Pile Design and Construction Practice

Sixth Edition

Michael Tomlinson and John Woodward

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Preface to the sixth edition

Two factors are driving the development of modern pile design and construction–the growth in demand for high-rise buildings and the subsequent requirement for ever-larger piles, frequently in areas with poor subsoils. New piling techniques and powerful piling rigs have effectively addressed the problems of producing piles to cope with the larger structural loads, and significant improvements have taken place in understanding the behaviour of piles. However, despite the advances in analytical and numerical methods using sophisticated computer software which allow theoretical soil mechanics solutions to be applied to aspects of pile design, much reliance still has to be placed on empirical correlations. The late Michael Tomlinson was an empiricist committed to the scientific method with extensive practical knowledge, and these principles and applications are still the backbone of practical pile design.

A guiding precept in this edition was therefore to keep to the spirit of MJT's work, retaining a substantial amount of his writings on the technicalities of pile design, particularly the demonstration of the basic principles using his hand calculation methods and the reviews of the extensive case studies. However, there are new codified design procedures which have to be addressed. For example, the formal adoption in Europe of the Eurocodes for structural design (and 'load and resistance factor design' more generally elsewhere) has led to new ways of assessing design parameters and safety factors. One of the main objectives in this edition has been to give an overview of the current Eurocode requirements combined with the practicalities of applying the new suite of British Standards which relate to construction materials and installation procedures. However, compliance with the more systemised Eurocode rules has not necessitated any significant changes to the well-established procedures for determining ultimate geotechnical values for routine pile design. For more complex structures, such as offshore structures and monopiles, the new design methods for driven piles in clays and sands, developed from the extensive laboratory research and field testing by Imperial College for example, represent an important practical advance in producing economical foundations.

The author wishes to thank David Beadman and Matina Sougle of Byrne Looby Partners for a review of the reworked examples, Chris Raison of Raison Foster Associates for comments on current Eurocode 7 pile design; Paul Cresswell of Abbey Pynford for his contribution on micropiles; Colin O'Donnell for comments on contractual matters; and Tony Bracegirdle, David Hight, Hugh St John, Philip Smith and Marina Sideri of Geotechnical Consulting Group for their reviews, contributions and inputs on many of the topics. Any remaining errors are the authors.

Many specialist piling companies 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. Thanks are due to the following for the supply of and permission to use photographs and illustrations from technical publications and brochures.

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The cover photograph shows two vertical travel box leads, 60 m long, as supplied by Bermingham Foundation Solutions company to Gulf Intracoastal Constructors, being erected to drive the 48 m long by 760 mm diameter steel piles for the pumping station at Belle Chasse, Louisiana. Pile driving was by the B32 diesel hammer (see Table 3.4) for vertical and 3:1 batter piles. With permission of Bermingham Foundation Solutions of Hamilton, Ontario.

Figure 4.42 is after Figure 4.47 on page 136 of 'Piling Engineering' 3rd edition 2009, by Fleming, Weltman, Randolph and Elson, published by Taylor & Francis, with permission.

Figure 9.25 is published with the permission of the Deep Foundations Institute as originally published in the DFI 2005 Marine Foundations Speciality Seminar proceedings. Copies of the full proceedings are available through Deep Foundations Institute, Hawthorne, NJ; Tel: 973-423-4030; E-mail: dfihq@dfi.org.

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Princes Risborough, United Kingdom

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Preface to the 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 worldwide 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, FIStructE, for his assistance with the calculations and his help in checking the text and worked examples.

Michael J. Tomlinson
Burton-on-Stather, United Kingdom
1977

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 rocks. 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 HISTORY

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 bridgeworks 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 were 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-place 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 resistance 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 1.1a). Thus, the safety factor against general shear failure of the spread foundation and its settlement under the design *applied load* (also referred to as the *working load*) can be predicted from knowledge of the physical characteristics of the 'undisturbed' soil with a degree of certainty which depends only on the complexity of the soil stratification.

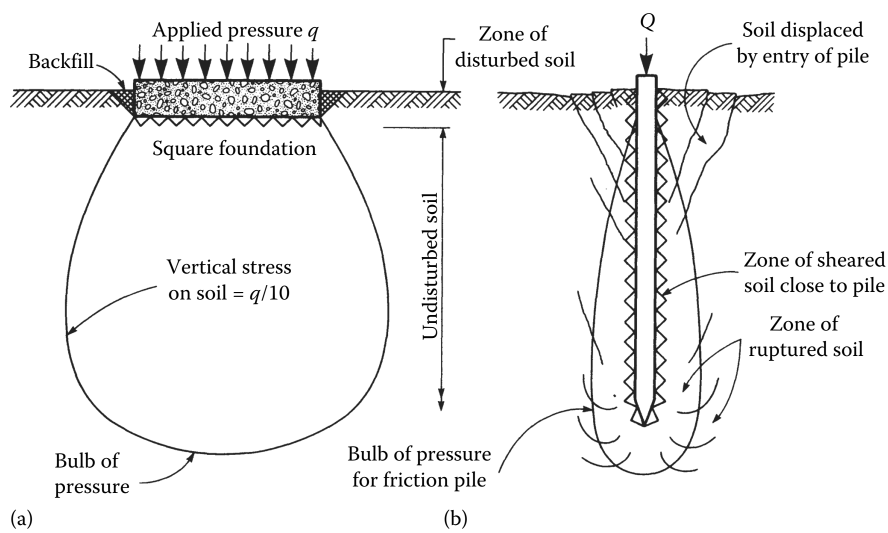

Figure 1.1 Comparison of pressure distribution and soil disturbance beneath spread and piled foundations: (a) spread foundation; (b) single pile.

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, jetting, vibration, jacking, screwing or drilling, the soil in contact with the pile face, from which the pile derives its support by shaft 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-gearing 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 shaft 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–soil movement, and to chemical or electrochemical 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.

Considerable research has been, and is being, carried out into the application of soil and rock mechanics theory to practical pile design. However, the effects of the various methods of pile installation on the carrying capacity and deformation characteristics of the pile and ground cannot be allowed for in a strict theoretical approach. The application of simple empirical factors to the strength, density and compressibility properties of the undisturbed soil or rock remains the general design procedure to determine the relevant resistances to the applied loads. The various factors which can be used depend on the particular method of installation and have been developed over many years of experience and successful field testing.

The basis of the *soil mechanics approach* to calculating the carrying capacity of piles is that the total resistance of the pile to compression loads is the sum of two components, namely, shaft friction and base resistance. A pile in which the shaft-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). The need for adopting adequate safety factors in conjunction with calculations to determine the design resistance of these components is emphasised by the statement by Randolph[1.1] 'that we may never be able to estimate axial pile capacity in many soil types more accurately than about ±30%'. 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

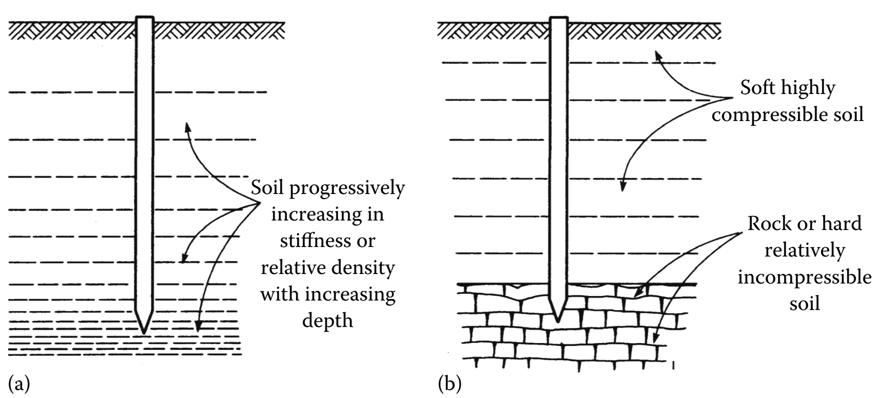

Figure 1.2 Types of bearing pile: (a) friction pile; (b) end-bearing pile.

problem to calculate that a precast concrete pile must be driven to a depth of, say, 20 m to carry safely a certain applied 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 rig must be correctly predicted.

Time effects are important in calculating the resistance of a pile in clay; the effects include the rate of applying load to a pile and the time interval between installing and testing a pile. The shaft-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. Because of the importance of such time effects both in fine- and coarse-grained 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 method of calculating the load-carrying capacity of piles mentioned earlier is based on a soil mechanics approach to determine the resistance of the ground to static loads applied at the test-loading stage or during the working life of the structure. Historically, all piles were driven with a simple falling ram or drop hammer and the pile capacity was based on the measurement of the ground resistance encountered when driving a pile. The downward movement of the pile under a given energy blow is related to its ultimate resistance to static loading. Based on the considerable body of experience built up in the field, simple empirical formulae were derived, 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 drawbacks to the use of these formulae when using diesel hammers due to the increase in energy delivered as the ground resistance increases and changes in hammer performance related to the mechanical condition and operating temperature. Driving tests on preliminary piles instrumented to measure the energy transferred to the pile head together with a pile driving analyser (PDA) can provide a means of applying dynamic formula for site control of working piles.

The more consistent hydraulic hammers overcome many of the problems of energy transfer and the availability of a large database of hammer performance and improvements in the application of PDAs has meant that under the right conditions, dynamic formulae can be reliable (see Section 7.3). Hence, the Eurocode for geotechnical design (EC7-1 Clause 7.6.2.5; see Section 1.5) allows the use of pile driving formulae to assess the ultimate compressive resistance of piles where the ground conditions are known. Also, the formula has to have been validated by previous experience of acceptable performance in similar ground conditions as verified by static loading tests on the same type of pile.

While the dynamic formula approach may now be more reliable, it can only be applied to driven piles and is being replaced by the use of pile driveability and stress wave principles. The basic soil mechanics design approach, and the associated development of analytical and numerical methods, can be applied to all forms of piling in all ground conditions.

1.5 INTRODUCTION OF EUROCODES AND OTHER STANDARDS

The Eurocodes[1.2], formulated by the transnational technical committees of the European Committee for Standardisation (CEN), are the Europe-wide means of designing works to produce identical, harmonised specifications for safe buildings, structures and civil engineering works. The United Kingdom, which adopted the European Public Procurement Directive of 2004 (2004/17/EC) through the Public Contracts Regulations of 2006, must ensure that all public projects in England, Wales and Northern Ireland are specified in terms of Eurocodes. Although there is no current legal requirement for structural design for private sector works to comply with Eurocodes, this is likely to change in the future under European trade directives.

The Eurocodes make a fundamental change to traditional UK design practice. They are not based on allowable stress and allowable capacity of materials calculated using overall (*global*) factors of safety, but on limit state design principles and partial factors applied to separate elements of the design, depending on the reliability which can be placed on the parameters or calculations. There are 10 structural Eurocodes made up of 58 parts which supersede the previous UK design standards, largely withdrawn by the British Standards Institute (BSI) in 2010. The main Codes of Practice, BS 8002 and BS 8004 dealing with foundation design and construction, are therefore no longer available. The concrete design standard, BS 8110 which was based on limit state principles, has also been withdrawn.

