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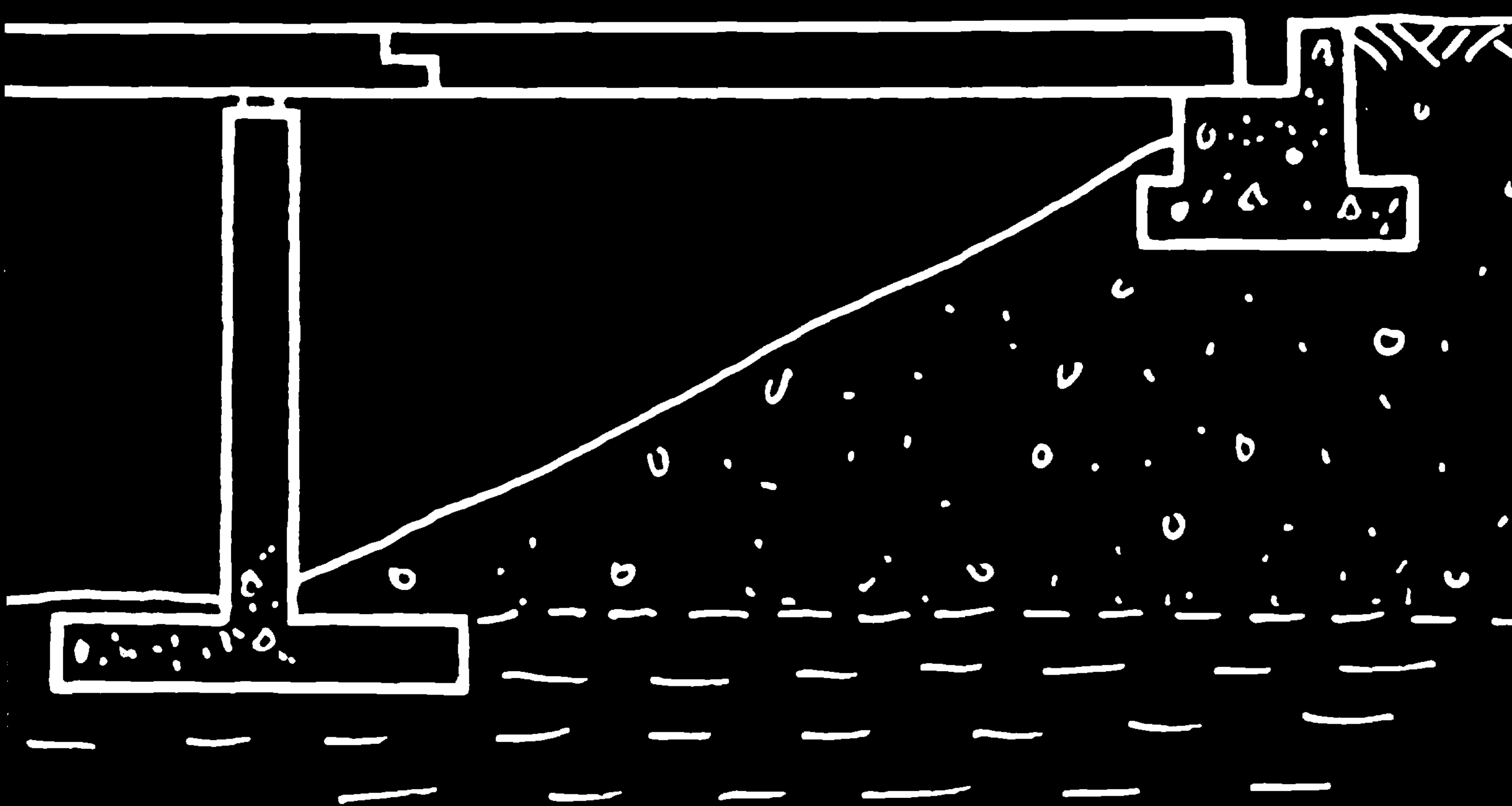
Bridge foundations and substructures



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Bridge foundations and substructures

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CONTENTS

INTRODUCTION

1 SURVEY OF CURRENT PRACTICE

This report on Bridge Foundations and Substructures summarises the findings of a survey of the attitudes of bridge designers in the United Kingdom. The objective of the survey was to gain some idea of the philosophies adopted by engineers in their choices of different types of foundations and substructures and of different methods of analysis. The report was also expected to highlight those areas where further guidance and research are required, and where financial savings might be effected.

The survey initially involved discussions during 1975/76 with about 120 experienced engineers within 46 organisations acknowledged below. During 1977 a considerable amount of additional advice and comment was added as a result of detailed criticism of a preliminary draft by the Steering Committee. The subsequent Draft Commentary on Current Practice in Design of Bridge Foundations and Substructures was published by the Building Research Establishment to stimulate public comment and it formed the text for a Seminar held at the Building Research Station in October 1977 attended by about 150 engineers. A considerable quantity of further comment, advice and criticism was collected at the Seminar and received in writing from bridge designers in the UK and abroad. This information has also been incorporated in this report.

2 OBJECTIVE OF REPORT

It became evident early on in the survey that for the majority of bridges there are too many independent and individual factors influencing design decisions to enable many useful generalisations to be made about the appropriate design or method of calculation. However, it was also evident that engineers in different organisations do have a similar attitude to many aspects of design, and it was possible to collect a very considerable amount of advice on good practice. As a result the report has concentrated on reproducing this advice and setting out the wide range of factors which influence selection of design, and method of calculation. At the same time an indication is given of the variety of solutions adopted and their problems. **Throughout the reports the statements of practice are based on the comments of one or more of the engineers interviewed on the survey.**

It is hoped that the report will prove to be a useful reference for both experienced bridge designers and newcomers. However concern has been expressed that such a compendium of experience might be thought a substitute for the experience itself and that it might be thought possible for the inexperienced to produce satisfactory designs using it alone. This they will not be able to do, and the report emphasises in many places the need to use judgement based on experience.

3 TYPES OF BRIDGE

This report has concentrated on the foundations and substructures of small and medium size road bridges over roads, rivers and railways. This group represents the majority of bridges built. Much of the discussion is relevant to other bridges (and other works) but in general these require further attention.

4 TEXTBOOKS AND REFERENCES

The textbooks listed below are recommended by bridge designers interviewed during the survey. They are referred to in the report by the authors' names. A few other specialist references are given in the various sections, and in general these are from published books. No attempt has been made to include comprehensive references of research papers, since it has not been possible to study them critically, and it is probably easier for designers to make direct use of commercial index systems, such as Geodex.

Soil mechanics and foundations

Terzaghi K and Peck R B. Soil mechanics in engineering practice. John Wiley, New York, 2nd ed, 1967.
Tomlinson M J. Foundation design and construction. Pitman, London, 3rd ed, 1975.
Lambe T W and Whitman R V. Soil mechanics. John Wiley, New York, 1969.
Tschebotarioff G P. Foundations, retaining and earth structures. McGraw-Hill, New York, 1973.
Peck R B, Hanson W E and Thornburn T H. Foundation engineering. John Wiley, New York, 2nd ed, 1974.
Little A L. Foundations. Arnold, London, 1961.
Bowles, J E. Foundation analysis and design. McGraw-Hill, New York, 1969.
NAVFAC DM-7 (1971). Design manual – soil mechanics, foundations and earth structures, US Naval Facilities Engineering Command, Washington DC.

Earth retaining structures

Huntington W C. Earth pressures and retaining walls. John Wiley, New York, 1957. (This book is out of print, but it provides much wise advice and is worth borrowing from a library for inspection.)

Textbooks on concrete with useful advice on foundations

Faber J and Mead F. Reinforced concrete. E & FN Spon, London, 1967.
Reynolds, C E and Steedman J C. Reinforced concrete designers handbook. Cement and Concrete Association, London, 8th ed, 1974.
Chettoe C S and Adams H C. Reinforced concrete bridge design. Chapman and Hall, London, 2nd ed revised 1951. (This book, which was published before the war, is out of print and out of date, but it provides some wise advice and is worth borrowing from a library for inspection.)
Military Engineering Vol XIV. Concrete, Part II. HMSO, London, 1964.

Undergraduate textbooks

Capper P L and Cassie W F. The mechanics of engineering soils. E & FN Spon, London, 5th ed, 1969.
Capper P L, Cassie W F and Geddes J D. Problems in engineering soils. E & FN Spon, London, 1971.
Scott C R. Soil mechanics and foundations. Applied Science Publishers, London, 2nd ed, 1974.
Simons N E and Menzies B K. A short course in foundations engineering. IPC Science and Technology Press, 1975.

Geology

Blyth F G H and de Freitas M H. A geology for engineers. Arnold, London, 6th ed, 1974.

Codes and Practice and specification

Civil Engineering Code of Practice No 2 (1951). Earth retaining structures.
British Standard Code of Practice CP2004, 1972. Foundations.
British Standard Code of Practice CP2001, 1957. Site investigation. Revised 1976 draft reference 76/11937.
British Standard BS 153, 1972. Specification for steel girder bridges.
British Standard BS 5400, 1978. Steel, concrete and composite bridges.
Department of Transport. Specification for road and bridge works. Her Majesty's Stationery Office. 1976.
Department of Transport. Notes for guidance on the specification for road and bridge works. Her Majesty's Stationery Office, 1976.

Department of Transport Memoranda related to this report are listed in Appendix D

5 ACKNOWLEDGEMENTS

The project received valuable assistance and detailed guidance and critism from an ad hoc Steering Committee comprising:

P Elliott	Department of Transport (BES) Chairman
J B Boden	Transport and Road Research Laboratory
L Clements	Scottish Development Department
R W Cooke	Building Research Station
S B Crutchlow	Department of Transport (E Int)
H G Frost	Department of Transport (E Int)
H A Hall	DTp, North Western Road Construction Unit
J B Holt	G Maunsell and Partners
A Knowles	DTp, Midland Road Construction Unit
W I J Price	Transport and Road Research Laboratory
Dr J E Spindel	British Railways Board
Dr W M C Stevenson	Department of the Environment for N.Ireland
J P Taberner	Costain Civil Engineering Limited
V G Towson	Department of Transport (BES)

In addition many comments and useful criticisms were received during the drafting of the report from:

T A Rochester	Department of Transport, Bridges
F Shields	Engineering (Design Standards) Division

During the survey of current practice meetings were held with engineers in the organisations listed below. Their interest, assistance and valuable advice are gratefully acknowledged.

Department of Transport:
- Bridges Engineering Divisions
- Engineering Intelligence Division
- Highway Engineering Computer Branch
- North Western Road Construction Unit, Head Quarters
- North Eastern Road Construction Unit, Head Quarters
- Midland Road Construction Unit, Head Quarters
- Midland Road Construction Unit, Stafford Sub Unit
- Eastern Road Construction Unit, Head Quarters
- South Eastern Road Construction Unit, Head Quarters
- South Western Road Construction Unit, Somerset Sub Unit
- South Western Road Construction Unit, Devon Sub Unit

Building Research Establishment
Transport and Road Research Laboratory

Scottish Development Department
Department of the Environment for Northern Ireland
British Rail (5 Regions)
London Transport
Devon County Council
Kent County Council
Kent County Council Highways Laboratory
West Yorkshire Metropolitan County Council
Construction Industry Research & Information Association
Ove Arup and Partners
Blyth and Blyth
Costain Civil Engineering Ltd
Robert J Curtis and Associates
R M Douglas Construction Ltd
Leonard Fairclough and Co Ltd
W A Fairhurst and Partners
Foundation Engineering Ltd
Frankipile Ltd
Freeman, Fox and Partners
Kier Ltd
Lind Piling Ltd
G Maunsell and Partners
Sir Alfred McAlpine & Son (Southern) Ltd
Scott, Wilson, Kirkpatrick and Partners
Dr F W Sherrell
Simplex Piling Ltd
Soil Mechanics Ltd
THI Group Services Ltd
Tarmac Construction Ltd
Taylor Woodrow Construction Ltd
Thorburn and Partners
M J Tomlinson Esq
Sir Owen Williams and Partners

CHAPTER 1

OVERALL PERSPECTIVE

1.1 Design process and priorities

This report has been set out to follow as far as possible the design process. The following pages of this Chapter report the statements of engineers interviewed during the survey (see Introduction) concerning the design priorities and the relevance of bridge foundations within the context of the whole contract, and general comments relating to economy in construction. Chapters 2, 3 and 4 discuss the assessment of site and factors that influence designers' choices of appropriate substructure and foundation. Chapters 5, 6 and 7 then report comments on the various methods used to design the chosen foundation. Finally Chapters 8 and 9 complete the design process with comments on details and anticipation of maintenance.

Although the survey was concerned solely with the design and construction of bridge foundations and substructures, several of the senior engineers interviewed prefaced their remarks with a number of general comments on attitudes, organisation and training of designers, which they considered as prerequisite for good design. Some of these recommendations are summarised in the following table.

Aspect	Comment
Overall perspective	Maintain a sense of proportion of the cost of the foundations, and of the deck, and consider the influence of their construction on the progress and cost of the rest of the contract. (See Section 1.2).
Construction simplicity	Aim to design works which can be constructed easily and comparatively quickly and which do not require many specialist trades and processes. (See Section 1.3).
Clarity of concept	A clear appreciation of all the main influences on the interaction of a foundation with the structure and soil environment is much more important than undue refinement of detailed calculations. When predictions cannot be exact make broad judgements of possible extremes of performance based on careful interpretations of soils data.
Autonomy of design team	Retain responsibility for decisions for all parts of the design within the design team: ie use specialists for advisers but not for making choice of design unless fully integrated into the team so that they are aware of the effects of their advice on the rest of the project.
Training of bridge designers	Train bridge designers to have a broad understanding of the behaviour of foundations and structure and of the construction and costs of both.

1.2 Bridge foundations in context of whole contract

Many engineers on the survey expressed the view that in order to obtain overall economy for a scheme it is essential to maintain a sense of proportion of the relative importance and cost of the various parts within the context of the whole project. The table below reports comments related to the significance of bridge foundations and substructures within the context of highway and bridge contracts.

	Comment
Bridges within highway contracts	Most of the bridges built in the last few years have been part of roadworks contracts. The cost of materials in the bridges is usually less than half of the cost of the bridges, and these often comprise only about a quarter of the total contract tender. Consequently savings of materials in bridges by a refinement of detail may not be significant in comparison with the contract total. However construction of some bridges can seriously interrupt the progress of other much more expensive parts of the contract, such as muck-shifting. For such bridges simplicity and speed of construction can be important design considerations and may affect the choice of structural form more than reduction of material content.
Bridge substructure and structure	Several design organisations pointed out that bridge substructures frequently cost more than the superstructures. When piling is required the cost of the piles alone is often comparable to that of the deck. If overall economy is to be obtained for the bridge it is important to develop the design of all parts simultaneously and not to give one part precedence by designing 'from top down' or 'from bottom up'. Some engineers feel that in the past piles were occasionally adopted unnecessarily, at significant expense, as a result of the designers first designing the decks with unduly tight limits set on differential settlement, and then having to provide piles to obtain sufficiently stiff foundations.
Comparison of designs	Several designers and contractors pointed out the difficulties of identifying the true cost of any part of a contract. Engineers warned of the dangers of drawing too fine conclusions about the relative merits of different designs purely from the basis of rates from contractors' tenders. Tenderers do not always try to make the rates for any part reflect the true cost of construction of the part or its complexity or interference with the rest of the contract. Tenders have to be prepared in a very short period and contractors often fill in all the rates on the basis of past experience. Then when this procedure is complete estimators pay attention to the more special features of the works including, the overall programme, the complexity of construction details of the structures, and their effect on the bridge programme and overall programme. At the same time they consider other commercial factors (such as the design organisation, the location and competition) and they take account of all these points in the spread of on costs on the net rates, or in a balancing item, or both. Consequently when a designer wishes to compare estimates for different designs he needs to make similar appraisals of construction complexity and interference to programme in addition to comparisons of material costs. (Although billed rates do not generally indicate the true distribution of cost within a contract, global estimates of costs based on them have proved sufficiently accurate and very useful for the overall planning of schemes. Several organisations maintain up-to-date costing systems for this purpose.)

1.3 Design for construction and maintenance

Several designers and contractors on the survey explained how significant savings can be achieved without detrimental effects on performance when attention is paid to methods of construction throughout the design process. The table below summarises their comments and suggestions.

	Comment
Speed of construction	Speed of construction is usually essential to achieve economy. Consequently the investment of considerable design effort may be justified to make details and procedures as simple and repetitive as possible. Opportunities for saving time can be seized during construction if the programme is flexible, as occurs when structures are designed so that the elements and details can be completed in a variety of sequences, and where restrictive procedures are not necessary.
Simplicity of construction	**The need for simplicity of construction, particularly in groundworks, was emphasised many times during the survey**. Simple works not only lend themselves to rapid construction but also reduce the contractor's dependence on many specialist trades and processes. Complex details in foundations can cause delays at the start of a project which have consequences throughout the contract. An awkward detail can have as much influence on construction costs as the form of the structure. Furthermore complicated details require a management effort out of all proportion to their importance. Even on relatively simple structures the work force need to complete two or three of a particular design before achieving a rapid turn-over. The aspects of simple construction most stressed were: 1 Use **materials which are readily available and easy to use.** 2 Construct as much as possible with **plant at existing ground level** to ease access and use natural crust and drainage. 3 Design excavations with a **level** bottom. 4 Use foundations with **simple shapes** and details (which can, if necessary, be adjusted easily to suite unforeseen ground conditions). 5 Form all **surfaces horizontal or vertical**, unless inclined or skew surfaces are essential for structural reasons or appearance. 6 Anticipate **re-use of formwork** without cutting or difficult alignment. 7 Where possible design structures to be **stable without propping** at all stages of construction. 8 Fix reinforcement and place concrete in **one plane at a time**. 9 **Use medium size reinforcement**, avoid both small bars and also heavy large bars which can be difficult to bend and fix accurately. 10 When walls are likely to be about 450 mm or wider, make them **wide enough for a man to get inside reinforcement**, and anticipate single pour construction. (Concern has been expressed by a few designers that while an increased adoption of more straightforward designs might lead to overall economy, the benefit would be felt only by the contractors and not by the owners since billed rates would not necessarily be adjusted. However other engineers argue that over a period of time contractors' rates in general alter to reflect changes in their average costs, while the level of profits is largely influenced by competition.)

	Comment
Site access and restrictions	See Section 2.2. Some designers have on occasions found it helpful to discuss with a potential contractor early in design the problems of access of heavy plant, and deployment on a restricted site.
Unforeseeable ground conditions and inclement weather	See Section 2.3 concerning soils investigation. Several designers advised the need for conservative designs for foundations because ground conditions can differ significantly from those predicted from the soils investigation. Furthermore, as explained in Section 8.1 and 8.2, construction of foundations can be severely hampered by inclement weather. Foundations which can be constructed quickly and which can be adapted to suit unexpected ground conditions are not only likely to be more economic but they could also perform better if the ground is prevented from softening from long exposure. Once construction is out of the ground progress is much more predictable (and faults can be corrected much more easily).
Temporary works	The cost of temporary works can influence the choice of bridge type. If the falsework requires expensive foundations, such as piles, then it may be that a form of structure which does not require such temporary support is more appropriate. Small details on the design can significantly help (or hinder) the fixing of temporary works.
Construction sequence drawings	Although the contractor is usually responsible for choosing the method of construction, several designers and contractors recommended that the designer should provide at least one suggestion for the method of construction when the design is novel or when the sequence is dictated by the design. Under some circumstances such information may be required to comply with Clause 14 (5) of the 5th Edition of the ICE Conditions of Contract.
Maintenance	Considerations for future maintenance that can be made during design are discussed in Chapter 9, while drainage details are discussed in Chapter 8.

CHAPTER 2

THE SITE

2.1 Site reconnaissance and desk study

Several designers stressed the importance of making the preliminary site reconnaissance and desk study right at the start of the design process before any decisions have been taken about type of structure and foundation. Such studies not only provide a large proportion of the essential information required for the design, so reducing the risk of expensive reappraisals later, but they also enable effective planning of the detailed soils investigation.

The Appendices of CP2001 incorporate several useful lists of the information that can be obtained.

	Comment
Site reconnaissance	The site reconnaissance of a bridge site is likely to include: 1 Assessment of problems of constructing bridge on the site, such as: access, obstructions, traffic, etc. 2 Study of local ground features which indicate soil characteristics and strata, natural drainage and groundwater regime: ie exposed soil strata in streams, vegetation etc. 3 Survey of existing structures on site and assessment of their influence on the ground and of the ground on them. 4 Survey of condition and type of neighbouring structures and engineering works. 5 Enquiries with the local residents, builders and statutory undertakers. 6 Trial pits with tractor mounted back hoe or hand auger.
Desk study	The desk study which proceeds simultaneously with the site reconnaissance generally includes: 1 Comparison of maps and surveys of site, with profiles of proposed works. 2 Information on previous uses of site (old maps useful). 3 Surveys of existing and abandoned services. 4 Analysis of geological maps and memoires of the Institute of Geological Sciences. 5 Inspection of aerial photographs including stereo pairs. 6 Discussions with the Institute of Geological Sciences and local organisations on available borehole records. 7 Discussions with water authorities, National Coal Board and so on, if appropriate. Some designers, when unfamiliar with the geology of the region, carry out these studies in consultation with an engineering geologist with local experience.

2.2 Influences of site on construction

The fundamental design decisions of form of structure and substructure and choice of foundations are based on considerations of what can be built and how it can be built most economically, as well as how it will perform. The following tables list some of the site characteristics which are found to affect construction.

Problem	Comment
Access	Special access may have to be constructed to an isolated bridge site. A contractor reported that only about half the bridges on a motorway route have simple access. Use of the route trace is often not practical because of the dominating demands of muck-shifting in good weather and because of impassability in bad.
Restricted space	On restricted sites the choices of foundation and bridge type are often controlled by what it is possible to build in space available and what plant can be used.
Piles	Piles may be needed to transmit heavy loads down through restricted space. Access for piling plant is a major problem. Large bored piles require room for boring rig, crane with casing and concrete trucks. Clearance of more than 15 m is often required, and in particular directions for raking piles. It is often advantageous to carry out piling from existing ground level to take advantage of access, and the probable hard crust for working of plant. Pile trimming may be necessary to a greater extent. There are some situations where fill is to follow when pile caps may be cast above or at ground level with advantage. Temporary works for piling over excavations or bank slopes can be very costly.
Poor ground	It is often advantageous to construct as much as possible from existing ground level, because the ground usually has a natural crust and established drainage. A viaduct over bad ground with poor ground level access may be best constructed over-hand. However access along the structure can then also be difficult.
Headroom	Overhead cables seriously restrict the height and choice of plant. It may be impossible to use piling rigs, cranes, concrete pumps, etc. It has on several occasions been found economic for construction to be sub-divided into small parts, sometimes prefabricated. The Factory Inspectorate can serve a prohibition notice to limit procedures. Consequently, if it is possible, it may well be worth trying to avoid siting a bridge under 400 KV cables.
Traffic interference	Interaction of construction with existing traffic on roads, railways and rivers can cause major disruption to the programme. If land is available it may be cheaper to spread works rather than suffer expensive delays.
Services	Congested services considerably complicate construction of foundations. Diversions are frequently late, and diversion of unexpected services can cause very expensive delays. On exceptionally congested sites it may be economic to prepare alternative or adjustable details which can be selected to suit conditions as found.
Groundwater	Lowering groundwater in excavations or piling can cause settlement of adjacent structures (see Section 2.5).
Noise	Designers and contractors pointed out the need to anticipate the effects of noise restrictions on piling operations. In special circumstances the restrictions could influence the choice of type of foundation or pile.

Problem	Comment
Bridges over railways	
General	**The design of a bridge over a railway has to be carried out in close cooperation with the railway authority from the start.** The requirements of the railway authority can radically affect construction procedures, timing and cost. If a bridge over a railway is part of a large project it can be advantageous to build it in an advance contract so that the special construction problems do not obstruct construction traffic access for muck shifting or disrupt the programme of the main contract. Several designers emphasised the importance of concentrating design effort into choosing the appropriate type of substructure and structure to facilitate construction under the particular constraints, rather than concentrating on refinement of detail.
Possessions	The influence and restrictions of the railway on construction depend on how busy the line is and what possessions are possible.
Footings	Spread footings usually have to be placed well outside 45° line down from edge rail, to avoid onerous restrictions and expensive temporary works. Restrictions are often also placed on the length of excavation that can be placed alongside the track. This can affect the length of base pours and reinforcement detailing. Suspect ground may warrant special caution as additional excavation or overdig may cut 45° line and so require restrictive and expensive temporary works.
Access to site	Contractors sometimes have difficulty obtaining access to both sides of track, and have only restricted access across.
Clearance	Lack of clearance for working space beside track can be very restrictive and construction may well be simplified if abutments are placed well behind minimum clearance allowed. Construction of abutments in cuttings can be particularly difficult, and under certain circumstances it can be cheaper to have an increased span to bankseats at the top of slopes For ordinary construction a space of about 2 m is needed between the fence and a wall. A small working space is possible if over-hand or simple construction is used, such as brickwork acting as permanent shutter to mass concrete poured in shallow lifts. Precast concrete, including post-tensioned H-blocks, has been used for facing abutments.
Piles	Methods of piling are likely to be severely restricted. Overhead electrification is likely to preclude all types of tall piling plant. Driven piles may not be accepted because of risk of plant falling on line. There may be insufficient clearance for large plant particularly for raking piles. Any risk of ground movement, particularly beneath control rods, is opposed by railway authority. Displacement piles can cause heave of track if driven in clay, or they can cause settlement if driven in gravel. Yet small cross-section driven piles can cause less disturbance on some soils than bored piles. Bored piles have advantage that they can often be placed by small tripod rigs. By placing pile caps as high as practical other track-side works are reduced.
Substructure	If cranes cannot be used then shutters and reinforcement may need to be prefabricated into elements which men can handle. Speed and economy ensue if construction in small parts can be programmed to provide continuous workload for men involved.
Structure	Decks are usually of precast or composite construction with beams placed during possession. (Aesthetic refinement such as edge cantilever, can often be omitted). Decks acting as props to strutted abutements are considered by some designers to be particularly inappropriate, because of extreme difficulty of placing deck before backfill, and virtual impossibility of replacing deck.

