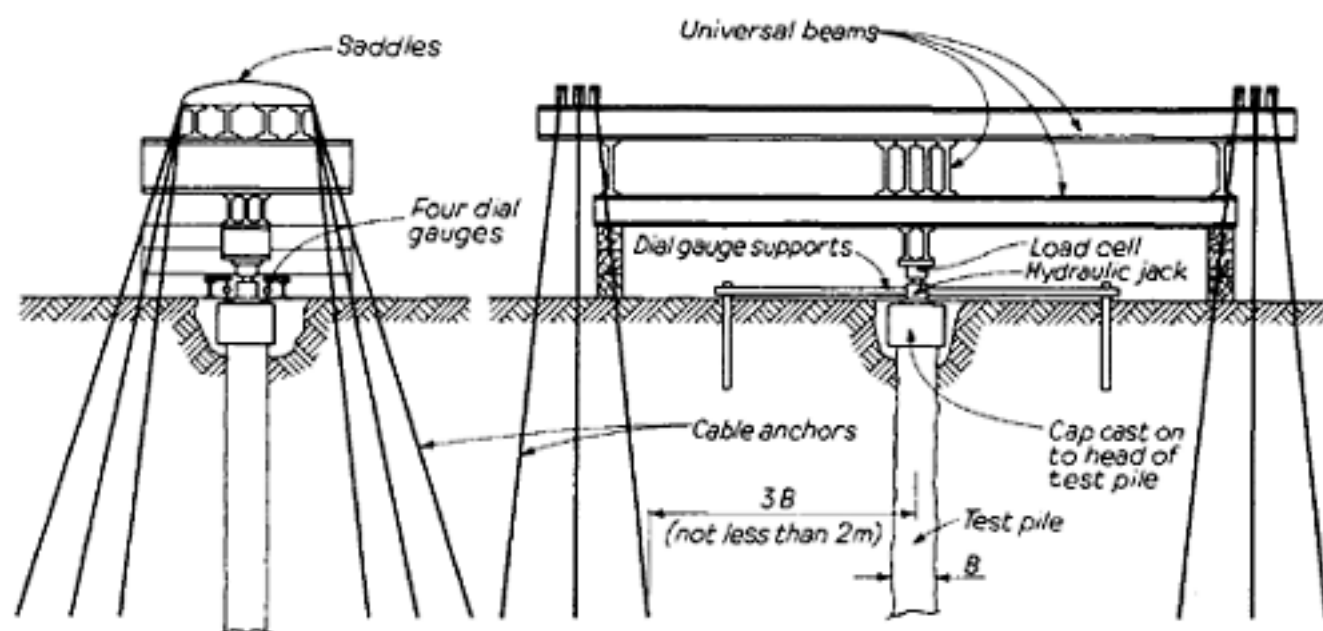


Fig. 11.10 Testing rig for compressive test on pile using cable anchors for reaction

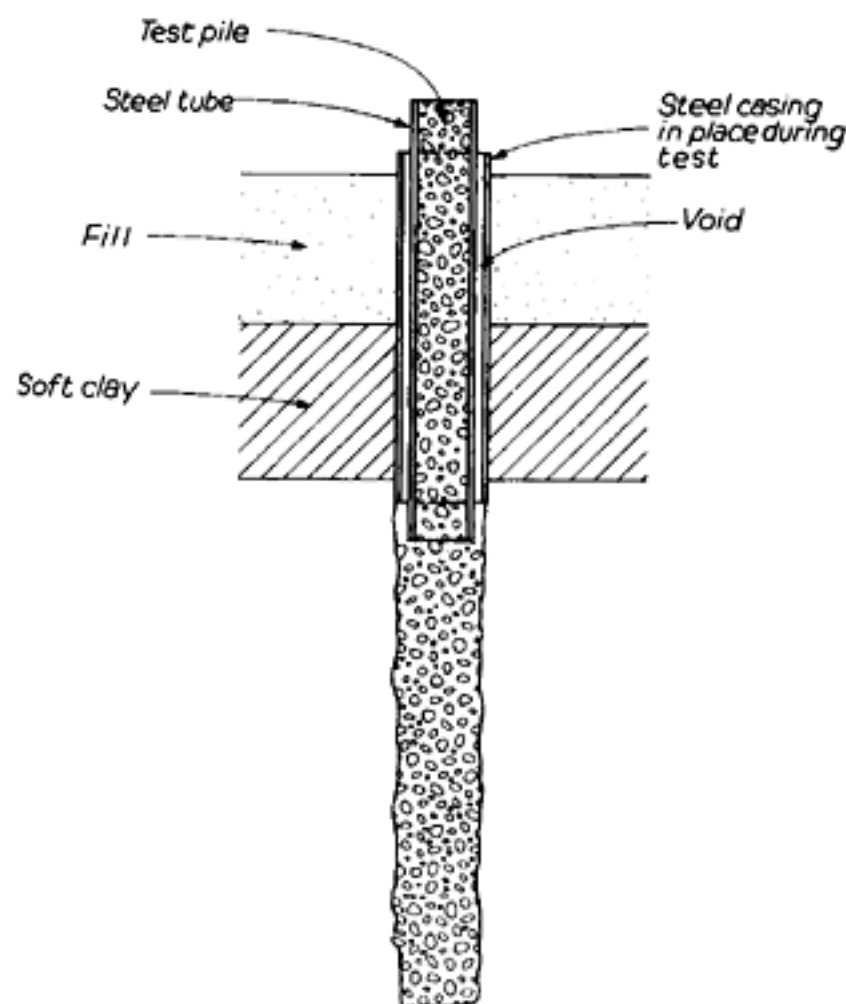


Fig. 11.11 Method of sleeving test pile to eliminate skin friction through fill

placed in pairs at each end of the girders, as shown in Figures 11.9 and 11.10. Permanent piles can be used as anchorages for ML tests on working piles but it is unwise to use end-bearing piles for this purpose when the skin friction will be low and the pile may be lifted off its seating. When using tension piles special threaded anchor bars extending above the pile head should be cast into the piles for attachment to the reaction girders. It is inadvisable to weld such bars to the projecting reinforcing bars because of the difficulty in forming satisfactory welds to resist the high tensile forces involved.

The hydraulic jack should have a nominal capacity which exceeds by 20% or more the maximum test load to be applied to the pile. This is necessary in order to avoid heavy manual pumping effort when nearing maximum load and to minimize the risks of any leakage of oil through the seals. The jacking force on the pile head should be measured by a load cell or pressure capsule since the pressure gauge fitted to most jacks is not sufficiently accurate, particularly when working towards the maximum capacity of the jack. However, the jack should have a pressure gauge mounted on the pumping unit which is calibrated to read in terms of the force on the ram. This gauge is necessary since the load cell mounted on the pile head may not be visible from the pumping position. For high-capacity piling tests, much heavy manual effort is saved by providing a mechanical pumping unit, and where CRP tests are being made a load pacer is a useful addition. The ram of the jack should have a long travel where piles are being loaded near to the failure condition. This avoids the necessity of releasing oil pressure and repacking with steel plates above the ram as the pile is pushed into the ground. Equipment is available for monitoring and restoring jacking loads at intervals of only a few seconds(11.12).

The settlement of the pile head under load can be measured optically by means of a surveyor's level reading onto graduated scales fixed to the pile in four positions or by laser beam producing an image on a photo-electric detector. Using the surveyor's level, if the scales are calibrated in 0.5mm intervals the movement of the pile can be measured to an accuracy of  $\pm 1$ mm which is adequate for most piling tests. An alternative method is to set a dial gauge on each of four reference points on the pile head. The dial gauges are clamped to a datum frame securely mounted well clear of the ground around the pile or the reaction support system, and the gauges are calibrated to read to 0.1mm. This order of accuracy is not realized in practice since wind, temperature effects and ground vibrations can cause the datum frame to move by much more than 0.1mm. However, it is helpful to be able to read to such an accuracy when making each increment of jacking force since the time-settlement curve can then be plotted accurately and the rate of decrease of movement is readily obtained. Levels should be taken on the datum frame before and after the loading test to check that the frame has not been displaced during the test. A linear potentiometer can be used to obtain the pile movements, which are read on a dial or print-out mechanism at an instrument station well clear of the pile. With a well designed kentledge support system the technicians should not be in a dangerous position under the

stack, but the working space is usually very confined, causing discomfort and fatigue to the technicians controlling a long-duration test.

Fleming<sup>(11.12)</sup> has reported that the equipment used to record pile head loads and settlements can be linked to a portable computer to plot load/settlement and time/settlement curves as the test is in progress. The data can be reproduced in the format of the test report and even used to analyse the pile behaviour throughout the whole range of loading.

Where piles have been designed by the methods described in [Chapter 4](#) it is very helpful to provide devices whereby the shaft and base loads can be evaluated separately. The load on the base of the pile can be measured by inserting load measuring devices in a cylindrical unit interposed between the pile base and the shaft. A typical installation consists of a ring of pillar-type load cells around the periphery of the unit recording to a data logger at the ground surface<sup>(11.13)</sup>. The distribution of skin friction on the pile shaft can be measured by fixing electrical-resistance strain gauges onto the interior surface of a hollow steel pile, or to a steel pipe embedded in a precast or cast-in-situ concrete pile. Gauges of this type can withstand the impact of pile driving and have given satisfactory service on piles which have remained in the ground for a year or more<sup>(4.5)</sup>.

A simple method described by Hanna<sup>(11.14)</sup> for obtaining the distribution of skin friction on the shaft of long hollow-section piles consists of installing metal rods down the interior of the pile. The rods are terminated at various levels as shown in [Figure 11.12](#) and are free to move in guides as the pile settles under load. By means of dial gauges mounted on the heads of the rods the elastic shortening of each length of pile between the toes of the rods can be measured. Thus the load reaching the pile shaft at the toe of each rod is given by the following expressions.

$$\text{Load at pile } P_3 = \frac{2AE\Delta_3}{l_3} - P \quad \dots(11.3)$$

$$\text{Load on pile at toe of rod 2 } P_2 = \frac{2AE\Delta_2}{l_2} - P \quad \dots(11.4)$$

where  $P_3$  and  $P_2$  are the loads reaching the pile at the level of the toe of the rod considered,  $A$  is the cross-sectional area of the pile shaft,  $E$  is the elastic modulus of the material forming the pile,  $\Delta_3$  and  $\Delta_2$  are the elastic shortenings of the pile between the pile head and rod 3, and the pile head and rod 2, respectively,  $l_3$  and  $l_2$  are the lengths of the rods, and  $P$  is the load on the pile head.