The BSI adopts and publishes, on behalf of CEN, the following *normative* standards for geotechnical design (with the prefix BS EN and the commonly used abbreviations):

EC7-1 BS EN 1997-1:2004 Eurocode 7: Geotechnical design, Part 1 General rules
EC7-2 BS EN 1997-2:2007 Eurocode 7: Geotechnical design, Part 2 Ground investigation and testing

EC7, which deals with the variable nature of soils and rock, differs in some respects from other structural codes where materials are more consistent in strength and performance. EC7 has to be read in conjunction with the following structural Eurocodes referenced in this text which bear on foundation design:

EC1-1 BS EN 1991-1-1:2002 Eurocode 1: Part 1-1 Actions on structures. General actions – Densities, self-weight, imposed loads for buildings
EC2-1 BS EN 1992-1-1:2004 Eurocode 2: Design of concrete structures, Part 1-1 General rules and rules for buildings
EC3-1 BS EN 1993-1-1:2005 Eurocode 3: Design of steel structures, Part 1-1 General rules and rules for buildings
EC3-5 BS EN 1993-5:2007 Eurocode 3: Design of steel structures, Part 5 Piling
EC4-1 BS EN 1994-1:2005 Eurocode 4: Design of composite steel and concrete structures, Part 1 General rules
EC5-1 BS EN 1995-1-1:2004 Eurocode 5: Design of timber structures, Part 1-1 General rules
EC6-1 BS EN 1996-1:2005 Eurocode 6: Design of masonry structures, Part 1 General rules
EC8-1 BS EN 1998-1:2004 Eurocode 8: Design of structures for earthquake resistance, Part 1 General rules
EC8-5 BS EN 1998-1:2004 Eurocode 8: Design of structures for earthquake resistance, Part 5 Foundations, retaining walls and geotechnical aspects

The objectives of the suite of Eurocodes are set out in BS EN 1990:2002, Basis of structural design, namely, to demonstrate structural resistance, durability and serviceability for the structure's designed working life. The clauses designated *principles* (*P*) in all Eurocodes are mandatory (i.e. *shall* clauses); the *informative* clauses indicate the means by which the principles may be fulfilled.

Each part of the Eurocode has to be read in conjunction with its corresponding *National Annex* (an *informative* document referred to here as the NA) which provides, within prescribed Eurocode limits, nationally determined parameters, partial factors and design approach to meet a country's particular conditions and practices for the control of its design process. The NA factors, published separately from the Eurocodes, are to be distinguished from those in *Annex A* (*normative*) in the Eurocode. The NA also sets out the procedures to be used where alternatives to the Eurocode are deemed necessary or desirable. Not all countries have produced NAs, but the UK Annexes for both parts of EC7 (and most of the other Eurocodes) are now applicable and importantly modify the parameters and factors published in Annex A. Designers therefore must be aware of the many variations to EC7 which exist in Europe when designing piles in one country for execution in another. Designers will be free to apply higher standards than given in the Eurocodes if considered appropriate and may use unique design factors provided they can be shown to meet the prime objectives of the Eurocodes. Such alternatives will have to be supported by relevant testing and experience.

Eurocodes introduce terms not familiar to many UK designers, for example *load* becomes *action* and *imposed load* becomes *variable action*. *Effect* is an internal force which results from application of an action, for example settlement. These and other new load conditions, *permanent unfavourable* and *permanent favourable*, require the application of different load factors depending on which of the *design approaches* and factor *combinations* are being used. The structural engineer is required to assess which actions give the critical effects and special care is needed when deciding on which actions are to be considered as separate variable actions; actions include temperature effects and swelling and shrinkage.

The United Kingdom has modified the EC7 partial factors in its NA to reflect established practice and has adopted Design Approach 1 (DA1) for foundations using partial factor combinations 1 and 2 in which the factors are applied at source to actions and ground strength parameters, requiring reliable and technically advanced soils testing laboratories. However, for *pile design*, the partial factors must be applied to the ground *resistance* calculations. This is inconsistent with the rest of EC7.

Clause 7 of EC7-1 deals with piled foundations from the aspects of actions 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 Clause 7, the designer is required to demonstrate that the sum of the ultimate limit state (ULS) components of bearing capacity of the pile or pile group (ground *resistances R*) exceeds the ultimate limit state design loading (*actions F*) and that the serviceability limit state (SLS) is not reached. New definitions of *characteristic* values (cautious estimate based on engineering judgement) and *representative* values (tending towards the limit of the credible values) of material strengths and actions are now given in BS EN 1990 and BS EN 1991 which must be considered when examining the various limit states (see Section 4.1.4). The use of cautious estimates for parameters can be important in view of the limitations imposed by the partial factors for resistance, especially for values of undrained shear strength at the base of piles. The representative actions provided by the structural engineer to the foundation designer should state what factors have been included so that duplication of factors is avoided.

EC7-1 does not make specific recommendations on calculations for pile design; rather, emphasis is placed on preliminary load testing to govern the design. Essentially, EC7-1 prescribes the succession of stages in the design process using conventional methods to calculate end-bearing resistance, frictional resistance and displacement and may be seen as the means for checking (*verifying*) that a design is satisfactory. This edition exclusively applies DA1 and the UK NA, and the reader who needs to consider DA 2 and 3 is referred to examples in Bond and Harris[1.3] which show the differences in design outcomes using the specified parameters from EC7-1. CIRIA Report C641 (Driscoll et al.[1.4]) highlights the important features of the Eurocodes applicable to geotechnical design using DA1 and the NA factors. The guide by Frank et al.[1.5] outlines the development of the code and gives a clause-by-clause commentary. The limit state and partial factor approach in EC7 should result in more economic pile foundations – particularly in the case of steel piles where the material properties are well defined.

The current EC7 procedures are not very amenable to the application of sophisticated computational developments in theoretical analyses, which in due course may produce further savings. In order to capitalise on these advances, two factors will have to be addressed: firstly, significant improvements in determining in situ soil parameters are required and, secondly, designers must have gained specialist expertise and competence to undertake the necessary modelling and be aware of the limitations. In any event, it is considered that a good understanding of the proven empirical geotechnical approach will be essential for future economic pile design, with continued validation by observations and publication of relevant case studies.

EC7 is to undergo a significant evolution over the next few years which should avoid the anomalies and difficulties in interpreting some of the current procedures; a new version will be published sometime after 2020.

New European standards (EN) have also been published dealing with the 'execution of special geotechnical works' (bored piling, displacement piles, sheet piles, micropiles, etc.) which have the status of current British Standards (and also designated BS EN). These, together with new material standards, are more prescriptive than the withdrawn codes and are extensively cross-referenced in this text. Selection of the design and installation methods used and the choice of material parameters remain within the judgement and responsibility of the designer and depend on the structure and the problems to be solved. Generally, where reference is made in Eurocodes to other BS, the requirements of the corresponding BS EN should take precedence. However, parts of existing standards, for example amended BS 5930: 1999 and BS 1377: 1997, are referred to in EC7-2 in respect of ground investigation and laboratory testing.

Where there is a need for guidance on a subject not covered by a Eurocode or in order to introduce new technology not in the ENs, BSI is producing 'noncontradictory' documents entitled 'Published Documents' with the prefix PD. Examples are PD 6694 which is complementary to EC7-1 for bridge design and PD 6698 which gives recommendations for design of structures for earthquake resistance; all come with the rider that 'This publication is not to be regarded as a British Standard'.

Geotechnical standards are also prepared by the International Standards Organisation (ISO) in cooperation with CEN. When an ISO standard is adopted by BSI as a European *norm*, it is given the prefix BS EN ISO. It is currently dealing with the classification of soil and rock and ground investigations generally and, when completed, the new set of ISO documents will supersede all parts of BS 5930 and BS 1377.

The UK Building Regulations 2010[1.6] set out the statutory requirements for design and construction to ensure public health and safety for all types of building; the complementary 'Approved Documents' give guidance on complying with the regulations. Approved Document A now refers exclusively to British Standards based on Eurocodes.

As noted earlier, some aspects of withdrawn standards are still referred to in the new BS ENs but designers should be aware of the risks of inappropriately mixing designs based on the new standards with withdrawn BS codes[1.7]. Designers should also be aware that compliance with a BS or BS EN does not confer immunity from the relevant statutory and legal requirements and that compliance with Eurocodes may be mandatory.

Working to code rules is only part of the design process. An understanding of the soil mechanics and mathematics behind the codes is essential, and designs and procedures should always be checked against comparable experience and practice. It is also important to avoid over-specification of design and construction as a result of applying new structural Eurocodes and the associated execution codes[1.8].

Alternative forms of limit state design, usually referred to as *load and resistance factor design* (LRFD), are being adopted and codified in many jurisdictions (see Section 4.10). Here, the factored load should not exceed the factored resistance, whereas the EC7-1 principle is that factored load should not exceed the resistance as determined by factored shear strength parameters (but note the previous comment for pile design).

A list of current and pending British Standards relating to geotechnical design is given in Appendix B.

1.6 RESPONSIBILITIES OF EMPLOYER AND CONTRACTOR

Contract conditions and procurement methods for construction in Britain for both main contracts and specialist work have changed significantly in recent years to meet new legal obligations and to implement the Eurocodes. These changes, which are considered in more detail in Section 11.2.1, have altered the relative responsibilities of the parties to a contract and the delegation of responsibilities to the parties' advisors and designers. Under the traditional piling contract arrangements, the employer's engineer is responsible for the overall design and supervision of construction. In this case, the engineer is not a party to the contract between the employer and contractor and must act impartially when carrying out duties as stated in the contract. With regard to the foundations, the engineer will have prepared, possibly with a geotechnical advisor[1.9], the mandatory Geotechnical Design Report and determined the geotechnical categories as required in EC7-1 and EC7-2 (see Section 11.1). The responsibility for the detailed design of the piles may then lie with the engineer or the piling contractor.

The New Engineering and Construction Contract (NEC3)[1.10], which is increasingly being used on major projects, does not provide for the employer to delegate authority to an engineer. A project manager is appointed under a contract with the employer to employ designers and contractors and to supervise the whole works, in accordance with the employer's requirements and instructions. The piles may be designed by the project manager's team or by the contractor.

The engineer/project manager has a duty to the employer to check the specialist contractor's designs, as far as practically possible, before approval can be given for inclusion in the permanent works. This will include determining that proper provision has been made by the piling specialist to cope with any difficult ground conditions noted in the ground investigation, such as obstructions or groundwater flow. Checks will also be made on pile dimensions, stresses in the pile shaft, concrete strengths, steel grades, etc. in accordance with specifications, relevant standards and best practice. However, the risks and liabilities of the piling contractor for his designs will not normally be reduced by prior approval. If the employer through the project manager provides the design, the risk for a fault in the design will generally fall to the employer.

The basic methods of undertaking the works either by employer-provided design or contractor design are outlined in Section 11.2.1. In all cases, the piling contractor is responsible for ensuring that reasonable skill and care has been and will be exercised in undertaking the piling works, usually confirmed in a form of warranty from the specialist.

The Eurocodes do not comment specifically on responsibility for checks, but require that *execution* is carried out by 'personnel having the appropriate skill and experience'; also that 'adequate supervision and quality control is provided during execution of the work, i.e. in design offices…and on site'. Here, 'execution' must be taken to mean both the design and construction of the piles. 'Adequate supervision' is not defined, but under the auspices of the Ground Forum of the Institution of Civil Engineers, a *Register of Ground Engineering Professionals*[1.9] has been developed to meet the European requirement to identify suitably qualified and competent personnel to address the issue.