Problem	Comment
Bridge over rivers General	**The water authority and other interested bodies, such as navigation authority and conservancy board, must be consulted from the start of design.** Their requirements, which may be stringent, can affect the spans, headroom, position and shape of substructures, clearance on banks for dredging plant and so on. A bridge whose piers or abutments are within the river can cause flooding for some distance upstream, as well as affecting scour and/or accretion upstream and downstream. It may also affect erosion and general stability of the river. Predictions can be uncertain and difficult to question.
Hydraulic Survey	The Hydraulic Survey needs to be planned by an engineer experienced in the effects of bridge crossings on river behaviour, and it requires as much care as the Soils Investigation. The survey should begin when the crossing is first contemplated as it may affect the location of the crossing, and because the longest possible record is required.
Floods	Floods and flood debris can pose a greater threat during construction of a bridge than during its life. The height and size of cofferdams may be restricted by the river authority to reduce their influence on floods. Some designers stressed the need to obtain information on likely flooding during construction at the design stage, and later make this available to tenderers (without interpretation).
Construction	Simplicity of foundation has been found on many occasions to be more important than material economy. Construction over water is much more expensive than on dry land. It can be economic to divert rivers and build bridges in the dry. Otherwise foundations have usually been constructed in cofferdams (care has to be taken about position of cofferdam if raking piles are used). Caissons are unlikely to be used for a small bridge. In recent years large bored piles, possibly with permanent thin steel skin, have been placed in casing from above water level. Pile caps have also been constructed above low water level. Choice of type of pile may be governed by what plant can be used and is available.
Access and clearance	Access for plant may be restricted to one bank. The benefits of overspanning small rivers are similar to those for railways.
Scour	Scour is the greatest single cause of bridge failures. Foundations need to be below the estimated lowest level of scour or bed protection works (gabions, rip rap, etc). Scour can be rapid in times of flood, and even in large slow rivers it can go deep and change unpredictably. Scour can be very much worse during flood turbulence than when it can be measured. Piers with streamlined edges cause less eddies and so less scour than circular columns. However there may be little benefit if the direction of flow alters by a few degrees during floods. A smooth faced footing accelerates side flow and invites scour. Marina mattresses, gabions and rip rap slow down flow and silting fills their voids; they can be inexpensive. Hard clays and shales soften with time and erode. Sheet piling used for cofferdams is often left in for scour protection. When a serious risk of scour is identified designers should consult an engineer experienced in river hydraulics at the earliest possible stage. The Hydraulics Research Station, Wallingford, Oxon, offer an advisory service.
Hidden river beds	River beds change their courses and a flood plane can have several hidden courses, possibly with flowing water. They can often be located by series of trial pits. Abutments may be founded on the sides of a buried valley, where positions of boreholes are critical.
Drawdown	Substructures and embankments subject to flooding are designed to drain rapidly or be impermeable, in which case stability during drawdown can be critical.
References	C R Neill (1973). Guide to bridge hydraulics. Published by University of Toronto Press for Roads and Transport Association of Canada. Department of Highways Ontario (1967). Prediction of scour at bridges, Report RR115.

2.3 Subsidence

In regions of potential subsidence the subsidence dominates other design restraints. Particular experience is required beyond scope indicated below.

Problem	Comment
Coal mining new workings. . old workings. . .	The majority of subsidence problems in the UK relate to coal mining. The National Coal Board have experience in prediction and control of settlement due to current **long-wall mining**, but often cannot specify directions of advance of coalfaces in future. Prediction of settlement and collapse due to upward migration of voids of old **pillar and stall** workings is usually not possible. If risk is unacceptable then old workings, and shallow seams that could have been worked, are **grouted**. The depth and spacing of grout injection depend on extent of old workings, and depth and soundness of rock above (an example is grouting to 20 m or 30 m at 6 m spacing below roadworks, 3 m spacing below structures, and 1.5 m spacing below special foundations). Grouting can only be done properly by a competent contractor, and can be very expensive. Water in workings can make grouting ineffective, and it can be difficult to control the flow of grout (it is usually progressed up dip of coalseam). It can also be difficult to check that grout has been successful.
Salt mining	Salt mining is much less predictable than coal mining, and can also cause very large subsidence.
Swallow holes	Swallow holes (generally at surface of chalk and limestone) can be difficult to identify (even with geophysical methods) unless they are visible to the eye or on aerial photographs. When found they are usually filled, grouted or bridged.
Structures	Structures are usually designed to **accommodate distortion** due to subsidence by articulating or flexing. **Simplicity and predictability** of performance are essential. It may be easier to predict the performance of a simple flexible structure than a three-dimensionally articulated structure with complex mechanism. **Many bridges have simply supported torsionally flexible decks.** Jacking pockets are generally provided. Considerable care is given to design of **joints**, rockers and bearings to accommodate the large movements. Parts are tied together to prevent separation. (For this reason some designers use strutted abutments for small bridges to ensure compression at joints, but such structures can be subjected to very high earth pressures during the compression phase of a subsidence wave.)
Substructures	**Substructures are designed to tolerate ground strain and cracks beneath.** Refined calculations are not warranted, but careful checks are made of limiting conditions of support, possibly at reduced factor of safety. **Spread footings** are generally used, often with movement permitted in controlled friction plane (of sand, or two layers of felt, or other selected material above a smooth blinding) to ensure that sliding can occur before instability from over-turning or bearing. If there is a risk of collapse above a local void the structure is designed to span or cantilever over. Walls are provided with reinforcement to resist frictional tensions, or else with plenty of movement joints. Culverts have been articulated. Piles are generally not appropriate unless mineworkings are very deep, or the piles are constructed (possibly within large sleeves) so that their performance is not affected by subsidence movements. Possibility of piles bearing above voids is checked.
Temporary works	Mineworkings can seriously affect foundations and cost of temporary works: a bridge design avoiding need for expensive temporary foundations may be appropriate.
Monitoring	Monitoring of structures vulnerable to unpredictable subsidence is important and reassuring.
References	Institution of Civil Engineers, (1972). Report on mining subsidence. The Institution, London. Bell, F G (1975). Site investigation in areas of mining subsidence. Newnes-Butterworths. Simms, F A and Bridle, R J (1966). Bridge design in areas of mining subsidence. Journal of the Institution of Highway Engineers, November 19–38. National Coal Board Mining Department (1975). Subsidence engineers handbook.

2.4 Soils investigation

Principles

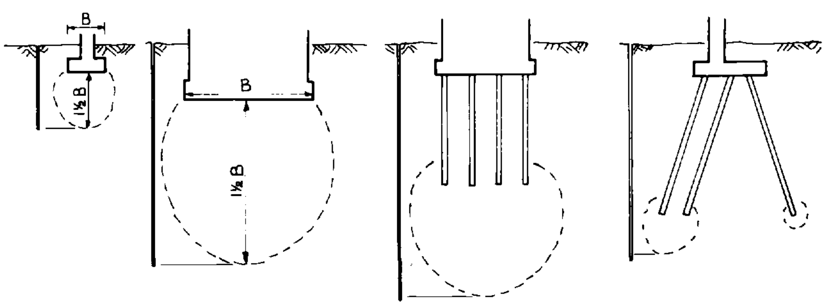
The soils investigation (see Burland et al reference to Section 5.5) is expected to yield:

1 A knowledge of the soil profile and groundwater conditions across the site, to the depth affecting foundations, set in the context of the local geology and tied in with local experience.

2 A detailed and systematic description of the soil in each stratum in terms of its visual and tactile properties. This should preferably be coupled with routine in-situ indicator tests for easy correlation with local experience and practice.

3 An estimate or determination of the mechanical properties of the relevant strata.

For the majority of small and medium sized bridges the decisions as to type and depth of foundations can be made primarily on the basis of (1) and (2) above. In general an investigation cannot be sufficiently precise to provide accurate quantitative information of ground behaviour and the designer has to **estimate the limits of behaviour between which the actual performance is likely to be.**

Subject	Comment
Planning of investigation	Several designers and geotechnical engineers stressed the importance of tailoring the scope of the investigation to suit the particular requirements of each bridge and foundation in turn, and not just following a stereotyped procedure. By considering the requirements of the particular foundations the risk of vital information being neglected is reduced, and there is also less risk of carrying out a large number of irrelevant tests. A deficient or stereotyped investigation could prove sufficient for the design of many small structures, but for a singificant minority an inadequate or misleading investigation could result in very expensive changes and delays during construction. In order to identify the particular requirements of each foundation, it is often found convenient to carry out the investigation in two stages: **Stage 1** **To obtain a broad picture of ground conditions and identify problems such as ground water.** It might include trial pits and one deep borehole per bridge with a large number of inexpensive tests such as SPTs to indicate variability. The conclusions should enable the depth of the preferred foundations to be identified and indicate the extent of further investigations required. **Stage 2** **A detailed study of competence,** variability and construction problems of the strata in which preferred foundations will be placed. This can involve further trial pits and two or more boreholes per foundation in a pattern to identify the soil strata in three dimensions along and across the site: but fewer are often found sufficient in regions where strata are predictable. Knowledge of the ground is required to at least bottom of the zone of soil influenced by loading (ie 'pressure bulb' shown in sketches below). The investigation may include special tests and installation of piezometers. If the preferred foundation cannot be identified with confidence, information will be needed for the whole range of strata which might provide support or affect performance. 

Subject	Comment
Planning of investigation (contd)	The continuity of site investigation between Stages 1 and 2 depends on the size of contract and planning of the design process. With close co-operation between site investigation supervision and bridge design, the two stages are often consecutive. It is generally considered advisable for the designer to be consulted before the first borehole on each site is abandoned. On a large contract it may be practical to complete Stage 1 boreholes at all bridge sites, while the designer chooses the preferred foundation types, and then to continue with the Stage 2 investigation. However such preferred procedures are not always practical during the planning stage of a project because of problems of programme or access. There can be occasions when the precise position of foundations, and possibly of a bridge, are not known at the time of the main soils investigation. Then the designer might cover the likely extreme positions of the bridge in plan with the trial pits and boreholes of the main investigation, and later when the design has progressed he can carry out further investigation possibly in the form of trial pits with in-situ testing only. **The soils information required for planning construction procedures can be quite different from that required for predicting the performance of the ground beneath foundations.** Information for construction is required by the designer to help him identify the type of bridge which can be built most economically, and by the contractor who must work in and on the ground. Ground characteristics which particularly affect construction include: the consistency of the material to be excavated; the strength of the ground alongside excavation; groundwater levels and potential flows; the strength of ground beneath temporary works; the strength of ground beneath heavy plant. While the collection of such information as part of routine soils investigations is encouraged, designers emphasise that it is unwise to provide the contractor with an interpretation of it in the Contract Documents.
Liaison with bridge designer	Designers and geotechnical engineers stressed the **importance of the bridge designer being intimately involved with the site investigation** from its initiation. He knows how critical or not critical ground movements are to the structure and he needs to convey the appropriate sense of proportion to the management of the investigation to ensure that their interpretive conclusions are based on correct assumptions. He also needs to check that his understanding of borehole log descriptions agrees with the contractor's by **personally carrying out a detailed visial inspection of split cores early on in the investigation.** It can be useful, if the design process is likely to be protracted, for the designer to write a brief report for later reference summarising the types of bridge envisaged during the investigation, together with observations on the shortcomings of the investigation. In contrast, on the occasions when a late change is made to the route the designer may need to anticipate the limitations of a rushed investigation and adopt a design for bridge and foundations which is conservative and simple.
Supervision	Designers and contractors on the survey emphasised most strongly the **importance of vigilant and competent supervision in the field.** The quality of an investigation cannot be checked later. Small details of great significance, such as waterbearing fissures and silt bands or boulder sizes,caneasily be overlooked. The engineer charged with making the detailed foundation recommendations, has the greatest motivation to demand an accurate investigation and so should have a day to day involvement and control over the field drilling and testing programme.

2.4 (contd) Soils investigation

Subject	Comment
Supervision (contd)	Some design offices check daily logs and preliminary test results immediately and maintain an updated log of ground conditions over the site. Thus they identify inconsistencies in time for checks to be made. Although this requires experienced senior staff it occupies little time if done as a routine. **Particular attention needs to be paid to the description of soils.** Proper descriptions can provide the most valuable information from the investigation. Poor quality descriptions can be misleading and result in considerable waste of money. The descriptions must be in accordance with the standard of CP2001 to ensure correct interpretation by later users.
Report writing	Two reports on the investigation are usually produced. The first, written by the contractor, collects together the factual information of borehole logs and tests. The second contains the interpretation of the facts and recommendations and is written either, by the design organisation if they have expertise and manpower, or by their consultant, or most commonly by the contractor on a consultancy basis. Whichever practice is followed **it is essential that there is close interaction between the person making recommendations and the designer to ensure that the recommendations correctly apply to the particular requirements of the bridge and the overall scheme.** Some designers have criticised reports for containing conservative generalisations, while contractors have complained that they could not obtain adequate information about proposed structures. **Once a report has been submitted it is difficult to relax restrictions without further investigation.** At the same time care has to be taken that the co-operation is not so close that the objectivity of recommendations is decreased. **The assumptions and calculations (or method) from which conclusions are drawn need to be included in an appendix to the report to facilitate verification.** Ultimate capacities, load factors and factors of safety need to be clearly stated and justified, and the level at which judgement has been used made clear. Care needs to be taken when interpreting descriptions of strength since there are significant differences between some textbooks and CP2001. Also due regard needs to be paid to the dependence of empirical relationships and factors of safety on particular test methods and sample sizes. Reports should not need to be long. Several designers observed that the current fashion for encyclopaedic volumes is not found helpful. If piling is involved it is convenient to contractors if the relevant section is written so that it can be separated easily from the rest for forwarding to piling subcontractors. At present bridge designers usually only issue the factual report to tenderers, because of the contractual risk of interpretations being proved incorrect during construction. Contractors complain bitterly at this practice since they feel that a general appraisal of groundwater and construction characteristics, by someone with firsthand experience of the region, could be of considerable value to them.
Report reading	**The reports need to be read critically**, starting with an assessment of the reliability and scope of factual information. **It is often forgotten that it is only possible to extract Class 1 samples, suitable for accurate compressibility and strength testing, from a small proportion of real soils**, as is indicated in Appendix A. Some geotechnical engineers feel that some structural engineers place too much reliance on numerical test results while not taking enough notice of qualitative observations.

2.5 Groundwater

The groundwater regime on a site has as much influence on construction and performance as the soil. The need for its proper investigation and assessment cannot be overstressed. It has in the past been overlooked on several occasions and unexpected groundwater problems have caused some of the most serious construction delays. The subject is discussed in detail by Tomlinson (Ch 11).

Problem	Comment
Assessment	The necessary information can often be obtained at little extra cost simply by taking more care with observations during normal site investigation. **Levels of water in boreholes** need to be recorded in detail by dipping at every start and stop of work, including breaks and breakdowns, (if possible with an intermediate reading to indicate whether rise is slowing down), and details given of casing levels. It is sometimes appropriate to order special standstills for water measurement. While simple borehole records are helpful they are not accurate and can be misleading because of water draining into the hole or being added to assist boring, or if level is thought stationary when it is moving. Also, **it is never clear at what level water is entering.** **Standpipes or piezometers** are needed if groundwater is likely to be a serious problem, or the possibility is recognised in preliminary borehole. The appropriate type of piezometer depends on soil type; the Casagrande type open standpipe is simple and often satisfactory. Installation of piezometers in correct strata require very close liaison between installer and designer. **Flow of water** through strata can be large, but can be difficult to estimate : a study of local geology is helpful. Pumping trials are unlikely to be necessary for foundations of small or medium size bridges. However a trial pit or augered shaft can give a rapid and direct indication of likely construction difficulties. Observations of groundwater conditions at a point, like those of soils, can be **misleading** about the regime in the ground en masse, and so should always be treated with **caution**. In heterogeneous soils conditions can change markedly.
Seasonal variations	Seasonal and yearly variations can be very large; so it is important to record standpipe levels regularly and for as long as possible.
Excavations	The flow of water into the majority of excavations for small and medium sized bridges can generally be controlled by pumps without much difficulty. But it is not uncommon to find one or more foundations on a contract in marginal ground conditions where it is difficult to predict in advance what problems will occur. Then very expensive delays can occur while the method of control is selected. The expense is generally much less when problems are anticipated. **If serious problems are likely then a different design might be appropriate.**
Permeable ground	The flow can be controlled by dewatering but the expense of excavation is increased enormously, possibly tenfold compared to in dry. Lowering of the water-table and/ or removal of fines can affect the stability and settlement of neighbouring structures. In some ground such as above gravel stratum it may be impossible, and it may be necessary to construct under water, possibly using mass concrete or hydraulic fill.
Moderate permeability	Pumping from open sumps is most commonly used because of its relatively low costs. But where the inflow is large there is a risk of instability of sides and base of excavation due to slumping and 'boiling'. Wellpointing can be used effectively in sands and sandy gravel. It has the advantage that water flow can be controlled to improve the stability of excavation. It has the disadvantages of expense and that equipment is not usually on site. Cofferdams are frequently practical where a cut-off in a substratum is achieved. They then have the advantage that they reduce the uncertainty of predicting the efficiency and effects of pumping.

Problem	Comment
Variable ground	None of above techniques may be effective in very variable ground, or fissured weak rocks, or mixtures of boulders and clay. Driving of sheeting may not be possible, and it may be necessary to use other expensive techniques, such as grouting.
Impermeable ground	Impermeable soils, including bedded and laminated rocks, can be vulnerable to heave at the bottom of excavations if subject to high water-table.
Piles	Construction of bored piles is made much more difficult by water ingress to shafts and many expensive delays have occured in past.
Performance	Groundwater affects the performance of bridge substructures by: **reducing bearing capacity of substrata;** **increasing lateral pressures** by flooding or perched water-table; **frost heave** of light structures; **flooding** of culverts below water-table in embankment; **reducing stability of bankslopes** supporting bank seats or abutments. (Horizontal and vertical boreholes for drainage can help stabilise a bankslope in shales, mudstones etc.) **The settlement of footings** is substantially increased if swelling and uplift has taken place at the base of excavation. The performance of bored **piles is affected if soil is loosened or cement washed out** during construction.
Corrosive water and soil	**Sampling and testing** of groundwater for corrosive conditions are often inaccurate due to exposure and contamination. If the initial site investigation indicates high acid or sulphate contents then much more careful sampling and rapid testing may be appropriate in the second stage investigation. From these results the need can be assessed for sulphate resisting cement, sheathing and so on. CP2004 gives advice on protective measures. See also Section 8.6. The risk of **sulphate attack** depends on the accessibility of the foundation to water. If the foundation is above the water-table or the water is not flowing then the risk is small (however there can be a significant flow of water through fissures in fissured clay). Sulphate contamination of clay can be very localised due to pockets of gypsum crystals. These have a low solubility, but sodium and magnesium sulphates may also occur and are highly soluble. Sulphate attack is more severe in acidic conditions. **Steel bearing piles** are unlikely to lose a significant thickness of metal from corrosion where driven into undisturbed soil which is not aerated by flowing water. Little information is available about the degree of corrosion of piles driven into fills and soils which might be aggressive because they contain oxygen or anaerobic sulphate producing bacteria. Some piles have been inspected in such ground after 30 years or so, and while most have suffered negligible loss of section a few can be expected to lose several mm during life of structure. Piles with protective coatings did not perform significantly better. It is known that parts of piles buried with disturbed soil are much more vulnerable than piles driven into undisturbed ground. Fortunately there is a sufficient built-in corrosion allowance in bearing piles for all but the most aggressive soils, since working load stresses are limited to only 30 per cent of yield to avoid damage during driving.

2.6 Advanced contract pile and plate bearing tests

	Comment
Advanced contract pile tests	
Guidance for design	Designers have found it extremely difficult on many sites to make realistic predictions of pile performance, unless they already have experience of tests on similar piles in virtually identical ground conditions. For the majority of small and medium sized bridges on piles this imprecision is generally accepted because it is more economic to adopt a conservative design than carry out an advanced contract pile test. However, where a contract includes a large number of piles a considerable saving may be achieved by improving the design using the results of advanced tests. It is generally not possible to achieve this from tests in the main contract because working piles are only tested to 1.5 times working load, and a delay waiting for testing and analysing special 'trial' piles is usually unacceptably expensive. See Section 7.7 for detailed comments.
Guidance on construction problems	An advanced contract pile test has the added benefit, which may be of much greater value, that particular problems in installation are recognised before the main contract and so reduce the risk of expensive unexpected delays in the main contract. On occasions a trial installation is done without test. Since the performance and installation problems are very dependent on plant and method it is advantageous if the advanced pile is installed with conventional equipment, as similar as possible to that likely to be used in the main contract. Then all the records of equipment, rates of work, casing, groundwater, collapses etc could be of considerable benefit to subsequent tenderers. However it is quite possible that the main contractor will wish to use a quite different pile type of a different subcontractor and then much of this benefit of the advanced contract is unavoidably lost. Unfortunately the nomination of the advanced contract piling contractor for the main contract can introduce worse contractual problems than the advanced contract avoids.
Plate bearing tests	
	Plate bearing tests (see CP2001 Section 5.5) are performed to assess the bearing capacity and modulus of the ground on a greater scale than is possible in conventional borehole investigations. They are useful for checking that piles are not necessary in uncertain ground conditions (a trial pile cannot indicate this). In heterogeneous soils they often indicate that much greater bearing pressures can be used than can be predicted from small scale test results while in fissured clays they can indicate that a lower bearing capacity should be assumed. Some series of plate bearing tests have been performed rapidly and inexpensively in augered holes and trenches using the rig/excavator above for load reaction. They have the limitations that tests are generally impractical in water bearing ground and the region of soil tested is only of same scale as the plate, so several tests at different depths may be required for large foundations. Reference on plate bearing tests: Marsland, A. (1973). Large in-situ tests to measure the properties of stiff-fissured clays. Building Research Establishment Current Paper CP1/73.

CHAPTER 3

CATALOGUE OF SUBSTRUCTURES

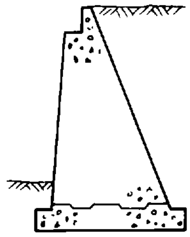
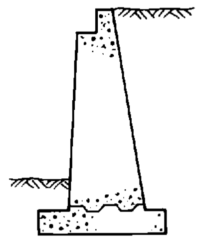
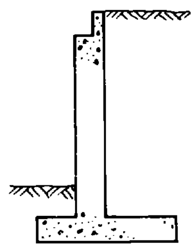
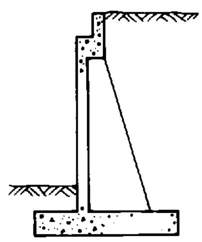
3.1 General considerations

This Chapter forms a catalogue of the most commonly used types of substructure, to provide a reference during the conceptual design stage of bridges. The table below lists some of the general comments relating to philosophy of design reported during the survey, while the subsequent tables contain the comments related to various particular types. Sketches are included of typical substructure shapes, but not of the wide variety of variations and combinations of features which may be adopted to suit a bridge to its particular constraints. Details such as shear-keys, sloping footings, ballast walls and so on are discussed in Chapter 8.

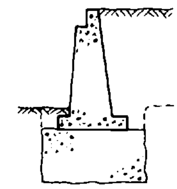
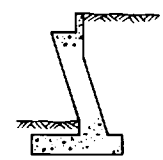
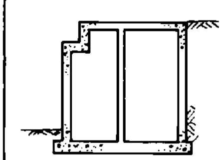
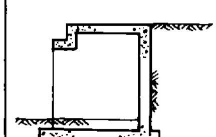
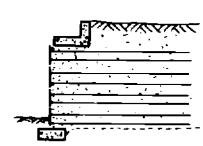
Subject	Comment
Interdependence of substructure, deck and approaches	Many designers emphasised the importance of developing the conceptual design of substructures in parallel with the design of the deck and approaches. Decisions related to span arrangement affect: the number, loading and size of foundations; the zones of soil providing support; relative difficulties of construction of, and access to, different foundation positions. The design loads on a particular foundation might be adjusted by a change in spans, or articulation, or form of deck construction. The design bearing pressure of a bearing stratum depends on the acceptability of the associated settlement and differential settlement which must be accommodated by the articulation and flexure of the structure (see Section 4.2). The approach embankment could settle more than the bridge (see Section 5.1). Consequently the differential settlement between embankment and abutment needs to be considered at the same time as that of the bridge when considering their effects on articulation, drainage, and ride of the road.
Construction stages	Several contractors pointed out the influences of complexity of structural form and details on the cost and time of construction. Simplifications aimed at reducing the number of construction procedures can lead to overall economy and better workmanship. The construction of works below ground level is often the most susceptible to delays because of its vulnerability to water from inclement weather and unforeseen ground conditions. Refinements of geometry and detail of foundations in the bottom of deep excavations are seldom economic (see Section 8.1.). Structures which are freestanding at all stages of construction are usually easier to build than structures which need propping. Speed of construction can be assisted if the design enables the bridge and earthworks to be completed by more than one sequence so that the programme can have flexibility to accommodate changes of circumstances.