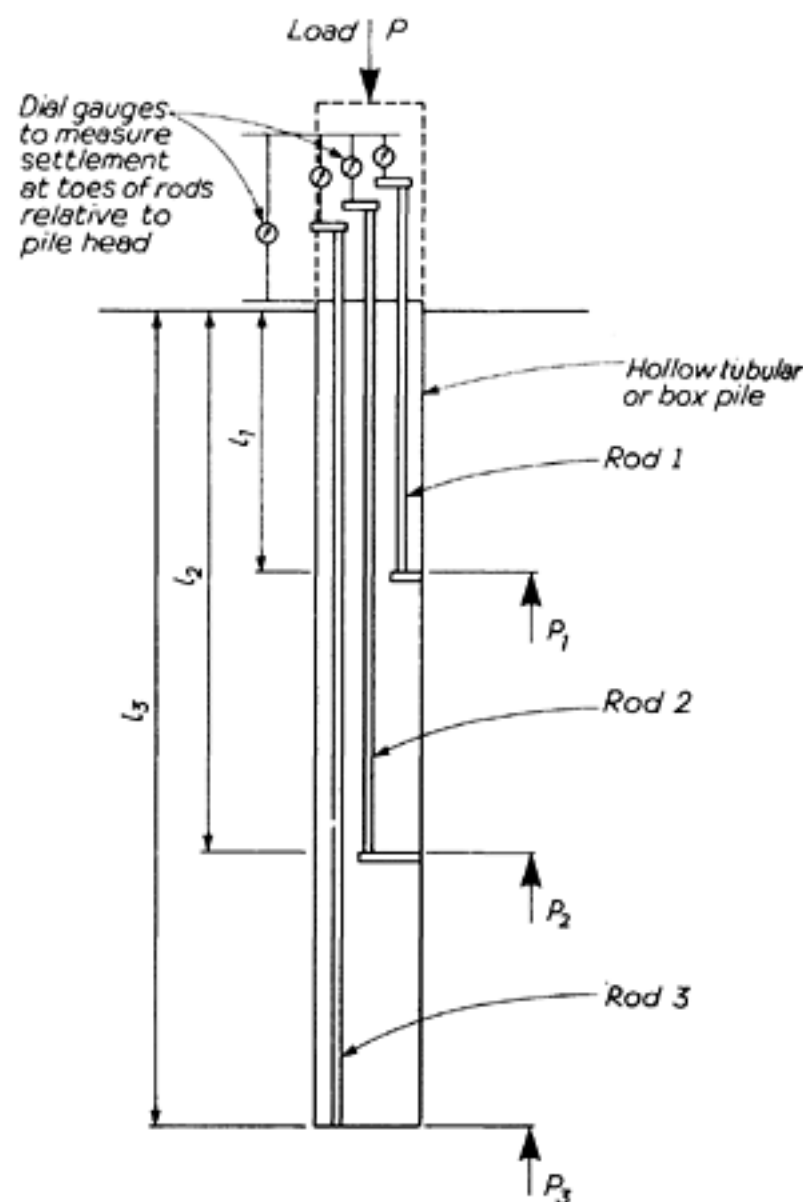


Fig. 11.12 Use of rod strain gauges to measure load transfer from pile to soil at various levels

*down pile shaft*

The load at the toe of rod 1 is obtained in a similar manner.

Where the rod strain gauges are used in the interior of a steel tubular pile filled with concrete the elastic shortening between each length of pile is that due to the elastic modulus of the composite section. Thus

$$\Delta_l = \frac{P \times l}{A_s E_s \left( 1 + \frac{A_c}{A_s} \cdot \frac{E_c}{E_s} \right)} \quad \dots(11.5)$$

where  $\Delta_l$  is the elastic shortening over length  $l$ ,  $P$  is the load on length  $l$ ,  $A_s$  is the area of steel,  $A_c$  is the area of concrete,  $E_s$  is the elastic modulus of the steel, and  $E_c$  is the elastic modulus of the concrete.

While these forms of instrumentation are used mainly for research-type investigations they can be adopted for the preliminary test piling to give useful design information at a relatively small additional cost.

Further guidance on the procedure for pile load testing is given by Weltman<sup>(11.15)</sup>.

A recent development in compression test loading is the *Statnamic* test developed jointly by the Berminghammer Corporation in Canada and TNO-IBBC in the Netherlands<sup>(11.16)</sup>. The load is applied to the pile head by the reaction of an explosive force designed to raise weights mounted on the pile head to a height of about 2.5m at accelerations of up to 20g. It is claimed that over 10MN of force is generated by the explosion of fuel in a combustion chamber beneath the stack of cylindrical weights. It is also claimed that the duration of the explosive reaction of about 100 milliseconds can reproduce the effect of static loading up to the working load, with little divergence at twice the working load; thus simulating a static rather than a dynamic load test of the type described in [Section 7.3](#). The force on the pile is measured by a load cell and the deflections of the pile by a laser beam and light-sensitive sensor.

#### 11.4.2 Interpretation of compression test records

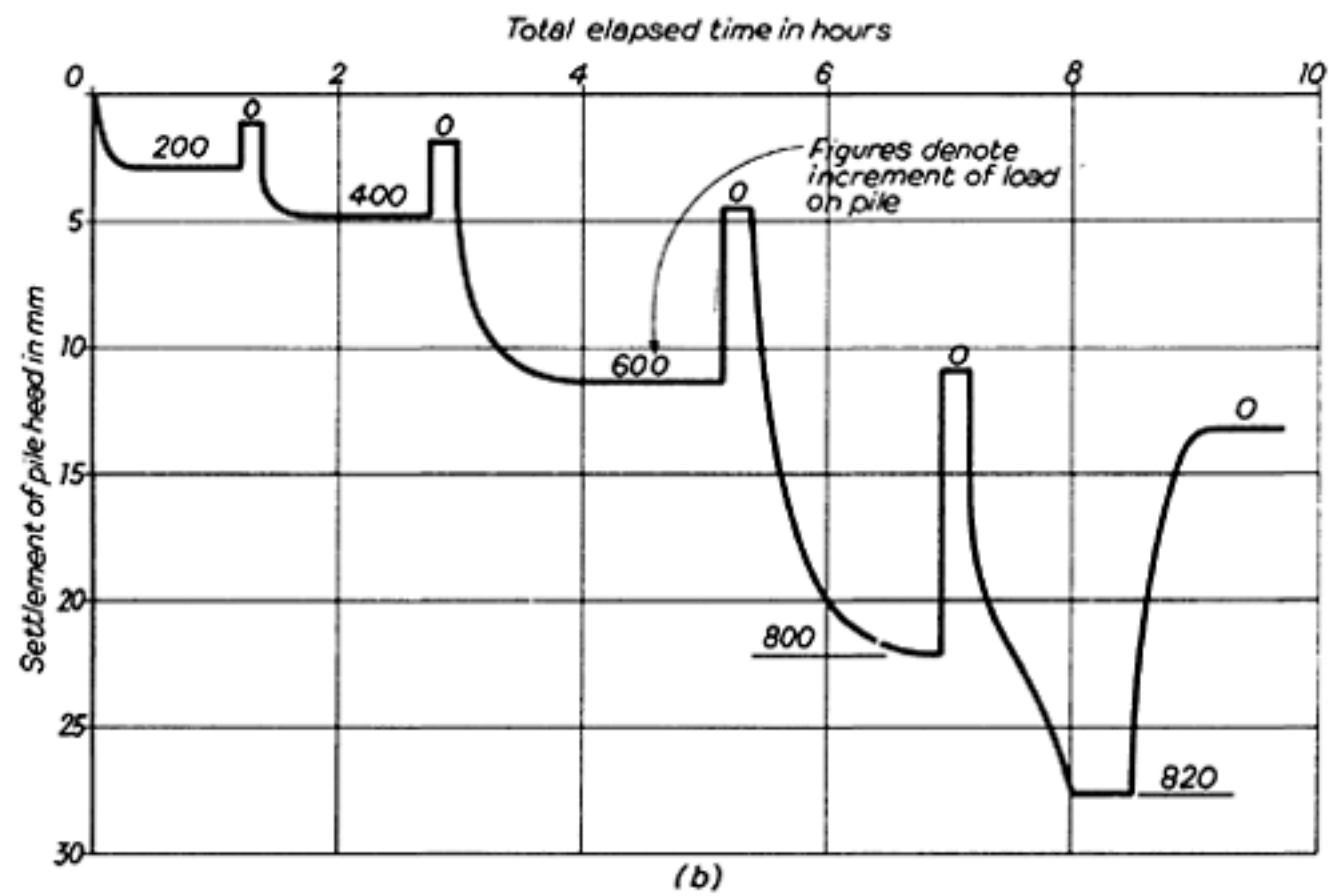
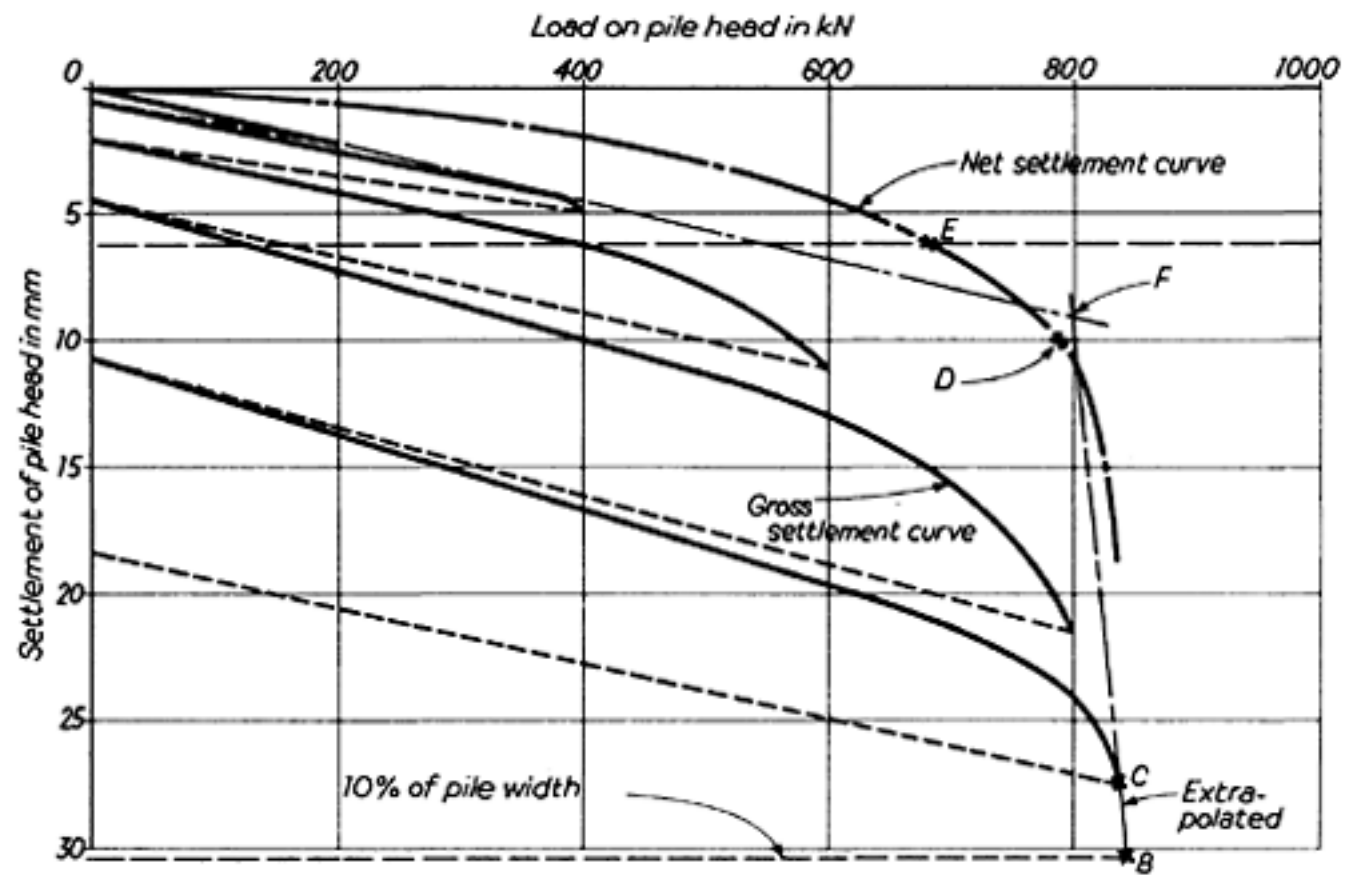
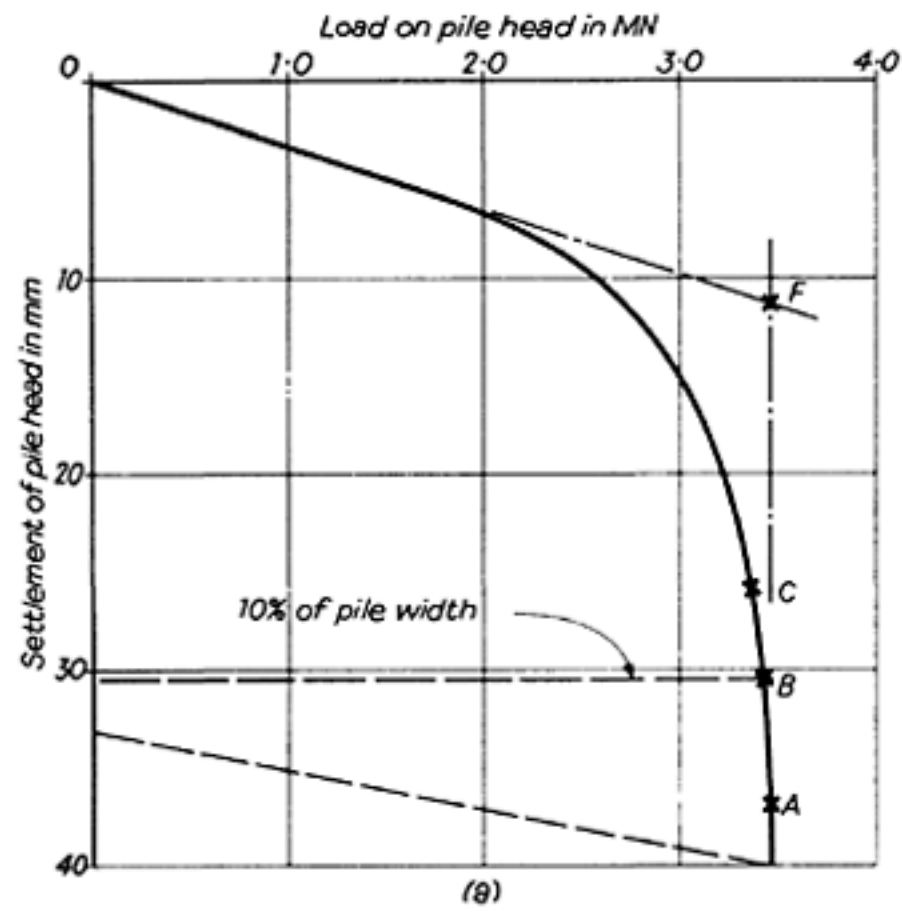
A typical load-settlement curve for the CRP test and a load-time-settlement curve for the ML test are shown in [Figure 11.13](#). The ultimate or failure load condition can be interpreted in several different ways. While there is no doubt that failure in the soil mechanics sense occurs when the pile plunges down into the ground without any further increase in load, from the point of view of the structural designer the pile has failed when its settlement has reached the stage that unacceptable distortion and cracking is caused to the structure which it supports. The latter movement can be much less than that resulting from ultimate failure in shear of the supporting soil.