The liability for dealing with unforeseen ground conditions should be explicitly addressed in the contract conditions. Similarly, the party liable for providing any additional piles or extra lengths compared with the contract quantities should be identified. If the piling contractor had no opportunity to contribute to the ground investigation, it would be reasonable for the contract to include rates for extra work and for payment to be authorised. Payment would not be appropriate if the piling contractor is shown to have been overcautious, but a decision should not be made without test pile observations or previous knowledge of the performance of piles in similar soil conditions. Contractor-designed piling has promoted the development of highly efficient and reliable piling systems, which means a contractor is less able to claim for extra payments.

Whichever form of contract is used, it is the structural designer's responsibility to state the limit for settlement of the foundation at the applied loads based on the tolerance of the structure to total and differential settlement (the serviceability). He must specify the maximum permissible settlement at the representative load and at some multiple in a pile load test, say, 1.5 times, as this is the only means that the engineer/project manager has of checking that the design assumptions and the piles as installed will fulfil their function in supporting the structure. 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 settlement should not be so large that the limit states are compromised (Section 4.1.4). It is unrealistic to specify the maximum movement of a pile under lateral loading, since this can be determined only by field trials.

The piling contractor's warranty is usually limited to that of the load/settlement characteristics of a single pile and for soundness of workmanship, but responsibilities regarding effects due to installation could extend to the complete structure and to any nearby existing buildings or services; for example, liability for damage 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. The position may be different if a building were to suffer damage due to the settlement of a group of piles as a result of consolidation of a layer of weak compressible soil beneath the zone of disturbance caused by pile driving (Figure 1.3). In the case of an employer-designed project, the designer should have considered this risk in the investigations and overall design and specified a minimum pile length to take account of such compressible layer. The rights of third parties in respect of damage due to construction are now covered by statute (see Section 11.2.1).

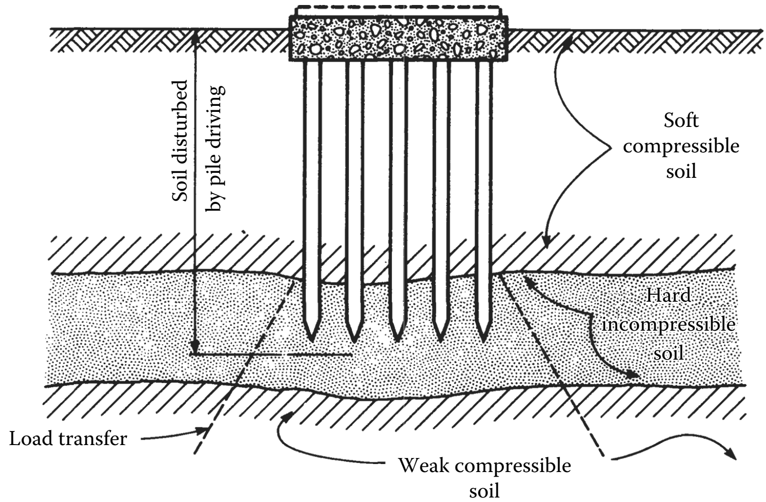

Figure 1.3 Pile group terminating in hard incompressible soil layer underlain by weak compressible soil.

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Chapter 2

Types of pile

2.1 CLASSIFICATION OF PILES

The traditional classification of the three basic categories of bearing piles is as follows:

1. *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. Large-diameter screw piles and rotary displacement auger piles are increasingly used for piling in contaminated land and soft soils.
2. *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.
3. *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. Continuous flight auger (CFA) piles have become the dominant type of pile in the United Kingdom for structures on land.

Eurocode 7 Part 1(1.2) (EC7-1, all Eurocodes are referenced in Section 1.5 and Appendix B) does not categorise piles, but Clause 7 applies to the design of all types of load-bearing piles. When piles are used to reduce settlement of a raft or spread foundation (e.g. Love(2.1)), as opposed to supporting the full load from a structure, then the provisions of EC7 may not apply directly.

Examples of the types of piles in each of the basic categories are as follows:

2.1.1 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

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

1. Steel tube driven and withdrawn after placing concrete
2. Steel tube driven with closed end, left in place and filled with reinforced concrete
3. Precast concrete shell filled with concrete
4. Thin-walled steel shell driven by withdrawable mandrel and then filled with concrete
5. Rotary displacement auger and screw piles
6. Expander body

2.1.3 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)
6. Steel sheet piles used as combined retaining wall and vertical load bearing

2.1.4 Replacement piles

1. Concrete placed in hole drilled by rotary auger, baling, grabbing, airlift or reverse-circulation methods (bored and cast-in-place or in American terminology *drilled shafts*)
2. Tubes placed in hole drilled as earlier 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

2.1.5 Composite piles

Numerous types of piles of composite construction may be formed by combining units in each of the preceding categories or by adopting combinations of piles in more than one category. For example, 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. Tubular steel casing with a spun concrete core combines the advantages of both materials, and fibreglass tubes with concrete or steel tube cores are useful for light marine structures.

2.1.6 Minipiles and micropiles

Both replacement piles and small-displacement piles may be formed as mini-/micropiles.

2.1.7 Selection of pile type

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

1. The location and type of structure
2. The ground conditions
3. Durability

Considering the first of these factors, 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 permanent and 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 applied loads. Augered piles are also suitable where it is desired to avoid ground heave, noise and vibration, that is, 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.

For the design of foundations in *seismic situations*, reference can be made to criteria in EC8-5 which complement the information on soil–structure interaction given in EC7-1. However, the codes and the recommendations in the British Standard Institute document PD 6698:2009 give only limited data on the design of piles to resist earthquakes. The paper by Raison[2.2] refers to the checks required under EC8-1 rules for piles susceptible to seismic liquefaction at a site in Barrow (see Section 9.8).

The second factor, *ground conditions*, influences both the material forming the pile and the method of installation. Firm to stiff fine-grained soils (silts and clays) 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 fine-grained 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.

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, a driven pile would be suitable. For hard driving conditions, for example in glacial till (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.

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 mobilising resistance to uplift or lateral loads.

When piling in *contaminated land* using boring techniques, the disposal of arisings to licensed tips and measures to avoid the release of damaging aerosols are factors limiting the type of pile which can be considered and can add significantly to the costs. Precautions may also be needed to avoid creating preferential flow paths while piling which could allow contaminated groundwater and leachates to be transported downwards into a lower aquifer. Tubular steel piles can be expensive for piling in contaminated ground when compared with other displacement piles, but they are useful in overcoming obstructions which could cause problems when driving precast concrete or boring displacement piles. Large-displacement piles are unlikely to form transfer conduits for contaminants, although untreated wooden piles may allow ‘wicking’ of volatile organics. Driving precast concrete piles will densify the surrounding soil to a degree and in permeable soil the soil-pile contact will be improved, reducing the potential for flow paths. End-bearing H-piles can form long-term flow conduits into aquifers (particularly when a driving shoe is needed), and it may be necessary for the piles to be hydraulically isolated from the contaminated zone.

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 groundwater 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 groundwaters. 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. Checklists for durability of man-made materials in the ground are provided in EC2-1 and complementary concrete standards BS 8500 and BS EN 206; durability of steel is covered in EC3-1 and EC3-5.

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 seawater or to disturbed soil must be protected against corrosion by cathodic means if a long life is required. Corrosion rates are provided in Clause 4.4 of EC3-5, and work by Corus Construction and Industrial[2.3,2.4] has refined guidelines for corrosion allowances for steel embedded in contaminated soil. The increased incidence of *accelerated low water corrosion* (ALWC) in steel piles in UK tidal waters is considered in Section 10.4. *Mariner grade* steel H-piles to ASTM standard can give performance improvement of two to three times that of conventional steels in marine splash zones.

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), workmanship and contract documentation are given in the Specification for Piling and Embedded Retaining Walls published by Institution of Civil Engineers[2.5] (referred to as SPERW). This document is generally consistent with the requirements in EC7-1 and the associated standards for the ‘Execution of special geotechnical works’, namely,

- BS EN 1536:2010 Bored piles
- BS EN 12063:1999 Sheet piling
- BS EN 12699:2001 Displacement piles
- BS EN 14199:2005 Micropiles

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 load carried. Consideration must also be given to the overall cost of the foundation work which will include the main contractor’s on-site costs and overheads.

Depending on the contract terms, extra payment may 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 and diameter on which to base a tender. Experience is also an important factor in determining whether the cost of preliminary test piling can be omitted and testing limited to that of proof loading selected working piles. In well-defined ground conditions and relatively light structural loads, the client may rely on the contractor's warranty that the working piles meet the specified load-carrying capacity and settlement criteria. However, the potential to save costs by omitting preliminary pile tests will be limited by EC7-1 Clause 7.6.2, which requires that pile designs based on calculation using *ground test results* (i.e. the measurement of soil properties) or on dynamic impact tests must have been validated by previous evidence of acceptable performance in static load tests, in similar ground conditions.

A thorough ground investigation and preliminary pile tests are essential in difficult ground. If these are omitted and the chosen pile design and installation procedures are shown to be impractical at the start of construction, then considerable time and money can be expended in changing to another piling system or adopting larger-diameter or longer piles. The allocation of costs resulting from such disruption is likely to be contentious.

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 will influence the choice of a particular piling system.

As a result of the introduction of new and revised codes and standards, considerable cross-referencing is now necessary to produce compliant designs. While it is not possible to deal with all the implications, this chapter provides a summary of some of the main points from the standards concerned with piling.

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 piling is also a low-cost, sustainable resource and may become more widely used as an alternative 'environmentally friendly' material when compared with steel and concrete[2.6]. To demonstrate that timber products come from managed and sustainable forests, recognised forest management certification should be provided to the user together with *chain of custody* statement. Timber piles used in their most economical form consist of round untrimmed logs which are driven butt uppermost. The traditional British practice of squaring the timber can be detrimental to its durability since it removes the outer sapwood which is absorptive to liquid preservative as BS 8417 (see Section 10.2). The less absorptive heartwood is thus exposed, and instead of a pile being encased by a thick layer of well-impregnated sapwood, 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 groundwater level, are resistant to fungal decay and have an almost indefinite life. However, the portion above groundwater level in a structure on land is liable to decay, and BS EN 12699 prohibits the use of timber piles above free-water level, unless adequate protection is used. The solution is to cut off timber piles just below the lowest predicted groundwater level and to extend them above this level in concrete (Figure 2.1a). If the groundwater 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 seawater 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

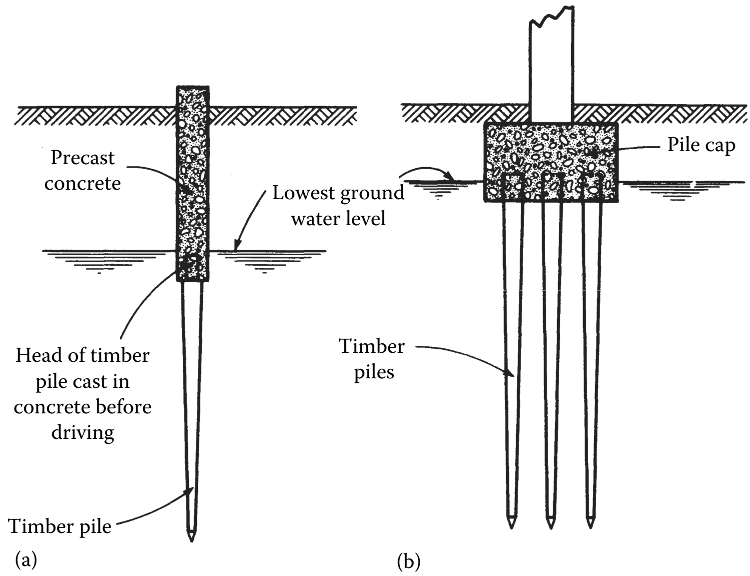

Figure 2.1 Protecting timber piles from decay by (a) precast concrete upper section above water level and (b) by extending pile cap below water level.

with preservative or greatly minimised 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 shaft 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 untreated timber (hardwoods are not treated because they will not absorb liquid preservatives).