3.2 Wall abutments

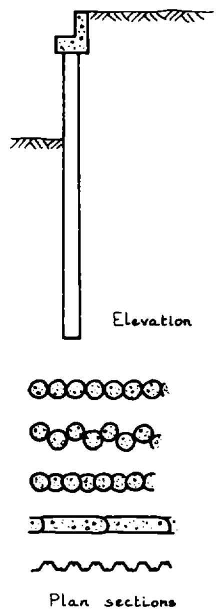
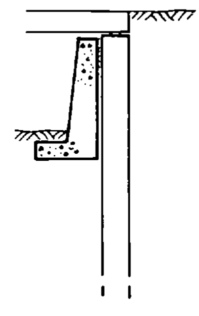
The stability of a retaining wall is usually calculated in terms of the forces acting on a vertical plane element of unit length. However economies can sometimes be made by considering the full structure as a single three dimensional body. A simple change of shape can improve stability as easily as an increase in weight. However, **intersections of component parts of the structure are possibly the most time consuming and expensive portions to construct and consequently are best kept to a minimum.** Reinforced concrete walls and bases are generally more satisfactory when designed with **more than the theoretically economic thickness.** Flexural stiffness is increased and steel fixing and concreting are much easier.

Type	Comment	Usage	Sketch
Mass	Mass concrete provides a simple form of construction. But the large quantity of concrete is relatively expensive. Sometimes when poured in several lifts the back face is stepped. Placing concrete under a sloping back shutter can be difficult unless men have access. The front face is often raked back to assist resultant load's line of action through central third of base. Lack of reinforcement may make it ideal for a small contract with little or no other steel fixing. Low strength concrete can be used with consequent benefit of less heat of hydration and cracking problems.	Common for walls up to 2 or 3 m. Rare above 6 m	
Semi-mass	Compromise between simplicity of mass concete and low material content of reinforced concrete. Can be the most economic form of construction if reinforcement details are very simple	Common	
Reinforced T	Most common form of construction. Often cheaper than mass concrete but relative merits are balanced. Minimum width of base is likely to be achieved with heel larger than toe. However in cutting situation a smaller heel is likely to be economic because of reduced excavation and working space, though sliding resistance is reduced. Complicated reinforcement details make construction slower than semi-mass. For single lift construction walls need to be wide enough for a man to stand between reinforcement to simplify construction and inspection.	Most common	
Counterfort	The complicated construction of counterforts and much formwork make this uneconomic for walls less than 10 to 12 metres in height. Can be economic for taller walls. If walls are too thin then construction in tall lifts is difficult. Compaction of fill between counterforts is difficult and not always satisfactory.	Seldom used except for very tall walls	

3.2(contd) Wall abutments

Type	Comment	Usage	Sketch
Abutment on fill	A deep foundation can often be achieved more economically by placing a small abutment on fill than by constructing a large abutment. In difficult ground conditions fill can be self-compacting, such as mass concrete, and placed without workmen entering the excavations. The abutment design can be standard with depth of fill determined on site (but the detail and range of possible changes need to be considered in design and shown on drawings since the stability of the combined structure must be checked). Speed and simplicity of construction can more than compensate for apparent cost of large quantity of material (See Section 4.1 concerning problems of excavation in difficult ground conditions.)	Used when appropriate	
Sloping abutments	Sloping abutments are sometimes used when less lateral clearance is required at the top compared with the bottom and for aesthetic reasons. Construction is much more difficult than for vertical walls, particularly if the bridge has any skew (when re-use of formwork is likely to be minimal). Temporary support of the wall during construction is much more satisfactory if it can bear on the footing in front.	Often used by some designers	
Cellular	A cellular abutment is an expensive complicated form of construction which is usually only used when very low even bearing pressures are needed and/or where piling can be avoided altogether. Cells are provided with positive drainage or filled with unsaturateable filling such as expanded polystyrene. Cellular abutments have been built with open backs with embankment slope inside. Compaction of the spill-through fill is then difficult to achieve. The design is discouraged by some designers who have used them before. Cellular abutments can also be filled with fill to provide a heavy anchorage for resisting horizontal forces. Internal structure can then be much simpler.	Rare, for special situations	 
Reinforced earth	Reinforced earth is appropriate for situations with embankments behind: but less likely to be suitable in cuttings or where ties interfere with boundaries and obstructions. The structure has a large tolerance for movement and so is ideal for sites with poor ground near the surface (but if poor stratum is deep then circular slip is not resisted by ties). Differential settlement between abutments and embankment should be smooth. A batter to the front face helps to hide forward movement of facing during construction.	No abutments have yet been built in UK, although a few have been built abroad.	

3.2 (contd) **Wall abutments**

Type	Comment	Usage	Sketch
Reinforced earth (contd)	The wall can be built overhand with a minimum working space. Damaged facing units (but not ties) if appropriately designed can in some situations be replaced during life without affecting stability. Caution is warranted in assessing the corrosion life of ties and fittings for abutments with long design lives. There is some concern about long-term erosion through gaps in some types of facing.		
Diaphragm, contiguous bored pile or sheet pile wall	These forms of structure are convenient for construction from ground level, when their high cost is compensated for by the speed of construction and lack of temporary works. They usually require some form of facing after excavation. If these abutments cannot stand as free cantilevers they can be propped by the deck or by a ground slab or ground beams, or tied back with anchors. Contiguous bored piles are often the more economic for small or isolated works. The piles can be in-line or staggered to increase the wall thickness and provide a tolerance for variation in pile size: or piles can be 'secant' for better water sealing, (reinforcement only in every other pile). Water seeping through needs to be collected. Diaphragm walling may be economic for large or repetitive works. It can be constructed with fair-faced precast elements which do not need subsequent covering. The wall may have to support a water table behind if adequate drainage cannot be provided. Steel sheet piling has generally been assumed by bridge designers to be unsuitable for permanent works with a long design life. However, recently it has been used to advantage for permanent abutments with protection/additional thickness provided against corrosion. In some cases sheet piles have been used for just retaining the earth while large piles behind support the vertical load. Economy is particularly evident when sheet piles are needed in any case for a temporary cofferdam. It is important not to use too small a section which cannot be driven into the ground.	For special situations.	
Large diameter piles with wall in front	Abutments have been constructed within railway embankments by boring two large diameter piles at each abutment and then sliding in a precast deck to rest on bearings on the piles. Subsequently a retaining wall of contiguous bored piles, or conventional construction in cofferdam is placed in front of the large diameter piles and then the dumpling is removed. Construction of the wall after placing deck is a difficult process.	For special situations.	

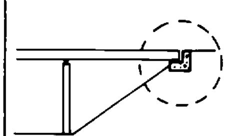
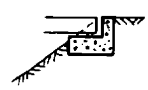
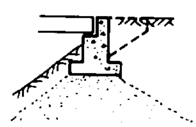
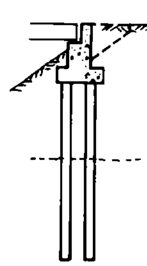
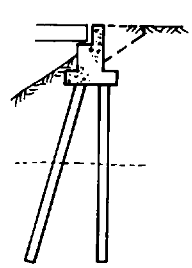
3.3 Open abutments

Open abutments, with the end spans of the deck bearing on seatings at the tops of embankments are often preferable and can look better than retaining wall types of abutment. The cost of the end span, pier and end support of a narrow bridge is likely to be less than that of a closed abutment with wing walls and retained fill. For a wide bridge the cost of wing walls is relatively less significant and the closed abutment is then likely to be more economic.

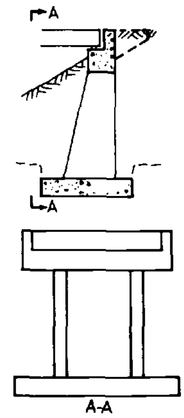
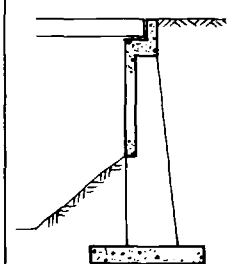
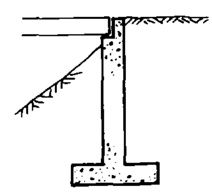
Open abutments are particularly suitable where the ground is not firm enough to support the heavy weight of a closed abutment or where horizontal forces must be kept to a minimum. Since the bank seat settles with the embankment the problems of settlement of backfill behind the abutment are reduced.

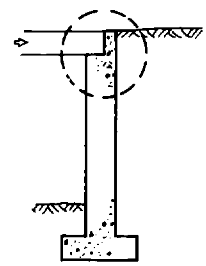
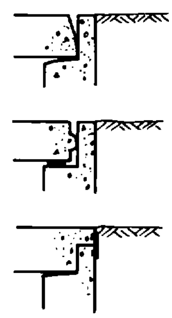
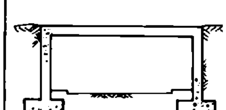
Open end spans above embankments can have the disadvantage that the contractor may have considerable difficulty in providing foundations for falsework for deck.

Access needs to be provided under end of span for maintenance of bearings and shelf drainage.

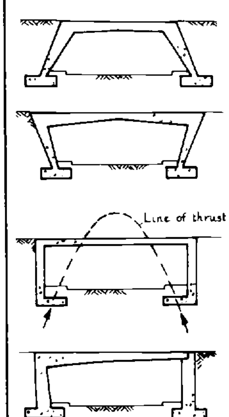
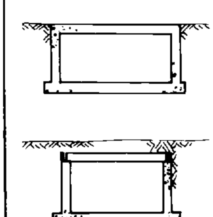
Type	Comment	Usage	Sketch
Bank seats	Bank seats provide simple economic structures in semi-mass or reinforced concrete. Ideal for bridges over cuttings where the bank seats can be founded on firm undisturbed ground. Some designers use them on embankments on specially compacted fill without piles, and much prefer them to skeleton abutments (but contractors dislike the need to complete earthworks before structure and complain at delays waiting for settlement). Settlement of fill is anticipated and bearing pressures are usually kept within range 100 to 200 kN/m^2: the lower figure being appropriate to better quality suitable material, while the higher figure is likely to require selected granular fill. The abutment with exposed bearing shelf is probably more economical than the buried type because of the reduced length of deck. The front edge of the footing is best kept well back from the slope because of erosion and frost action and because compaction of the fill on the slope is slight. Wing walls are constructed as cantilevers or on footings.	Often used in cuttings. Less common on embankments.	  
Bank seats on piles	Bank seats are placed on piles beside cuttings and on embankments when the ground or fill is not strong enough. However settlement of the embankment can subject the piles to downdrag settlement and loads. They are found more convenient if the piles can be placed at the same time as the other piles in the contract (usually at start). They can be uneconomic if the piles restrict the construction sequence or require remobilising plant. For the embankment situation the skeleton abutment (on or without piles) can be more economic and less restrictive on construction sequence, and also more easily designed to resist laterial loads. Installation of raking piles beside bank slopes can be impractical and expensive depending on the plant used. Bank seats have been restrained with deadman anchors while vertical piles support only vertical loads.	Used occasionally	 

3.3 (contd) Open abutments

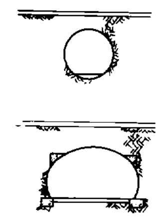
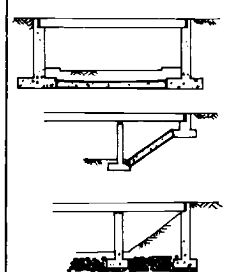
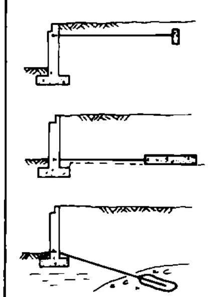
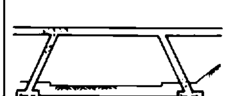
Type	Comment	Usage	Sketch
Buried skeleton	Buried skeleton abutments are more expensive and less popular than bank seats, though often preferred in embankments because of the lesser risk of settlement. They have advantage that construction can be independent of embankment, particularly if designed so that the bearing shelf can be placed before or after compaction of fill beneath. It can be difficult to compact fill properly between the legs, and in time the fill behind can settle into the poorly compacted zone.	Ofted used in embankments	
	Partial skeleton abutments are occasionally used for very tall abutments or where foundations are very deep. They can be very expensive if construction is not simple. If face wall is more than half total depth a buried wall is likely to be more satisfactory.	Rare	
Buried wall	A simple buried wall can be more economic than a complicated skeleton abutment but the cost benefit is unlikely to be reflected in tenders. It has the benefit that proper compaction of the backfill is more likely to be achieved.	Little used	

Type	Comment	Usage	Sketch
Strutted or vertical beam abutments	Many small bridges of spans up to 15 m (and a few of greater span) have been constructed with the deck acting as a strut to the top of abutments. With the lateral support provided by the deck the substructure and foundations can be narrower and simpler than for free-standing cantilever walls. In special situations such as where installation of raking piles is difficult or where abutments are abnormally tall, the lateral support is particularly useful. Usually the walls are monolithic with the footings but some have been designed with hinges at top and bottom, so forming a mechanism which is only stabilised by the back fill. Various simple details for deck seating have been used with bearings as simple as sheets of felt or lead. However, care is taken that the reaction and thrust of the deck act at design points of contact and do not cause spalling elsewhere. Some decks have been designed with details to avoid need for abutment shelf drainage. Strutting of abutments is not recommended for skews greater than 20° but effect of skew depends on ratio of length/width. Strutted abutments can have some serious shortcomings particularly for bridges at the larger end of the range. If the structure is not stable until complete with backfill expensive restrictions on construction may be necessary. The substructures might have to be strutted and braced, or else the deck might need to be placed prior to backfilling which then has to be carried out simultaneously behind both abutments. Replacement of the deck or bearings (whose lives have been found in past to be less than the substructures) could be very difficult. Earth pressures are likely to equal or exceed at-rest pressure and since calculations are imprecise sections should be robust. See Section 6.1 for discussion on design pressures. For larger spans earth pressures are likely to be much greater than at-rest pressures due to contraction/expansion cycles of the deck.	Often used for small bridges, but popularity decreasing. Very seldom used for spans exceeding 20 m.	 
Portal frames	Portal frames are used for the same advantages as strutted abutments, but for a much larger range of spans. It is possible to use a more slender deck than for a simply supported span, but the portal frames are generally more expensive. It is also necessary for the foundations of portals to have a greater resistance against sliding since horizontal movement of the footings can overstress the top corners of the portal.	Occasionally used.	

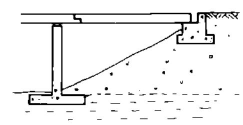
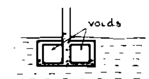
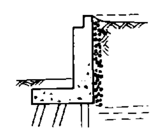
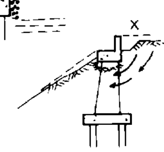
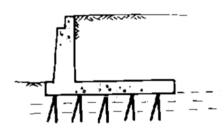
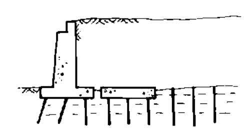
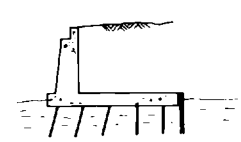
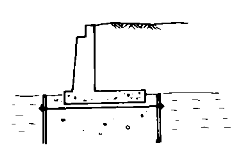
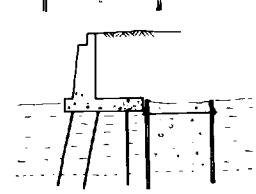
3.4 (contd) Strutted, portal and box structures

Type	Comment	Usage	Sketch
Portal frames (contd)	Some portals are designed with hinges at the bottom of the legs to improve the predictability of the structure. However the construction sequence is considerably more complicated when the legs cannot stand as free-standing walls. (It is also necessary to protect the hinges from vehicle impact.) The effective stiffness of the footings and stiffening by the backfill are difficult to predict and consequently many portals are designed for the worst conditions of both 'free' and 'fixed' feet. Differences in reinforcement are usually small. It is also generally difficult to predict the intensity of earth pressures, and the design has to be checked for very different extreme values. Some portals have been designed with a soft filler down the backs of the legs, and a few have even been provided with separate retaining walls behind, to separate the uncertainties of earth pressure from predictions of structural action: it is unlikely that such expedients are economic or significantly improve the accuracy of calculations. Some portal frames have reputedly had the legs sloped back to reduce the earth pressures: again it is unlikely that this had achieved economy since the span and bending moments have been increased. The footings of some portals have been placed in front of the legs to move the effective points of support inwards and so reduce span bending moments.		
Box structures	Box structures are often economic for small spans. Accordingly they have been used frequently for culverts and when the toes of cantilever abutments would be close together. They can also be economic for much larger structures on poor ground, particularly when they avoid the need for piles. In addition to economy they have the advantage that differential settlement between structure and embankment is likely to be less than when structure is on piles. The cost of the contractor's falsework for the deck is reduced because the lower slab can be used as a firm foundation. The structure can be constructed as an open U with simply supported deck. This is particularly appropriate when a pre-cast deck is used. For small structures in particular, it is important that the design of sections and reinforcement should aim at simplicity of construction rather than economy of materials. Box structures are occasionally installed under busy roads or railways by the method of thrust boring. The high construction cost can be justified by the avoidance of disturbance to traffic.	Often used for small structures	

3.4 (contd) Strutted, portal and box structures

Type	Comment	Usage	Sketch
Flexible corrugated metal structures	Flexible corrugated metal structures of circular or derivative cross-section have been used for many culverts and small underpasses, and a few with spans up to 12 m. Greater spans are possible. These structures have the advantages of low material cost and speed of construction. (If necessary one can be pre-assembled alongside an existing embankment and installed during a weekend possession). They also have the advantages that embankment construction is continuous over the top and settlement can occur without the discontinuities that are common between embankment and rigid structures.	Often used for culverts up to 6 m span	
Ground struts	Sometimes on very poor ground it is convenient to prevent lateral movement of abutments by ground struts or slabs. Raking piles can be avoided. The struts are made flexible if vertical settlement beneath the under-road is anticipated: even rock-fill blankets have been used effectively as struts. The disadvantages of ground struts are that they interfere with services beneath the under-road and there can be a risk of their being excavated by mistake in the future.	Used only for exceptional reasons	
Anchors and ties	Ground anchors are often not practical for retaining walls because of the problems of land easements, but they can be practical for abutments. They are treated with suspicion by many designers particularly for cohesive soils and where ground water might affect grouting.		
	When ties are used for deadman anchors or for tying retaining walls back-to-back provision has to be made for relative settlement of wall and fill, and consideration given to construction details which do not interfere with satisfactory and simple placing of fill. It is also necessary to check that the structure can tolerate the movement necessary to mobilise the anchor resistance.		
	In areas of mining subsidence the foundations of some bridges are tied together to prevent separation. However the effects of ground strain on ties, and anchors need checking. Ties between foundations are occasionally used on arch structures to prevent separation if the ground is not stiff enough: it is sometimes questionable whether the arch is then the correct choice of structure.		
	Anchors and ties, like ground struts, can be vulnerable to accidental damage during excavation for services at a later date.		

3.5 Abutments above highly compressible strata

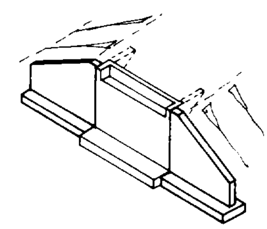
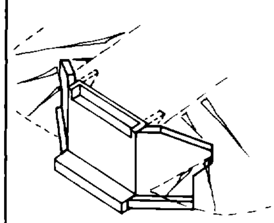
Type	Comment	Usage	Sketch
	Abutments above highly compressible strata are usually designed to accommodate or prevent large vertical and horizontal movements and possibly a deep seated slip failure. The problem is minimised when the embankments can be built well in advance of the bridge, possibly with surcharge, so that much of the movement occurs prior to construction of structures. If substantial movement cannot be avoided then the deck and substructures are usually of lightweight design and able to accommodate vertical and horizontal movement. Tilting of substructures is minimised when the bearing pressures are uniform under footings. Piles are sometimes used only to provide vertical support while being allowed to move laterally with the embankment. Buoyant foundations with or without cellular substructures are very occasionally used if little or no additional load can be carried by the ground but the cells may need to be very large. If movement of the bridge cannot be tolerated the substructures are designed to be independent of or resist the movement of the ground due to embankment loading. The abutments are sometimes designed with a smooth back face (possibly with single size gravel fill interface) so that the embankment downdrag is minimised. Ledges, such as under skeleton abutment shelves, are avoided since voids form underneath into which the fill behind collapses. Piles raked backwards beneath a settling embankment also have to be avoided since they are likely to be bent. The lateral drag effects of ground flowing through piles is sometimes reduced by placing piles within outer cases with a clearance (possibly filled with soft filler). Abutments have been designed with heels extended to support the embankment behind and to provide a longer flow path for the soil underneath. On some very soft ground it has been necessary to design abutments like quays with a sheet pile cut-off wall. Other solutions have involved bulk replacement of alluvium within cofferdams. Embankments have also on a few occasions been supported on seperate relieving platforms on piles, or partially supported by uncapped piles. In a few special situations lateral spreading of embankments has been reduced by mattresses and ties. **It is important to consider the overall stability of the structure and embankment** in addition to overturning, bearing and sliding.	Used on exceptionally poor ground.	        

3.6 Wing walls

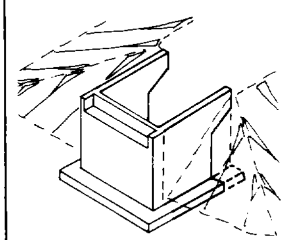
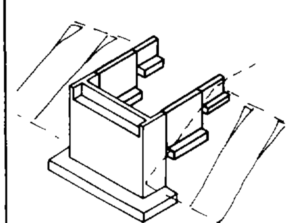
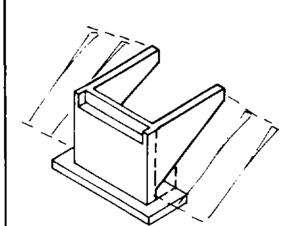
The cost of wing walls can be a significant fraction of the total cost of substructures, particularly for narrow bridges. They can also have a significant effect on appearances, which can be ruined by a marginal saving. The choice of geometry is usually controlled by the topography of the site, construction restrictions and construction sequence. **If possible it is advantageous if the wing walls can be designed so that the contractor can construct them before or after the deck.** Often they can be structurally independent of the abutment, in which case the joints between need to be designed to permit significant relative movement with generous features to hide relative tilts. Differential settlement with abutment can be large due to differences in bearing pressure.

Wing walls cantilevered back from abutment walls, or attached to strutted abutments or portal frames, are **partially restrained against movement and can be subjected to earth pressures greater than active.** Several abutments have suffered cracking at the joints with wing walls, and consequently many engineers give such abutments robust sections of conservative design to transfer the complex loads.

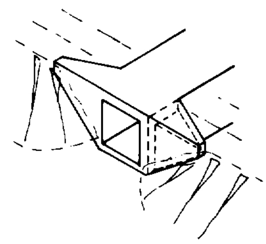
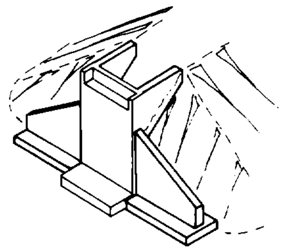
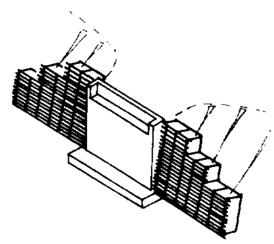
Wing walls are often constructed with a batter to the front face and the section tapering towards the top. When the height also decreases towards the ends access for construction can be complicated by the front and back shutters impinging unless they are specially cut to fit particular walls. To achieve economy through re-use of formwork it may be necessary to build the wall with constant thickness. The base dimensions can often be designed constant without loss of economy, if the wall is designed as a whole and not just on a unit length basis. Construction is also usually much simpler if the wing wall bases are at the same level as the abutments.