With reference to [Figure 11.13](#), some of the recognized criteria for defining failure loads are listed as follows.

1. The load at which settlement continues to increase without any further increase of load (Point A).
2. The load causing a gross settlement of 10% of the least pile width (Point B).
3. The load beyond which there is an increase in gross settlement disproportionate to the increase in load (Point C).
4. The load beyond which there is an increase in net settlement disproportionate to the increase of load (Point D).
5. The load that produces a plastic yielding or net settlement of 6mm (Point E).
6. The load indicated by the intersection of tangent lines drawn through the initial, flatter portion of the gross settlement curve and the steeper portion of the same curve (Point F).
7. The load at which the slope of the net settlement is equal to 0.25mm per 10kN of test load.
8. The City of New York (1985 Building Code) defines the allowable load on a pile as one-half of the load at which the net settlement after the removal of the load does not exceed 0.25mm per 10kN of test load where the net settlement is the gross settlement due to the test load, minus the rebound after removing 100% of the test load, or one half the load causing a net settlement of 19mm where the net settlement is as defined above, less the elastic shortening of the pile due to the total test load.

Criterion 3 is defined more precisely by the Swedish Piling Commission as the load which gives twice the movement of the pile head obtained for 90% of the load.

Eurocode 7, Section 7.6.3, prescribes a method for assessing design pile loads from the load-settlement



*Fig. 11.13 Compression load tests on 305×305mm pile*

- (a) Load-settlement curve for CRP test for pile on dense gravel*
- (b) Load-settlement and time-settlement curves for pile on stiff clay*

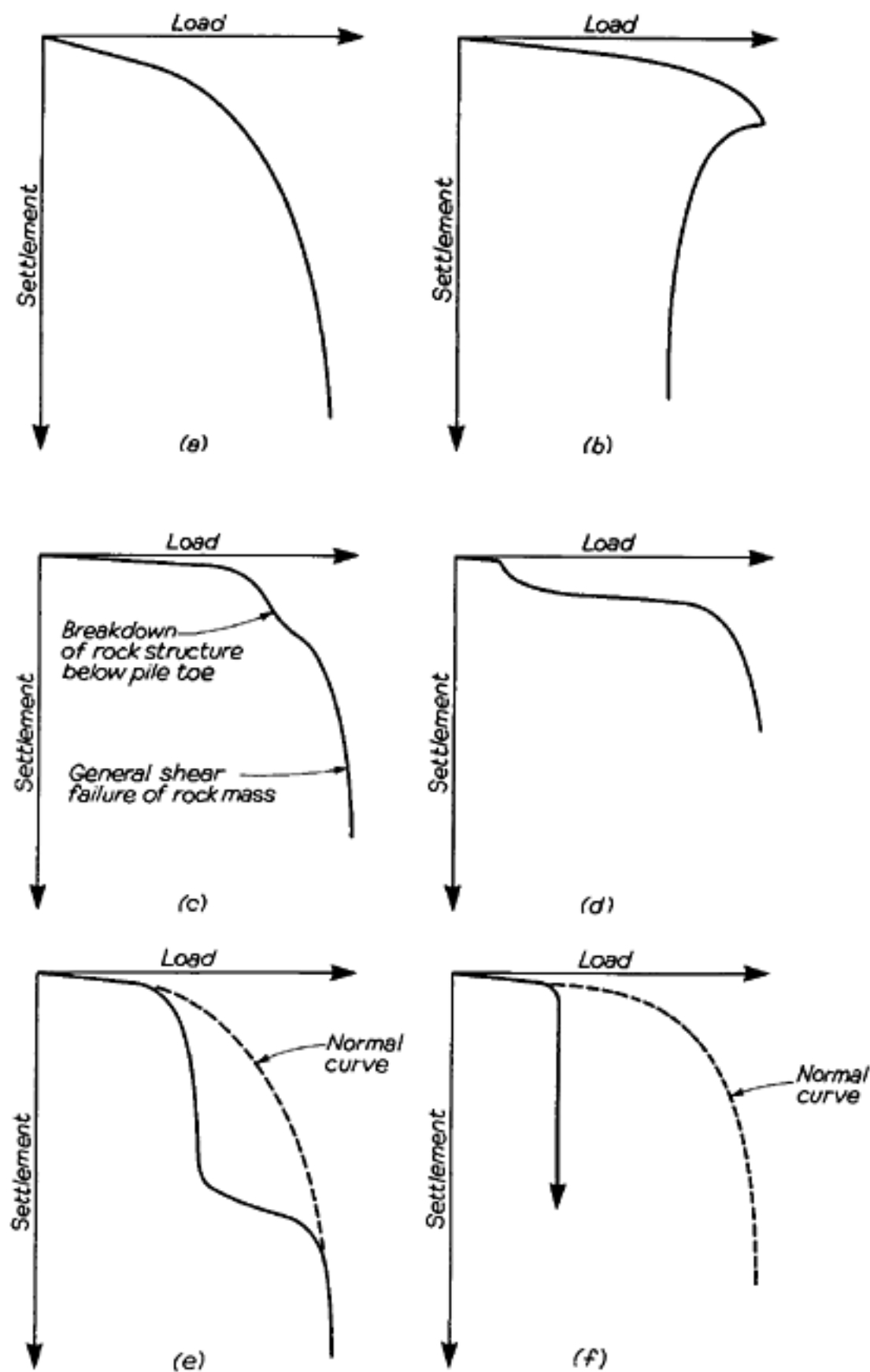


Fig. 11.14 Typical load-settlement curves for compressive load tests

- (a) Friction pile in soft-firm clay or loose sand
- (b) Friction pile in stiff clay
- (c) Pile end bearing on weak porous rock
- (d) Pile lifted off seating on hard rock due to soil heave and pushed down by test load to new bearing on rock
- (e) Gap in pile shaft closed up by test load
- (f) Weak concrete in pile shaft sheared completely through by test load

curves. The design loads are referred to as  $F_{ULS}$  and  $F_{SLS}$  for checks on the ultimate and serviceability limit-states,

respectively. The code should be consulted for details of the method.

With experience the load-settlement curve from a compression test can be used to interpret the mode of failure of a pile. A defective pile shaft is also indicated by the shape of the curve. Some typical load-settlement curves and their interpretation are shown in [Figure 11.14](#).

A method of analysing the results of either CRP or ML tests to obtain an indication of the ultimate load is described by Chin

(11.17). The settlement  $\Delta$  at each loading stage  $P$  is divided by the load  $P$  at that stage and plotted against  $\frac{\Delta}{P}$  as shown in [Figure 11.15](#). For an undamaged pile a straight line plot is produced. For an end-bearing pile the plot is a single line ([Figure 11.15a](#)). A combined friction and end-bearing pile produces two straight lines which intersect ([Figure 11.15b](#)). The inverse

slope of the line gives the ultimate load in each case. Chin describes how a broken pile is detected by a curved plot ([Figure 11.15c](#)).

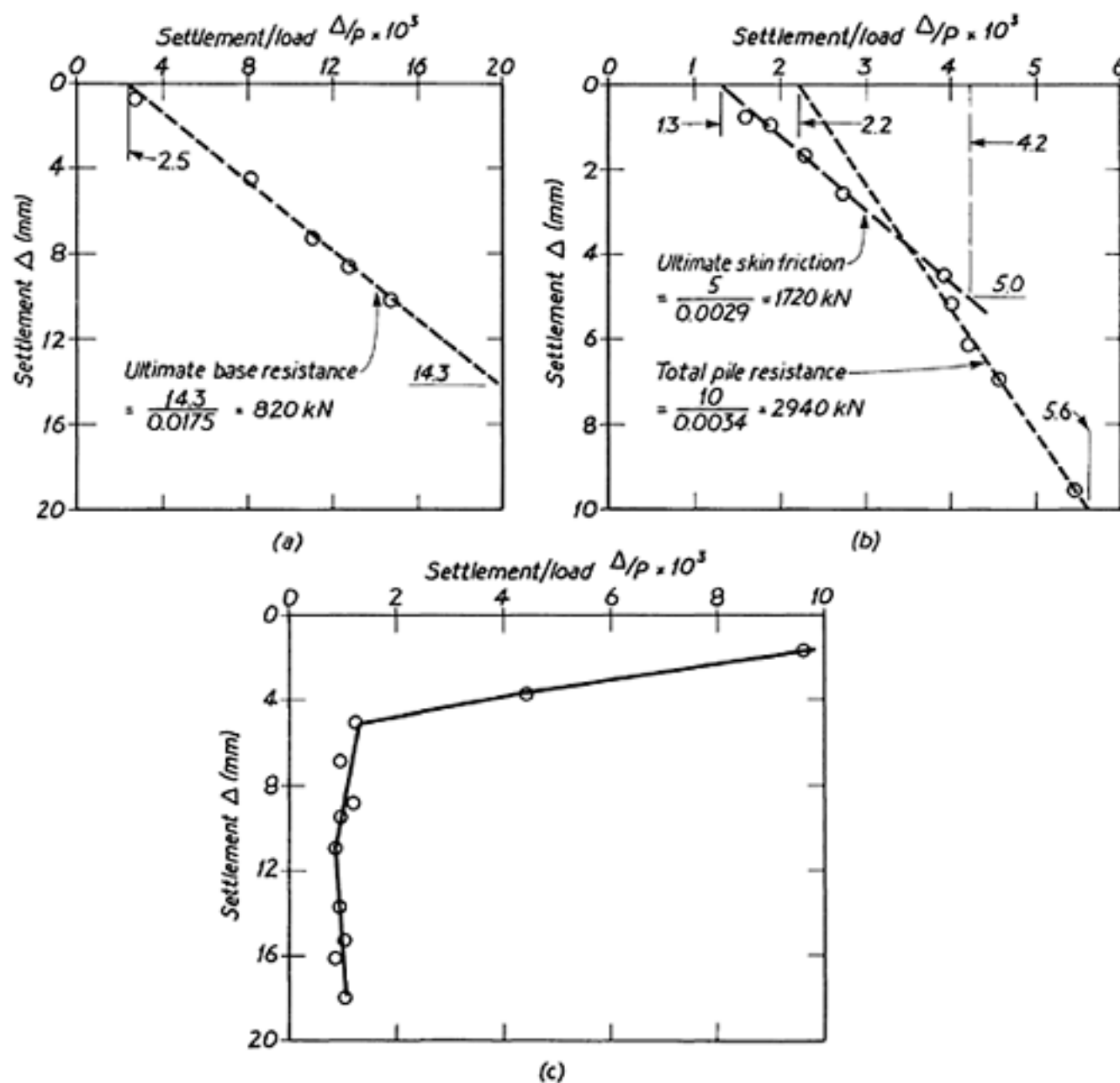


Fig. 11.15 Analysis of load settlement curves from pile loading test (after Chin<sup>(11.17)</sup>)

- (a) End bearing pile
- (b) Friction and end bearing pile
- (c) Broken pile.