BS 5268-2, which provided the allowable design stresses for compression parallel to the grain for the species and grade of green timber being used, has been withdrawn. The replacement Eurocode EC5-1 provides common rules for calculating stresses which apply to the design of timber piling. Reference must also be made to BS EN 338 for characteristic values for all timber classes as described under common and botanical names in BS EN 1912. The design load and design compressive stress parallel to the grain are then calculated using the EC5 National Annex partial factors for timber for verification against failure. (See McKenzie and Zhang[2.7].)

Examples of commercially available timbers which are suitable for piling are shown in Table 2.1. The values given for hardwoods, such as greenheart, are considerably higher than those of softwoods, and generally, timber suitable for piles is obtained from SS grades or better. The timber should be straight-grained and free from defects which could impair its strength and durability. To this end, the sectional dimensions of hewn timber piles must not change by more than 15 mm/m, and straightness shall not deviate more than 1% of the length.

The stresses quoted are for timber at a moisture content consistent with a temperature of 20°C and relative humidity of 65%. Timber piles are usually in a wet environment requiring the application of reduction factors (k_{mod}, see Section 7.10) to convert the code stress properties to the wet conditions. When calculating the stresses on a pile, allowance must be made for

Table 2.1 Summary of characteristic values of some softwoods and tropical hardwoods suitable for bearing piles (selected from BS EN 1912 Table 1 and BS EN 338 Table 1)

Standard name	*Strength class*	*Grade*	*Bending parallel to grain ($f_{m\,k}$) (N/mm²)*	*Compression parallel to grain ($f_{c,0\,k}$) (N/mm²)*	*Shear parallel to grain ($f_{v\,k}$) (N/mm²)*	*5% modulus of elasticity ($E_{0.5}$) (kN/m²)*
British spruce	GS	C14	14	16	3	4.7
European redwood	GS	C16	16	17	3.2	5.4
Canadian western red cedar	SS	C18	18	18	3.4	6.0
British pine	SS	C22	22	20	3.8	6.7
Douglas fir–larch, United States	SS	C24	24	21	4	7.4
Jarrah	HS	D40	40	26	4	10.9
Teak	HS	D40	40	26	4	10.9
Ekki	HS	D70	70	34	5	16.8
Greenheart	HS	D70	70	34	5	16.8

GS is visually graded *general structural* softwood to BS 4978:2007; HS is visually graded hardwood to BS 5756:2007; SS is visually graded *special structural* softwood to BS 4978:2007.

The UK gradings apply for timber used in the United Kingdom and abroad.

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 the taper on a round log.

Typical pile lengths are from 5 to 18 m carrying applied loads from 5 to 350 kN. The maximum capacity of the pile will be limited by the set achievable without causing damage. Large numbers of timber piles, mainly Norwegian spruce, are driven below the water table in the Netherlands every year for light structures, housing, roads and embankments.

As a result of improved ability to predict and control driving stresses, BS EN 12699 allows the maximum compressive stress generated during driving to be increased to 0.8 times the characteristic compressive strength measured parallel to the grain. While some increase in stress (up to 10%) may be permitted during driving if stress monitoring is carried out, it is advisable to limit the maximum load which can be carried by a pile of any diameter to reduce the need for excessively hard driving. 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 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. It is now common practice to use a pile driving analyser (PDA) which can measure the stress in the pile during driving to warn if damage is likely to occur.

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 mobilise the required 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 applied 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. Cases have occurred where the measured 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 minimised 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 to 1.5 m. This necessitates the use of a heavy hammer (but preferably less than 4 tonnes), 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 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 (Figure 2.2a and b). 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 18 m. If longer piles are required, they may be spliced as shown in Figure 2.3. A splice near the centre of the length

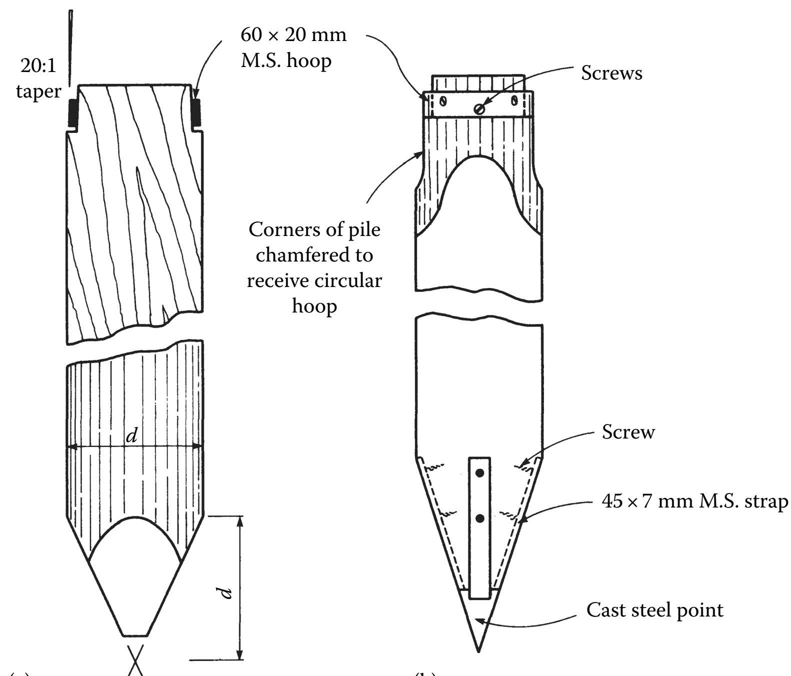

Figure 2.2 Protecting timber piles from splitting during driving. (a) Protecting head by mild steel hoop. (b) Protecting toe by cast-steel point.

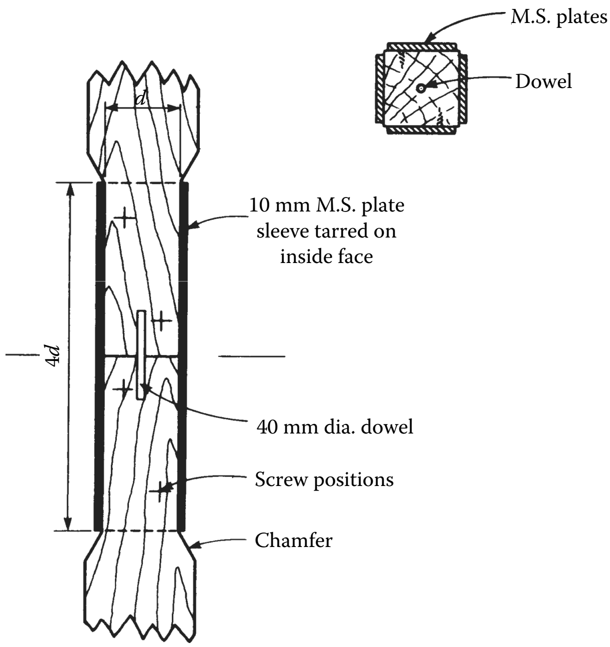

Figure 2.3 Splice in squared timber pile.

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.

2.2.2 Precast concrete piles

Precast concrete piles have their principal use in marine and river structures, that is in situations where the use of driven and cast-in-place piles is impracticable or uneconomical. For land structures, unjointed precast concrete piles can be more costly than driven and cast-in-place 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.
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, especially where high-quality concrete is required. 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

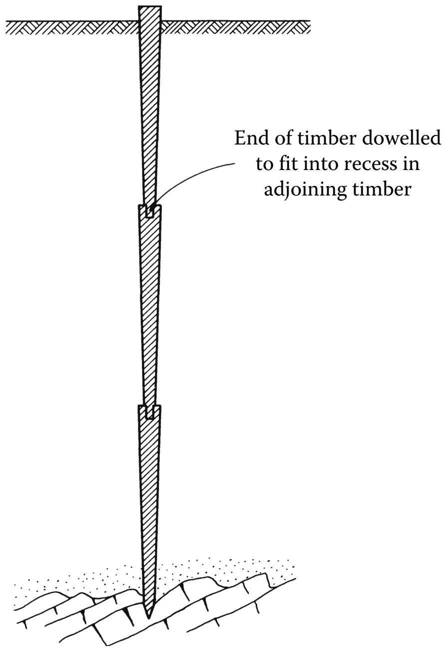

Figure 2.4 Splicing timber piles in multiple lengths.

stratum, savings in the overall quantity of concrete compared with cast-in-place piles can be achieved since higher 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 essential. The problem of varying the length of the pile can be overcome by adopting a jointed type as Section 2.2.3.

Piles can be designed and manufactured in ordinary reinforced concrete or in the form of pretensioned 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, but prestressed piles may be required for hard driving conditions. 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 can be manufactured with a hollow core in hexagonal, octagonal or circular sections. The interiors of these piles can be filled with concrete after driving 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. Hollow-core piles can be readily inspected for breakages in difficult driving and can be strengthened by infilling with structural reinforced concrete when considered for reuse. Where piles are designed to carry the applied loads mainly in end bearing, for example 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, up to 1.6 times the shaft width with a minimum length of 500 mm or equal to the width of the enlargement.

Precast and prestressed piles have to be designed not only to withstand the loads from the structure but also to meet the stresses and other serviceability requirements during handling, pitching and driving and in service as stated in the relevant material Eurocodes and the associated National Annexes. To avoid excessive flexibility while handling and driving, the usual maximum unjointed lengths of square section piles and the range of load-bearing

Table 2.2 Typical capacity and maximum lengths for ordinary precast concrete piles of square section (subject to reinforcement)

Pile size (mm²)	*Applied load (kN)*	*Maximum length (m)*
250	200–300	12
300	300–450	15
350	350–600	18
400	450–750	21
450	500–900	25

capacities applicable to each size are shown in Table 2.2. (See also Figure 7.2 for maximum lengths at various lifting points.)

EC2-1 provides common rules for concrete for building and civil engineering which are not very different from the withdrawn BS 8110 in terms of general design approach, but the replacement codes contain significant cross-references which now have to be considered for concrete design. Concrete performance, quality and production are subject to BS EN 206-1, which must be read in conjunction with the United Kingdom's complementary rules for strength and exposure classes, cover, etc. in BS 8500-1 and BS 8500-2 as designated in Table 2.3. The minimum concrete class for precast and prestressed piles specified in BS EN 12794 clause 4.2.2.1 is C35/45 and can be deemed suitable for hard driving conditions. (Note the strength classification in EC2 is based on denoting the minimum characteristic strength of a *cylinder* at 28 days/minimum characteristic *cube* strength at 28 days in N/mm², i.e. $f_{ck\ cyl}$ and $f_{ck\ cube}$ represented, e.g. as C35/45.) BS 8500 recommends strength classes of concrete C45/55 in tidal splash zones as in Table 2.4. The strengths in BS EN 13369 dealing in general with precast concrete products are not appropriate for most piling applications, but the reinforcement requirements have to be adhered to (as below).