Type	Comment	Usage	Sketch
Wing walls parallel to abutment	Wing walls parallel to the abutment face are in general not as economic in terms of materials as angled walls but are usually the simplest type of walls to build. They have advantages that abutment can be built quickly with skew only affecting details of the abutment shelf. For bridges constructed in existing embankments they are likely to cause least disturbance to embankment. Back-fill can be properly compacted with roller traversing from side to side. To ensure compaction at sides of carriageway it is usually necessary to have additional horizontal length at top of wall. They are often designed monolithic with abutment. Sometimes the parapet is continued a short distance behind the abutments and is supported on short cantilevers off the back of the abutment. These cantilevers are usually most conveniently constructed in trenches after completion of backfilling.	Common	
Wing walls at angle to abutment face	Generally the most economic wing walls in terms of materials are on lines bisecting angles between over-road and under-road. The extreme ends can be cantilevers if the walls are of reinforced or semi-mass concrete construction. But small cantilevers can be more trouble than they are worth. The wing walls can be totally cantilevered off the abutment wall if they are not too long and the abutment is robust. Horizontal tops to walls and parapet cantilever detail are the same as for walls parallel to abutment face.	Common	

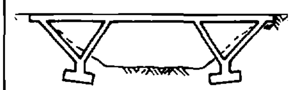
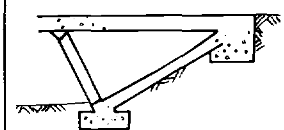
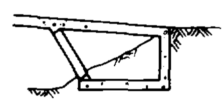
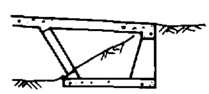
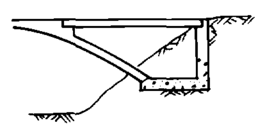
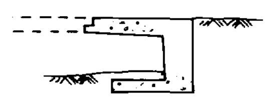
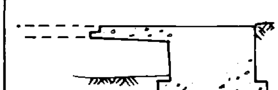
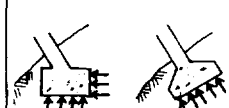
3.6 (contd) Wing walls

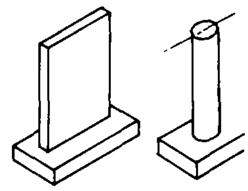
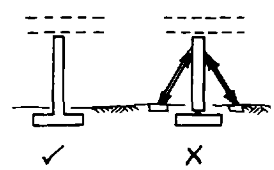
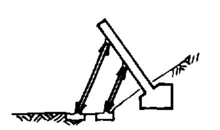
Type	Comment	Usage	Sketch
Wing walls parallel to over-road	Less economic than angled walls but continue line of deck and provide support for parapets. Such wing walls may not be appropriate when they or their excavations impinge on adjacent structures, services or traffic diversion. They can cause sight line problems where roads merge near the end of a bridge. Also if road is curved the continuation of a long bridge chord line can restrict the road curvature. In addition they have disadvantages that proper compaction of backfill in the corners is seldom achieved and that forming of exposed edges of skew abutments requires special care. When the wing walls are structurally attached to the abutment advantage can be taken of the stability of the box structure. On narrow bridges or where used at same time as counterforts the abutment wall can be designed as spanning transversely. If such a box structure is founded on piles, including rakers, care needs to be taken that the displacement of the piles under the wing walls is not incompatible with the displacements of the piles under the abutment. It is sometimes found best to have only vertical and forward raking piles, or to separate structurally wing walls if their piles rake sideways.	Common	
	When construction is in a cutting it is sometimes economic to split the wing walls in parts founded at different levels. However this is not appropriate if the founding material cannot stand unsupported in steps, and the economy is questionable if each height of wall requires a new shutter. Occasionally when an abutment is supported on piles short lengths of the wing walls are cantilevered off it while the remainder of the wing walls are independent and supported on footings.	Uncommon	
Cantilever wing walls parallel to over-road	An abutment with large cantilever wing walls looks similar, when complete, to the abutment with wing walls on footings parallel to the over-road. It has the advantage that the whole structure is supported on a single footing which can settle as a single body. It has the same problems during construction, but in addition to poor compaction in the corners it has the disadvantage that the compaction is difficult against the cantilevers because of flow of material underneath. It has on occasions been found necessary to construct a mass concrete wall underneath to provide a firm boundary wall for compaction against. Voids or filler have sometimes been left under the cantilevers to permit backward tilting: but	Common often with sloping abutments	

3.6 (contd) Wing walls

Type	Comment	Usage	Sketch
Cantilever wing walls parallel to over-road (contd)	they are generally not necessary since such tilting usually accompanies even greater settlement of the embankment. The abutment and cantilever wing walls need to be robust and stiff to prevent outward bending.		
Cantilever wing walls to box and portal structures	When cantilever wing walls are attached to a portal frame or box structure the stiffness of the wing walls affects the distribution of moments at the edges of the abutment wall and deck slab. Care has to be taken that the wall has the appopriate reinforcement to transmit the forces attracted by this stiffness. The wing walls of box and portal structures can be thickened up at the top to cantilever horizontally from the deck which is a very strong member in-plane.		
Double wing walls for high abutments	A high embankment is sometimes most economically retained by an abutment with two sets of wing walls. The upper pair are cantilevers parallel to the over-road and retain fill which spills round to the top of the lower wing walls, which are parallel to or angled to the abutment face.	Rare	
Crib walls	In general wing walls are of the same form of construction as the abutments,ie mass, semi-mass or reinforced concrete. The latter is necessary for the more complicated structures with large cantilevers. Reinforced earth or sheet piling can sometimes be used for wing walls to a concrete abutment. Crib walling is also suitable for wing wall construction and can be economic. (It is not thought suitable for abutments.) Its rustic appearance makes it more acceptable in rural situations than urban situations. Construction requires careful supervision since the appearance can be unsatisfactory if setting out and element tolerances are not carefully controlled. Cracking and spalling of elements can occur if elements are laid with stress concentrations. The backfill needs to be drained and crib filling needs careful selection since some granular materials flow out.		

3.7 Other substructures

Type	Comment	Usage	Sketch
	The diagrams illustrate some other substructures which have been used for particular situations. They are usually not the most economic solution, though they can sometimes be justified for aesthetic reasons. Bridges with V supports can be difficult to build and consequently expensive, particularly when they have only two footings. In addition to needing very complicated temporary works control of fill at the ends has been a problem. However with simple footings and the bank slope struts poured against the embankment construction need not be so complicated. (Some designers make the strut concave to maintain contact with the ground). V supports have been used for cantilever and drop-in span decks. End-fixity has also been obtained by abutments with large toes to footings and by massive anchor blocks. Arches and arching structures with raking piles ideally require very firm foundations. Some have been built on less suitable ground with large buried substructures to mobilise or balance horizontal reactions. The decks have to be analysed for the effects of lateral movements of foundations. The contract documents for these complicated structures benefit from the inclusion of a construction sequence drawing, with detailed attention paid to interference of falsework for raking piers and deck, and to striking sequence. Otherwise serious differences of opinion are likely during the contract. Footings transmitting inclined loads to the ground are much easier to construct with horizontal and vertical faces than with bearing face normal to line of thrust.	Rare, special cases	       

Type	Comment	Usage	Sketch
	The simplest piers to construct, and most economic, are usually vertical with uniform rectangular or circular section and fixed at the bottom to the foundations. Inspection and placing of concrete is facilitated if the pier is large enough for a man to climb down inside. Standardisation of details and reinforcement in piers of a bridge or several bridges can lead to overall economy. Shaping of piers can have aesthetic advantages. Complicated geometry, such as variation in section, is likely to increase cost unless considerable re-use of shutters is possible without restrictions on order of construction.	Common	
	Some designers place little confidence on single columns founded on single large diameter piles because of the serious consequences of setting out errors. A pile group of three or more smaller piles is preferred. In the design of piers on piles or footings the rotational stiffness of the foundations may have to be calculated to determine the effective height of the pier. Narrow footings often have a sufficiently low stiffness to behave like pins, at the same time being wide enough for stability during construction. Bearings at the bottoms of the piers are generally not popular unless they can be made rigid during construction. Construction above the bearing is complicated and subsequent temporary propping of the piers complicates construction of the superstructure. They also have to be protected against vehicle impact.		
	Raking piers on simple foundations require firm ground to resist the arching forces. All stages of construction are considerably more difficult and expensive than for vertical piers and subsequent temporary propping complicates construction of the superstructure. Their higher costs can often only be justified for high bridges where the rake significantly reduces the span.		

CHAPTER 4

CHOICE OF FOUNDATION

4.1 Choice of footings or piles

Many designers try to use spread footings whenever they can. They try to avoid piles because of the considerable expense and also because of the greater risk of encountering unforeseen and expensive technical and contractual problems during construction. Even in the marginal situation where identifiable costs favour a piled foundation, many designers go to considerable trouble to design deck and substructures to work on spread footings with low bearing pressure and possibly with significant movements. The amount of differential settlement that is considered acceptable is discussed in Section 4.2 and 5.5.

The following tables list some of the ground conditions and construction considerations for which footings or piles have been found to be advantageous. **For sites with variable strata the designers often have to work through a range of depths and sizes of foundation** to identify the most economic with acceptable settlement characteristics. The choice is not always straightforward. Sometimes it is necessary to choose between a foundation which can be constructed to a verifiable quality without disturbing the ground but with a slightly unsatisfactory factor of safety, and one at greater depth with a higher theoretical factor of safety but with less certainty abouts its quality. **The decision is also affected by the overall nature and scale of the contract**, and it may not be economic or practical to select what might appear to be the most appropriate foundation for each bridge if viewed in isolation. None of the generalisations following are correct for every bridge.

Ground	Footings preferred	Piles preferred
Stiff clay, or medium dense dry sand or gravel	Footings are often found to be straightforward and appropriate for bridges for reasons of cost, reliability and ease of construction.	Piles may have to be used where very heavy concentrated loads have to be transmitted to ground.
Firm clays or loose dry gravels or sand and gravel	Some designers prefer to use special structures on footings with bearing pressures as low as 100 to 150 kN/m^2 possibly with significant settlement (most usually occurs during construction).	Piles often adopted. Settlement is less than that of footings, but often not as much less as anticipated (settlement of a pile group is often as much as $^1/_2$ or $^1/_3$ of that of a footing and takes place rapidly.)
Stiff stratum at moderate depth with deep water table	Spread footings at high level supported on mass concrete or granular fill, with formation on firm stratum, have been found by several designers to be less expensive than deep footings.	
High water table in permeable ground		Many designers and contractors advise against large excavations below the water table. Dewatering can increase the cost of excavation by as much as tenfold, and claims for unforeseen conditions are likely. Driven or cased bored piles are likely to be appropriate, but installation can also be difficult.
Stiff ground overlying soft ground eg gravel over clay	Shallow footings can be designed to make use of load spreading quality of the stiff ground. Piles can give rise to driving or boring problems and by concentrating load above or in clay can cause larger settlements.	

Ground considerations

Ground	Footings preferred	Piles preferred
Soft silty clays, peat, and uncompacted fills	Spread footings can be used on fill if ground can be preconsolidated by surcharge. Buoyant/compensated foundations can be more economic than piles, if firm stratum is very deep, and if excavation and construction are not too complicated.	Piles are usually adopted. Measures have sometimes had to be taken to prevent damage from lateral loading due to adjacent embankment. (See Section 3.5).
Interbedded sand and silt layers	Settlement of footings has often been overestimated. Excavation has been hindered by water in layers, but installation of bored piles has also given many problems.	
Loose sands increasing in strength with depth	Improving existing ground (Section 4.3) has sometimes been found cheaper than piles. But the difficulties of prediction and monitoring the performance of the special techniques can make them more expensive and troublesome than a structural solution.	Driven piles compact the soil and can provide high load capacity with minimum settlement. Settlements of footings can be larger than predicted if not artificially compacted. Single size sands can be impossible to compact.
Chalk	Unless chalk is at depth spread footings are generally satisfactory. In the past piles were used more often. But it has been found that even soft chalk consolidates quickly under load (usually during construction) and material which looks in poor condition can take higher loads than anticipated.	Where upper surface is at unpredictable depth due to swallow holes, piles may be economic because they can be driven to different depths without delays in contract. Chalk softens during driving or boring and recovers some strength during following weeks. It can be worth delaying tests and testing piles of different lengths to determine the optimum.
Unpredictable and impenetrable ground, such as boulders or rock with clay matrix	Both excavation for footings and installation of piles can be very difficult, with predictions of movement unreliable. Shallow footings can be the more economic even with conservative estimates of bearing capacity and settlement.	Steel H piles have been used and driven either to deep firm stratum or to sufficient depths to mobilise adequate friction.
Compacted fill	Well compacted deposits of suitable or selected fill are generally found to be as good as, if not better than, natural deposits of the same material.	
Steeply dipping rock substratum	Mass concrete fill can grip stepped interfaces where piles might glance off.	If stratum is at depth then precast concrete piles with rock shoes or steel piles may have to be used.
Deep seated instability and mining subsidence	Footings supporting an articulated/flexible structure are likely to be appropriate, since their performance can be anticipated with greater confidence than other foundations. Piles are not effective in preventing deep seated instability. They are also likely to be very expensive if they have to resist or be protected from loading by moving ground. Furthermore it can be important not to 'dowel' the structure into ground subjected to large strains. (See Section 2.3).	
Over rivers and estuaries		Piles are usually preferred irrespective of ground conditions.

4.1 (contd) Choice of footings or piles
Construction considerations

Construction consideration	Footings preferred	Piles preferred
Cost	A large footing can be justified if it avoids the need to use piles. For the average motorway bridge the tender price for piles is about the same as the price of the deck or the price of the substructures. If the pile depth exceeds 30 m their price can be more than the combined price of deck and substructures.	If piles have to be used savings can sometimes be obtained by raising the pile cap to ground level.
Delays and consequential costs	Some designers try to avoid piling except when absolutely unavoidable primarily to avoid repeating unfortunate experience in the past when piling involved serious contractual problems and unforeseen consequential costs. The consequential costs can be large because the piling delays, being at the start of the contract, can affect much subsequent construction. But it must be remembered that to a certain extent such problems result from the unsatisfactory and unpredictable nature of the ground which motivated the choice of piles in the first place.	
Site restrictions	Spread footings can usually be constructed without requiring special access for heavy plant. See Section 2.2 concerning influences of site on construction.	Boundary constraints, such as adjacent buildings, steep slopes, railways and rivers can make construction of spread footings difficult.
Quality control	Quality of ground and foundation can be checked easily for footings. In contrast an engineer cannot inspect soil in-situ beside small bored piles or piles cast under water or bentonite. He cannot see what happens to piles on removal of casing, and it is probable that a number of piles constructed with ground water problems are defective. The quality of piles is very dependent on quality of the piling gang and supervision.	Driven piles have advantages where ground water causes difficulty with bored piles.
Modification	Simple mass and reinforced concrete footings can be modified to suit excavated ground conditions with minimal delays. In contrast modifications to pile layout or type can have very expensive contractual consequences.	Pile lengths can be modified to suit uneven depth of substrata.
Alternative design	Some designers consider it a disadvantage of piles that they are sometimes forced to accept alternative designs for piles for apparent economic benefit while risking worse contractual problems and reliability.	
Construction programme	Contractors have much greater flexibility in the order and timing of construction of footings than of piles. They are often restricted as a result of their contract with the subcontractor in the number of rigs than can be used for piling and by the high cost of remobilising piling plant.	

4.2 Differential settlement criteria

The choice between footings and piles for foundations is sometimes controlled by what differential settlement can be tolerated by the bridge deck. Criteria for acceptable differential settlement differ enormously between design offices, particularly for foundations for continuous structures.

Deck	Criteria
Continuous deck	Some designers aim to design a continuous deck for a differential settlement of about 1 in 800 (25 mm on 20 m span). But others expect differential settlement to be controlled to as little as 1 in 4000 (5 mm on 20 m span). The deck designed for 1 in 800 is likely to be slightly more expensive since differential settlement has to be considered as a primary load, comparable to live load and temperature. However, this cost can be very small in comparison with the cost of providing the very stiff foundations required for 1 in 4000, which are likely to require piles unless on hard rock. In addition to cost, the very stiff foundations have the disadvantage that the structure does not move with adjacent earthworks to the detriment of the smooth ride of the road.
Simply supported decks	Foundations for simply supported structures are frequently designed for differential settlements of the order of 1 in 800. 1 in 200 (which is just visible) has been necessary on occasions. Then careful attention must be given to the effects of settlement on drainage alignment and headroom. In areas of mining subsidence much larger movements have to be anticipated. (See Section 2.3). The determination of acceptable bearing pressures to control settlement is discussed in Section 5.4, and the prediction of settlement in Section 5.5.

4.3 Ground improvement

If the upper strata at a bridge site are weak, then the possibility of strengthening the ground may prove more economic than founding on lower, firmer strata. The better known techniques of ground improvement are summarised below. The limitations of some of the techniques are not clearly understood and the prediction and monitoring of their effectiveness can be difficult. Preparation of the specification can also be difficult and an advance trial may be required in addition to test loading.

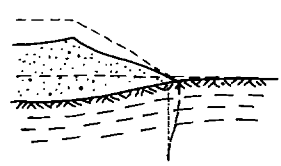
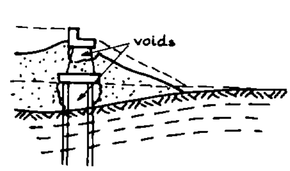
Technique	Comment
Surcharge	Surcharging with embankment has been used on several occasions to speed up settlement and increase the bearing capacity of soft clays. It is usually of most benefit to end supports comprising bank seats which spread their load through the embankment and do not subject the soft sub-strata to concentrated loading. It has the disadvantage that the time required for settlement to occur may seriously strain the contract programme or even warrant an advanced contract for the embankment, which is often not pratical. However experience in many British conditions has indicated that settlement often occurs more quickly than expected and can be virtually complete by the end of the contract.
Sand-drains	Sand-drains have been found effective on occasions for accelerating settlement. However they have been used unnecessarily on other occasions when the natural drainage of the ground has been much more effective than originally estimated from laboratory tests.
Vibrocompaction	Vibrocompaction has been used satisfactorily for improving the bearing capacity of granular materials, such as loose sand, below concentrated foundations as well as embankments. Bearing pressures of up to 300 to 400 kN/m^2 have proved practical on vibrocompacted gravel. (The technique has been used unnecessarily and with no effect when the density of the existing ground has been underestimated).
Vibro replacement	Loose sand has been consolidated by vibration and further strengthened by stone columns inserted by vibro replacement. The effectiveness of the technique for soft clays has not yet been established.
Dynamic compaction	Dynamic compaction, using a dropping weight, has been used for compacting a variety of soils. The range of application and effectiveness is not clearly understood.
Grouting	Grouting has been used frequently for stabilising fissured rock, particularly in regions of old mineworking. But the techniques can be very costly and unpredictable if the dip and fissures are not advantageous. Other soils are very seldom ideal for grouting and many engineers are opposed to it because of the difficulty of making the grout go where it should and not go where it shouldn't. Much damage can be done by using excessive pressure.
Reference with lists of other references	Institution of Civil Engineers (1976). Ground treatment by deep compaction. 154 pp.

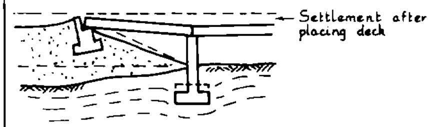
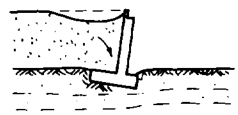
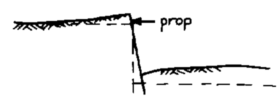
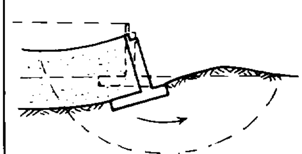
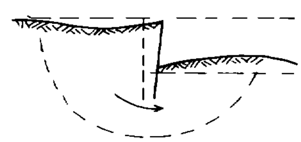
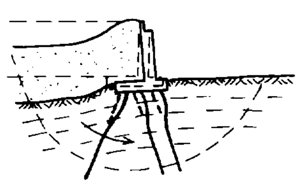
CHAPTER 5

SPREAD FOOTINGS

5.1 Global behaviour

Many designers and geotechnical engineers encourage the practice of considering the global behaviour of the complete bridge and adjacent earthworks before starting the design and detailed calculations for individual foundations. The following sketches illustrate some of the possible global movements that can occur around bridges. The true behaviour can be very different from the simplifying assumptions of calculations. For example, abutments are considered in calculations to overturn forwards although they frequently settle backwards due to settlement of the embankment behind. The global movements due to earthworks can be much greater than the settlement calculated for an individual foundation. It is often found difficult to make precise calculations and sometimes only broad judgements are possible. However several designers mentioned the benefit of making separate calculations for favourable and unfavourable assumptions so as to estimate possible limits of behaviour between which the actual performance should be.

Comment	Surcharge situation	Excavation situation
The 'dish' of settlement induced by an embankment starts outside the toe and does not reach the maximum until beyond the brow. The lateral movement of the ground is generally much less than the vertical, unless the upper stratum is so soft that it 'flows'. Because the zone influenced by an embankment is so much greater than that influenced by individual footings, the settlement due to an embankment on soft ground can be much greater than that due to footings even though the bearing pressure may be less.		
The zone of heave or rebound due to removal of ground in a cutting is the reflection of the settlement due to an embankment. Is is probably of smaller magnitude, but even with rock it can be significant for a stiff structure such as an arch.		
If the brige is supported on rigid foundations when the adjacent embankment settles then pile cap and piles can be subjected to heavy loads from embankment arching and downdrag as well as from lateral movements. Also voids can form beneath ledges and later the embankment collapses locally into the voids. The global behaviour of pile foundations is discussed in greater detail in Section 7.1.		

Comment	Surcharge situation	Excavation situation
The construction sequence has serious effects on differential settlement experienced by the deck. The end supports can settle more than piers during construction due to embankment settlement, but later the piers can settle more because most of their load results from the deck which is constructed last.		
A retaining wall supporting an embankment on stiff ground is likely to lean forwards, or slide forwards. The movements behind are predominantly due to slip of the active wedge.		
In a strutted excavation the rebound due to unloading can exceed other movements. The ground behind the wall can move down during excavation and later move up over an area extending about 2 times the depth.		
A retaining wall supporting an embankment on compressible ground can lean backwards as it settles due to settlement of embankment behind, and the ground in front can heave due to the circular strain of ground.		
The zone of strain round a cantilever wall supporting an excavation can mirror that of the retained embankment with the heave accentuated by circular movements.		
The piles supporting a retaining wall can also be thought of as resisting the circular motion of the embankment and ground. If the ground is soft the piles can be subjected to high lateral pressures due to the soil movement as well as vertical loads due to the arching of the settling embankment.		

5.2 Loading on substructures and foundations

The table below lists the various design loadings for substructures mentioned by designers during the survey. The loadings are not considered to act all at the same time, and various combinations are identified during design. Many of the loads are the same as those specified for the design of bridge structures in British Standard BS 153 and Department of Transport Memorandum BE 1/77 (see Appendix D). These will eventually be superseded by British Standard BS 5400. Care has to be taken not to confuse the limit state design procedures of BS 5400 with the traditional permissible stress procedures currently used in foundation design in CP2004 and CP2.

The relative importance of loads of different duration depends on the time-dependent behaviour of the supporting soil, as discussed in Section 5.3. Sections 5.4 and 5.5 describe the different methods designers have to adopt for calculation of bearing capacity and settlement to take account of the different behaviours of non-cohesive and cohesive soils under 'dead' loads and 'live' loads.

Loading	Comment
Dead load	(i) self weight of substructure; (ii) weight of fill supported by foundation; (iii) dead load and superimposed load of superstructure.
Earth pressure	Continuously acting on completed substructure but likely to fluctuate in intensity with substructure movement, vibration, water table and so on. See Section 6.1 concerning relationship between earth pressure and movement and restraint of substructure.
Differential settlement	Differential settlement of embankment relative to substructure transfers load to substructure and foundations. Differential movements of supports of indeterminate bridge change reactions.
Hydraulic	Piers in rivers are subjected to lateral forces from change of direction of water flow. Forces increase in times of flood.
Flood	Drawdown condition following flooding of embankment. Also flooding of retaining wall drainage membrane from behind or above.
Creep and shrinkage	Creep and shrinkage movements of superstructure can affect reactions and thrusts on substructure.
Temperature	Temperature changes in superstructure can alter reactions and thrusts or apply displacement to supports (which can in turn alter earth pressures). Temperature changes in parts of substructures and ground affect the forces and differential movement between them. Information on temperatures is in the standards listed above.
Traffice on bridge	Detailed information is included in standards listed above for loading from: (i) vertical gravitational load incorporating impact allowance; (ii) centrifugal load acting radially; (iii) longitudinal loads, which can act towards or away from substructure.
Traffic on abutment	Live load surcharges to represent vertical traffic loading are given in the standards listed above. Horizontal loads due to braking and traction can act on substructure through fill and pavement. The designers who mentioned this did not find CP2 or other references much help and had to rely on first principles.
Wind	Detailed information on wind loads on superstructure and piers is included in the standards listed above.
Impact	Piers are vulnerable to impact from vehicles, trains and river craft. The standards listed above provide guidance for piers alongside highways. Massive abutments and foundations are much less vulnerable and not likely to fail, though some displacement can occur from severe impact.
Exceptional	Snow, ice packs, earthquake and so on.
Construction	Combinations of any of the above loads can be critical for incomplete substructures during stages of construction together with loads due to temporary works, stored materials, moving loads, and possibly also accompanied by reduction of support.

5.3 Undrained and drained behaviour

Many engineers are uncertain when it is appropriate or necessary to consider the stability of a foundation separately under undrained and drained conditions.

When saturated soil is subjected to a sudden increase (or decrease) in load the particle packing cannot change suddenly due to the presence of porewater and the soil strength cannot change from its initial undrained value. Over a period of time, which depends on the ground permeability, the soil consolidates (or swells) and the strength changes to the drained value appropriate to the degree of compression. In any construction situation with partially saturated soils and loads of differing durations, the soil condition lies between the extremes of undrained and drained. However for design purposes it is convenient to check the stability for the worse of idealised undrained and drained conditions.