### 11.4.3 Uplift tests

Uplift or tension tests on piles can be made at a continuous rate of uplift (CRU), or an incremental loading basis (ML). Where uplift loads are intermittent or cyclic in character, as in wave loading on a marine structure, it is good practice to adopt repetitive loading on the test pile. The desirable maximum load for repeated application cannot be readily determined in advance of the load testing programme since the relationship between the ultimate load for a single application and that for repeated application is not known. Ideally a single pile should be subjected to a CRU test to obtain the ultimate load for a single application. Then two further piles should be tested; one cycled at an uplift load of, say, 50% of the single-application ultimate load, and the second at 75% of this value. At least twenty-five load repetitions should be applied. If the uplift continues to increase at an increasing rate after each repetition, the cycling should be continued without increasing the load until failure in uplift occurs. Alternatively an incremental uplift test can be made with say ten repetitions of the load at each increment.

A typical load-time-uplift curve for an ML test is shown in [Figure 11.16](#). The criteria for evaluating the failure load are similar to those described in 11.4.2.

Eurocode 7, [Sections 7.5](#) and 7.7.2.2, prescribe a method of deriving the design tensile capacity,  $Q_t$ , of a single pile from

tension tests. The load causing an uplift of 10% of the equivalent shaft diameter is defined as the limit load  $Q_{tL}$ . The mean

and lowest values of  $Q_{tL}$  obtained from a number of load tests are reduced by a variable factor (which depends on the number

of tests) to obtain the characteristic capacity  $Q_{tk}$ . The design capacity  $Q_t$  is then obtained from  $Q_{tk}/\gamma_m$ . The material factor  $\gamma_m$

is specified as 1.6.

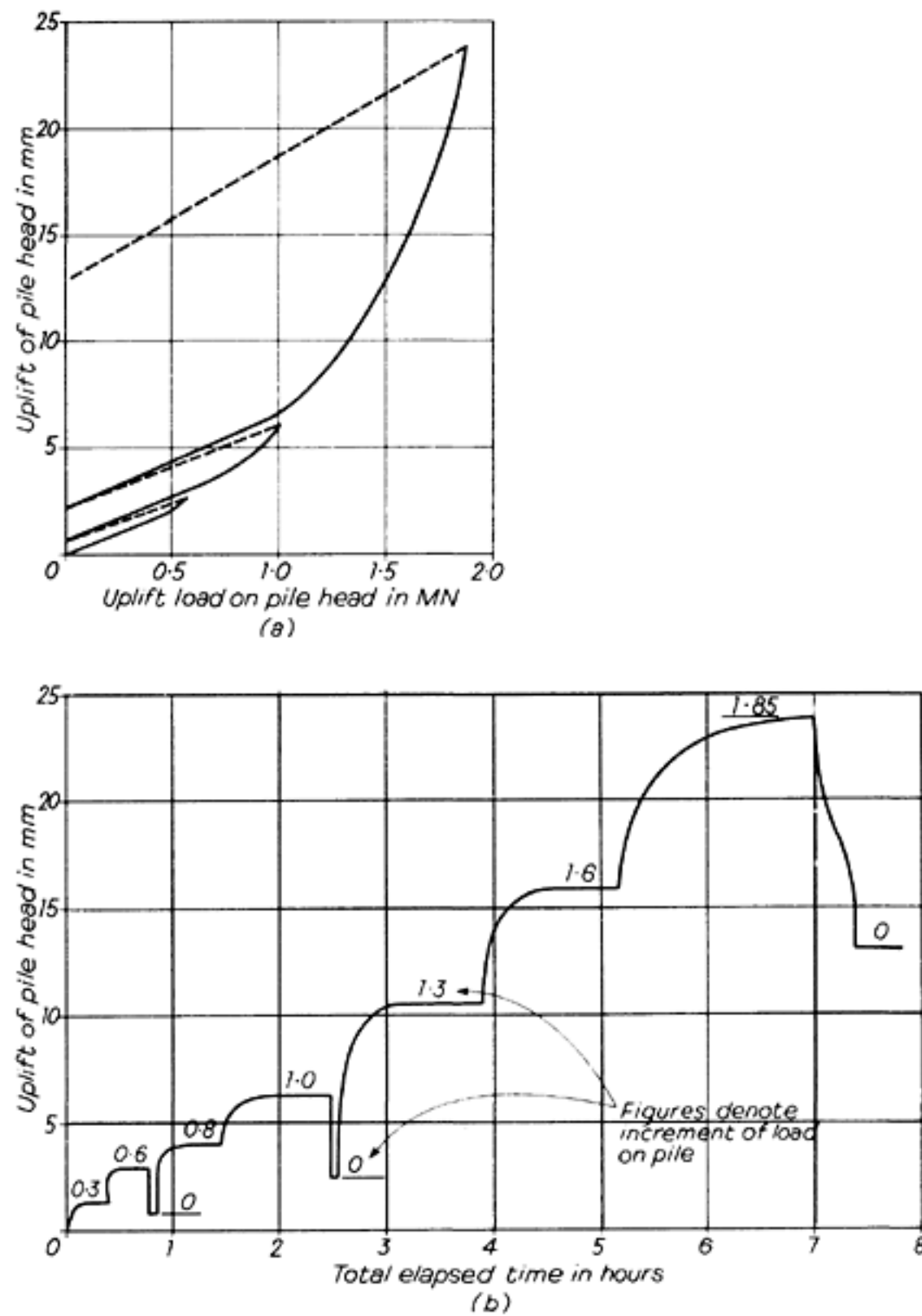


Fig. 11.16 Uplift load on test pile (ML test)

- (a) Load-uplift curve  
(b) Time-uplift curve

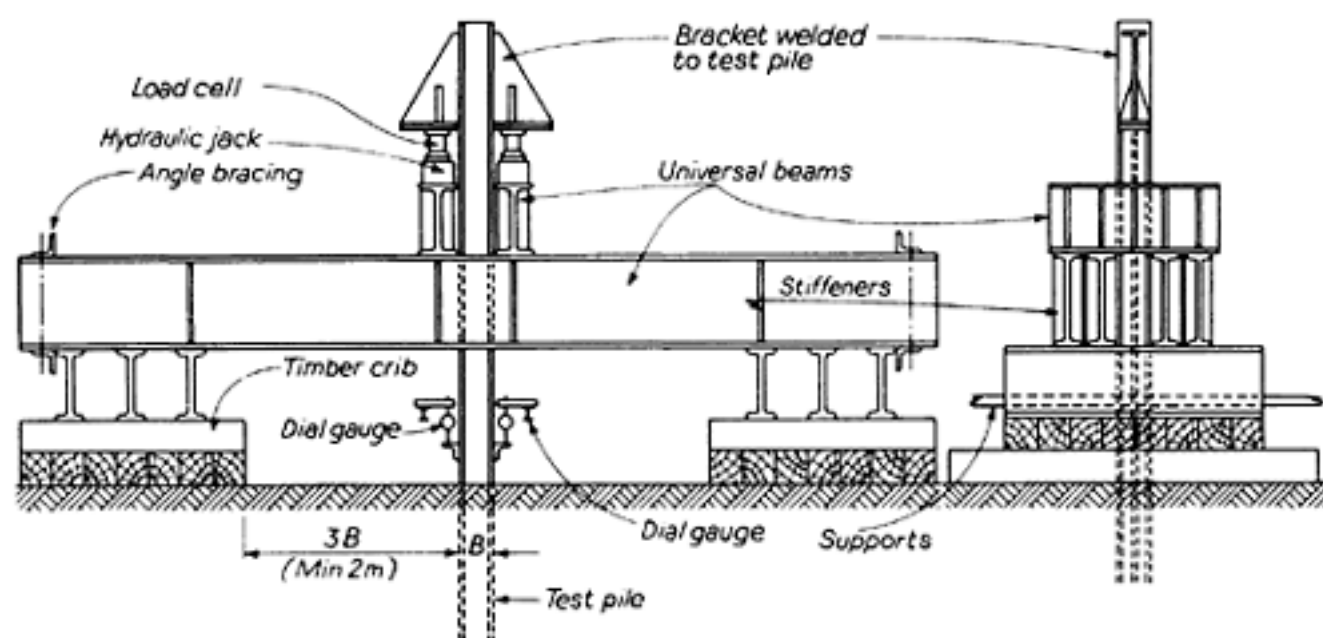


Fig. 11.17 Testing rig for uplift test on H-section pile using ground as reaction

A loading rig for an uplift test is shown by [Figure 11.17](#). This utilizes the same components as the compressive load testing

rig shown in [Figure 11.9](#). The methods used for measuring the jacking force and the movement of the pile head are the same as those used for compressive tests. It is particularly important to space the ground beams or bearers at an ample distance from the test pile. If they are

too close the lateral pressure on the pile induced by the load on the ground surface will increase the skin friction on the pile shaft.

#### 11.4.4 Lateral loading tests

Lateral loading tests are made by pulling a pair of piles together, or jacking them apart. If the expected movements are large, for example when obtaining the load-deflection characteristics of breasting dolphin piles, a 'Tirfor' or block and tackle can be employed to pull the piles together and a graduated staff used to measure the horizontal movement, as shown in [Figure 11.18](#). Where the lateral loads on piles are of a repetitive character, as in wave loading or traffic loads on a bridge, it is desirable to make cyclic loading tests. This involves alternately pushing and pulling of a pair of piles, using a rig of the type shown in [Figure 11.19](#). Instead of a pair of piles a single pile can be pushed or pulled against a thrust block ([Figure 11.20](#)). Where pushing methods are used restraining devices should be provided to ensure that the jack and strut assembly does not buckle during the application of load.

Where possible the lateral movement of the pile heads should be measured by dial gauges mounted on a frame supported independently of the test piles, as shown in [Figure 11.20](#). This may not be feasible in marine piles since the oscillation of the piles and the structure supporting the frame in waves and currents may make it impossible to obtain readings with sufficient accuracy. Measurements made of the curvature of a pile by lowering an inclinometer down a tube fixed to the wall of a hollow pile or cast centrally in a solid pile are helpful in checking the assumptions made on the point of fixity as described in [Chapter 6](#). The Building Research Establishment have developed highly accurate microlevels only about 25mm long which can be mounted in a probe and lowered down a sleeve cast into the pile. The slope of the pile at the head can be measured by extending the pile above ground level by a stiff indicator rod. Dial gauge readings are made at the top and bottom of this rod. It is also helpful when testing piles in marine structures to make two sets of tests, applying the load at two different levels, say at the head and just above low water of spring tides. This provides two sets of curves relating deflections to bending moments.