Table 2.3 Summary of exposure classes as BS 8500-1

Exposure class	*Class description*	*Examples applicable to piling*
XO	No risk of corrosion or attack	Reinforced concrete exposed to very dry conditions
XC	Carbonation-induced corrosion	Reinforced concrete buried in soil Class AC-1
XD	Chloride-induced corrosion (not from seawater)	Reinforced concrete immersed in chloride conditions
XS	Chloride-induced corrosion (from seawater)	Reinforced concrete below mid tide level
XF	Freeze–thaw attack	Concrete subjected to frequent splashing with water and exposed to freezing

Note: Each class is subdivided depending on the severity of attack as shown in Table 2.4.

Table 2.4 Typical concrete grades and cover suitable for exposures

Strength class	*Exposure class*	*Water/cement ratio*	*Cement content (kg/m³)*	*Nominal cover (mm)*
25/30	XC2 (non-aggressive)	0.65	260	25–50 + Δ_c
35/45	XS1 (airborne salt)	0.45	360	35 + Δ_c
45/55	XS3 (intertidal wet/dry)	0.35	380	45 + Δ_c

BS EN 12794 Table 3 gives detailed production tolerances and defines two classes of precast piles – *Class 1* with distributed reinforcement or prestressed piles and *Class 2* with a single central reinforcing bar. Foundations in naturally aggressive ground conditions/brownfield sites/contaminated land are not covered in EC7-1, and the recommendations in BRE Special Digest 1[2.8] (SD1) and BS 8500-1 should be followed for both in situ foundation concrete and precast units.

High stresses, which may exceed the handling stresses, can occur during driving, and it is necessary to consider the serviceability limit of cracking. EC2-1 Clause 7.3 allows for maximum crack widths of 0.3 mm in reinforced concrete elements taking account of the proposed function of the structure and exposure of precast and prestressed elements. It has been UK practice to require cracks to be controlled to maximum widths close to the main reinforcement ranging from 0.3 mm down to 0.15 mm in an aggressive environment, important when considering laterally loaded and tension piles. Annex ZA to BS EN 12794 deals with the CE marking of foundation piles and the presumption of fitness for the intended use. (All timber, precast and steel piles will have to be so marked for use on European construction sites from 2013.)

In EC2-1 Clause 4.4, nominal cover to reinforcement is defined as $c_{nom} = c_{min} + \Delta c_{dev}$ where c_{min} is dependent on bond requirements or environmental conditions as detailed in Tables 4.1 through 4.5 of EC2. Δc_{dev} allows for deviations, set at 10 mm in EC2 NA, but may be reduced where strict QA/QC procedures are in force. Cover required in BS EN 12794 is c_{min} but the value of Δc to satisfy the environmental conditions defined in BS 8500-1 and BS EN 206-1 is shown in Table 2.4 for two classes of concrete specified for precast piles with an intended life of 50 years and 20 mm maximum aggregate. UK practice would indicate that for well-controlled production, Δc should be 5 mm generally and 10 mm in marine exposures.

Although the XC2 classification in BS 8500 for reinforced concrete in non-aggressive ground allows a minimum strength of C25/30, this is not appropriate for piles as noted earlier. The durability of concrete in aggressive ground is considered in Section 10.3.1.

Concrete made with ordinary Portland cement (CEM 1) is generally suitable for precast piles at the above-mentioned strengths in normal exposures. Table 1 of BS EN 197-1 gives the composition of the main types of cement which address all the exposure classes, and the groups in Table A1 of BS 8500-2 show the comparisons with the SD1 *ACEC* exposure grades. For example, cement to address Class XS3 given earlier is limited to types CEM 1, IIA (with fly ash), IIBS (with ground granulated blast furnace slag), and SRPC. Note the codes no longer refer to pfa (*pulverised fuel ash*) and 'flyash' may be other ash from power stations, not necessarily pfa.

BS EN 12794 (Annex B9) states that for Class 1 piles, longitudinal reinforcement shall be a minimum diameter of 8 mm with at least one bar placed in the corner of square piles; circular section piles shall have at least 6 bars 8 mm diameter placed evenly around the periphery. Transverse reinforcement must be at least 4 mm diameter depending on the pile diameter, and the pile head must have a minimum of 9 links in 500 mm. Percentages of transverse steel are specified for hollow-core piles. BS EN 12794 refers to BS EN 13369 for the quality of reinforcement and prestressing steel to be used, which in turn refers to other standards, such as BS EN 10080 steel for reinforcement of concrete and BS 5896 for prestressing wire and strand. The specification and grades of steel given in BS 4449 steel for the reinforcement of concrete, as revised in 2009, complement BS EN 10080. EC2-1-1 in Annex C states that the code applies only to reinforcement with characteristic yield strength (f_{yk}) in the range 400–600 N/mm^2. Other steels, including plain bars, may be used provided they conform to Annex C requirements. Ribbed bars in 500 N/mm^2 steel, classified as A, B or C depending on the steel ductility and the ratio of f_{tk}/f_{yk}, are the most common grade used in the United Kingdom. Users of reinforcement are referred to data sheets provided by UK CARES, the third-party certifying body for reinforcing steels, for additional clarification.

The diameter of main reinforcing steel in the form of longitudinal bars may have to be increased depending on 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 (see Table 7.2). Design data for various lifting conditions are dealt with in Section 7.2. In some cases, the size of the externally applied lateral or uplift loads may necessitate the provision of more main steel than is required by lifting considerations. In hard driving conditions, it is advantageous to place additional transverse steel in the form of a helix at the head of the pile to prevent shattering or splitting. The helix should be about two pile widths in length with a pitch equal to the spacing of the link steel at the head. A design for a precast concrete pile for use in 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 in Figure 2.5b. The design of a typical prestressed concrete pile in accordance with UK practice is shown in Figure 2.6. Square and octagonal piles are usually fabricated up to 600 mm wide.

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

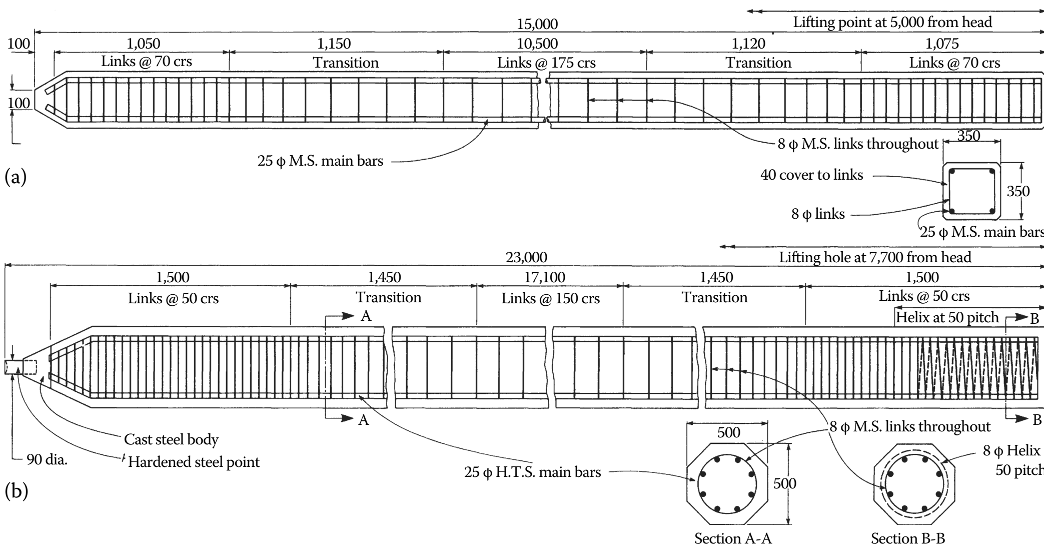

Figure 2.5 Design for precast concrete piles (a) 350 mm square pile, 15 m long (b) 500 mm octagonal pile, 23 m long.

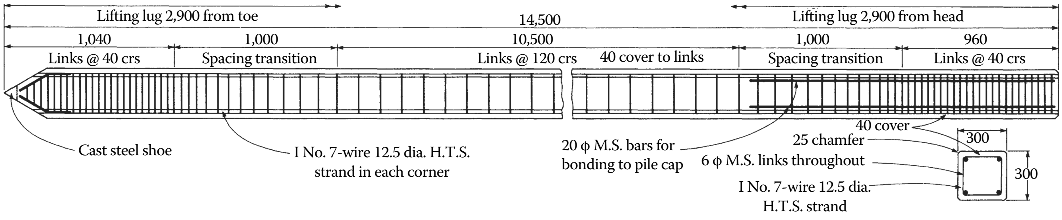

Figure 2.6 Design for prestressed concrete pile.

slender units to be lifted and driven. However, slenderness is not always advantageous since a large cross-sectional area may be needed to mobilise sufficient resistance in shaft friction and end bearing and additional lifting points required for pitching. 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. Prestressed concrete piles of hollow cylindrical section are manufactured by centrifugal spinning in diameters ranging from 900 to 2100 mm and lengths up to 40 m. For optimum driving performance, the prestressing force, after losses, is usually between 7 and 10 N/mm^2.

Prestressed concrete piles should be made with designed concrete mixes of at least Class C35/45, but as noted earlier, account should be taken of the special exposure conditions quoted in BS 8500 and BS EN 206-1. Minimum percentages of prestressing steel stipulated in BS EN 12794 are 0.1% of cross-sectional area in mm^2 for piles not exceeding 10 m in length, 0.01% cross-sectional area × pile length for piles between 10 and 20 m long, and 0.2% for piles greater than 20 m long. The high concrete strength required for prestressed piles means that they can withstand hard driving and achieve high bearing capacity. However, it may be desirable to 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 overdriven to achieve an arbitrary set given by a dynamic pile-driving formula. BS EN 12699 limits the calculated stress (including any prestress) during driving of precast piles to 0.8 times the characteristic concrete strength in compression at time of driving; a 10% increase is permitted if the stresses are monitored during driving (e.g. with a PDA).

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

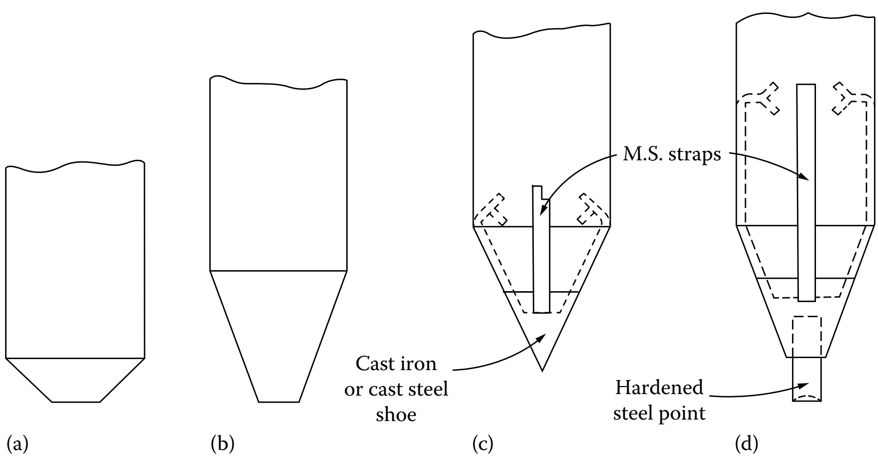

Figure 2.7 Shoes for precast (including prestressed) concrete piles. (a) For driving through soft or loose soils to shallow penetration into dense granular or firm to stiff clays. (b) Pointed end suitable for moderately deep penetration into medium-dense to dense sands firm to stiff clays. (c) Cast-iron or cast-steel shoe for seating pile into weak rock or breaking through cemented soil layer. (d) *Oslo* point for seating pile into weak rock.