Foundations are usually designed by considering all loads as permanent. For permeable materials, such as gravels, this practice is generally realistic because soil test strengths and the dominating loads are virtually all long term in comparison with the time necessary for the dissipation of excess pore pressures in the deposit. For materials of lower permeability the practice is usually conservative because under conditions of increased loading the undrained soil strength is less than the drained strength after consolidation and dissipation of excess pore pressure. However there are conditions some of which are listed below under which the practice can be unsafe. and there are also conditions under which a fully drained effective stress analysis can provide a more realistic explanation of the behaviour and stability of the ground which leads to an economy in design.

The subject is discussed in detail in most textbooks, cf Terzaghi and Peck, Articles 17 and 18. But unfortunately many naturally occuring soil desposits have heterogeneous and complex characteristics and structure and it is often virtually impossible to predict the response of the deposit to change of load or the time for dissipation of excess pore pressure from potential failure zones. The application of concepts of undrained and drained behaviour is then far from clear.

The table below indicates some idealised conditions and conditions under which added caution is warranted or benefit might be obtained from a drained effective stress analysis.

Ground	Undrained/'short-term'	Drained/'long-term'
'Sand gravel'	Impact	Virtually all loads except impact.
'Clay'	Impact, traffic, wind and short-term erection. Temperature, flood and hydraulic fluctuation on slow draining deposits.	Dead load, earth pressure, differential settlement, hydraulic and long-term erection. Temperature, flood and hydraulic fluctuation on quick draining deposits. Permeable bands in many deposits enable much of pore pressure dissipation to occur during construction. Strength increases with consolidation.
Removal of overburden		Removal of overburden permits swelling and reduction of effective stresses so that **drained strength is less than undrained.**
Fissured over-consolidated clays		**Slip can occur on polished slickenside** with low 'residual' angle of friction. Fissures can drain quickly so that relevant performance can be effectively drained.
Weathered marls, mudstones and shales	These weak rocks are really heavily consolidated clays and silts and behave as such. Measurement of 'undrained' strength can be difficult because of heterogeneity. But short term **strength of clay in band or fissure can be critical.**	Effective stress parameter from drained tests can be more consistent and reliable than undrained test results.
Chalk	Strong under stationary load, but the fabric is destroyed by vibration/mechanical action.	Consolidates quickly. Strength of disturbed chalk increases with time as some fabric reforms.

5.4 Bearing pressure

There are significant differences in current practice of offices and within offices in methods of predicting loading and bearing capacity. The table below summarises some of the methods and comments reported during the survey.

The bearing capacities of sites are reported to be frequently underestimated, often because borehole information is inadequate. Several designers felt that foundations are sometimes unnecessarily deep: sometimes 3 or 4 m deep when 1 m would do.

Acceptable bearing pressures are usually controlled by settlement. Often a total settlement of 50 mm is acceptable, though design rules for bearing pressure often relate to only 25 mm.

Ground/ foundation	Method/parameter	Comment
Non-cohesive soil	Maximum safe bearing capacity to exceed maximum load.	Assumed by most designers.
dry	Presumed value from CP2004, CP2 or Tomlinson Ch2; also other codes and textbooks.	Commonly used. Some designers consider that values near top of ranges could be used more often.
	Terzaghi and Peck empirical chart of allowable bearing pressures (for 25 mm settlement) vs SPT N-value. (Terzaghi and Peck, Article 54, and many textbooks.)	Commonly used, but practice differs re correcting N-value (see Tomlinson p 99) or not correcting. **Several engineers stressed the fact that the SPT test is only a crude index test for comparing different soils, and that undue refinement of theory can be misleading.**
	Sutherland (1974), Peck Hansen and Thornburn, Lambe and Whitman, and others, charts relating N-value bearing pressure and settlement.	Used by a few. Recent authors indicate that bearing pressures up to twice those indicated by Terzaghi and Peck are satisfactory for 25 mm settlement.
	Terzaghi and Peck, and Meyerhof bearing capacity factors. (Terzaghi and Peck, Article 33, Tomlinson Ch2).	Not generally appropriate since settlement governing.
	Load test.	See Section 2.6.
below or near water table	Half bearing capacity on dry ground.	Commonly used (see Tomlinson p 98/99).
Cohesive soil	Factors of safety adopted to limit settlement and local failure.	Many designers use F of S of 3 for dead load alone and 2 for combined dead load and live load: but some increase latter figure if ground is heterogeneous or if loading is inclined. Lower figures are very occasionally adopted when large settlements can be tolerated and predicted with confidence. Some use F of S of 3 for all loadings. (Terzaghi and Peck, Article 53). An overstress of 25 per cent is sometimes accepted with abnormal live loads if F of S is above 3.

Ground/ foundation	Method/parameter	Comment
Eccentric and inclined loads	Allowable pressure	Many engineers are sceptical of commonly accepted pressure distributions and question whether pressures quoted for uniform vertical loads are applicable to peak pressures under eccentric and inclined loads. Textbooks and codes differ. There is little advice on settlement. Site investigation reports seldom consider it.
	CP2 § 1.47	Commonly used.
	CP2004 § 2.3.2.3.7	Inconsistent with CP2 and reported impractical for retaining structures.
	Meyerhof's equations (see Lambe and Whitman p 211).	Ultimate limit state approach. Used by a few designers.
Footings on slopes or dipping strata		Generally the effect of slope or dip has to be calculated from first principles.
Reference	Sutherland, H B (1974). Granular materials. Review Paper Session I. Settlement of structures. Pentech Press. London, 473–499.	

5.5 Movements

Settlement

Total settlement does not usually cause problems in highway construction unless bridge headroom becomes insufficient. It is the **differential settlement** between parts of foundations of a bridge and between bridge and embankment that generally affects performance and use most. However differential settlement can normally only be estimated from total settlement predictions. Unfortunately predictions of settlement are usually problematical and often inaccurate.

Aspect	Comment
Routine design	During the routine design of foundations for small bridges on stiff soils the settlement is generally controlled by the choice of Factor of Safety for bearing pressure, as discussed in Section 5.4.
Site investigation	Geotechnical engineers stressed that when settlements larger than 50 mm are anticipated due to soil compression the site investigation needs to pay special attention to them and it is not satisfactory to rely on allowable pressures which are based on the assumption that settlement is related to strength and controlled by Factor of Safety.
Data assessment	**A very careful assessment of data is considered much more important than refinement of calculation.** The relevance and reliability of information needs questioning, and it is useful to plot all data against depth to identify anomalies and variations. Laboratory measurements of the compressibility of many natural deposits are often inaccurate.
Case records	Engineers in several parts of the country have found that **case records** of large foundations on particular ground conditions generally **provide the only accurate** guide for the prediction of settlement of new foundations on similar ground. It is often found that the stiffness of undisturbed ground en-masse greatly exceeds predictions based on tests. Considering the importance of settlement and the unnecessary cost in conservative designs, it is surprising that few observations are made of bridge settlement, and very few predictions are checked.
Loading	Some designers simply make a conservative calculation of settlement under total dead load and live load. Others take much care and assess possible limits of performance and consider the effects of sequence and duration of loads. A major part of the loads on abutment foundations acts early in construction, while most of the load on pier foundations is only applied when the deck is added. Short-term loadings have little effect on the settlement of clay: some designers consider 10 per cent of live load as effective with dead load. However settlement of non-cohesive soils is rapid and it is general practice to consider full live load with dead load.
Differential settlement acceptable magnitude	Section 4.2 reports the wide range of differential settlement criteria adopted by designers of bridge decks. Some expect the total settlement to be not more than 25 mm. In contrast when 1 in 800 is adopted for the slope of differential settlement its magnitude can typically be 25 mm on a 20 m span, and the acceptable total settlement can be as large as 50 mm (note dependence on span).
calculation	Differential settlements due to differences in strata can be calculated, but are subject to the combined errors of the separate settlement calculations. Differential settlements due to variations in a single heterogeneous deposit can seldom be predicted and are often arbitrarily assumed to be half the total settlement. (Lambe and Whitman Ch 25 and reference below report studies). Predictions for critical stages of construction can be calibrated and modified from comparisons of predictions and performance of earlier stages.

Settlement (contd)

Aspect	Comment
Differential settlement minimisation	Differential settlement of foundations on similar ground is minimised by some designers by **designing all foundation to settle uniformly and by same amount.** This may require different bearing pressures for footings of different sizes and in different relative position. The accuracy of the settlement prediction may be questionable, but errors should be consistent and differential settlement small. To achieve this reduction or control of differential settlement it is sometimes necessary to make the foundations larger than are strictly required for stability. The extra cost may be well justified if the differential settlement loads on the deck are reduced, or if piles are avoided.
Heave	See Section 5.1 for mechanisms due to embankment or cutting, Section 2.5 for action of groundwater, and Appendix B for effects of piling. A structure across a deep cutting can be subjected to significant long term heave due to removal of vertical and lateral loads in ground. Some soils, such as chalk and mudstones, soften at the surface when exposed in excavations, but compress again quickly during construction of substructures.
Duration of settlement	Settlement of bridge foundations has on several occasions been found to occur much more quickly than that of adjacent embankments, and to be largely complete by the end of construction. This is frequently quicker than anticipated because of quick drainage through silt layers and fissures. However homogeneous clays can be very slow. Predictions of rate of settlement have been found to be more reliable when based on in-situ permeability measurements rather than laboratory tests.
Reference	Burland, J B, Broms, B and De Melo, V F B. (1977). Behaviour of foundations and structures. State of the art report, Session II, Proceedings 9th ICSMFE, Tokyo, Vol 2, pp 495–546.

Horizontal movements

Aspect	Comment
Prediction	See Section 6.1 concerning relationship between earth pressure and horizontal movement of walls. In general horizontal movement is unlikely to be excessive if the Factor of Safety against sliding exceeds 2. Horizontal movements of abutments have generally been small and not noticed, but when problems have arisen they have been difficult to correct. Predictions are likely to be even more unreliable than settlement predictions unless they are based on field observations (of which few have been published). The horizontal movement of ground under a foundation half way up an embankment has been observed to be greater than the horizontal movement at the top.

CHAPTER 6

ABUTMENT EARTH PRESSURES AND STABILITY

6.1 Active and at-rest earth pressures

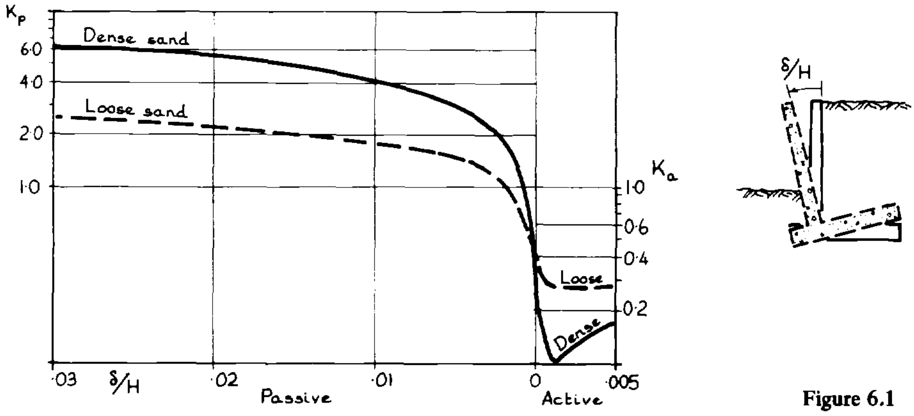
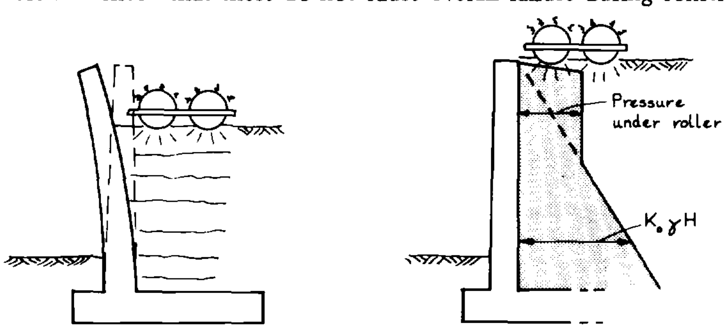
The majority of designers use the concept of equivalent fluid pressure or nominal earth pressure coefficient when calculating earth pressures on the abutments of small and medium size bridges. The calculation is usually straightforward since abutments generally support fill of one consistency with a horizontal surface. Field measurements (see reference) have indicated complex non-linear distributions of pressure on abutments. However interpretation and reapplication of such results for routine design are generally most conveniently performed by 'fitting' a linear relationship and deducing equivalent fluid pressures or earth pressure coefficient. Simple calculations are generally preferred because of the ease with which the designer can take account of the details and three dimensional structure of an abutment. However **for large structures and where ground conditions are abnormally complex designers expect to make more detailed calculations with Coulomb wedge and/or Rankine stress analysis and possibly with detailed consideration of soil-structure interaction.**

Although the application of the concept of equivalent fluid pressure is simple the **selection of intensity appropriate to a particular structure is not simple and requires a careful study of the interaction of the structure and its environment.** The following tables report the earth pressure intensities adopted by some designers for abutments with different restraints.

<table>
<tr><th></th><th>Comment</th></tr>
<tr><td>Active pressures on freestanding abutments</td><td>Active pressures on drained freestanding abutments are usually calculated using an
Equivalent fluid pressure = $5H\ kN/m^2$, where H is in metres
(= $30\ H\ lb/ft^2$, where H is in feet)

when (i) the wall can move by tilting or sliding;
(ii) the wall has positive drainage (see Section 8.6);
(iii) the backfill is a freedraining granular material (see Section 8.7).

Because of the simplicity and popularity of the rule of '5H' care has to be taken that it is not applied to walls and situations that differ significantly from those for which it was developed. The rule was developed over a long period of time and found satisfactory for simple structures which could move and which were never subjected to the heavy compaction by modern plant.</td></tr>
<tr><td>Other backfills</td><td>Designers assume higher equivalent pressures than $5H\ kN/m^2$ for less satisfactory fills, as indicated in Terzaghi and Peck (Ch 8). It can prove economic to design for suitable fill in regions where selected granular fill is very expensive. But it is usually very expensive to retain fill of high equivalent fluid pressure and a bank seat is sometimes found to be more economic. Equivalent fluid pressures lower than 5H kN/m^2 (in accordance with CP2) are occasionally used for particular fills with properties which can be relied upon.</td></tr>
</table>

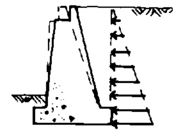
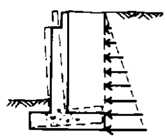
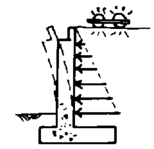
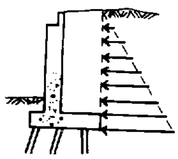
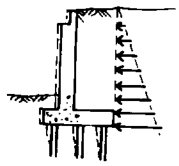
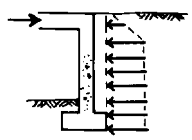
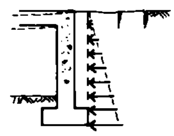
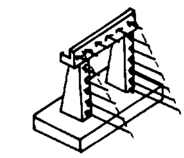
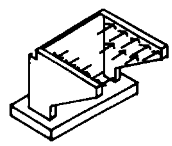
	Comment
High pressures on restrained abutments Dependence of pressure on movement	Abutments and parts of abutments which are fully or partially restrained against lateral movement are generally designed for pressures greater than active. An example of the relationship between earth pressure coefficient and displacement of walls supporting sand (as a fraction of height) is illustrated in Figure 6.1. The chart is relevant to walls which move bodily or rotate (or flex) about the bottom, but not for rotation about the top.  **Figure 6.1**
Strutted abutments and portal frames	Strutted abutments and the legs of portals are propped at the top. Consequently, the wall moves little and earth pressure at-rest is often assumed for their design. However some engineers consider it more appropriate to design for redistribution with the trapezoidal pressure distribution recommended by CP2 Clause 1.4343 for strutted excavations.
Abutments on piles	Earth pressure at-rest is assumed by some designers to act on abutments which are supported rigidly on piles. An intermediate coefficient between active and at-rest is appropriate when some lateral movement can occur, as is the case for many piled foundations.
At-rest pressure coefficient	The coefficient of earth pressure at-rest K_0 is generally estimated to be in the range between 0.4 (for well graded granular) to 0.7 (for less suitable backfill). Some designers assume for routine design that K_0 is 1.5 times the active pressure coefficient given in CP2, though occasionally 2 times active is thought appropriate. (These factors are not appropriate for walls designed to retain undisturbed over-consolidated clay without displacement, in which case the horizontal earth pressure can exceed the vertical overburden pressure).
Cantilever T-walls and wing walls Effects of compaction	Earth pressure at-rest is assumed by some engineers when designing the stem of conventional cantilever T walls for working conditions, and cantilever wing walls; while active pressure is assumed appropriate for checking the factor of safety for ultimate stability of the whole structure. Higher pressures than active have been observed on instrumented structures but stability failures have been few. It can be argued that when the fill is compacted behind a stem above a heel the lateral pressure beneath the roller approximates to the compaction pressure of the roller. At depths below which $K_0\gamma H$ exceeds the compaction pressure the intensity is assumed to be equal to $K_0\gamma H$. If a heavy roller is used it may be appropriate to consider pressure higher than at-rest and check that these do not cause overall failure during construction. 

	Comment
Vibration	There is little information on the influence of traffic vibration on earth pressures. Many designers assume that abutments will be subject to some vibration and follow the recommendation of CP2 that active pressure should be considered to act normal to the back or virtual back of the wall. Heavy vibration is likely to cause earth pressures greater than active, and an argument similar to the above for compaction may be appropriate. There is also little information available about the effects of intermittent horizontal loads from braking and temperature effects.
Range of possible behaviour	Several designers recommended that for circumstances when prediction of the probable behaviour is not straightforward, it is worth trying to **identify the limits of the range of possible behaviour by making alternative calculations based on favourable and unfavourable assumptions.**
3-Dimensional form	**The design of the 3-dimensional form of an abutment can sometimes have a much greater influence on the stability than variations in earth pressure.** Several designers emphasised the importance of choosing the optimum geometry of structure and arrangement of joints to achieve economy.
Load combination	One designer reported that the adoption of earth pressures greater than active for various degrees of restraint, as discussed above, has not necessarily resulted in heavier structures because the more detailed analysis has also indicated that the worst possible load combinations are less than the cumulative sum of possible loads previously assumed.
Reference	Jones, C J F P and Sims, F A (1975). Earth pressures against the abutments and wing walls of standard motorway bridges. Geotechnique Vol 25, No 4, 731–742.

6.1 (contd) Active and at-rest earth pressures

Summary of comments on earth pressures coefficients for abutments with freedraining fill

Refer to Section 6.1 for explanations.

Wall type	Calculation	Assumed restraint	Effective earth pressure coefficient	Sketch
1 Mass wall freestanding	Overall overturning, bearing and sliding	Small relieving movements possible	K_a	
2	Wall design		K_a	
3 Cantilever T-wall freestanding	Overall overturning, bearing and sliding	Small relieving movements possible	K_a	
4	Wall stem design	Stem restrained against base during compaction	K_o	
5 Cantilever T-wall on raking piles	Pile loads	Wall restrained against horizontal movement	K_o	
6	Wall stem design	As 4	K_o	
7 Cantilever T-wall on vertical piles	Pile loads	Wall partially restrained against horizontal movement	$K_a < K < K_o$	
8	Wall stem design	As 4	K_o	
9 Strutted abutment, portal frame	Wall stem design	Retained fill arches	Redistribution as CP2, Clause 1.4343	
10	Portal frame deck span sagging	Relieving effects of earth pressure reduced by shrinkage	0.75 K_a for low estimate	
11 Skeleton abutment (see Section 6.3)	Overall over turning, bearing and sliding	Fill arches on to columns	2 K_a on column width As 3 for wall part	
12	Column design		2 K_a on column As 4 for wall part	
13 Cantilever wing walls	Wing wall design	Walls restrained by abutment	K_o	

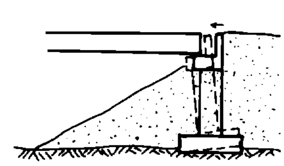
Notes:

1. Coefficient K refers to 'effective' earth pressure. Water pressure needs to be considered as well if drainage is poor (see Section 8.6), and at least to drainage level.
2. Many designers assume active earth pressure acts normal to virtual backs of abutments (and to stem), as recommended by CP2 for walls subject to vibration.
3. The form of an abutment in 3 dimensions has a major influence on stability. See Section 3.2.

6.2 Passive pressure

	Comment
Risk of removal of passive support	Passive resistance is ignored by most engineers in front of toes to abutments and retaining walls when they consider sliding and overturning stability, unless the footing is very deep. It is ignored because of the risk of a trench being excavated in front of the wall at some time in the future. Furthermore, the resistance of the ground for at least a metre beneath the surface is likely to be affected by weathering.
Construction precautions	If passive pressure does have to be assumed then the construction sequence has to be tightly specified and controlled to ensure that the ground does not soften during construction. And even when it is ignored many engineers aim to benefit from any resistance that can be obtained and specify that the backfill of working space and overdig in excavation in front of the toe should be in mass concrete.
Displacement requirement	To develop full passive resistance large displacement must occur, as illustrated in Figure 6.1 of earth pressure coefficients against wall displacement. For this reason passive resistance when it is considered, such as in front of a shear key, is often attributed a value well below its theoretical value at large displacements. Some engineers attribute to it a value in stability calculations equal to about 0.7 of the safe vertical bearing pressure for the soil.

6.3 Skeleton ('open' or 'spill-through') abutments

	Comment
Variety of methods	A wide variety of different arbitrary rules are used by designers for computing pressures on skeleton abutments. (See footnote about survey). The picture is further complicated by the fact that the design details also differ markedly with very different spacings and widths to columns. The range of methods is illustrated by the four listed below in order of magnitude of assumed earth pressures. 1 No lateral earth pressures on columns in stable embankments with slope 1 in 2 or shallower. It is argued that a skeleton abutment is not likely to reduce the stability of the embankment, nor is it likely to have much influence on the movements of the embankment. Consequently some movement of the abutment is anticipated, similar to a bank seat on vertical piles, and some provision is made for the narrowing of the deck expansion joints. Lateral movements of abutments half way down embankments are reported to be greater than movements at the crest.  2 Chettoe and Adams recommend that columns be designed for an active pressure immediately behind them plus an arbitrary allowance of up to 100 per cent addition. 3 Huntington recommends that no reduction from full active pressure over the gross width be made if the openings are less than twice the width of the legs, and the fill in front shall not be considered as providing greater resistance than active, with reduction due to the descending slope taken into account. 4 Full active pressure over the gross width.
Footnote on survey	During the survey 20 statements of practice were recorded relating to skeleton abutments, amongst which there were 12 different methods of calculation. The extremes of practice 1 and 4 above were recorded most times, 4 times each.

	Comment
Lateral pressures	During construction lateral pressures on culverts can build up in a similar manner to those on strutted abutments. Consequently at-rest pressures are assumed to develop by some designers. When culverts are below a perched water-table in an embankment, where drainage may not be practical, the possibility of high excess pore-water pressure is usually considered. Lateral forces also act on buried structures near the surface as a result of the traction and braking loads ('longitudinal loads') along the road above. The amount of load transmitted to the structure depends on the relative stiffness of the structure and covering fill. The effects are usually ignored if the depth of cover exceeds the span. It may be possible to use the road slab to transfer these loads past the buried structure, but the road slab has to be designed structurally for this function.
Vertical pressures	The vertical earth pressure on the roof of a culvert has often been calculated as equal to the weight of overburden directly above, and some engineers have thought that some reduction is possible due to arching of the soil above. But other engineers point out that **the earth pressure on top of a concrete culvert is likely to be greater than the weight of overburden** above since the culvert is stiffer than the fill at either side. The amount of additional load attracted to the culvert depends on the relative stiffness of the culvert and fill and also on the stiffness of the ground beneath. Vertical pressures have been measured on a culvert deep in a rock fill dam equal to 2 times the weight of overburden.

6.5 Stability of retaining walls

There are differences in the Factors of Safety used by different design offices for similar structures. Some engineers are very critical of the Code of Practice CP2 for being more conservative than several more recent textbooks. However, as indicated below, the lower Factors of Safety of other texts are usually countered by more conservative assumptions elsewhere. Comparison with American, Australian and South African practice indicates that the CP2 Factors are not unduly conservative. It is also argued by some designers that CP2 is appropriate and needs to be conservative and simple for routine use by structural engineers for small and medium size bridge abutments. The additional cost resulting from conservatism is not likely to be large. The following table summarises the methods reported during the survey. (Bearing capacity is discussed in Section 5.4.)