Typical load-deflection curves for cyclic tests are shown in [Figure 11.21](#). Eurocode 7, [Section 7.8](#),



*Fig. 11.18 Lateral loading test on two steel tubular piles forming part of breasting dolphin*

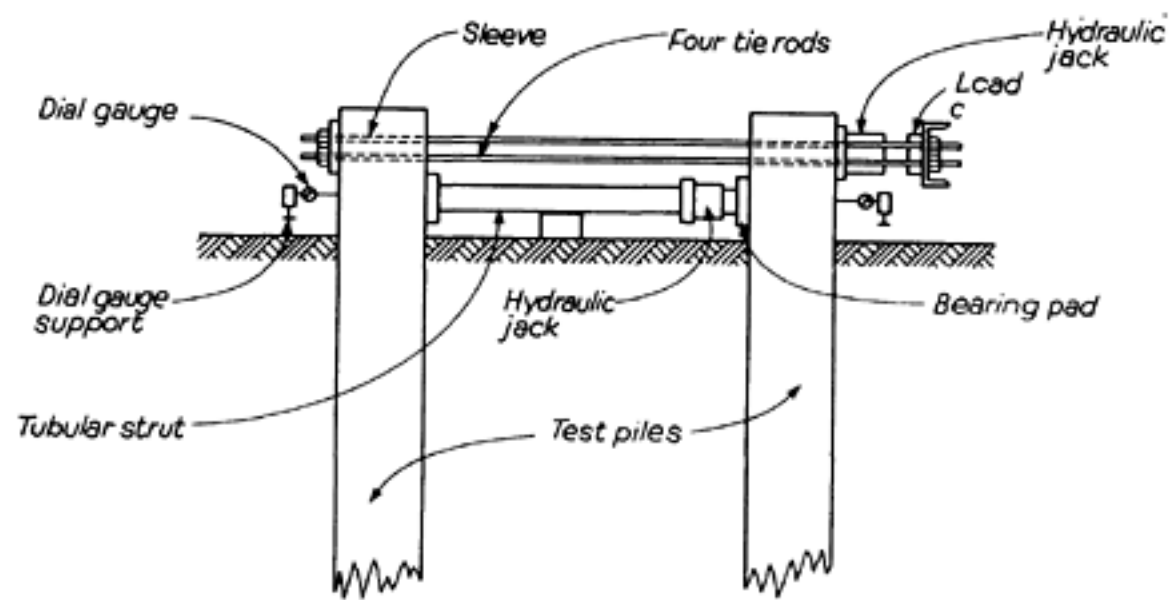


Fig. 11.19 Testing rig for push and pull lateral loading test on a pair of piles.

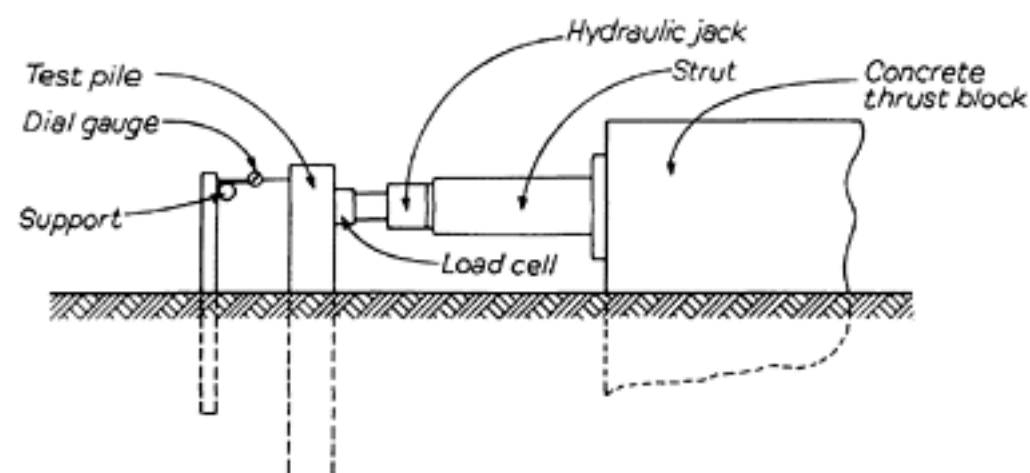


Fig. 11.20 Testing rig for lateral loading test on single pile

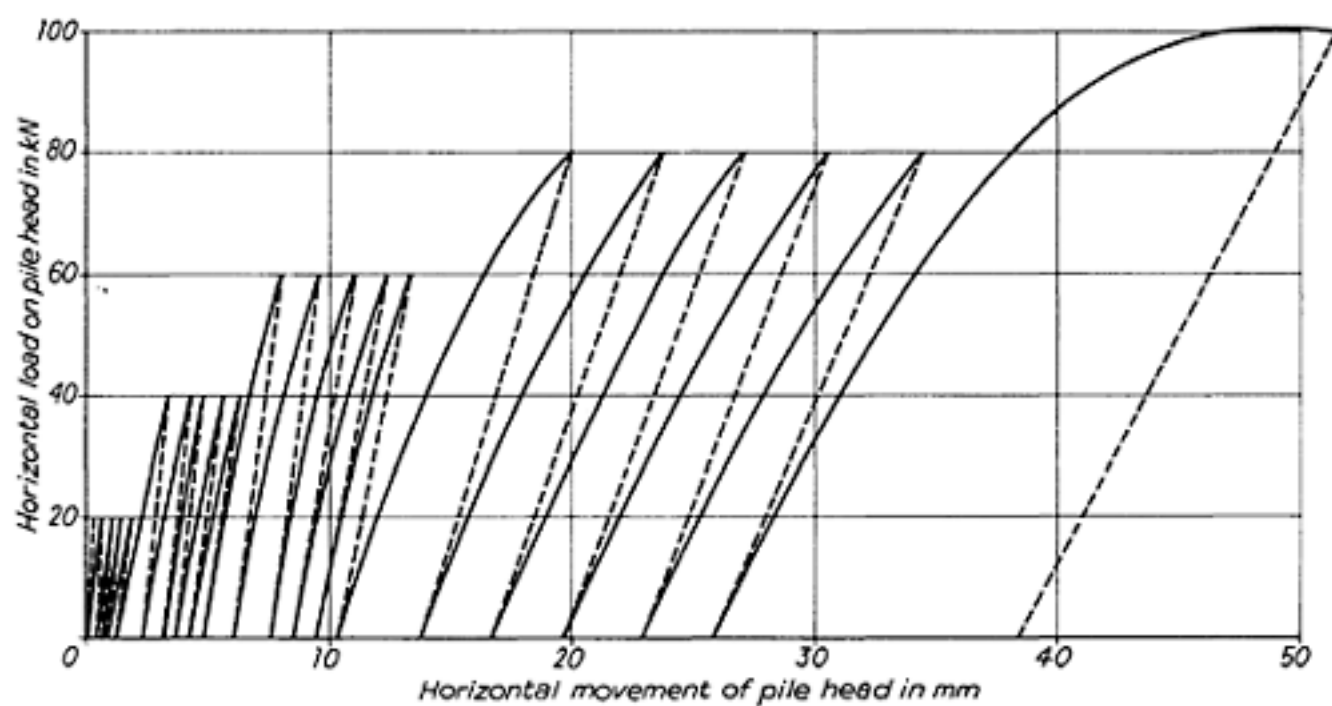


Fig. 11.21 Load-deflection curve for cyclic horizontal loading test on pile (some load cycles omitted for clarity)

does not prescribe a method of deriving the ultimate limit-state design transverse load  $F_{tr}$  as is the case for compression and uplift loading.

### 11.5 Tests for the structural integrity of piles

From time to time doubts are thrown on the soundness of pile shafts. Excavations for pile caps may show defective conditions of the type illustrated in [Figure 3.38](#) and [Figure 3.39](#), and questions are immedi-

ately asked about the likelihood of similar defects at greater depths and in other piles on the site. Where preformed piles, such as precast concrete or steel tubular sections are used, defects can readily be explored by lowering inclinometers down guides fixed to the interior (see 2.2.4) or by inserting a light or TV camera down the interior of a hollow pile. In Holland it is the practice to check the soundness of precast prestressed concrete piles by embedding a thin electric cable down the shaft. After driving, a test for electrical continuity is made quite simply by incorporating a light bulb in the circuit. If the pile shaft is broken it shows as a break in the circuit. Such explorations cannot readily be made with cast-in-place concrete piles. Weltman<sup>(11.18)</sup> has listed the following of integrity testing methods.

1. Excavation.
2. Exploratory boring and drilling.
3. Closed circuit television.
4. Acoustic.
5. Radiometric.
6. Seismic.
7. Dynamic response.
8. Integral compression test.

A complete pile can rarely be examined economically by method 1. Piles are frequently installed in soft or loose ground, making excavation difficult and costly particularly below the water table. It would be unthinkable to examine all piles on a site by excavation down the shaft.

Drilling, either by open hole methods using a percussion drill or rotary rock roller bit, or by rotary coring, can be used in piles of medium to large diameter, but it is difficult to keep the drill hole within the confines of the shaft of a small-diameter pile. If it is possible to flush an open hole clear of dirty water an inspection can be made by TV camera to look for cavities or honeycombed concrete. Heavy water losses when the drill hole is filled with water also indicate defective concrete. A cored hole provides a better indication of soundness, and compression tests can be made on the cores, but the method is more costly than open hole drilling. Caliper logging down a drill hole gives an indication of overbreak caused by weak concrete or cavities.

Sonic pulse equipment can also be lowered down drill holes, when irregularities in the trace of the sonic log indicate a defective shaft. Pairs of ducts can be formed down a pile shaft at the time of placing the concrete and various logging devices used to scan the concrete between the ducts. These include sonic pulse measurements, gamma-ray logging, and neutron emissions. The latter methods described by Weltman are believed to be reliable but are costly since they involve the use of skilled technicians and the transportation to site and operation of nuclear testing devices with their elaborate safety precautions. The seismic method consists of dropping a weight on the pile head and observing the time of return of the seismic wave reflected from the toe. This method is quite widely used and has been shown by experience to give reasonably reliable results. The method does not give reliable results in jointed precast concrete piles, however. The dynamic response method consists of mounting a vibrating unit on the pile head and interpreting the oscillograph of the response from the pile. This method is again quite widely used. Test loading is a positive method and its value in detecting defects in a shaft is illustrated in Figures 11.14e and 11.14f. However, test loading all the piles on a site is a costly operation, particularly if heavy kentledge loads are required. The method of shaft compression has been described by Moon<sup>(11.19)</sup>. A rod or cable is anchored at the base of the pile in a sleeve, and by jacking the tension member against the pile head the ability of the pile shaft to carry the design compressive load can be checked. It is possible to recover the tension members after each test. The main advantage of specifying integrity testing of all or randomly selected piles is that it encourages the contractor to keep a careful check on all the site operations. However, the methods do not replace the need for full-time supervision of the piling work by an experienced engineer or inspector.

The limitations of integrity testing were demonstrated by experiences of a field trial competition in Holland<sup>(7.5)</sup>. Ten different precast concrete pile shapes with different forms of defect were installed in drilled holes. The average score from 12 specialist firms competing in the trials was four correct identifications out of the ten shapes, but the suitability of the test conditions has been criticized.