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 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 with a Brinell hardness of 400–600 was employed. The 89 mm point was machined concave to 12.7 mm 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.

The strict requirements imposed by BS EN 12699 and BS EN 12794 mean that precast and prestressed piles are now usually made in factory conditions using precision steel moulds on firm reinforced concrete beds. Distortion in timber forms and when tier casting (Figures 2.8 and 2.9) and the difficulty in squaring the drive end can then be eliminated. Moulds can be stripped as soon as crushing tests on cylinders/cubes (cured using the same methods as for the pile) indicate that the piles have reached 60% of the required 28-day strength. For example, Aarsleff Piling produced 600 mm square precast piles up to 14.3 m long for the Channel Tunnel Rail Link (CTRL) using purpose-built steel moulds in their factory in Newark. The sides of the moulds were locked together using a combination of cams and hydraulic rams which, after the concrete had reached an initial set of 24–28 N/mm^2 in 21 h, were operated to release the 12.5 tonne pile. A typical steel mould is shown in Figure 2.10.

There are situations when it is appropriate to set up pile production on a construction site, for example where established factories are remote from the site, where the number of piles justifies the costs of setting up a casting yard, or where there are transportation restrictions. In Bangkok, 17,000 × 500 mm diameter prestressed, precast hollow cylindrical piles, 10–14 m long with 100 mm thick wall, were required for the depot of the new Mass Rail Transit system[2.9]. A casting yard was established adjacent to the site to fabricate the pile elements, using centrifugal spinning and 24 h autoclave curing followed by a period of ambient wet curing to give minimum strength of 50 N/mm^2. At peak production, 19 rigs were on-site driving 95 piles per day. Another type of prestressed pile was used for the Oosterschelde

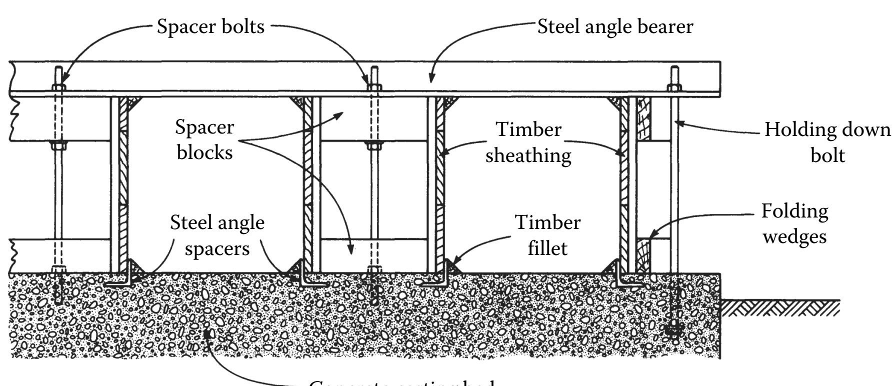

Figure 2.8 Timber formwork for precast concrete piles.

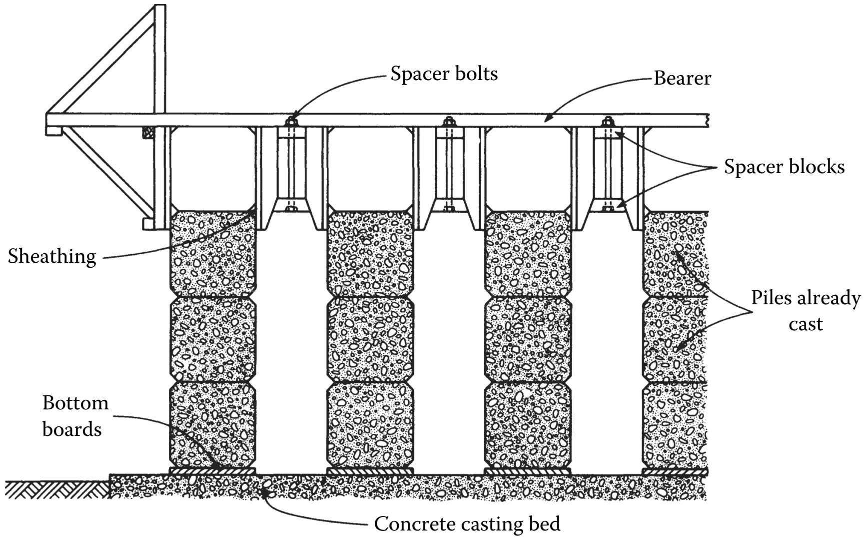

Figure 2.9 Casting precast concrete piles in tiers.

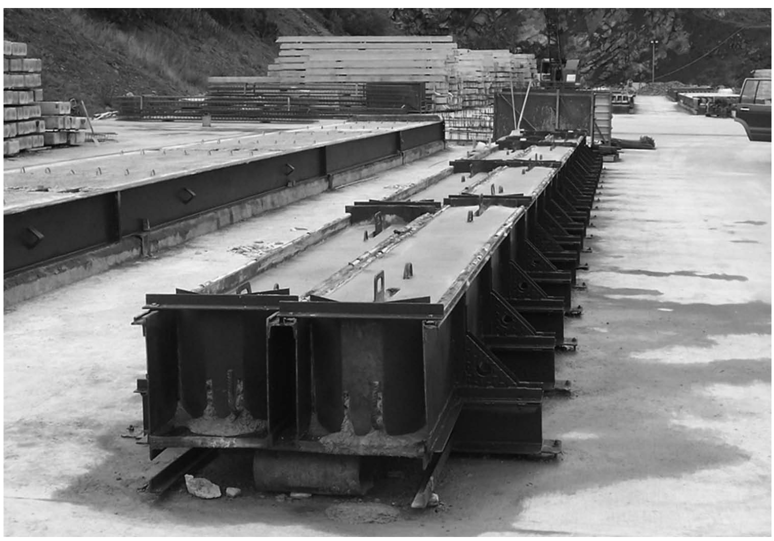

Figure 2.10 Steel moulds in pile casting yard.

Bridge in the Netherlands. Here, 4 m diameter prestressed concrete cylinder piles were made as vertically cast segments and then joined longitudinally to form 60 m long piles for installation by crane barge and caisson-sinking methods.

All precast 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

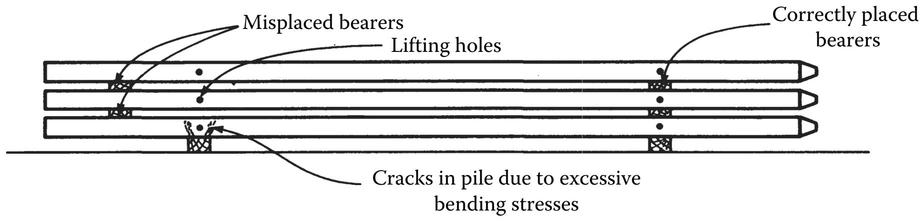

Figure 2.11 Misplaced packing in stacks of precast concrete piles.

only at the lifting positions, as, if they are misplaced, there could be a risk of excessive bending stresses developing and cracking occurring (Figure 2.11).

One of the principal problems associated with precast concrete piles is unseen breakage due to hard driving conditions. Jointed precast concrete piles when driven through soft or loose soils on to hard rock are particularly susceptible to damage. 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. Where such conditions are expected, it is advisable to provide a central inspection hole in test piles and sometimes in a proportion of the working piles. A check for deviation of the pile from line can be made by lowering a steel tube down the hole. If the 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 tube jams in the hole, an inclinometer is used to 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 with a central hole. Deviation from the production straightness of the axis of the pile should be limited to a maximum of 0.2% of the pile length.

Breakages are due either to tensile forces caused by easy 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 situations, 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 use of the PDA will assist in determining actual stresses along the pile (Figure 7.3b) for comparison with the calculated stresses; remedial actions then include changing the hammer, reducing the stroke and changing the cushioning.

The precautions for driving precast concrete piles are described in Section 3.4.2, and the procedures for bonding piles to caps and ground beams and lengthening piles are described in Sections 7.6 and 7.7.

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. BS EN 12794 defines pile joints in four classes, Class A to Class D, depending on whether the pile is used in compression,

Table 2.5 Dimensions and properties of square section piles as manufactured by Balfour Beatty Ground Engineering in the United Kingdom

Square section (mm)	*Maximum section length (m)*	*Typical applied load (kN)*
190	8	350
235	14	500
270	15	800
350	13.5	1200

Note: Resistance to applied load is dependent on dimensions of pile and soil properties.

tension or bending and the impact load test to be applied to verify the static design calculations. If the pile joint satisfies the impact and bending tests, then the ultimate capacity of the joint is 'identical' to the calculated static bearing capacity. A segment 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 possible to drive the jointed piles to 40 m in soft ground.

Balfour Beatty Ground Engineering produces and installs typical Class 1 precast piles in a range of segment lengths and square sections as shown in Table 2.5 normally in C45/55 concrete. The precast concrete units are locked together by a steel bayonet-type joint to obtain the required bending and tensile resistance, and a rock shoe incorporating an Oslo point may be used (Figure 2.7d).

Other types of jointed precast concrete piles include the *Centrum* pile manufactured and installed by Aarsleff Piling in the United Kingdom using C40/50 concrete and rigid welded reinforcement cages in varying lengths from 4 to 13 m in square sections from 200 to 400 mm. Lengths greater than 4 m for the 200 and 250 mm sections can be jointed using a single locking pin driven horizontally into locking rings in the joint box. The *multi-lock* ABB joint with four bayonet locking pins is used for the larger sections and provides a degree of pretensioning to the joint (Figure 2.12). Depending on the length, section and joint used

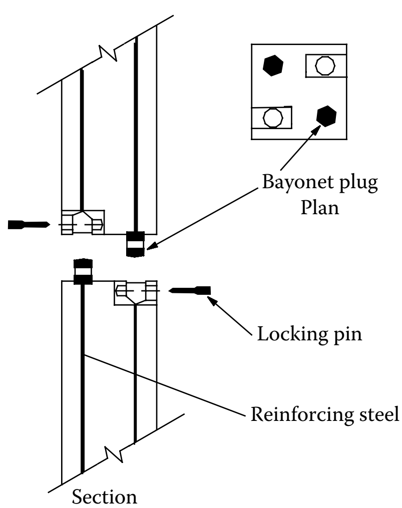

Figure 2.12 Typical locking pin joint for precast concrete pile.

and the ground conditions, capacities up to 1200 kN in compression and 180 kN in tension are possible. In addition to the above-mentioned 14.3 m long 600 mm square piles, Aarsleff produced 600 mm square jointed segmental piles up to 3.5 m long for low-headroom work on CTRL.

RB precast square concrete piles with a single central bar (as Class 2 given earlier) are made and installed by Roger Bullivant Ltd. They are available in a range of capacities (depending on ground conditions) from 200 kN for the nominal 150 mm square section to 1200 kN for the 355 mm square pile, in lengths of 1.5, 3 and 4 m. The standard joint for the limited tensile and bending capability is a simple spigot and socket type bonded with epoxy resin with each pile length bedded on a sand/cement mortar. Special joints (such as the Emeca joint) and pile reinforcement can be provided as needed to resist bending moments and tension forces.

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 due to bending caused by any deviation from alignment, the pile may break up into separate units with a complete loss of bearing resistance (Figure 2.13). This type of damage can occur with keyed or locked joints when the piles are driven heavily, for example in order 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, and tests to check bending, tension and compression capabilities should be carried out for particular applications. However, even joints made from steel castings require accurate contact surfaces to ensure that stress concentrations are not transferred to the concrete.