Aspect of stability	Factor of safety		Comment
	CP2	Other	
Sliding	F approx 2		Resistance provided by base friction/cohesion and passive earth pressure.
		$F \geqslant 1.5$	Some designers follow recommendation of Terzaghi and Peck (Article 46) and other authors for F = 1.5. But these generally assume lower angles of base friction than CP2 and ignore passive resistance. 2.0 is used when passive resistance is considered.
Over-turning	Resultant in middle third		Gravity walls
	$F \geqslant 2$		Other walls
		$F > 2.25$	Used by some engineers when there is uplift at the heel.
		See comment	Some engineers follow recommendation of Huntington that over-turning stability should be controlled by keeping resultant thrust on foundations for all types of wall within the limits:
			1 For walls on firm soil, resultant within middle third.
			2 For walls on rock, or bottom of vertical stem of solid gravity wall resting on footing, resultant in middle half.
			3 For walls founded on very compressible soil, resultant thrust should be at or behind the centre of the base to avoid forward tilting.
		Meyerhof's equation	A few designers are using the theory of Meyerhof (see Lambe and Whitman p 211) to check the ultimate limit state for combined overturning, sliding and bearing under resultant inclined eccentric load.
Slip failure			It is necessary to check a deep seated slip failure when the soil strata at depth are weaker than those checked for sliding stability.
	$F \geqslant 1.25$		Soil strengths based on back analysis of failure of the same type of soil.
	$F \geqslant 1.5$		Soil strengths based on tests.
			Such a check may be necessary for retaining walls on piles as well as on footings, particularly if the piles are used to support the retaining wall above a layer of soft clay. Huntington provides more detailed advice.

Aspect of stability	Factor of safety		Comment
	CP2	Other	
Slip failure (contd)			**The slip stability of bank seats has to be checked.** The value of the appropriate factor of safety depends on whether the slip is close to the structure or is a deep seated embankment slip Slip conditions during construction should not be overlooked; piles to bank seats and legs of skeleton abutments have failed in the past during construction due to slip of embankment into excavations for piers.
Reference	In addition to textbooks listed in Introduction some designers use: Recommendations of the Committee for Waterfront Structures EAU 1970. English Translation 2nd Edition 1971. Wilhem Ernst & Sohn, Berlin.		

CHAPTER 7

PILE FOUNDATIONS

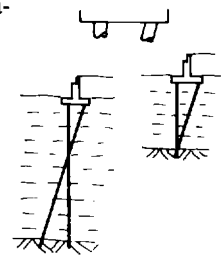
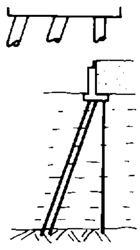
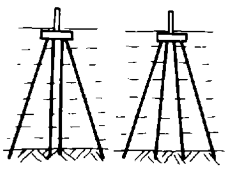
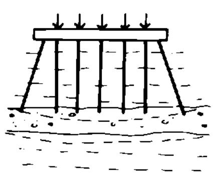
7.1 Global behaviour

Several designers stressed the benefit of considering the global behaviour of pile foundations with their environment before starting detail design and calculations. Broad judgements are made to assess primary influences and possible modes of movement.

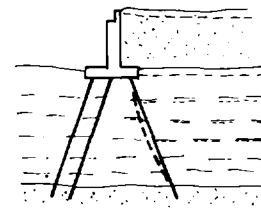
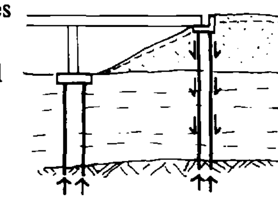
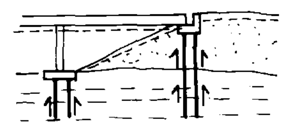
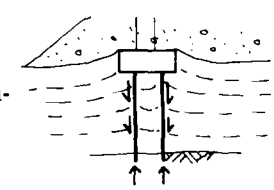
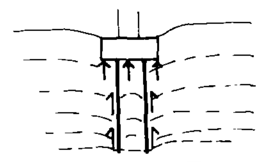
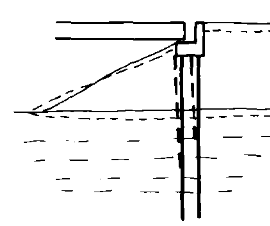
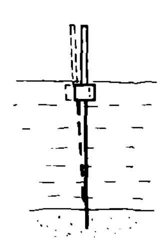
Our understanding of the precise interaction of a pile foundation and its environment is still rudimentary, and a designer has to rely heavily on his past experience and observation of the satisfactory performance of similar installations. Fortunately a considerable body of expertise has been built up over the years in the practical design of pile foundations. Nonetheless occasions do occur when there is doubt about the probable performance of a pile foundation and some designers then find it helpful to make alternative assessments and calculations for favourable and unfavourable assumptions.

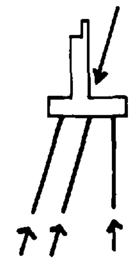
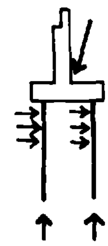
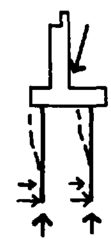
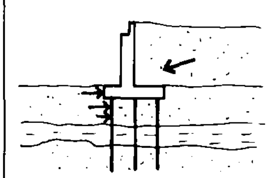
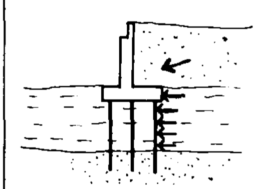
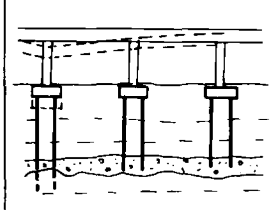
Designers in general use simple conservative design methods (see Section 7.6). Many feel that they are unlikely to obtain a clear understanding of the possible overall complexities except for the simplest of pile arrangements. Even so it is not always clear what is conservative and confidence can usually only be based on the apparently satisfactory performance of traditional designs.

The following tables summarise some of the possible primary influences and modes of movement considered by designers in the survey.

Behaviour	Comment	Sketch
Overall concept	Some designers encourage the drawing of the whole of the foundation in its environment. Pile groups are sometimes designed with thought given only to the arrangement of the piles at the cap, with lengths selected separately to provide the necessary bearing capacity. What appears a sensible arrangement of piles at the cap can look totally different if the piles are drawn to full length (the top sketch was taken from a text book).	
	The piles can be spaced at the cap and appear likely to distribute the load to the substrata; while in fact they unnecessarily concentrate the loads at depth. A bridge pier may be more efficiently supported by a fan of piles than by a group with all central piles vertical and edge ones raked. 	
Pattern of displacement	The pattern of displacements needs to be considered carefully, particularly if the group includes piles of different rakes. A pile group under a vertical load is likely to cause a 'dish' of settlement in the sub-strata; raking piles at the edges and corners of a vertical group may bear in ground that settles less than piles beneath the centre and so be subjected to much greater axial and bending loads. Failure of such piles has been known to take place.	

7.1 (contd) Global behaviour

Behaviour	Comment	Sketch
Pattern of displacement (contd)	**Relative movements between ground and a group with different rakes can induce large secondary effects which cannot be predicted.** Settlement of an embankment behind an abutment with piles raked backwards can subject these piles to severe bending. For this reason raking piles back under an embankment is often discouraged. The effects of relative settlement between ground and vertical piles are likely to be less severe and more easily predicted.	
Downdrag	Settlement of an embankment and sub-stratum relative to piles causes downdrag (negative skin friction). Only in exceptional circumstances is this likely to cause failure of the pile material but it does increase settlement. If the settlement of the upper strata relative to the bearing strata is small downdrag is not likely to be a problem since a smaller settlement of the piles will relieve it. Bitumen slipcoat is sometimes used to reduce downdrag on driven piles. However some engineers question its effectiveness and its ability to survive driving through some soils. It can be cheaper to use larger piles with strength to carry the downdrag. Its significance is discussed by Tomlinson (p 40) and influence of pile spacing by Broms (Reference Section 7.4).	
	If the structure is articulated so that the settlement does not impair performance then it may be practical to let the piles settle with the embankment.	
Pile cap load	The contribution of the pile cap to the bearing capacity of the foundation depends on much the same global interaction of foundation and environment as that which influences negative skin friction. If the ground around the piles is settling relative to the piles, ie the conditions for which downdrag occurs, then the ground will settle away from beneath the pile cap so that the cap only receives support from the piles. On the other hand if the piles settle relative to the ground around, then the cap will progressively pick up greater load. Since all the weight of the cap is initially applied to the ground when the concrete is wet, the bearing load between cap and ground should then increase from this level. Some designers have been able to make significant savings in piles when the caps did not have to be supported by them.	 
Lateral movement	Pile groups are sometimes designed so that they can move laterally with the surrounding soil. A pile group may be better able to perform the function of supporting vertical loads if it is able to move laterally without having to react with rakers, anchors or a propping deck. Although there is little guidance available for the prediction of lateral movements it is often found possible to provide more than adequate tolerance in the expansion joints. It is reported that a foundation half way down an embankment can move more laterally than points at the crest or toe.	
	Lateral movements have been anticipated for piles under a pier which could only be supported on a single line of piles between railway tracks. The piles were designed to flex with the pier during temperature movements of the deck. (When large movements are expected the change of eccentricity of the load may need to be checked).	

Behaviour	Comment	Sketch
Group design	Designers differ in their demands for the performance of piles. Some only require that a pile should be able to support an axial thrust, with moments resisted by the stiffening effects of the soil. To transmit inclined loads to sub-strata a group of such piles has to have the geometry of a triangular truss. They are not appropriate if the surrounding ground is too soft to provide lateral stability, and it is then necessary for the piles to have strength in bending.	
	If the piles are designed to support moments then inclined loads can be carried by frame action (rather than truss) and it is possible for all the piles to be vertical. Many engineers concerned primarily with foundation construction consider raking piles unsatisfactory and prefer to have all their piles vertical, while engineers concerned with abutment design often prefer to use raking piles with moment capacity and rely on the combined truss and frame action of the group. Whatever the geometry, the horizontal load on the pile groups may be transmitted to the ground by sub-grade reaction near the top of the piles (possibly assisted by the sliding resistance of the cap) and/or by the portal action of the group transmitting horizontal forces to the lower ends of the piles. The precise force system depends on the relative stiffness of the piles and the surrounding soil. Methods of pile group analysis are discussed in Section 7.6. At present no method of analysis is able to cope with all the common applications of piles and the most appropriate method can only be selected after a consideration of the relevance of its assumptions to the particular situation.	 
Lateral loads	**The most serious deficiency of all current methods of group analysis for abutments is that they do not consider the influence of the embankment behind on lateral loading of the piles.** It is reasonable to assume that a hard upper stratum can resist horizontal forces with subgrade reaction passive pressures on the fronts of piles. But if that layer of ground is soft, then it might apply unfavourable active pressures on the backs of the piles. Even when the ground is firm the strain under the edge of the embankment may cause the ground to displace forwards relative to a stiff pile group. There is a wide range of different force systems between the extremes of relative stiffness. During design it is usually only possible to make qualitative assessments of the limits of ground stiffness for which passive resistance or active loading are relevant from considerations of the global displacements.	 
Factor of safety	It may be necessary to study global behaviour when selecting the factors of safety appropriate for pile design. In a large group some variation of pile capacity must be expected and a sub-standard pile is unlikely of itself to jeopardise the performance of the group. But if failure of an isolated foundation could lead to progressive failures of neighbouring foundations then a higher factory of safety may be appropriate.	
	An abutment on very short piles may need to be checked for overturning stability, as if on a footing.	

	Comment
Specialist experience	Selection of pile type and estimation of performance is an aspect of bridge design for which the **majority of designers find it necessary to make use of a specialist.** Wide experience is required of the various types of pile in different ground conditions. The choice is influenced not only by cost but also by differences in performance, which depend on soil and groundwater conditions and also on the type of plant used and competence of the gang. The installation process can increase the strength of some soils and decrease the strength of others. The choice may not even be straightforward to an expert, if he does not have personal experience of identical piles in similar strata locally, and he may well request a trial installation to confirm his choice.
Consultation with contractors	Piling contractors have the greatest fund of experience of installation of their types of pile and many designers **seek their advice early on in the design process.** But it has to be appreciated that commercial considerations can influence contractor's advice. However those offering a wide range of pile types may be able to advise without prejudice on the most appropriate one offered. Some designers have found it advantageous to have recourse to an independent expert during the tender period, when alternatives are likely to be proposed, to assist in selection and in preparation of special specifications.
Responsibility for design	**The majority of designers expect to take overall responsibility for the pile foundation and only make up their minds after studying all sources of information and experience available.** The adequacy of the chosen type is ensured by load tests and proper supervision. (One organisation has a break clause in the contract to the effect that if the initial load test is unsatisfactory the piling contractor can be paid for his work and asked to leave the site). However it is sometimes not clear during a contract whether difficult ground conditions were unforeseeable and who is responsible for the additional cost. The difficulties can result from poor quality plant and labour for which the contractor is reponsible. Some designers, when faced with the possibility of a very expensive delay, find it worth calling in an adviser with experience of installation problems and possibly paying for trials of different methods such as varying the length of casing, even when the responsibility is thought not to be theirs.
Vigilant supervision	**The importance of vigilant supervision of installation cannot be stressed too strongly** (see Thorburn & Thorburn (1977)).
References	Whitaker T. (1976). The design of pile foundations. Second Edition, Pergamon Press. Weltman A J and Little J A. (1977). A review of bearing pile types. Report PG1, CIRIA. London. Thorburn S and Thorburn J Q. (1977). Review of problems associated with the construction of cast-in-place concrete piles, Report PG2, CIRIA, London.

7.3 Contract documents

Many complaints were received during the Survey from piling contractors about the unsatisfactory quality of contract documents. The table below summarises comments.

Problem	Comment
Presentation	**All information relating to piling needs to be concentrated in a few pages and drawings** so that it can be easily separated and issued to subcontractors. At present subcontractors often find they are issued with several different sets of incomplete documents by the different main contractors. Such unnecessary duplication is avoided by one design organisation by making the piling information directly available to *bona fide* piling subcontractors. It is usually found impractical for tenderers to take advantage of a note that the full Site Investigation Report is available for inspection at the designer's office. Both the office and the site are likely to be at some distance and senior engineers can neither spare days for travelling nor can they absorb the information at a glance.
Specification differences	**There is confusion between different specifications** (ICE, DTp, GLC, RIBA, Federation of Piling Specialists, and so on) and piling operatives do not always know the differences.
Clarity	The specification for piles is often scattered over several drawings as well as the Specification. Uncoordinated scattered notes can result in unnecessary small differences in construction throughout contract. (One contract was reported to involve 8 different grades of concrete in piles because of small variations in notes on drawings). To avoid confusion some designers write their complete specification on the pile drawing.
Soil information	**The quality of soil information supplied to piling subcontractors is often poor.** Apparently many tender enquiries are accompanied by borehole records which do not reach or extend below the bottom of piles. Often little information is available about ground-water, casing depths and boring problems.
Unforeseen ground conditions	Contractors at tender stage often tend to be optimistic and not look for problems, but when problems are encountered they look for all the inadequacies they can. They try to play on the inadequacy of borehole records and the designer's staff do not always have sufficient knowledge to reject questionable claims. **It is in the client's best interest to have a clear idea of piling problems before the tender is let.**
Measurement	**The bill of quantities needs to be presented in a form suitable for the piles to be used.** It is not possible to cover with a single list of items the fundamental differences in method of construction of different piles or the different consequencies of changes during construction. Some designers have overcome this problem by providing separate drawings and documents for each type likely to be offered (only allowing the subcontractor to tender with the documents most appropriate). Others design for one likely type and provide separate documents only when an alternative is offered.
Reinforcement	The documents often do not indicate whether or how reinforcement should be changed if pile length differs from design, and the method of measurement can confuse the issue by including reinforcement in rate per metre. **Changing the length of a cage at the last minute is usually impractical and often deleterious.** The length of reinforcement and details must be indicated.
Length	Designers generally consider it advisable to overestimate the lengths of piles in tender documents. **However possible shortening is often not achieved** because tests have not been completed or the designer (and checker) is not at hand to approve the change: a delay can quickly cost as much as the extra length. If a subcontractor anticipates shortening he might adjust his tender to obtain most payment for the early stages.
Alternative designs	Attitudes of designers differ towards alternative piles offered by contractors. Some designers accept them readily, while others consider it is easier to contain claims during construction by sticking to the initial design. **If alternative designs are to be encouraged, then design loads must be stated on the drawing.** If the design loads are dependent on the adopted method of analysis then this needs to be stated also.

7.4 Pile behaviour
Axial loading

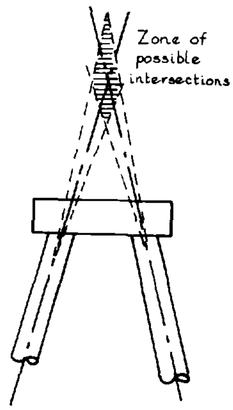
Aspect	Comment
Design	Highly sophisticated design methods are generally considered irrelevant because of the variability of soil properties. Differences in apparent soil behaviour resulting from differences in testing procedures and sample size can influence predictions much more than refinements in design method, and consequently deserve the greater attention. **Only experience of similar site conditions provides definite guidance to behaviour** and only load testing provides confirmation of capacity. In addition to general textbooks in the Introduction the references below give guidance and lists of other references.
Factor of safety	**Factors of safety used in design have not been standardised** and practices differ. Many engineers use a global factor of safety of 3 during design when no test results are available, and 2 when test results are available for the same type of pile in the same ground. Other engineers check pile capacities on site on the basis of tests using a factor of safety of 2.5 for shaft friction and 3 for end bearing. Large diameter piles in clay are frequently designed for the worst of an overall factory of safety of 2 or partial factors of 1 for shaft friction and 3 for end bearing.
Settlement of group	Prediction of settlement of piles and group is considered by many designers to be problematical and liable to error. In clays the friction at working load can be a very large proportion of the total so that load settlement behaviour is dominated by the mechanism of friction development. Friction on the shafts of end-bearing piles may provide a substantial proportion of the support at working load since it may be mobilised by smaller displacements than the end bearing. The quality of installation work can have a marked effect on the load/settlement behaviour of both driven and bored piles. Prediction of settlement of pile groups is reviewed by Broms (1976). The short term settlement in a loading test of a single pile is of little help in predicting the long-term settlement of a group unless it is end bearing on rock or very dense granular soil where elastic compression of the piles is the main component of settlement. The term 'group factor' is used in different ways by different engineers, and application is often arbitrary. When applied to the reduction of bearing capacity per pile because of the neighbours it is generally less than 1 for groups in clay and also possibly for driven piles in dense sands if vibration loosens particle packing. It is greater than 1 for loose sand because piling causes densification. 'Settlement ratios' are considered more useful by some engineers and relate settlement of group to that of single pile (see Whitaker reference below).
Tension piles	Designers differ in their assumptions concerning the tensile resistance obtained from skin friction with the ground. Some assume it can be effective in some form for all piles in tension; others assume that it cannot be relied on for long term loads in clay. A few designers ignore it completely and only consider the weight of the pile. Such differences in assumptions can have significant influence on the layout and cost of a group resisting large horizontal forces. Broms (1976) reports on research into differences in skin friction of piles in tension and compression.
References	Broms B. (1976). Pile foundations – Pile groups. 6th European Conference on soil mechanics and foundation engineering, Vienna, pp 103–132. Tomlinson M J. (1977). Pile design and construction practice. Viewpoint Publication, London. pp 448. Thorburn S and MacVicar R S. (1971). Pile load tests for failure in the Clyde alluvium. Conference on behaviour of piles, I.C.E. London, pp 1–7. Burland J B and Cooke R W. (1974). The design of bored piles in stiff clays. Ground Engineering, Vol 7, No 4, pp 107–114. Skempton A W. (1959). Cast-in-situ bored piles in London clay, Geotechnique, London, 1959, 9 (4) 153–173. Whitaker T. (1976). The design of pile foundations. Second edition. Pergamon Press. Institution of Civil Engineers (1977). Piles in weak rock, pp 233.

Lateral loading

Aspect	Comment
Relevance	Designers generally assume that lateral loading of piles, whether active or passive (subgrade reaction), can be dismissed in the design of the majority of small and medium size bridges which are not on exceptionally soft ground.
Active loading	Piles for bridge abutments may need to be designed to resist active lateral loading **when the substratum is soft and moves forwards relative to the piles due to surcharge effects of the embankment behind.** (Huntington provides advice for calculation of this active pressure and slip circle analysis of abutment on piles). Even if there is a more than adequate factor of safety against slip, piles under an abutment may be subjected to some bending due to long-term creep flows of the ground in the potential slip-direction. So **there may be active loads on piles even in stiff ground,** but of small magnitude because displacements are small.
Subgrade reaction	In recent years several methods of pile group analysis have come into use which rely to some extent on subgrade reaction to resist horizontal loads. Subgrade reaction can usually be relied upon for support when the piles move towards the supporting ground and the ground is stiff. **But since it can be very difficult to predict how piles and soil move relative to each other many engineers are not prepared to place confidence in subgrade reaction for lateral support of foundations** subject to long-term loading. Also the displacement necessary to develop the required passive resistance can be unacceptable. There are structures such as pier foundations subject to lateral live loads for which subgrade reaction can provide adequate lateral support. The methods of analysis of Broms (1976) and methods of beam on elastic foundation have been used on many occasions. It is often found that only large diameter piles can develop the required capacity against horizontal loads economically. If the top of the pile is in weathered ground, then the stiffness of this may have to be ignored, or at least considered as much less than that of deeper ground. On the other hand if the pile is beneath a cap it may be reasonable to assume the full stiffness of the ground, possibly with additional passive resistance and restraining effect of the caps. Sometimes different stiffnesses for the ground have to be considered for long-term and short-term loading.
Load test	Some engineers consider it just as important to load-test piles laterally as well as vertically if the consequences of unsatisfactory performance jeopardise the structure. In the past tests have been designed for the particular installation, usually by jacking two piles apart. It has not always been clear how the piles have reacted at depth, or what the maximum moments were.
References	Broms B. (1976). Stability of flexible structures (piles and pile groups). 6th European Conference on soil mechanics and foundation engineering. Vienna, p 230–269. Tomlinson M J. (1977). Pile design and construction practice. Viewpoint Publications, London, pp 448.

7.5 Raking piles

The installation of raking piles is generally fraught with many more problems than vertical piles, as discussed in the table below. As a result engineers concerned primarily with design and construction of foundations generally try to avoid their use. But most designers concerned with the whole bridge like to see inclined loads resisted by a triangle of forces in a truss-like arrangement of piles.

Problem	Comment
Cost	The cost of raking piles in poor ground can be as much as two times that of vertical piles. It can be sensible to use vertical piles of twice the size if difficult construction problems can be avoided. But in stiff ground which can stand unsupported the problems of rake are not so significant.
Maximum rake	The maximum practical rake depends on the ground conditions and method of installation. Many design offices **limit the maximum** rakes to the following values: 1 in 8 for large diameter piles 1 in 5 for small diameter bored piles 1 in 4 for small diameter driven piles Piles can sometimes be installed at greater rakes (1 in 3 is often possible) but not all contractors have the appropriate equipment. In special situations steel piles have been driven at very much greater rakes to provide an oblique prop or anchor.
Tolerances	**Tolerances can have a significant effect on the theoretical distribution of loads in a group.** While vertical piles are generally expected to be within a tolerance of 75 mm at piling platform level and within 1 in 75, raking piles are often expected to be only within 1 in 25 for rakes up to 1 in 6 with greater tolerance at greater rakes. Tighter tolerances may not be obtainable. If raking piles are installed from a level significantly different to formation then the 75 mm lateral error is increased by the effect of the error in the rake. It is not uncommon for piles to be outside specified tolerances. If a group of piles is drawn with the centre lines shown for positions of extreme tolerance, it is found that the ranges of positions of possible intersections (and load eccentricity) is large. If the lines of piles wander off course, ie 'banana', the ranges are even larger.  Several engineers pointed out that during the design of a group there is no point in trying to fix positions of piles more accurately than they can be constructed.
Rig working space and movement	Most rigs can only rake in particular directions and must have working space on the appropriate side of each pile. If alternate piles are raked in different directions, then it can be difficult to move the machine without tracking over piles. It may be appropriate to stagger lines of different rake to avoid interference of rig and surplus lengths. Where manoeuvring is difficult it can be worth designing the group so that the rig can work along the group.
Bored piles	**The installation of bored piles is made more complicated if not impossible by rake because-** 1 Local collapses can occur forming voids above the bore before a casing can be placed. 2 Driving the casing on a rake can be more difficult. 3 It is difficult to get the cage of reinforcement concentric without bars sagging with inadequate cover beneath. 4 Concrete has to be placed by a tremie to be sure of quality. This is seldom done and it is difficult to demand it unless it has been specified. It is difficult to tremie concrete deeper than 20 m. 5 On withdrawing a casing concrete is pulled up more than with vertical casings and the risk of voids forming is greater.
Driven piles	Driving piles on a rake does not have such serious problems as boring. But the **reduction in driving efficiency** can be much greater than acknowledged by the contractor. A check should be make that piles driven on a rake do not meet apparent refusal or adequate set at a higher level than vertical piles in the same strata.
Load tests	The performance of raking piles should only be checked by tests on raking piles since many of the problems of installation do not occur with vertical piles. A test on a vertical pile may be useful for proving the load carrying capacity of the ground, but it may well give little indication of the quality of installation of raking piles.