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# APPENDIX

## Properties of materials

### 1 Cohesionless soils

	Density when drained above ground water level $\gamma$ (Mg/m <sup>3</sup> )	Density when submerged below ground water level $\gamma$ sat (Mg/m <sup>3</sup> )	Angle of shearing resistance $\phi$ (degrees)
Loose gravel with low sand content	1.6–1.9	0.9	28–30
Medium dense gravel with low sand content	1.8–2.0	.0	30–36
Dense to very dense gravel with low sand content	1.9–2.1	.1	36–45
Loose well-graded sandy gravel	1.8–2.0	.0	28–30
Medium-dense well-graded sandy gravel	1.9–2.1	.1	30–36
Dense well-graded sandy gravel	2.0–2.2	.2	36–45
Loose clayey sandy gravel	1.8–2.0	1.0	28–30
Medium-dense clayey sandy gravel	1.9–2.1	1.1	30–35
Dense to very dense clayey sandy gravel	2.1–2.2	1.2	35–40
Loose coarse to fine sand	1.7–2.0	1.0	28–30
Medium-dense coarse to fine sand	2.0–2.1	1.1	30–35
Dense to very dense coarse to fine sand	2.1–2.2	1.2	35–40
Loose fine and silty sand	1.5–1.7	0.7	28–30
Medium-dense fine and silty sand	1.7–1.9	0.9	30–35
Dense to very dense fine and silty sand	1.9–2.1	1.1	35–40

### 2 Cohesive and organic soils

	Density when drained above ground water level $\gamma$ (Mg/m <sup>3</sup> )	Density when submerged below ground water level $\gamma$ sat (Mg/m <sup>3</sup> )	Undrained shear strength (kN/m <sup>2</sup> )
Soft plastic clay	1.6 –1.9	0.6 –0.9	20–40
Firm plastic clay	1.75–2.0	0.75–1.1	40–75
Stiff plastic clay	1.8 –2.1	0.8 –1.1	75–150
Soft slightly plastic clay	1.7 –2.0	0.7 –1.0	20–40
Firm slightly plastic clay	1.8 –2.1	0.8 –1.1	40–75

Stiff slightly plastic clay	2.1 –2.2	1.1 –1.2	75–150
Stiff to very stiff clay	2.0 –2.3	1.0 –1.3	150–300
Organic clay	1.4 –1.7	0.4 –0.7	—
Peat	1.05–1.40	0.05–0.40	—

3 Rocks and other materials

Material	Density (Mg/m <sup>3</sup> )
Granite	2.50
Sandstone	2.20
Basalts and dolerites	1.75–2.25
Shale	2.15–2.30
Stiff to hard Marl	1.90–2.30
Limestone	2.70
Chalk	0.95–2.00
Broken brick	1.10–1.75
Solid brickwork	1.60–2.10
Ash and Clinker	0.65–1.00
Pulverized fuel ash	1.20–1.50
Loose coal	0.80
Compact stacked coal	1.20
Mass concrete	2.20
Reinforced concrete	2.40
Iron and steel	7.20–7.85

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# Name index

Aas, A.S., [153](#)  
 Adamchak, J. C., [312](#)  
 Adams, J.I., [191](#), [211](#), [213](#)  
 Anderson, D.F., [342](#)  
 Anderson, W.F., [401](#)  
  
 Baldi, G., [198](#)  
 Ballisager, C.C., [114](#)  
 Bannerjee, P.K., [251](#)  
 Barkan, D.D., [331](#)  
 Barthelemy, H., [122](#)  
 Baxter, J.W., [289](#)  
 Beattie, D., [284](#)  
 Beebe, K.E., [319](#)  
 Beetstra, G.W., [355](#)  
 Berezantsev, V.G., [117](#)  
 Beringen, F.L., [153](#)  
 Bermingham, P., [401](#)  
 Bessey, G.E., [365](#)  
 Bjerrum, L., [3](#), [30](#), [113](#), [152](#), [177](#), [191](#), [348](#)  
 Bolognesi, A.J.L., [86](#)  
 Bond, A.J., [105](#)  
 Brassinga, H.E., [127](#)  
 Bretschneider, C.L., [307](#)  
 Britt, G.B., [50](#)  
 Broadhead, A., [305](#)  
 Broms, B.B., [124](#), [223](#), [233](#), [349](#)  
 Bryden-Smith, D.W., [199](#)  
 Broug, N.W.A., [127](#), [352](#)  
 Browne, R.D., [368](#)  
 Bruce, D.W., [369](#)  
 Brzezinski, L.S., [192](#)  
 Buchanan, N.W., [349](#)  
 Burbidge, M.C., [180](#)  
 Burford, D., [198](#)  
 Burland, J.B., [104](#), [134](#), [168](#), [178](#), [180](#), [187](#), [199](#)  
 Bustamente, M., [153](#)  
 Butler, F.G., [135](#), [178](#), [187](#)  
  
 Campanella, R.G., [198](#)  
 Carrier, W.D., [170](#)  
 Cearn, P.J., [377](#)  
 Chandler, R.J., [154](#)  
 Chapelaar, J.C., [310](#)  
 Chellis, R.D., [362](#)  
 Chin, Fung, Kee, [353](#), [395](#)  
 Chow, Y.K., [190](#), [192](#)  
 Christian, J.T., [170](#)  
 Christoffersen, H.P., [183](#)  
 Claessen, A.J.M., [154](#)  
 Cole, K.W., [192](#)  
 Cooke, R.W., [108](#), [112](#), [196](#)  
 Cooper, D.H., [319](#)  
 Cox, W.R., [244](#)  
 Coyle, H.M., [153](#)  
 Curtis, R.J., [94](#), [95](#), [193](#)

Dailey, L.W., [307](#)  
Dalamatov, B.L., [343](#)  
Davies, T.G., [250](#)  
Davies, W.W., [275](#)  
Davis, A.G., [154](#)  
Davisson, M.T., [232](#), [241](#)  
De Beer, E., [148](#)  
de Gijt, J.G., [127](#)  
den Haan, E.J., [50](#)  
Dent, G.E., [301](#)  
Derbyshire, P.H., [45](#)  
Devata, M., [355](#)  
Di Biagio, E., [153](#)  
Dobie, M., [154](#)  
Domone,P.L.J., [368](#)  
Duncan, P.T., [135](#)  
Durgunoglu, H.T., [114](#)  
Dutt, R.N., [121](#)

Easton, B.J., [401](#)  
Eglinton, M.S., [365](#)  
Ehlers, C.J., [315](#)  
Elson, W.K., [152](#), [223](#), [255](#)  
Evans, J.H., [312](#)

Fawcett, A., [97](#)  
Fellenius, B., [151](#)  
Fleming, W.G.K., [50](#), [92](#), [112](#), [194](#), [199](#), [278](#), [392](#)  
Fox, E.N., [170](#), [175](#)  
Freeman, C.F., [219](#)

Gallagher, K.A., [257](#)  
Gallois, R.W., [198](#)  
Geddes, W.G.N., [97](#)  
George, A.B., [154](#)  
Gerwick, B.C., [27](#), [75](#), [315](#)  
Gill, H.L., [241](#)  
Gill, S.A., [141](#)  
Glanville, W.H., [275](#)  
Goble, G.G., [277](#)  
Gold, L.W., [342](#)  
Gooch, M.N., [199](#)  
Goodman, R.E., [139](#)  
Gouvenot, D., [153](#)  
Grabham, F.R., [50](#)  
Gregersen, O.S., [128](#)  
Grime, G., [275](#)  
Gruteman, M.S., [128](#)

Hagerty, A., [190](#)  
Hall, F.R., [355](#)  
Hambly, E.C., [345](#)  
Hanna, T.H., [30](#)  
Hansen, J.Brinch, [225](#)  
Hanson, W.E., [114](#)  
Hawkins, A.B., [289](#)  
Healy, P.R., [97](#), [111](#), [135](#), [142](#)  
Hedborg, C.E., [371](#)  
Heijnen, W.J., [128](#)  
Hill, L., [124](#)  
Hobbs, N.B., [141](#), [145](#)  
Hodges, W., [154](#)  
Holt, W.B., [289](#)  
Hooper, J.A., [195](#)  
Horvat, E., [152](#)  
Horvath, R.G., [145](#)  
Hsieh, T.K., [331](#)

Hughes, J.M.O., [400](#)  
Humphries, E.F., [289](#)  
Hutchinson, J.N., [219](#)

Irish, K., [331](#)  
Irwin, W.W., [342](#)  
Isemoto, N., [121](#)  
Ishihara, K., [153](#)

Jackson, W.H., [319](#)  
Janes, M., [401](#)

Jamiolkowski, M., [258](#)  
 Jardine, R.J., [105](#)  
 Jezequel, J.F., [258](#)  
 Johanessen, I.J., [152](#)  
 Johnson, J.W., [319](#)  
 Jonson, W., [355](#)  
 Jones, D.A., [113](#)  
 Joustra, K., [377](#)  
 Jorden, E.E., [154](#)  
 Journeaux, N.L., [145](#)

Kalra, J.C., [178](#)  
 Kee, R., [198](#)  
 Kinder, G., [97](#)  
 King, D., [311](#)  
 Kinoshita, S., [343](#)  
 Kishida, H., [121](#)  
 Klajnerman, D., [257](#)  
 Knill, J.L., [355](#)  
 Koop, F.B., [244](#)  
 Kraft, L.M., [113](#)  
 Kulhawy, F.H., [115](#), [139](#)

Lane, P.F., [92](#), [278](#)  
 Lea, F.M., [365](#)  
 Leach, B.A., [154](#)  
 Lee, D.J., [289](#)  
 Leung, C.F., [198](#)  
 Levesque, M., [354](#)  
 Little John, G.S., [219](#)  
 Lizzi, F., [334](#)  
 Lord, J.A., [143](#), [145](#), [187](#)  
 Lunne, T., [183](#)  
 Lyons, G.G., [113](#)

Macphie, H.L., [199](#)  
 Malkin, A.B., [355](#)  
 Mallard, D.J., [154](#)  
 Marchetti, H., [401](#)  
 Marsland, A., [401](#)  
 Martin, R., [153](#)  
 Massarch, K.R., [349](#)  
 Matlock, H., [240](#), [243](#)  
 McAvoy, R.P., [257](#)  
 McCabe, R., [353](#)  
 McKenzie, A., [377](#)  
 McKinlay, D.G., [401](#)  
 Medland, J.W., [154](#)  
 Meigh, A.C., [125](#), [182](#), [187](#)  
 Meyerhof, G.G., [111](#), [114](#), [131](#), [150](#), [169](#), [213](#), [256](#)  
 Mitchell, J.M., [97](#)  
 Mitchell, J.K., [114](#), [115](#), [153](#)  
 Moon, J., [319](#)  
 Moon, R., [400](#)  
 Moore, J.E., [153](#)  
 Moretto, O., [86](#)  
 Morley, J., [369](#)  
 Morganstern, N.R., [342](#)  
 Morison, J.R., [308](#)  
 Mundy, F.S., [289](#)  
 Murdock, L.J., [104](#), [114](#)

Newmark, N., [307](#)  
 Nordlund, R.L., [119](#)

North-Lewis, J.P., [85](#)

O'Brien, M.P., [319](#)

O'Neill, M.W., [86](#)

O'Riordan, N.J., [87](#)

Ono, T., [355](#)

Ostenfeld, C., [355](#)

Osterberg, J.O., [141](#)

Padfield, C.J., [178](#), [197](#)

Pasqualini, E., [258](#)

Peck, R.B., [114](#), [129](#), [166](#), [190](#), [376](#)

Pells, P.J.N., [139](#), [145](#), [211](#)

Penner, E., [342](#)

Pettit, J.J., [153](#)

Peuch, A.A., [211](#)

Pinches, G.M., [289](#)

Pink, S., [154](#)

Poskitt, T.J., [153](#)

Poulos, H.G., [136](#), [167](#), [178](#), [185](#), [250](#)

Powderham, A.J., [194](#)

Prakash, S., [255](#)

Prasad, G.D., [257](#)

Price, D.G., [355](#)

Radhakrishna, H.S., [211](#)

Radhakrishnan, K., [198](#)

Randolph, M.F., [107](#), [251](#)

Rausche, F., [277](#)

Reddaway, A.L., [152](#)

Rees, T.C., [153](#)

Reese, L.C., [86](#), [95](#), [112](#), [224](#), [236](#), [244](#)

Reid, G.O., [307](#)

Reid, W.M., [349](#)

Richart, F.E., [331](#)

Rigden, W.J., [108](#), [110](#)

Roberts, W.K., [50](#)

Robins, P., [154](#)

Robertson, P.K., [198](#)

Robinson, K.E., [232](#)

Rojas, E., [50](#)

Romanoff, M., [369](#)

Rosenberg, P., [145](#)

Shector, L., [199](#)

Shelbourne, H., [70](#)

Sherif, G., [179](#)

Sherrell, F.W., [154](#)

Sherwood, D.E., [97](#)

Shields, D.G., [258](#)

Shimmi, Y., [153](#)

Shimoni, J., [319](#)

Sillett, D.F., [199](#)

Simms, L.G., [289](#)