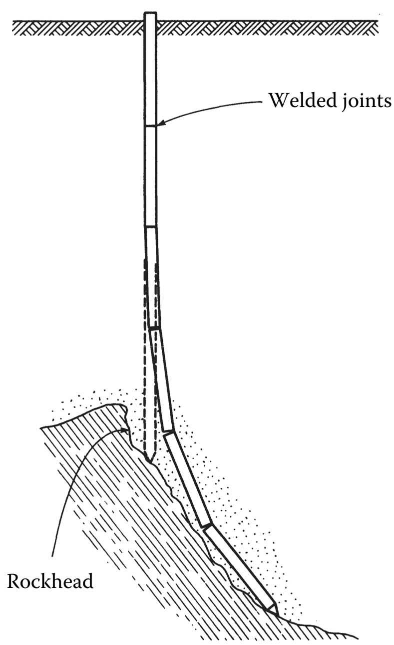

Figure 2.13 Unseen breakage of precast concrete piles with welded butt joints.

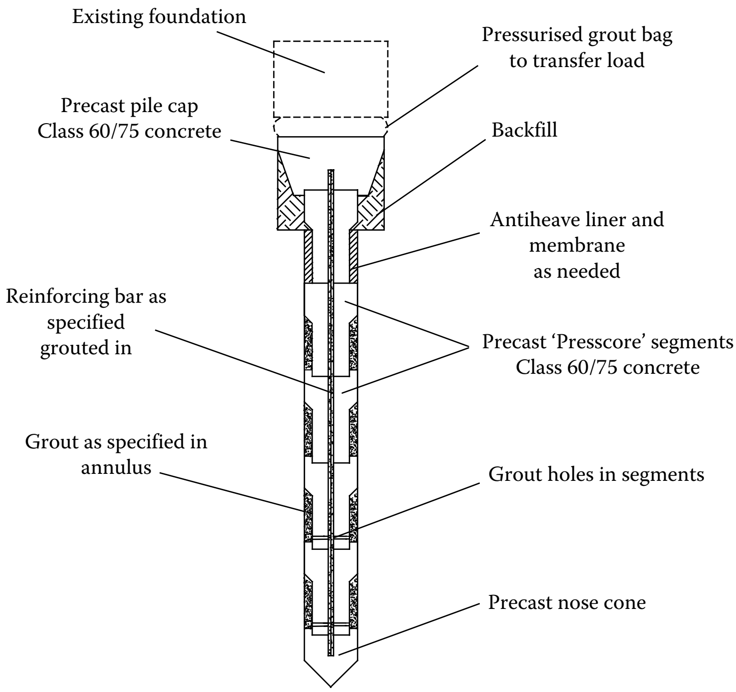

Figure 2.14 Presscore pile. (Courtesy of Abbey Pynford Foundation Systems Ltd., Watford, England.)

The *Presscore* pile developed and installed by Abbey Pynford PLC is a jointed precast concrete pile consisting of short units which are jacked into the soil. The concrete in the pile units and precast pile cap is 60 N/mm^2, and a reinforcing bar can be placed through the centre of the units (Figure 2.14). On reaching the required bearing depth, the annulus around the pile is grouted through ports in the units. The use of jacked-in piles for underpinning work is described in Chapter 9.

A high-strength cylindrical precast pile, 155 mm diameter and 1 m long, was developed in Canada for underpinning a 90-year-old building in Regina(2.10). The segments were cast using steel fibre-reinforced concrete with a 28-day compressive strength of 90 N/mm^2 and steel fibre content of 40 kg/m^3. Each segment was reinforced with four steel wires (9 mm) welded to a steel wire circumferential coil. Recesses were provided at each end of the segment and stainless steel rods connected each segment to form the joint. Hydraulic jacks with a capacity of 680 kN reacted against a new pile cap, and as each segment was jacked down, the next segment was screwed and tensioned on to the connecting rod. The required 600 kN pile capacity was achieved at depths ranging from 11 to 13 m.

2.2.4 Steel piles

Steel piles have the advantages of being robust, easy 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 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

Figure 2.15 Box piles using Z-sheet pile sections in fabrication yard. (Courtesy of Maxx Piling Ltd., Shenfield, UK.)

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.

Types of steel piles include plain tubes, box sections, box piles built up from sheet piles, H-sections and tapered and fluted tubes. Hollow-section piles can be driven with open ends as Figure 2.15. 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 Section 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 seabed in marine structures or in disturbed ground can be protected by cathodic means, supplemented by bituminous or resin coatings (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 (Section 4.3.3).

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, but testing should be carried out as required in

BS EN 12699. 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 seabed 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 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. 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 3.7) from which to install the piles is then advantageous.

Long lengths of steel tubular piles for offshore petroleum production platforms can be handled in a single length on large crane barges. Where this is not practical, they can be driven by underwater hammers, but for top-driven sectional piles, a pile connector is a useful device for joining lengths of pile without the delays which occur when making welded joints. The Frank's Double Drive Shoulder Connector (Figure 2.16) was developed in the United States for joining and driving lengths of oil well conductor pipe and can be adapted for making connections in piles up to 914 mm diameter. It is a pin and box joint which is flush with the outside diameter (OD) and inside diameter (ID) of the pile, with interlocking threads which pull the pin and box surfaces together. The joint is usually welded on to the steel pipe, not formed on the pipe ends. 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 follower for the final stages of driving, can be cut off for reuse.

Where large steel tubular piles need to be spliced to drive below ground level and are required to carry compressive loads only, splicing devices such as those manufactured

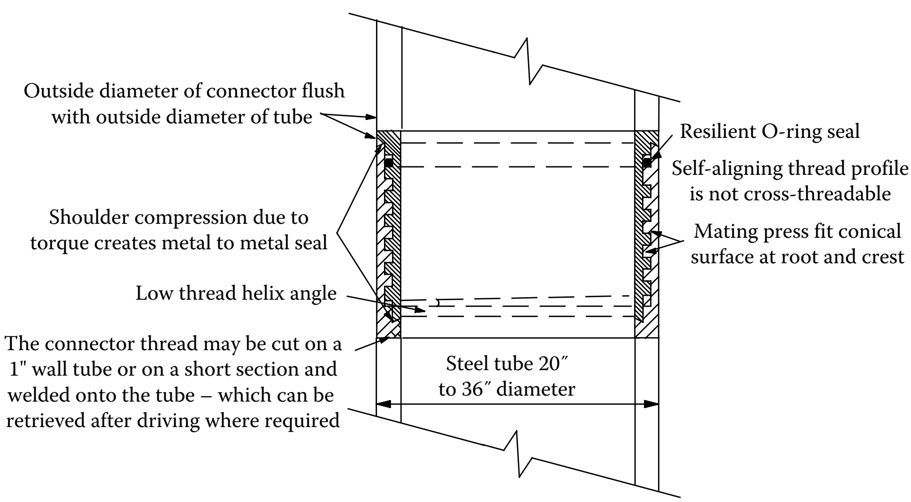

Figure 2.16 Schematic arrangement of Frank's *Double Drive Shoulder Connector.*

by the Associated Pile and Fitting Corporation of the United States (APF) or Dawson Construction Plant in the United Kingdom 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 splicer can also be used for cylindrical precast piles. Splicers are also available for H-piles in compression and consist 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. 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 minimising drag and oscillation from waves and currents (Sections 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 to order in Britain by Deepdale Engineering in a range of ODs up to 4000 mm in standard carbon steel and high-tensile steels to BS EN 10025-2 with wall thickness from 10 to 50 mm. ArcelorMittal produces a standard range of piles up to 3 m diameter and 25 mm wall thickness and up to 53 m long (without splices). The tubes are manufactured as either seamless, spirally welded or 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. The joints are then welded from both sides. In the longitudinally welding process, a steel plate is cut and bevelled to the required dimensions and 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 on the plate thickness that can be handled by particular machines. 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 62 mm by the longitudinal welding process. Special large-diameter piles can be manufactured by the process.

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 seabed, can be in a thinner mild steel or high-tensile steel depending on the severity of the driving conditions. The 1.3 m OD steel tubular piles used for breasting dolphins for the Abu Dhabi Marine Areas Ltd. tanker berth at Das Island were designed by BP to have an upper section 24 mm in thickness, a middle section 30 mm in thickness, and a lower section of 20 mm in thickness. The overall length was 36.6 m. As an economic alternative to tubular steel piles for turbine bases at a wind farm on a reinstated open-cast coal site in County Durham, Aarsleff installed 36 340 mm OD recycled, high-grade oil well casings through unpredictable backfill to toe into sandstone bedrock at each base. The additional stiffness of the casings allowed the use of a 4 tonne accelerated impact hammer to overcome obstructions to

Table 2.6 Dimensions and nominal applied loads for typical concrete-filled cased piles using light-gauge tubes

Internal diameter (mm^2)	*Area of concrete (mm^2)*	*Typical capacity (kN) for ordinary soil*[a]	*Typical capacity (kN) for rock*[b]
254	50,670	150	200
305	72,960	300	350–460
356	99,300	400	500–650
406	129,700	500	600–850
457	164,100	650	800–1,000
508	202,700	800	1,000–1,300
559	245,200	1,000	1,250
610	291,800	1,200	1,500

[a] Ordinary soil – sand, gravel or very stiff clay.
[b] Rock, very dense sand or gravel, very hard marl or hard shale.

driving and achieve a set of 25 mm in 10 blows. Sections of the threaded and collared casing could be joined to produce the maximum depth of 21 m.

Light spirally welded mild steel tubular piles in the range of sizes and typical capacity listed in Table 2.6 are widely used for lightly loaded structures, usually driven by a drop hammer acting on a plug of concrete in the bottom of the pile (see Section 3.2). These piles, known as *cased piles*, are designed to be filled with concrete after driving. Extension tubes can be welded to the driven length to increase penetration depth. Roger Bullivant Ltd. provides thicker wall tubes for cased piles from 125 to 346 mm diameter with up to 10 mm wall section for top driving of the pile. If piles have to be spliced, a special compression joint is needed for driving. Pile capacities claimed range from 350 to 1250 kN depending on ground conditions. In countries where heavy timbers are scarce, cased piles have replaced timber piling for temporary stagings in marine or river work. Here, 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. Continuity steel is usually inserted at the top of the pile to connect with the ground beam or pile cap.

Steel box piles are fabricated by welding together trough-section sheet piles such as the CAZ and CAU sections made by ArcelorMittal in double, triple or quadruple combinations or using specially rolled trough plating. Larssen U-section piles and Hoesch Z-sections, both rolled by Hoesch, are also suitable for box piles. 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.

The *MV* pile consists of either a steel box section (100 mm) or H-section fitted with an enlarged steel shoe to which a grout tube is attached. The H-pile is driven with a hammer or vibrator, while grout is injected at the driving shoe. This forms a fluidised zone along the pile shaft and enables the pile to be driven to the deep penetration required for their principal use as anchors to retaining walls. The hardened grouted zone around the steel provides the necessary frictional resistance to enable them to perform as anchors.

H-section piles, hot rolled in the United Kingdom to BS 4-1 as universal bearing piles (Figure 2.20a), 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. The Steel Construction Institute's *H-Pile Design Guide*, 2005,(2.11) is based on limit state design as provided in the Eurocodes and, in addition to describing H-piles in detail, makes reference to the offshore industry's recommended practice for steel tubular piles based on North Sea experience as described in the *ICP Design Methods for Driven Piles in Sands and Clays* (see Section 4.3.7).