Comment	Problem
Lack of knowledge	Many designers expressed the opinion that current knowledge of the behaviour of pile groups is inadequate. For this reason they felt that a conservative approach is warranted.
Selection of simple arrangement	**Several designers stressed that it is much more important to choose a suitable arrangement of piles to resist inclined loads than to select the best method of calculation.** For the majority of small and medium size bridges on piles **very simple arrangements of piles are selected.** It is generally considered difficult to estimate the possible performance of a complicated group with certainty or to decide what is conservative.
Considerations of statics	**Many designers determine the position of piles from a simple consideration of** statics with piles in only two directions. It is often found that if pile positions and rakes are selected purely from static considerations then whatever method of calculation is used for the group, the applied loads are primarily resisted by axial forces in piles, with secondary moments and shear forces relatively small. Appendix C summarises a simple design method used in several offices for selection of pile position, using the elastic centre. Many consider this method of design sufficient in itself; some refine the arrangement using a computer analysis.
Non-uniform load distribution	It is often assumed that the loads on the piles in any row are uniformly distributed. However if the bearing stratum settles it is likely that the piles **at the ends of rows support greater loads than piles in the middle** of rows. To ensure that end piles shed load to others without failure some designers check that the compression strength of the end piles and shear strength of the pile cap exceed the ultimate resistance of the soil on the piles.
Methods of calculation	Some commonly used methods of calculation are reviewed in the next table. **Most designers prefer to use the simple traditional** methods of analysis of CP2 for routine design. Many feel that the more complicated methods can seldom be shown to predict loads more reliably or economically. And it is concluded from the satisfactory performance of most pile structures in the past that the traditional methods are satisfactory for simple pile groups in average ground conditions. However some form of frame analysis is usually considered necessary for the more complicated arrangements of pile groups and for simple arrangements in very soft ground in order to predict the moments in the piles.
Comparisons of methods	Comparisons of predictions from the different methods of analysis for an arbitrary pile group indicate enormous differences in performance which undermine the confidence of some designers in every method. However some designers have found that if the group is designed by one method to support the loads optimally by axial forces then predictions from different methods indicate relatively small ranges for axial loads with only significant ranges in the secondary actions of pile moments and shears. Even so the similarity of prediction does not necessarily imply they are correct; none of the analyses considers the significant influence of ground movements due to embankment surcharge.
Factors of safety	Comparison between loads calculated by methods which consider a cross-section of a group (methods 1 to 5 of next table) and loads from a 3-D method (7 of next table) is not straightforward since the latter predicts variations in pile loads along each row. Such variations may correspond more closely with reality but it is not clear how such load predictions should be used in design. Ductility of the ground causes redistribution so that local overstressing does not lead to failure. A lower factor of safety is appropriate for the piles subjected to the peak loads in a row than is appropriate for the average capacity of the whole row. Past performance has indicated that current factors of safety for average capacity are generally satisfactory; so it is unnecessarily conservative to apply the same factors of safety for the exceptionally loaded piles. Since method 7 is likely to be used more extensively in the future consideration is urgently required as to what factors of safety are appropriate for individual piles.
Subgrade reaction	Methods 4 to 7 of the following table all make use of subgrade reaction to some extent to resist lateral loads. While this added restraint may be appropriate for a pier foundation, it is not always clear whether it is appropriate for an abutment where the flow of ground due to embankment surcharge may well apply active loads to the backs of piles (see Section 7.1 and 7.4). However when the upper stratum is firm enough to provide subgrade reaction, raking piles might be reduced or avoided.

7.6 (contd) **Pile groups**

Summary of methods of calculation of loads in pile groups

Method	Reference Details Comment	Usage
1 Static method (hand)	CP2 p 154. A system of axial pile loads is calculated which are simply in equilibrium with applied loads. Some designers first consider the pile cap as a footing and determine distribution of bearing pressures, and then place piles as necessary to carry loads. In the static method and the rivet group method the primary forces are axial loads, and no account is taken of secondary moments due to flexure of piles. The methods are thought not appropriate by some designers when upper strata are exceptionally soft and pile bending is significant: a frame analysis is then used.	Commonly used for routine design
2 Rivet group (hand)	CP2 p 151 with simple assumption below for raked piles. Piles are first considered vertical at their positions under the cap and an elastic distribution of axial load is found in equilibrium with vertical component of applied load at its position under the cap. Then a number of piles are raked so that the horizontal components of their axial loads balance the horizontal component of the applied load. See comment on static method.	Commonly used for routine design
3 Elastic centre (hand or computer)	CP2 p 155 The piles and cap are considered to form a truss or frame. The cap is assumed to be rigid while the piles are elastic and are supported on rigid supports at their feet (or equivalent point of support in the length of the piles). The cap/pile and pile/support connection can be pinned or fixed. If the connections are pinned the system is then a truss with only axial loads calculated for the piles. If the connections are fixed bending moments are also calculated. Some engineers design for the worst conditions of pinned and fixed connections.	Commonly used for routine design
4 TAMS (computer)	References and example in Appendix of: Dixon H H & Berry D W (1970). Extensions to the Chania-Sasumua water supply scheme for Nairobi. Proc. I.C.E, Vol 45, January, pp 35–64. Piles are assumed embedded in soil and the stiffnesses of their tops subject to lateral and axial loads are calculated. Then a stiffness analysis is carried out for the cap supported by springs with lateral and axial stiffnesses of piles.	Used by a few for routine design
5 Plane frame (computer)	Several bureaux and 'in house' computer stiffness programmes are used for pile group analysis. Some are ordinary structural analysis programmes while others have been specially written for pile groups. A wide variety of different assumptions are possible, and are used, relating to: pile/soil axial interaction, lateral interaction, effective supports, cap stiffness, fixity of connections.	Used by a few for routine design. Also used for complex conditions
6 Space frame (computer)	Three dimensional analysis with facilities of plane frame. Used only for exceptional three dimensional problems.	Exceptional cases only
7 Elastic half-space (computer)	Department of Transport Highway Engineering Computer Branch, HECB/B/7-PGROUP. A three dimensional flexibility analysis in which piles are assumed embedded in an elastic half space, and pile/soil interaction occurs at a discrete number of points on each pile. This method has been available for a short period and has been found useful by a few designers. But it is considered by many to be more complicated than is warranted from current knowledge of pile behaviour.	Used by a few for routine design or checking. Also used for complex conditions
Footnote on survey	Of 21 statements of practice recorded during the Survey: 17 used the simple methods for routine design (static method 4, rivet group 7, elastic centre 6), but of these 5 used a frame analysis for special conditions; 3 used TAMS or plane frame for routine design and 1 used PGROUP. Some designers used one method for design and another for checking.	

7.7 Pile tests

Aspect	Comment
'Trial' piles to investigate performance	'Trial' piles are generally loaded to failure, ie large displacements, in order to obtain as much information about the ground as possible. (Tests to only 1½ times working load seldom provide information about ground capacity.) The constant rate of penetration test is being used increasingly, because it is quicker than incremental tests and for trial piles often gives a clearly defined failure load. It is usually found impractical to use early tests of piles in main contract as investigatory since the cost of delays waiting for results can be considerable, and often potential economies in design cannot be achieved because information is too late. Maximum benefit is most likely to be achieved if installation and performance can be checked in an advanced contract (see Section 2.6). One advantage of an advanced contract is that piles can be left for a longer time before testing; so enabling the soil's strength to recover. Testing in a rush during the main contract can seriously undervalue long term performance. The number of 'trial' piles required depends on the differences of ground conditions over site. If the working piles bear on a rock substratum of known competence 'trial' piles may not be needed.
'Test' piles to check quality of construction	'Test' piles are generally working piles which are loaded to 1½ times working load. Some engineers consider them to be of questionable value, except where gross defects are discovered, since they give no indication of defects which cause long term deterioration. Results can be dependent on the position of the pile in the group. A pile in the middle of a group can appear much stronger than piles at the edge, particularly if driving has compacted ground between. Most designers retain the right to test any pile during the contract. It is not unknown for the last piles placed to be of the worst quality because of the rush of the contractor to leave site. The number of tests required depends on size of contract and difficulties of installation (see Appendix B). In general designers expect to test one pile per abutment or small bridge. On a small contract it may be cheaper to make the design more conservative and not test. It can be very difficult to test large diameter piles, but in general their quality can be inspected visually more satisfactorily.
Load application	For small piles kentledge has been found convenient and reasonably representative of future load form. On soft ground attention has to be paid to temporary support of kentledge. For large piles kentledge can be uneconomic and unsafe and reaction is often provided by tension piles or anchors. Attention has to be given to spacing and interaction in the ground between test pile and anchor piles to ensure that the test pile is transferring load to ground in the same way as the working pile. With kentledge or tension piles the reaction should be applied to the pile top by a jack.
Raking pile tests	See Section 7.5 concerning need to test on rake. Raking piles have been tested with kentledge on two piles of opposite rake, and also by reaction with raking tension piles.
Dynamic driving records	Dynamic pile driving formulae or analyses based on the wave equation have not been found to provide a reliable indication of pile capacity. However a set empirically calibrated for one pile on a site can be used for comparison of other similar piles in other parts of the site if ground conditions are virtually identical and driving procedures are the same.
Integrity testing	Methods are reviewed in reference below. In general direct methods by excavation or coring are the most trusted but are expensive. The indirect methods relying on sonic, electronic or radiation measurements are less reliable and require interpretation. They have the inherent disadvantages of non-destructive methods for concrete above ground together with problems of interpretation of unknown shapes in unknown soil. Consequently many engineers are sceptical of their use at present. However several techniques are progressively being developed and some useful applications have been carried out.
Reference	Weltman A J. (1977). Integrity testing of piles – A Review. Report PG4, CIRIA, London.

CHAPTER 8

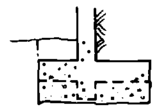
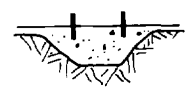
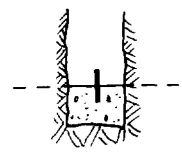
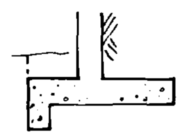
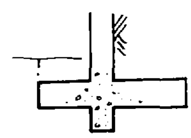
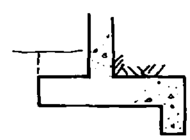
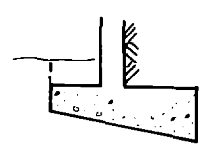
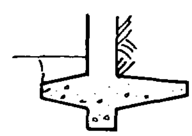
DETAILS

8.1 General comments

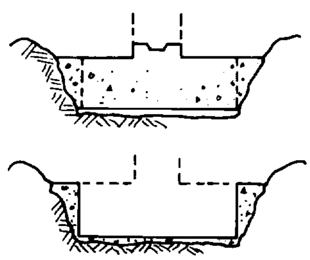
The careful design of substructure details can have as much influence on economy of construction as the basic choice of type of structure.

	Comment
Influence on construction programme	Contractors find that small details can cause delays in construction with consequential cost out of all proportion to the importance of the detail. This is most likely to happen when the design of the detail restricts the contractor's choice of construction sequence so that he cannot progress other much larger parts of the contract.
Construction during inclement weather	Construction of foundations is dominated by weather conditions. Contractors beseech designers to consider during design and preparation of specifications the problems of excavating to line and level, blinding, and stabilising vertical faces during periods of inclement weather. A foundation that can be completed in a week is not only likely to be very much more economic than one that requires several processes taking several weeks, but it is also likely to perform better since the ground will not be softened as much by exposure and long-term release of pressure.
Construction simplicity	Designers and contractors stress the importance of simplicity in design of details as well as in design of overall structural form. Several designers have found that optimisation of designs with respect only to assumed cost-rates and not simplicity of construction tends to result in compact and complex details which are not only difficult to build to the desired quality but also take an undesirably long time to complete. For these reasons reinforced concrete walls and bases are generally more satisfactory when designed with more than the theoretically economic thickness. Flexural stiffness is increased and steel fixing and concreting are much easier.

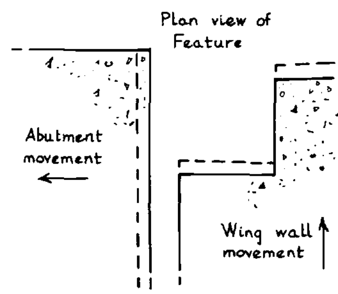
8.2 Excavation shape and shear-keys

	Comment	
Level formation	The simplest abutment foundation has a level formation. Sliding must be resisted by friction under the base, if passive resistance is ignored in front of the toe. Often a wider base is needed for sliding stability than for bearing or overturning stability. For this reason some designers dimension the base for the latter criteria and provide additional resistance to sliding by a shear-key or by inclining formation. However the cost of additional width of base or greater depth is often no more than that of the other remedies and the construction problems and ground softening can be less severe. (To minimise ground softening some designers give the formation a very small cross fall to improve drainage.)	 
Ground disturbance from shear-key	**The process of constructing a shear-key frequently loosens and softens the very material on which reliance is placed** for passive resistance. With most ground conditions a level formation can be excavated simply and quickly. But if a trench 1.5 m deep is subsequently excavated and is open for a week or so, then some slippage or loosening usually occurs. Sometimes, the trench acts as a drain for the rest of the excavation. Unfortunately shear-keys are most needed in soils which are least able to stand during construction without softening. If shear-keys have to be used then it may be best to design them so that they can be excavated and concreted as a continuous process with the blinding and with simple dowels inserted to transfer horizontal loads. The edges can be battered so that they are stable without movement. Alternatively the shear-key can be placed in a trench from a higher level with concrete poured as excavation proceeds. Precise control of dimensions is seldom important.	 
Shear-key in front	**Shear-keys are generally not placed at the front** of footings because of the risk of the lateral resistance being later removed by excavation for services. Furthermore softening of the ground under the toe is much more critical than elsewhere since it reduces the overturning stability. For the same reason **the toe needs to be at least 1 m below ground level** to avoid softening due to frost action, and **land drains should not be placed near the toe.**	
Under middle	Shear-keys placed under the middle of footings push during sliding against soil that is under heavy vertical compression. Excavation can be simpler than at either edge of the base since a machine can straddle the trench. If the reinforcement in the shear-key is continuous with that in the walls construction is more complicated since the shear-key then has to be constructed with the accurate line and levels of the wall.	
At back	The shear-key at the back also obtains lateral resistance from ground subjected to vertical compression. The possibility of sliding up inclined planes may need to be checked as well as sliding horizontally. Excavation at the back of the base is likely to cause less disturbance than under the middle, but the depth of excavation on the retained side is greater.	
Inclined formation	A footing with inclined formation can have simpler details than one with a shear-key. The inclined formation requires deep excavation of the retained side, but the formation should not suffer the same softening that occurs during the construction of a reinforced concrete shear-key. However excavation and construction on a slope is time consuming and expensive, and it may be cheaper to construct the full width at the lower depth. Increased tolerance and cover are often found desirable.	
Complex	Some designers and contractors consider that the time required to construct complex foundations not only leads to unnecessary softening and disturbance of the ground but is also unnecessarily expensive.	

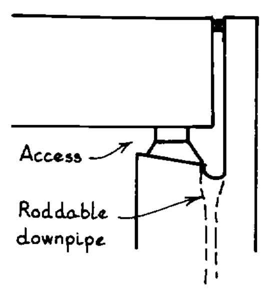
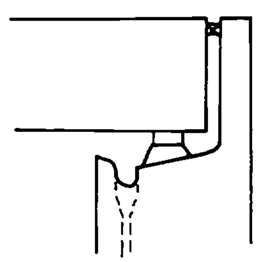
8.3 Substructure form and reinforcement

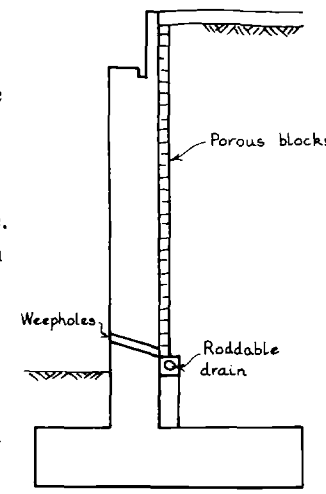
Detail	Comment
Form	
Shape	Several designers and contractors consider that a simple shape is not only likely to be cheaper to build, but also more likely to be to the standard of quality required. It is much easier to build with surfaces plane and vertical. If a wall is curved in plan then construction with a batter can be very complicated. If a wall has batters to the faces and a reducing height then formwork extending above the two faces impinges and restricts access for men and materials. Cutting the formwork to fit exactly can greatly increase the cost by preventing re-use. If an abutment has a forward or backward batter to the front face and wing walls in different planes then it can be very difficult to make the formwork at the corner and also difficult to support it adequately. Haunches at the intersection of retaining walls and bases have been used often in the past, but do not appear to be very common at present. They complicate the construction at the kicker and compaction of the concrete may be poor. It is possible that they improve the performance of the wall/base joint, but detailing is complicated. Haunches are sometimes used at the corners of portal frames and culverts to reduce shear stresses and so avoid links.
Pour size	It is becoming increasingly economic to construct abutments and piers in single large pours from footing to bearing shelf. Some designers thicken up such walls a little, if necessary, to make it easier for men to climb inside the formwork. The standards of workmanship of formwork, steel fixing, cleaning, and concrete pouring and inspection are all likely to be better than in intricate and slender structures.
Tolerances	Careful thought needs to be given to the **tolerances**; as to which matter and which do not. Many dimensions of substructures are not critical. Some tolerances are controlled by the need for parts of the construction to fit together, some by the ride of the road, and some by the need for good visual appearance.
Formwork	
Cost	The **cost of formwork is a significant proportion of the total cost of substructures** and considerable savings ensue where multiple re-use is practised. Wastage is reduced if walls are designed so that lengths between construction joints are standardised and if possible related to the dimensions of standard sheets of plywood. The heights of walls, levels of features and steps in levels between footings should be selected so that as much as possible can be constructed with a single shutter without adjustment or unnecessarily restricting the order of the construction. It is important during the design of tapered piers to consider the restrictions on construction sequence that might result from having to modify the formwork to suit different heights.
Bases	Bases can sometimes be constructed most economically by casting concrete directly against ground; ie without formwork, and possibly using expendable temporary supports. Another method is to construct a concrete box in advance with internal forms: the method is useful where there is a lot of repetition and the formwork can be struck quickly and re-used with steel fixing following in a clean dry space. 
Reinforcement	Contractors find it is much **easier to fix reinforcement in vertical and horizontal** planes than inclined planes. Fixing of corners and joints is much quicker if one plane can be fixed at a time. One contractor has found that it takes about twice as long to fix the reinforcement in abutments and piers than in foundations because of the problems of knitting heavy bars together in different directions. Fixing is made much more difficult if a crane has to be used to support heavy bars in one horizontal direction as they are threaded through bars in another plane. Control of dimensions can be difficult with heavy bent bars because of bending tolerances. It is also much easier to fix medium sized bars than numerous small bars. Congestion should be avoided.

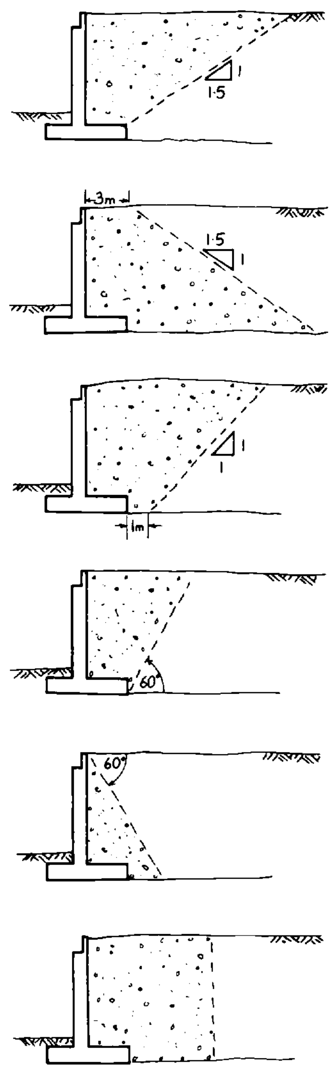
8.4 Construction and movement joints

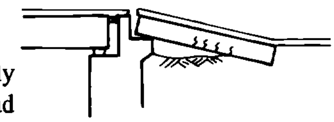
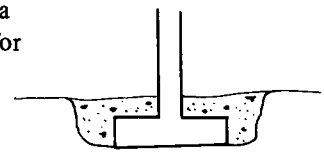
Detail	Comment
Construction joints	Contractors find that **positions of construction joints and restrictions on lengths of pour have significant influence on the programme.** Consequently if designers have stringent requirements they need to be stated clearly in the contract documents and not be left simply to the approval of the resident engineer. The positions are usually chosen to suit shrinkage and temperature movements of concrete, particularly if crack inducers are not spaced regularly along the pour. However for walls the maximum length of pour is often controlled by the size of shutter that can be lifted by the available cranes. Frequently a length of pour of about 10 m is found most practical. Making construction joints with stop-ends, and scabbling before the subsequent pour is expensive. Sometimes it is even necessary to incorporate a water bar to prevent penetration through the joint. Removal of laitenance is not always practical. It may be more practical and cheaper to form a proper movement joint where strength across the joint is not required. But a movement joint requires exact location with exact reinforcement scheduling and so will not allow minor variations of construction sequence.
Movement joints	There do not appear to be any definitive general guidelines concerning the positioning of movement joints. Some designers try to avoid them if possible, and only locate them at changes of direction and section to absorb shrinkage movement, and at changes of load intensity (as between wing walls and abutments) and of ground condition to avoid settlement stresses. Elsewhere they try to control cracking by reinforcement and crack control joints. Frequently movement joints are designed to accommodate relative movements of 25 mm in all directions. The joints in footings and walls often have a shear-key, with wall joints also having a water-bar. In areas of mining subsidence wider joints are sometimes provided and the shear-keys omitted so that they do not pick up loads for which they are not designed. Care has to be taken at joints between parts of a structure which are likely to move in different directions. At the joint between a wing wall and an abutment, tilting of the wing wall can be in a different direction to that of the abutment, and a shear-key might provide a prop against active pressure movements and be subjected to large loads. Where significant movement is anticipated some designers provide a generous feature (say 75 mm) to hide it. 
Crack control joints	Crack control joints at about 3 m centres have been used by some designers to control the spacing of cracks in very long walls due to shrinkage and thermal movements. Concrete is poured in monolithic form across them and cracks are induced during curing by deep grooves on the opposite faces. Cracking may be assisted by plastic tubes up the middle of the wall and partial discontinuity of reinforcement. A special water bar can be attached to the back shutter or internally to cover the crack once it has occurred. Crack control joints should give the contractor more flexibility, as within reason he may form construction joints where he chooses. Turton C D. (1974). To crack or not to crack. Concrete, Vol 8, No 11, Nov, pp 32–36. Hughes B P. (1973). Early thermal movement and cracking of concrete. Current Practice Sheet No 3PC/06/2. Concrete, Vol 7, No 5, May, pp 43–44.
Seals and provisions for leakage	Many designers appear to be disillusioned **about the effectiveness of polysulphide and other plastic seals.** Many proprietary seals appear attractive on paper but are very difficult to install properly under site conditions, and several types deteriorate in a relatively short time. There has in the past been a tendency to follow the lead of manufacturers to use bigger and more complicated types of joint. However many designers now try to design as simple a joint as possible. Adequate provision is made for drainage of water that does leak through in order to avoid discolouration of visible structure. Drainage channels and pipes are large enough to take the full flow of water that might enter them and also large enough to be properly cleaned out during construction and later. A 25 mm diameter hole is likely to block.

8.5 Top of abutment

Detail	Comment
Curtain wall form	**Abutment curtain walls are frequently designed with complexity of detail out of all proportion to their importance.** If the curtain wall is supported on a corbel off the back of the abutment, then the cost of making the shutters and fixing intricate reinforcement may be much greater than constructing a thicker abutment wall with plane back face. Even quite a large thickening of the main wall can be economic if it avoids the need for falsework and a soffit shutter on a large over-hang. The saving in time can be of greater significance. In addition compaction of the backfill against a plane face is likely to be more satisfactory than around ledges. Expensive delays can arise if the backfill has to be compacted up to the level of a corbel soffit prior to construction of corbel and curtain wall. Contractors have sometimes found that insufficient clearances have been provided for proper construction of corbels.
Loads	**Curtain walls are subjected to concentrated vertical and horizontal** loads from the wheels of vehicles. These loads cause larger local moments than might be predicted from considerations of equivalent overburden. Several engineers design curtain walls to resist these moments, having assumed some simple spreading of the loads through the structure, such as at 45°.
Reinforcement	On post-tension bridges the curtain wall has to be detailed so that it can be constructed after the deck has been post-tensioned. If not detailed appropriately the reinforcement may get in the way of the jack.
Long abutment	The **reinforcement along the top** of an abutment is often light in comparison to that in the footings; it is sometimes appropriate in long walls to increase this reinforcement at the top to provide bending strength or crack control in the event of the ends of the abutment settling more than the centre.
Wing walls to bearing shelf	The **small wing walls at the ends** of the bearing shelf are also commonly designed with **unnecessarily complicated** details. It is not always clear at the design stage whether it would be more convenient to construct these before or after placing the deck. If possible they should be designed so that the construction sequence can be determined during the contract.
Drainage of bearing shelf	Some designers consider that in the past **bearing shelves on some abutments have not had adequate drainage**, as is demonstrated by the staining over the outside face. A considerable quantity of water can flow down through the expansion joint, and some designers consider it wise to provide drainage of sufficient capacity to carry the full flow that might enter in the event of joint seals being ineffective. They also design the channel along the bearing shelf to be accessible for cleaning. A few designers place the channel in front of the bearings to make access simple. (If access is very difficult it may not even be possible to clear out construction debris completely.) The falls along the channel need to be more than adequate since nominal falls can be obscured by unfavourable tolerances. The effluent pipes need to be roddable. While many engineers slope these pipes down into the abutment backface drainage layer, an increasing number are providing independent roddable down-pipes to soakaways. Some designers provide drainage channels along the tops of piers if there are joints in the deck which could leak if they deteriorate.  