Simpson, B., [50](#)

Skaven-Haug, S.V., [343](#)

Skempton, A.W., [112](#), [169](#)

Sliwinski, Z., [112](#), [124](#)

Smith, E.A.L., [276](#)

Stanley, R.G., [353](#)

St. John, H.D., [2](#), [153](#), [211](#)

Steinbrenner, W., [170](#)

Stephen, S.C., [307](#)

Stermac, A.G., [348](#)

Stevens, R.F., [110](#)

Stroud, M.A., [175](#), [376](#)

Sturrock, K.R., [97](#)

Sullavan, R.A., [315](#)  
Sutherland, H.B., [154](#)

Taylor, P.T., [153](#)  
Teh, C.L., [190](#)  
te Kamp, W.C., [126](#)  
Terzaghi, K., [130](#), [166](#), [170](#), [223](#), [376](#)  
Thorburn, J.Q., [97](#)  
Thorburn, S., [97](#)  
Thornburn, T.H., [114](#), [376](#)  
Tominga, M., [153](#)  
Tomlinson, M.J., [153](#), [154](#), [401](#)  
Tonnisen, J.K., [50](#)  
Touma, F.T., [86](#)  
Tryde, P., [312](#)  
Tsyrovitch, N.A., [342](#)  
Turner, M.J., [113](#)  
Turner, R.M., [139](#)

Sainsbury, R.N., [311](#)  
Samuel, R.H., [98](#)  
Saito, A., [153](#)  
Schaaf, S.A., [319](#)  
Schmertmann, J.H., [182](#)  
Schultze, E., [179](#)  
Scott, I.D., [85](#)  
Selby, K.G., [355](#)  
Semple, R.M., [108](#)  
Sharrock, M.J., [178](#), [197](#)

Van den Hoonard, J., [355](#)  
Van Hooydonk, W.K., [153](#)  
Van der Noot, H.J., [199](#)  
Vesic, A.S., [115](#), [128](#), [136](#)  
Vogelaar, L.J.J., [355](#)

Wallays, M., [148](#)  
Walker, W.P., [331](#)  
Ward, W.H., [198](#)  
Warner, M., [319](#)  
Webb, D.L., [154](#)  
Welch, R.C., [247](#)  
Weltman, A.J., [97](#), [111](#), [135](#), [393](#), [400](#)  
Wex, B.P., [348](#)  
Wheeler, P., [277](#)  
Whitaker, T., [112](#), [401](#)  
White, E.E., [335](#)  
Whitman, R.V., [331](#)

Whittle, R.T., [284](#)  
Wiegel, L., [307](#)  
Williams, A.F., [145](#)  
Williams, N.S., [73](#)  
Wilson, L.C., [154](#)  
Windle, D., [153](#)  
Wong, Y.K., [198](#)  
Wood, R.H., [289](#)  
Woods, R.D., [355](#)  
Wootton, J.R., [319](#)  
Wroth, C.P., [107](#), [168](#), [400](#)  
Wyllie, D.C., [139](#), [147](#), [219](#)  
Wynne, C.P., [50](#)  
  
Yaron, S.L., [319](#)  
Yeats, J.A., [87](#)

# Subject index

Abutments, bridge, [345–9](#)  
 Acid attack, [368](#)  
 Adfreezing, [342–3](#)  
 Adhesion, *see* Skin friction  
 Adhesion factor, [104–13](#)  
 Allowable load, [102–3](#)  
 American Petroleum Institute, [121](#), [218](#), [257](#), [315](#)  
 Anchors, [214–21](#), [388](#)  
 Arbed piles, [31](#)  
 Attrition, [350](#)  
 Auger, [42](#), [79–81](#)  
 Auger-injected piles, [44](#)

Barges, [315](#)  
 Base grouting, [86–7](#)  
 Base resistance,  
     bored-and-cast-in-situ piles, [112](#), [124](#)  
     driven piles, [110](#), [120](#), [125](#)  
     driven-and-cast-in-situ piles, [110](#), [123](#)  
     in cohesionless soils, [114–29](#)  
     in cohesive soils, [103](#), [110](#), [112](#)  
     in layered soils, [131–3](#)  
     in rocks, [138–45](#)  
     observations of, [392–3](#)  
     related to static cone tests, [125–9](#)  
 Basements, [193–8](#)  
 Bauer piling system, [87](#), [345](#)  
 Belling bucket, [80–1](#)  
 Bentonite slurry,  
     concreting under, [86](#), [93](#)  
     drilling with, [85](#), [95](#)  
     effects on skin friction, [112](#), [124](#)  
 Berthing structures, *see* Marine structures  
 Bitumen coating, [152](#)  
 Block failure, [168](#)  
 Bored-and-cast-in-place piles,  
     base resistance of, [112](#), [124](#)  
     continuous flight auger, [44–5](#)  
     description of, [45–6](#)  
     durability of, [365–6](#)  
     enlarged bases for, [42](#), [80](#), [112](#), [212–14](#)  
     in cohesionless soils, [124](#)  
     in cohesive soils, [111–13](#)  
     in mining subsidence areas, [340](#)  
     in rocks, [144–8](#)  
     installation of, [79–87](#), [90–1](#), [192–3](#)  
     lateral loads on, [247](#)  
     reinforcement of, [44](#)  
     skin friction on, [111–13](#), [124](#)  
     underpinning with, [332–9](#)  
     working stresses on, [46](#)  
 Box piles, [28](#)  
 Bridge loading, [345–6](#)

Caisson pile, *see* Drilled-in tubular pile  
 Calcareous soils, [122–3](#), [142](#)  
 Camkometer, [171](#), [247–8](#), [377](#)

Cap block, [72](#)  
Caps and capping beams,  
for over-water bridges, [353–5](#)  
protection of, [365–8](#)  
structural design of, [280–9](#)  
Cased piles, [28](#)  
Cathodic protection, [372](#)  
Chalk, [141–3](#), [187](#)  
Characteristic loads, [4](#)  
Characteristic strengths, [4](#)  
Cohesionless soils,  
bored-and-cast-in-place piles in, [124](#)  
driven-and-cast-in-place piles in, [123–4](#)  
driven piles in, [119–23](#)  
loss of ground in, [192–3](#)  
pile groups in, [179–85](#)  
skin friction on piles in, [117–19](#), [124](#)  
Cohesive soils,  
bored-and-cast-in-place piles in, [111–13](#)  
driven-and-cast-in-place piles in, [110–11](#)  
driven piles in, [103–10](#)  
heave in, [190–2](#)  
pile groups in, [168–79](#)  
skin friction on piles in, [103–10](#)  
Composite piles, [47](#)  
Computer programs, [167–8](#), [250–3](#)  
Concreting piles,  
generally, [90–5](#)  
in permafrost, [344](#)  
in water-bearing ground, [93–5](#)  
under bentonite, [86](#), [93](#)  
Cone test, *see* Static and Dynamic cone  
Constant rate of penetration, [386–8](#)  
Constant rate of uplift, [396](#)  
Contiguous piles, [339](#)  
Continuous dynamic cone test, [377](#)  
Continuous flight auger, [44–5](#), [81](#)  
Contracts for piling, [380–3](#)  
Control of installation, [383–6](#)  
Corrosion of steel piles, [369–72](#)  
Current forces, [309–11](#), [350](#)  
Cyclic loading, [211](#)  
  
Deep pile groups, [185–6](#)  
Defects in pile shafts, [90–3](#), [399–400](#)  
Deflection,  
due to surcharge loading, [346–8](#)  
 $p$ - $y$  curves, [241–7](#)  
under lateral loads, [233–53](#), [398–9](#)  
Degaussing, [383](#)  
Design bearing capacity, [101](#)  
Design verification load, [386](#)  
Diaphragms, [318–19](#)  
Diesel pile hammers, [62](#)  
Differential-acting hammer, [61–2](#)  
Dilatometer, [378](#)  
Dolly, [72](#)  
Double-acting pile hammer, [61–2](#)  
Dragdown, *see* Negative skin friction  
Drilled-in tubular piles,  
description, [45–7](#)  
for marine structures, [304–5](#)  
Drilling equipment,  
for site investigations, [374–6](#)  
flight auger, [81](#)  
grabbing, [81](#)  
reverse circulation, [83](#), [315–16](#), [350](#)  
rotary power auger, [79–81](#)

tripod, [83–5](#)

Driven-and-cast-in-place piles,

- advantage, [35](#)
- base resistance of, [110](#), [123](#)
- equipment for installing, [76–8](#)
- Franki, [24](#)
- Fundex, [39](#)
- in cohesionless soils, [123–4](#)
- in cohesive soils, [110](#)
- in rocks, [143–4](#)
- installation of, [90](#)
- lateral loads on, [247](#)
- Positive, [40](#)
- Raymond, [40–1](#)
- shell types, [39–41](#)
- skin friction, [110–11](#), [123](#), [143](#)
- Vibrex, [39](#)
- Vibro, [39](#)
- Vibroform, [39](#)
- West's, [40–1](#)
- working stresses on, [42–3](#)
- Driven displacement piles,
  - in cohesionless soils, [123–4](#)
  - in cohesive soils, [103–10](#)
  - in rocks, [138–43](#)
  - skin friction on, [103–10](#), [117–19](#)
- Driving cap, [72](#)
- Driving tests, [275–7](#), [383–5](#)
- Driving stresses, [275–7](#)
- Drop hammers, [57](#)
- Dutch cone tests, *see* Static cone tests
- Dynamic cone test, [377](#)
  - see also* Continuous dynamic cone
- Dynamic pile formulae, [3–4](#), [276](#)
- Eccentric loading on piles, [232](#), [253–7](#), [277–8](#)
- Effective stress (pressure), [103](#), [150](#)
- Efficiency formulae, [166](#)
  - of piling hammers, [66–7](#)
- Elastic continuum, [250–3](#)
- Elastic modulus,
  - see* Deformation modulus
- End bearing resistance,
  - see* Base resistance
- Enlarged bases,
  - for bored-and-cast-in-place piles, [42](#), [81](#), [112](#), [212–14](#)
  - for driven-and-cast-in-place piles, [37](#)
  - for marine structures, [317](#)
  - for precast concrete piles, [15](#)
- Equivalent raft, [168](#)
- Esch-Belval piles, [28](#)
- Eurocode, [101–3](#), [108](#), [112–14](#), [212](#), [213](#), [383](#), [388](#), [393](#), [396](#), [398](#)
- Europile, [23–4](#)
- Failure load, [101](#)
- Fender piles, [353](#)
- Filled ground,
  - pile groups in, [189–90](#)
  - single piles in, [148–51](#)
- Fondedile piles, [334](#)
- Frames, for piling, [51–2](#)
- Franki piles, [24](#), [37](#), [336](#)
- Friction, skin, *see* Skin friction
- Frozen ground, piling in, [342–4](#)
- Fundex piles, [39](#)
- Gamma ray logging, [400](#)

Geophysical surveys, [374](#)  
Glacial till, piles in, [111](#)  
Grabbing rigs, [81](#), [95](#)  
Ground beams, [280–1](#), [284–8](#)  
Ground heave, [190–3](#), [382](#)  
Ground water,  
    concreting under, [93–5](#)  
    observations of, [357](#), [374](#), [379](#)  
Groups of piles,  
    block failure of, [168](#)  
    eccentric loading on, [253–7](#)  
    heave effects, [190–3](#)  
    in filled ground, [189–90](#)  
    in rock, [186–9](#)  
    in soils, [166–86](#)  
    lateral loads on, [253–7](#)  
    negative skin friction on, [189–90](#)  
    optimization of settlements in, [196–8](#)  
    settlement of, [170–86](#)  
    ultimate resistance of, [168–70](#)  
Grouting, skin, [122–3](#)  
Guides, for piling, [53–7](#)  
  