Corus (part of the Tata Group) produces a range of broad flange H-piles in sizes from 203 mm × 203 mm × 45 kg/m to 356 mm × 358 mm × 174 kg/m; the ArcelorMittal HP range is similar. 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 × 305 mm in section were driven to depths of up to 49 m 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 × 368 mm H-piles were compared with those of precast concrete piles of similar overall dimensions. Both types of pile were driven by a Delmag D-25 diesel hammer (see Table 3.4). Although the driving resistances of both types were roughly the same to a depth of about 14 m (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 14.9 m, whereas the H-piles were driven on to 29 m without serious damage, even though driving resistance had increased to 0.5 mm/blow at the end of driving. Three of the H-piles were loaded to 3000 MN without failure, but three of the precast concrete piles failed at test loads of between 1100 and 1500 MN.

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 onto their outer periphery when the increased capacity is provided by a combination of friction and end bearing on the T-sections (Figure 2.18b). This method was used to reduce the penetration depth of 1067 mm OD tubular steel piles used in the breasting dolphins of the Marine Terminal in Cromarty Firth. A trial pile was driven with an open end through 6.5 m of loose silty sand for a further 16 m into a dense silty sand with gravel and cobbles. The pile was driven by a MENCK MRB 1000 single-acting hammer with a 1.25 m 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/200 mm at 22.6 m penetration. The pile was then cleaned out and plugged with concrete but failed under a test load of 6300 kN.

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

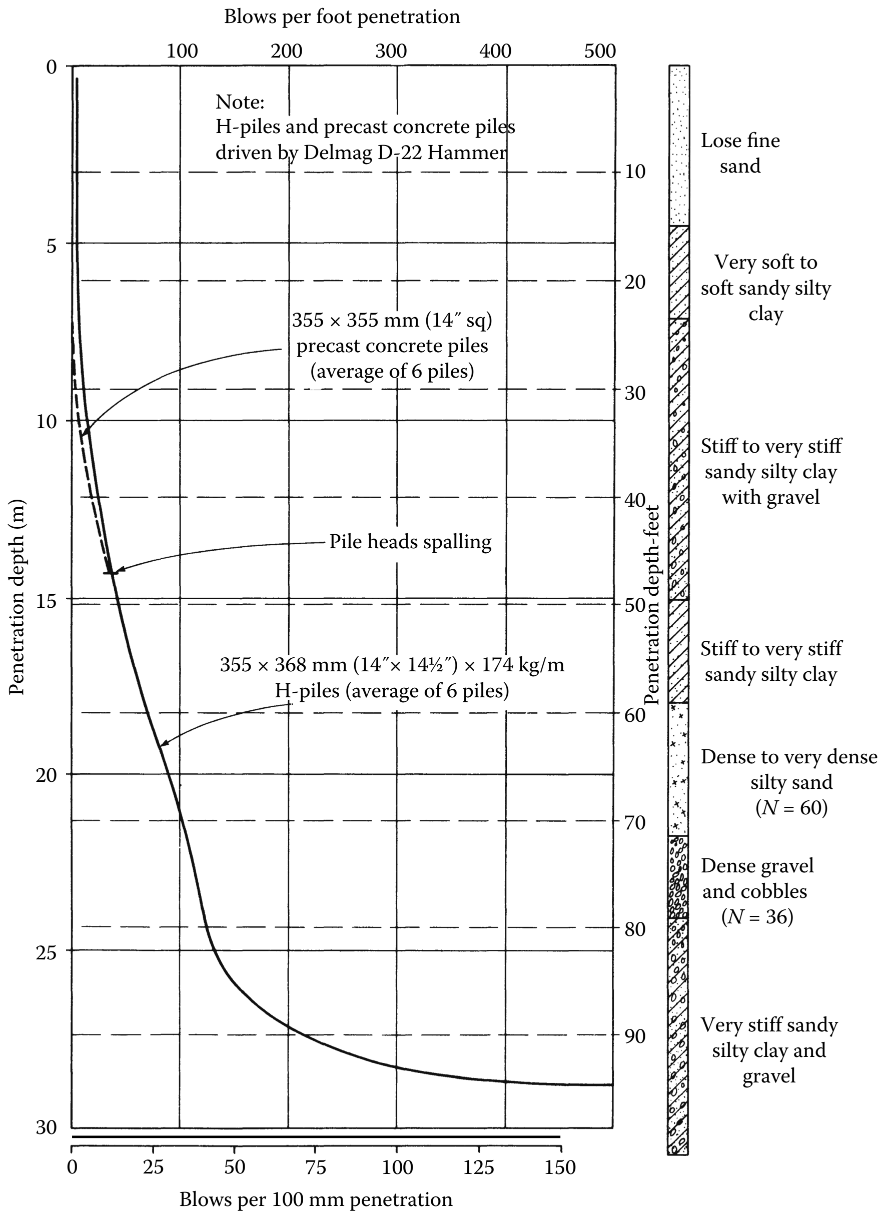

Figure 2.17 Comparison of driving resistances of 355 × 355 mm precast concrete piles and 355 × 368 mm H-section piles driven into glacial clays, sands and gravels in Hartlepool Nuclear Power Station.

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 × 0.303 × 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/200 mm at 19 m below seabed. The winged pile did not fail under the test load of 6300 kN.

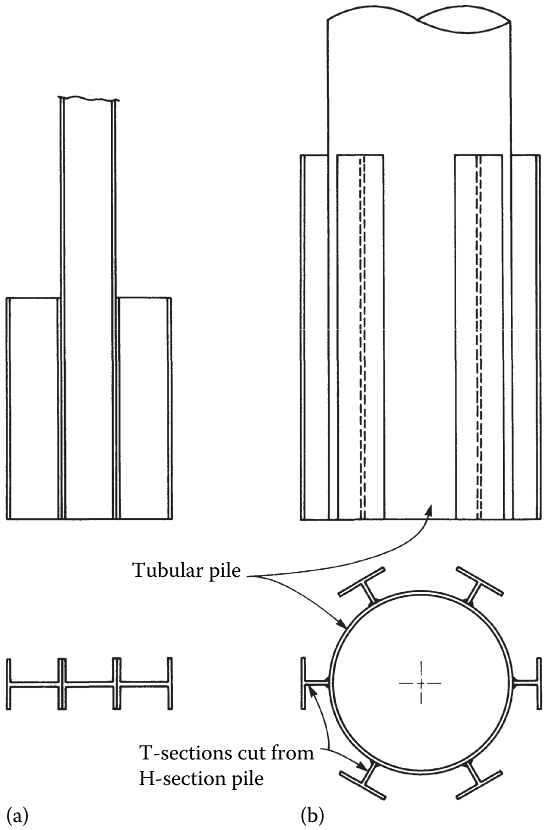

Figure 2.18 Increasing the bearing capacity of steel piles with welded-on wings (a) H-section wings welded to H-section pile and (b) T-section wings welded to tubular pile.

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. From his research, Bjerrum[2.12] recommended that any H-pile having a radius of curvature of less than 366 m 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 coarse-grained soil (sand and gravel) 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. A deviation of about 500 mm was observed of the toes of H-piles after they had been driven only 13 m through sands and gravels to an end bearing on sandstone at Nigg Bay in Scotland. Such damage can be limited by careful monitoring during driving using a PDA. EC3-5 defines the slenderness criteria for assessing buckling where the soil does not provide sufficient lateral restraint.

The curvature of H-piles can be measured by welding a steel angle or channel to the web of the pile. 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[2.13] at Lambton Power Station, Ontario, where 305 and 355 mm H-piles that were driven through 46 m of clay into shale had deviated 1.8–2.1 m from the vertical with a minimum radius of

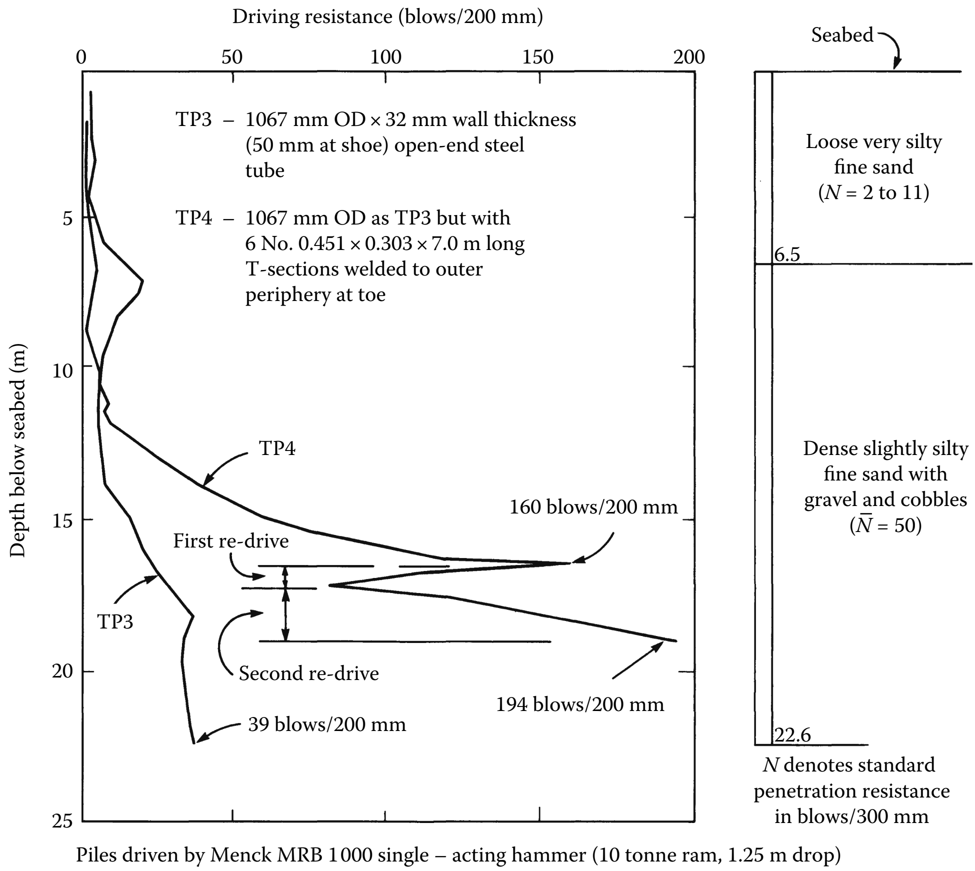

Figure 2.19 Comparison of driving resistance of open-ended plain and winged tubular steel piles at Britoil Tanker Terminal, Cromarty Firth.

curvature of 52 m. 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.

H-piles can be spliced on-site, either horizontally prior to installation to produce the desired length or to extend a driven section, using 100% butt weld to ensure full development of the strength of the section. End preparation using oxy-cutting to form either V or X bevels depending on alignment is usually acceptable[2.14]. The reuse of extracted H-piles is allowed under BS EN 12699, provided that the material complies with the design requirements, particularly in respect of durability and being undamaged.

Peine piles are broad-flanged H-sections rolled by Hoesch with bulbs at the tips of the flanges (Figure 2.20b). Loose clutches ('locking bars') are used to interlock the piles into groups suitable for dolphins or fenders in marine structures. They can also be interlocked with the Hoesch–Larssen sections to strengthen sheet pile walls. The ArcelorMittal HZ piles have tapered flange tips for interlocking.

The Monotube pile fabricated by the Monotube Pile Corporation of the United States 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 345 N/mm^2 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 203 mm,