H-section piles,  
    advantages of, [28](#)  
    curvature of, [30](#)  
    for machinery foundations, [331–2](#)  
    for underpinning, [337](#)  
    Peine, [31](#)  
    skin friction on, [110](#), [120](#)  
    strengthening toes of, [32–3](#)  
    wings for, [29–31](#)  
Hammers, piling, [57–72](#)  
Heave, ground, [190–3](#), [352](#)  
Helmets, [72–4](#)  
Hercules piles, [23](#)  
High-alumina cement, [367](#)  
Hoesch piles, [28](#)  
Horizontal load, *see* Lateral loads  
Hush piling, [72](#)  
Hydraulic hammers, [60–1](#)  
  
Ice forces, [311–12](#), [342–4](#)  
Inclined loading, [232](#)  
Impact forces, [301–4](#), [350–1](#)  
Integrity, structural, [399–400](#)  
  
Jacket structures, [313–15](#)  
Jacking piles, [336–7](#)  
Jetting piles, [74–5](#)  
Jetties, *see* Marine structures  
  
Krupp piles, [28](#)  
  
Laboratory testing, [379–80](#)  
Larssen piles, [28](#)  
Lateral loads,  
    from bridge abutments, [345–6](#)  
    on pile groups, [253–7](#)  
    on raking piles, [253](#)  
    on single piles, [221–53](#)  
Lateral load tests, [398–9](#)  
Leaders, [52–4](#)  
Limit states, [4](#)  
Load-settlement curves, [393–6](#)  
Load transfer to piles, [100–1](#)  
Load-uplift curves, [396–7](#)  
Loading tests,

- compression, [386–95](#)
- constant rate of penetration, [386–8](#)
- constant rate of uplift, [396](#)
- lateral, [398–9](#)
- maintained load, [386–8](#), [396](#)
- uplift, [396](#)

Loss of ground due to piling, [192](#), [382](#)

M.V. piles, [28](#)

Machinery foundations, [303–7](#)

Made ground, *see* Filled ground

Maintained load test, [386–8](#), [396](#)

Marine structures,

- current forces on, [309–11](#)
- durability of piles in, [361–3](#), [368–9](#), [370–2](#)
- ice forces on, [311–12](#)
- impact forces on, [301–4](#)
- wave forces on, [306–9](#)
- wind forces on, [311](#)

Mass factor, [186](#)

Menard pressuremeter, [247–50](#), [377](#)

Micropiles, [48](#)

Miga piles, [214](#), [336](#)

Mining subsidence, [339–42](#)

Minipiles, [48](#)

Modulus,

- deformation, of cohesionless soils, [182–4](#)
- deformation, of rocks, [147](#), [186–9](#)
- of subgrade reaction, [223](#)
- pressuremeter, [247–8](#)

Monotube piles, [31–2](#)

Mooring dolphins, [306](#)

Mooring forces, [305–6](#)

Negative skin friction, [148–52](#), [189–90](#), [347–8](#)

Noise control, [67–72](#), [382](#)

Open-endpiles, resistance of, [109–10](#), [121–2](#)

Organic acids, [368](#)

Oscillation of piles, [310–11](#)  
 Oscillators, casing, [81](#)  
 Oslo point, [17](#), [33](#)  
  
 PMI piles, [24](#)  
 P-y curves, [241–7](#)  
 Packing, [72–4](#)  
 Painting, [371](#)  
 Pali Radice, [334](#)  
 Partial safety factors, [4](#), [101](#), [108](#), [112](#), [134](#), [212](#)  
 Peine piles, [31](#)  
 Permafrost, [343–4](#)  
 Permeability tests, [379–80](#)  
 Plate bearing tests, [134](#), [136](#), [378–9](#)  
 Platforms, petroleum production,  
     *see* Marine structures  
 Plugging,  
     with concrete, [110](#)  
     with soil, [109–10](#), [121–2](#)  
 Portland cement concrete, [365–6](#)  
 Positional tolerance, [96–7](#), [278](#), [382](#)  
 Positive piles, [40](#)  
 Power augers, [79–81](#)  
 Precast concrete piles,  
     breakage of, [21–3](#)  
     concreting of, [17–21](#)  
     cracks in, [15](#)  
     driving of, [88–9](#)  
     driving stresses on, [275–7](#)  
     Hardrive, [23](#)  
     Hercules, [23](#)  
     jointed types, [23–4](#)  
     lengthening of, [278–9](#)  
     lifting of, [272–5](#)  
     prestressed, [15–16](#)  
     reinforcement of, [15](#), [273–4](#)  
     shoes for, [17–19](#)  
     structural design of, [272–9](#)  
     working stresses on, [19](#)  
 Pressure distribution, beneath pile groups, [165–8](#), [175](#)  
 Pressuremeter, [171](#), [247–8](#), [377–8](#)  
 Prestressed concrete piles,  
     advantages of, [16](#)  
     casting of, [17–21](#)  
     cylindrical, [21](#)  
     for marine structures, [312](#)  
     structural design of, [274](#)  
 Pretest piles, [335–6](#)  
 Pruyn point, [33](#)  
 Pyrites, [286](#)  
  
 Raking piles,  
     bending of, [253](#)  
     calculating loading on, [255–6](#)  
     for bridges, [345–6](#)  
     for marine structures, [300](#), [306](#)  
     installation of, [95–6](#)  
     resistance to lateral loads of, [209–10](#), [255–6](#)  
 Raymond piles, [40](#)  
 Reinforcement,  
     for bored-and-cast-in-place piles, [44–5](#)  
     for driven-and-cast-in-place piles, [37–8](#)  
     for precast concrete piles, [15](#), [273–4](#)  
 Replacement piles, *see* Bored and cast-in-place  
 Responsibilities, contractual, [5–6](#)

Reversed circulation drilling, [83](#), [315–16](#), [350](#)

Rock,

anchoring piles to, [214–21](#)

deformation moduli of, [147](#), [186–9](#)

drilled piles in, [47](#)

base resistance in, [138–45](#)

quality designation, [145](#)

settlement of piles in, [147–8](#), [186–9](#)

site investigations in, [375–80](#)

skin friction on piles in, [142–5](#)

Rockwell connector, [26](#)

Rotary auger, [79–81](#)

Rotary table, [83](#)

Safety factors, [4](#), [101](#), [108](#), [112](#), [133–4](#), [152](#), [212](#)

Sampling soils, [375](#), [379](#)

Sand, *see* Cohesionless soils

Scour, effects of, [126–8](#), [350–2](#)

Secant piles, [339](#)

Seismic logging, [400](#)

Selection of pile type, [7–9](#), [48–9](#)

Serviceability limitstate, [4](#)

Set of piles, [275–7](#), [383–5](#)

Setting out, [382](#)

Settlement,

of pile groups, [170–90](#)

of single piles, [133–8](#), [147–8](#)

Shear connectors, [217–18](#)

Sheathing, [152](#), [363](#), [368](#)

Shell piles,

description of, [39–42](#)

installation of, [90](#)

Positive, [40](#)

Raymond, [40](#)

West's, [40–1](#)

working stresses on, [43](#)

Shoes,

for concrete piles, [17–19](#)

for steel piles, [32–3](#)

for timber piles, [12](#)

Single-acting hammers, [60](#)

Site investigations for piling, [373–80](#)

Skin friction,

effects of overburden on, [106](#)

in cohesionless soils, [117–19](#), [124](#)

in cohesive soils, [103–11](#)

in layered soils, [133–4](#)

in rock, [142–5](#)

negative, [148–52](#)

observations of, [392–3](#)

on bored-and-cast-in-place piles, [113–24](#)

on driven piles, [103–10](#), [117–19](#)

on driven-and-cast-in-place piles, [110–11](#), [123](#), [143](#)

on grouted piles, [122–3](#)

Small displacement piles, [8](#), [110](#), [120–2](#)

Sonic logging, [400](#)

Spacing of piles, [169–70](#)

Specifications, piling, [9](#), [92–3](#), [95](#), [195](#), [380–3](#), [386](#)

Spiral welding, [27](#)

Splicing piles, [278–9](#), [315](#)

Standard penetration tests,

related to deformation modulus, [179](#)

related to  $\phi$ -values, [114](#)

techniques for, [376–7](#)

Static cone penetration test,

related to  $\phi$ -values, [114](#)

related to base resistance, [124–9](#)

related to compressibility of soils, [182–5](#)

- related to skin friction, [125](#)
- techniques for, [377](#)
- Statnamic tests, [393](#)
- Steel,
  - for anchors, [216](#)
  - for piles, [33–5](#)
  - reinforcing, *see* Reinforcement
- Steel piles,
  - advantages of, [49](#)
  - Arbed, [31](#)
  - box, [29](#)
  - cathodic protection of, [372](#)
  - concrete filling of, [24](#)
  - connectors for, [26](#)
  - corrosion of, [369–72](#)
  - driving of, [89–90](#)
  - Esch-Belval, [28](#)
  - for marine structures, [312](#), [370–2](#)
  - H-section, [28–31](#)
  - Hoesch, [28](#)
  - Krupp, [28](#)
  - Larssen, [28](#)
  - M.V., [28](#)
  - Monotube, [31–2](#)
  - painting of, [371](#)
  - Peine, [31](#)
  - shoes for, [32–3](#)
  - splicing of, [278–9](#), [315](#)
  - tubular, [25–8](#)
  - Union, [28](#)
  - welding of, [27](#), [279](#), [315](#), [383](#)
  - working stresses on, [33–5](#)
- Step tapered, [40](#)
- Stress grades, timber, [11](#)
- Stress wave theory, [275](#)
- Sulphate (and sulphuric acid) attack, [365–7](#)
- Sulphate resisting cement, [366](#)
- Supersulphated cement, [367](#)
- Surcharge loading on pile groups, [346–8](#)

Tapered piles,  
     description of, [31–2](#), [40](#)  
 Temporary compression, [385](#)  
 Timber piles,  
     damage to, [11–12](#)  
     driving of, [11](#), [87](#)  
     durability of, [357–63](#)  
     for fenders, [304](#)  
     preservation of, [360–1](#)  
     shoes for, [12](#)  
     splicing of, [12](#), [279](#)  
     working stresses on, [12](#)  
 Time effects,  
     in clays, [113–14](#)  
     in cohesionless soils, [129](#)  
 Tolerances, positional, [96–7](#), [278](#), [382](#)  
 Tremie concreting, [93–4](#)  
 Trestle guides, [54–7](#)  
 Tripod rigs, [83–5](#), [95](#), [332–3](#)  
 Tubex piles, [39](#)  
 Tubular piles, steel,  
     cleaning out, [89–90](#)  
     descriptions of, [25–8](#)  
     driving caps for, [74](#)  
     for marine structure, [304–5](#), [315–19](#), [370–2](#)  
     plugging of, [109–10](#), [121–2](#)  
  
 Ultimate resistance,  
     of piles in groups, [168](#)  
     to compressive loads, [100–2](#)  
     to lateral loads, [211–57](#)  
     to uplift loads, [210–21](#)  
 Underpinning with piles, [332–9](#)  
 Under-reamed piles, [42](#), [81](#), [112](#), [212–14](#), [317](#)  
 Underwater concreting, [93–5](#)  
 Underwater hammers, [61](#)  
 Union piles, [28](#)  
 Unloading effects, [127](#), [352](#)  
 Uplift on piles,  
     due to frost action, [209](#), [342–3](#)  
     due to swelling clays, [208–9](#), [286–7](#)  
     resistance to, [210–21](#)  
  
 Vane tests, [376](#)  
 Vibratory pile hammers, [63–4](#)  
 Vibrex piles, [39](#)  
 Vibro piles, [39](#)  
 Vibroform piles, [39](#)  
 Vibro-replacement, [64](#)  
  
 Wave forces, [306–9](#), [350](#)  
 Welding, [27](#), [279](#), [315](#), [383](#)  
 West's piles, [25](#), [40–1](#)  
 Winches, piling, [54](#)  
 Wind forces, [311](#)  
 Winged piles, [29–31](#)  
 Working load, specified, [386](#)  
 Working stresses,  
     on bored-and-cast-in-place piles, [46](#)  
     on driven-and-cast-in-place piles, [42–3](#)  
     on shell piles, [43](#)  
     on steel piles, [33–5](#)  
     on timber piles, [12](#)