Detail	Comment
Drainage of abutment back face	
Need	Many designers stressed the importance of **positive drainage down the back face of abutments.** Not only does such drainage prevent the building up of high pressures against the wall, but the drainage also prevents saturation of the fill behind the abutment. During a storm, water flowing through the porous sub-base of the road and service ducts meets an obstruction at the abutment and water collects which must be drained away. During construction such flooding of the fill can cause ponding and increases subsequent settlement. Drainage needs to be more than adequate to remove the full flow that could enter during a storm.
Capacity	The difference in cost between inadequate drainage and more than adequate drainage is small, and because of the importance of this item skimping is usually considered not justified. Several designers stressed that the **drainage system needs to be positive, testable and maintainable.** Currently, a porous block layer draining to a pipe capable of being cleared by rods is generally considered the most satisfactory form of drainage membrane. A 450 mm wide dry stone wall may be even more satisfactory in regions where the material and skilled labour are readily available. A layer of sand is difficult to place satisfactorily and even when it accords with a standard specification it may not be sufficiently permeable. The specification for concrete blocks should not require too high a strength otherwise their permeability is likely to be inadequate.  In addition to providing a positive drainage system, many engineers also incorporate weep holes to provide relief for drainage in the event of pipe blockage, and visible evidence of satisfactory drainage. They are sloped backwards to prevent weeping and staining when drainage is satisfactory.
Design water pressures	Designers often assume that there will be no water pressures affecting stability if the wall has adequate drainage. However it has to be remembered that the drainage layer is not necessarily at the effective back of the wall in the stability calculation. Sometimes **consideration has to be given to water pressures** if the backfill is not permeable or if the abutment is constructed against an existing impermeable slope. If adequate drainage cannot be provided behind the abutment then some water pressures must be considered. In any case CP2 recommends that it be considered up to the level of the drainage outlets.
Drainage of bearing shelf	See Section 8.5.
Waterproofing	The **backfaces of most abutments** are given some form of protective coatings: often two coats of bitumastic paint. Opinions differ as to the value of such coatings; some designers consider they achieve nothing and are a complete waste of money, while others consider that they provide an inexpensive additional protection to reinforced concrete even if not full waterproofing. It is thought that coatings reduce seepage of water through the concrete, which in turn reduces the amount of dirt which sticks to the front face. Such protection is not adequate for structures which are likely to be below the water table and must hold water out, such as subways, and these have to be properly tanked and drained. Clause NG 1601 of Notes for Guidance on the Specification for Road and Bridge Works gives advice on protection of concrete in corrosive ground.

Detail	Comment
Abutment backfill	
Material	**Designers in most parts of the country expect to use a free-draining fill to obtain the benefits of low earth pressure and greater predictability of performance.** Granular fill also has the advantage that **compaction in confined places is easier.** In some regions selected granular fill does not exist naturally and can be very expensive. In such situations it can be more economic to design the abutments to support the higher pressures of the less satisfactory fill available. But many designers remain cautious about using such fill because there is little reliable information on the design pressures. Relatively impermeable materials, such as chalk, have been used but serious settlement has occurred on occasions after inundation. In some regions pulverised fuel ash is available at an economic price and has been used successfully as a standard backfill behind abutments. Where shale and rock fill have been used it has often been necessary to limit the maximum particle size, particularly if piles have to be placed through the fill. Whatever fill is used, some thought should be given to its interaction with the embankment and abutment; an abutment on piles has failed by backward overturning caused by the settlement of a rigid mass of cemented fill.
Zone of free-draining fill	Designers' practices differ as to the size of zone specified for the free-draining fill. The sketches indicate some examples: not all are buildable. The most appropriate shape of the zone depends on the construction sequence: so it is unwise to have too rigid requirements in the design. For walls in a cutting only a small zone may be required. **Whatever shape is designed it is essential that it should be constructable with proper compaction.** Some engineers pointed out that it can be impossible to compact fill to the 60° slope that is sometimes indicated to suit a Coulomb wedge. Some engineers design the interface to have a slope of 1 in 1½ or less so that the face can be properly compacted if one material is placed in advance of the other. 
Compaction	It can be very difficult to prevent settlement of the backfill behind abutments, and many designers stressed the **need for thorough compaction.** It appears that sometimes even proper compaction does not eliminate it. However contractors are reported to often underestimate the care that is needed to achieve proper compaction, and supervision is not always adequate. Compaction in the corners of a structure is difficult and can only be done by small plant. Some designers specify an increase in the number of passes of compaction plant above that generally specified for the particular material in order to achieve the same density at least as the approach embankment. (If it is possible to pass a large roller along the back of an abutment then care should be taken that compaction pressures do not greatly exceed the design active pressures). Adequate compaction is particularly difficult to achieve beneath parts of a structure, such as under wing walls, and where there is no firm background against which to compact. It has been found advantageous on occasions to place mass concrete walls under parts of skeleton abutments and cantilever wing walls to provide such a firm background.

Detail	Comment
Run-on-slabs	**Designers and clients generally advise against the use of run-on slabs** except for special situations. They used to be more popular but serious problems of maintenance have arisen. It has been found that making up road levels after embankment settlement has generally been easier and less expensive where bridges have not had run-on slabs. It is considered more economic to rely on proper compaction of backfill and to make up the road surfacing as settlement occurs. 
Overdig and working space backfill	Backfilling to overdig and working space around foundations is usually done with selected granular fill, where small quantities are involved, unless the fill has to transmit bearing pressures or passive earth pressures in which case concrete fill is generally used. Some engineers consider that if there is a risk that backfilled excavations might act as reservoirs for water which could soften foundations, then fill with a clay content may be more appropriate. 

CHAPTER 9

PERFORMANCE, MAINTENANCE AND REPAIR

9.1 Monitoring of performance

Monitoring of performance has in the past only been carried out in special circumstances when problems have been anticipated. However several engineers during the Survey expressed a requirement for more case histories of performance of different types of bridges in the various regional conditions. The table below summarises some of the comments on monitoring performance and on possible procedures.

Aspect	Comment
Value	Regular measurements of line and level of bridge substructures and adjacent ground are of value because: 1 The designer (and others) can develop a much better understanding of the global behaviour of different types of bridge and their environments, and so gain an idea of which designs perform best and which calculations are relevant. 2 The case histories of full size structures on various ground conditions provide a reliable basis for predictions for future bridges in the region. 3 Observations during construction identify the speed and extent of movements, and so provide a basis for deciding if procedures can be speeded up. A significant saving may be possible in a contract involving a long pause or considerable repetition. 4 A warning is obtained of excessive movement and of insufficient support of particular points. Without a history of regular readings it is impossible to tell within a short period after a problem has been identified the difference between long-term movements and short-term fluctuations and errors.
Procedure	Measurements require careful planning, preferably at the design stage, so that all parties involved are fully aware of what is being done. One person should be responsible for the planning, installation, reading and maintenance of measurement points and particularly datum points. Several series of observations have been rendered useless because of uncertainties about datum positions. The frequency of measurements needed depends on the rates of movement. If measurements are initially taken frequently, such as once a month, then it quickly becomes apparent how much less (or more) often the measurements are really needed.
Equipment	The instruments and measurement points need to be simple, reliable, stable, inexpensive and easy to install, and above all robust and durable. A few organisations are equipped with precision levels for monitoring settlement. However for the majority of bridges properly calibrated site equipment should be satisfactory. Structures often settle more than is anticipated or appreciated without significant signs of distress and measurements with an accuracy of only about 3 mm can still be of considerable value.
Contract implications	Monitoring procedures can interfere with construction procedures. Positions of datum points and required sight lines need to be indicated on contract drawings otherwise additional costs can arise.
Reference	Symposium on field instrumentation. Butterworths, London, 1973. Burland J B. (1977). Field measurements – Some examples of their influence on foundation design and construction. Ground Engineering, Vol 10, No 7, October.

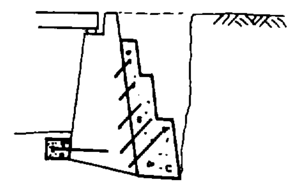
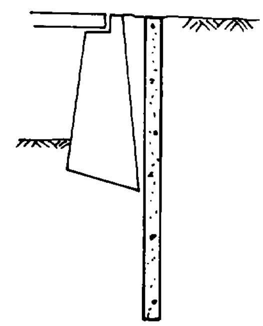
9.2 Inspection and maintenance considerations during design

Several engineers stressed the importance of considering future inspection needs during design. Although considerable effort is usually made to minimise the need for future maintenance it is considered prudent to make access provisions for subsequent inspection, cleaning and repair. The table below lists comments on aspects of maintenance which affect substructures.

Subject	Comment
Inspection facilities	During the design of substructures, consideration needs to be given to what facilities will be required for inspection and maintenance of: 1 Damage from scour. The river authority should be consulted. See Section 2.2. 2 Cleaning and testing of drainage to back face of abutment. See Section 8.6. 3 Cleaning and testing of drainage to abutment shelf. See Section 8.5. 4 Cleaning of drainage of other joints and crevices. 5 Inspection of deck bearings and anchorages and provision of pockets for jacking deck for repairs. Bank seats and buried abutments need sufficient headroom (900 mm minimum) and level standing area, with steps for access up paved slopes. Tall piers may need inspection galleries and access thereto. 6 Effects of horizontal movements of abutments above soft ground. Expansion joints and bearings can often be designed to accommodate greater movements than anticipated from calculations without much increase in cost. 7 Damage from subsidence, particularly mining subsidence. Reduction of headroom can be a problem due to settlement of substructures and resurfacing of under-road.
Reliance on maintenance	Designers generally try to design their bridges so that the maintenance required is a minimum. However it has on a few occasions been advantageous to place a continuous structure on subsiding foundations and make regular adjustments of levels at supports by jacking. Several designers stressed that this should only be done when there is no doubt about the maintenance being carried out.
Feedback of information	Several designers stressed the need for feedback of information on maintenance problems to the design office, even though the designers responsibility may terminate at the end of the contract maintenance period. Liaison with the maintenance authority was felt to be beneficial because: 1 Feedback of the defects in construction and performance provides guidance for improvements in future design. 2 Observations of performance such as settlement of real structures provide the only reliable basis for the prediction of the performance of similar structures on similar ground conditions. (See Section 9.1). 3 An understanding of the performance of old structures can greatly broaden a designer's perspective. For example the performance of slender masonry retaining walls of traditional construction can defy calculations. The difference between safe and unsafe is evident when drainage becomes defective.
References	Department of Transport. Technical Memorandum BE4/77. Inspection of highway structures. British Railways Board. CE Handbook No 6, Examination of structures. London Transport Executive. Manual of inspection of bridges, structures and buildings.

9.3 Repairs

The following notes list some of the comments made during the Survey about repairs to substructures.

Problem	Repair
Improvement to stability of abutments and walls	The stability of abutments and retaining walls has been improved by: 1 Anchoring to deadman or ground anchors. 2 Increasing toe. 3 Increasing back (possibly with deadman anchors for temporary condition).  4 Constructing diaphragm wall behind. 5 Underpinning by hit-and-miss method or small bored piles or pali radiche piles (defective piles have been inspected from shafts and tunnels).  6 Treating scour of rocky and granular river beds by refilling with concrete bags spiked together with reinforcement bars. Scour of silty river beds has required underpinning. 7 Grouting. Care has to be taken that grout pressures do not exceed existing ground pressures otherwise more harm can be done than good.
Replacement or widening of bridge	When a bridge has to be replaced by another on the same ground advantage can sometimes be taken of the fact that the old bridge has already consolidated the ground beneath. A borehole alongside the old foundations will not necessarily indicate the state of the ground beneath the foundations and it may be necessary to prove the ground by drilling. If a bridge has to be widened then differential settlement between new foundations and old foundations can be a problem.
Refurbishing of masonry bridges	Masonry bridges have been refurbished by: 1 Repair of scour damage to river bed round foundation. 2 Grouting of voids in substructures. Care has to be taken that grout does not block drainage and ducts. 3 Repointing of substructures. 4 Repointing of arch. 5 Arch is supported while road surface and fill are removed: fill is replaced by concrete with light reinforcement. 6 The road is resurfaced and waterproofed with fillet at parapet to throw water away from the parapet.

CHAPTER 10

ADVICE AND INFORMATION NEEDS

10.1 Dissemination of information

Survey of advice and information needs	Many of the engineers interviewed during the Survey (see Introduction) mentioned the aspects of design of bridge foundations and substructures for which they required advice or more information. These needs are reported on the following pages in four main groups: 1 Guidance on a particular aspect of a design from someone who has experience of similar problems on other projects. 2 Advice which might be considered part of a general lecture course on design of foundations and earth retaining structures. 3 Particular shortcomings in current knowledge which might be filled by a research project or by a clear presentation and critical review of existing information. 4 Improvement throughout the industry of a common practice or technique which is currently imprecise.
Finding the adviser	**Most of the information needs were of category 1 above and were concerned with obtaining guidance on particular aspects of designs, such as information on the behaviour of a particular regional soil.** In many cases it was thought that the necessary information already existed and that the real problem was in finding the 'expert' who might be of most help. Several engineers thought there was a need for a national centre for geotechnics which might provide a reservoir of technical expertise but which would also redirect enquiries to experts outside, particularly for problems of a regional, practical, or contractual nature. It was felt that often the 'expert' was likely to be someone with broad design and construction experience who could assess the importance of the problem, and then advise on how to avoid or solve it.
Updating this report	This report could help designers keep up to date with current practice in the design of bridge foundations. It is hoped that designers will report developments in practice and deficiencies of this document to: The Head of the Geotechnics Division, Building Research Station, Garston, Watford WD2 7JR.

10.2 Advice and information needs

1 Guidance on particular aspects of design and construction

Some of the following needs might be helped by general research projects. But in most cases the problems depend significantly on the details of the project,and specific advice based on existing experience is likely to be of more direct benefit.

Subject	Need	Related pages of this report
Site investigation and soil properties	1 Advice on appropriate drilling and sampling methods for various types of ground.	10, 83, 84
	2 Properties and behaviour of soils for different regions, eg Gault clay, Oxford clay, soils of the Midlands, and so on.	
	3 Appropriate methods and procedures of field and laboratory testing for assessment of design parameters of various soils, eg plate and pressuremeter testing, interpretation of oedometer tests for settlement.	15, 83, 84
	4 Guidance on the properties of various types of fill with different degress of compaction.	See Index
	5 Advice on hydraulic surveys.	8
	6 Advice on survey for, and design of bridge over old mine-workings.	9
Choice of foundation, design and analysis	7 Rationalisation of load combinations.	41
	8 Advice on acceptable levels of differential and total settlement.	36, 45
	9 Guidance on appropriate methods of estimating total and differential settlement.	45, 46, 60
	10 Guidance on the appropriateness of piles or spread footings for different situations.	33, 34, 35
	11 Advice on buoyant foundations and other alternatives to piles on soft ground.	26, 33, 34
	12 Assessment of significance of soft lenses of soil at depth.	33, 34
	13 Selection of appropriate undrained and drained soil parameters for design under various types of loading.	42
	14 Advice on ground treatment, eg when can vibroreplacement rock columns be used under abutments and bank seats.	37
	15 Advice on the design and construction of foundations on fill.	See Index
Pile foundations	16 Choice of pile for various ground conditions.	58
	17 Information on capability and shortcomings of piling plant (some engineers considered this more important than advice on choice of pile).	58, 62
	18 Ranges of skin friction of piles in various soils.	60

1 (contd) Guidance on particular aspects of design and construction

Subject	Need	Related pages of this report
Pile foundations (contd)	19 Longterm load transfer along pile shafts, especially clay soils.	
	20 Guidance on design and analysis for piles subjected to lateral loads.	61
	21 Bending of piles in ground and effective point of fixity.	65
	22 Advice on measurement of subgrade reaction.	61
	23 Advice on pile group design and optimum arrangement.	63, 64
	24 Advice on testing vertical and raking piles.	65
Retaining walls and abutments	25 Design of cantilever diaphragm walls.	20, 47
	26 Design of footings for vertical and lateral load.	43
	27 Guidance on choice of lateral pressures in relation to soil type, arrangement and stiffness of support system, and method of construction.	47–50
	28 Guidance on the design of skeleton (spill-through) abutments.	50
	29 Advice on design of flexible corrugated metal structures.	25
Methods of construction	30 Advice on construction methods and costs.	2, 3, 4
Details	31 Advice on corrosion of, and chemical attack on, foundations.	24, 72
Monitoring	32 Advice on what to measure, when and how.	75

2 General advice which might be included in a lecture course on foundations and earth retaining structures

Soil investigation and properties	1 Standardisation of soil description and logging.	12
	2 Advice on validity of various tests for different soil types and loading conditions.	83, 84
	3 Techniques of laboratory and in-situ testing and their interpretation.	83, 84
Soil/structure interaction	4 Case records of the global behaviour of structure, foundations and ground.	39, 40, 55, 56, 57
	5 Relationships between earth pressure and displacement.	48
Pile foundations	6 Occurrence of downdrag.	
	7 Pile group behaviour and analysis	63, 64,
	8 Piles subject to lateral loads and lateral ground movement.	61

10.2 (contd) Advice and information needs

3 Particular shortcomings in current knowledge which might be filled by a research project or by a clear presentation and critical review of existing information

Subject		Need	Related pages of this report
Monitoring	1	Observations of the movements of more bridges to determine the global behaviour of different types of structure on the various regional ground conditions of this country.	39, 75
Soil investigation and properties	2	Methods of assessment of Young's modulus, shear modulus and subgrade reaction of ground.	63, 64
	3	Properties of typical soils of Midlands and North.	
Footings	4	Assessment of allowable bearing pressure under footings supporting eccentric and inclined loads.	44
Earth pressures	5	Observations of earth pressures and associated movements on all types of retaining walls and substructures, including reinforced earth.	50
	6	Information on earth pressure resistance to fluctuating braking and temperature loads.	49
	7	Development of limit state theory for earth retaining structures.	
Pile foundations	8	Efficiency of pile driving hammers, particularly on rake.	58, 62
	9	The ability of piles to carry load in tension.	60
	10	Observations of loads in full size pile groups.	63
	11	Contribution of pile cap to pile group performance.	56, 63, 64
	12	Development of a general method of pile group analysis which considers embankment/abutment/cap/pile/soil behaviour. Method should cover all situations or should comprise a number of simple methods with proven ranges of application of adequate coverage.	64
	13	Factors of safety appropriate to more rigorous pile group design methods.	63, 64
	14	Testing of piles to relate to group behaviour.	65

4 Improvement required throughout industry of a common practice or technique which is currently imprecise

	1	Soil and rock description and general site investigation procedures and reporting.	
	2	In-situ testing, particularly of gravel.	83
	3	Sampling soils and weak rocks, and subsequent testing.	83
	4	Predicting of settlement and ground movement.	45
	5	Intregrity testing of piles.	65
	6	Detection of services under ground particularly where several lie close together.	6

Quality	Properties that can be reliably determined
Class 1	Classification, moisture content, density, strength, deformation and consolidation characteristics.
Class 2	Classification, moisture content and density.
Class 3	Classification and moisture content.
Class 4	Classification.
Class 5	None; sequence of strata only.

Soil type	Boring and sampling	In-situ testing	Strength and compressibility
Gravel Cobbles Boulders	Trial pit much more satisfactory. Advancing bore and sample recovery difficult, needs shell and casing with chisel for boulders. Class 4 samples from trial pit. Class 5 samples from bore-holes because fines washed out.	SPT gives some indication of relative density in gravel: unreliable for coarse gravel and cobbles. Dutch cone cannot penetrate dense gravel and coarse material. Field plate, shear and density in-situ tests are the most reliable but require dry pit. In-situ permeability gives some indication of fines: pumping trial better.	Direct measurement only possible from field plate and field shear tests.
Sand	Boring using shell below water: water might be added to assist. Sand with water tends to 'blow' up borehole so loosening sand below. Samples generally Class 5 deficient in fines. Class 4 from split spoon. Class 3 or 2 from piston sampler if possible.	Dutch cone best method for assessing relative density. SPT gives some indication but can be erroneously low due to loosening from 'blow'. Field plate tests etc. best as above. In-situ permeability and pumping tests for perme-ability.	Direct measurement only possible from field plate and field shear tests. Approximate values of strength can be estimated empirically from SPT and Dutch cone.
Silt	Boring using shell, or clay cutter above water. Class 5 if fines washed out, but Class 3 if recovered intact. Class 2 possible from piston sampler.	Field tests best as above. Dutch cone best method for assessing relative density. SPT gives some indication but can be erroneously low due to loosening from 'blow'.	As for sand.

Soil type	Boring and sampling	In-situ testing	Strength and compressibility
Normally consolidated and lightly over consolidated clays	Boring using clay cutter when dry, and possibly shell below water. Class 4 for intact lumps from shell and clay cutter. Class 2 from open-drive sampler. Class 1 from piston sampler.	Field tests best as above. Vane test useful. Dutch cone useful. In-situ permeability to test clay with silt bands if rate of consolidation important. Probing to establish thickness.	Field tests best. Laboratory tests of shear strength and compressibility satisfactory from Class 1 samples. Large samples needed for greater reliability Approximate estimates of strength possible from Dutch cone and SPT calibrated empirically from field tests.
Over consolidated clays	Boring using clay cutter when dry, and possibly shell below water. Class 1 of firm to stiff material from open-drive sampler and piston sampler: Class 2 or 3 of stiff and hard material. Class 4 and 5 from shell and clay cutter.	Field plate, shear and density in-situ tests needed for accurate assessment, but require dry pit. Dutch cone, SPT and pressuremeter can be used.	Field tests best. Laboratory tests of shear strength and compressibility practical of Class 1 samples, but results affected by relative scales of sample and fissure geometry: considerable scatter in undrained test results. Drained tests, even of Class 2 samples, have less scatter. Approximate estimates from Dutch cone and SPT calibrated empirically from field tests may be more reliable than laboratory undrained tests.
Clay with gravel or cobbles	As for gravel, etc, above except that Class 4 samples of stiff/hard clay can be recovered by rotary core drilling.		
Weak rock	Boring using clay cutter, shell or chisel may be possible. Class 5 from shell. Class 4 from clay cutter. Class 3 or 4 from sampler. Rotary core drilling of harder rocks. Cores likely to be deficient of weak bands and weak materials.	Field tests best as above. Pressuremeter and SPT give rough indication of strength and compressibility. Percussion drilling monitored by engineering geologist can indicate fissure, void and soft layer geometry.	Field tests best, and estimates based on visual inspection in shafts calibrated from field tests and foundations elsewhere. Approximate estimates of strength possible from SPT calibrated empirically.

APPENDIX B COMMENTS ON PILE INSTALLATION

The following comments of design engineers were recorded during the Survey.

Comments on driven piles

Comments/advantages	Warnings/disadvantages
Driven piles are used on more bridges than bored piles. Segmental driven precast concrete piles have increasingly been found economic. Every pile is 'tested' or at least compared with load-tested piles by the driving process. Steel, precast concrete, and cased cast-in-place piles have assured cross-section.	Driving to a set is usually not reliable. The dynamic driving formulae are found to be unreliable and empirically derived sets are dependent on ground conditions. In some ground, such as chalk, a poor set can be adequate if soil strength increases with time. In contrast in silty soils high pore pressures develop at the base and resist driving, so giving the appearance of hard ground (on redriving next day driving may be easier), but under long term loading settlement can be large. **The pile's capacity and appropriate set/depth can only be verified by load test.**
Type of hammer needs to be selected to suit ground conditions. For example, at the limits of range of soil types: a slow hammer works well in clays but not sands and gravels, while a vibrating hammer works well in saturated sands and gravels and not in clays.	In built-up regions noise restrictions may limit choice of type. Some types are difficult to extend/cut-back if depths driven differ from those anticipated. Closely spaced piles have to be monitored to **ensure driving of one does not lift the adjacent one.** Redriving may be necessary. Also shear in soil at side during driving may later relax and in doing so lift piles. If piles **meet refusal at different depths** then a check should be made that base bearing loads of the higher ones do not get transferred to deeper ones by 45^{o} spread of load. Fixed length precast concrete piles are only appropriate in known uniform conditions since otherwise problems are encountered in extending/shortening. Precast concrete piles and shell piles have been known to break during driving into dense gravel and Keuper Marl.
It is often worth using high tensile steel, even if not needed for load capacity, since driving is easier with less tearing. Steel H piles can usually be relied on to drive through a dense layer if required.	**Some very expensive delays** have resulted following use of steel H piles to bear on weak rocks and shales, **when piles have been designed on basis of steel section capacity rather than rock's capacity.** The piles did either not meet refusal or reduced capacity of neighbours as a result of splitting rock.
Steel H and cased piles have advantage of low mobilisation cost and can be placed by main contractor. (It may be worth down-grading nominal capacity to compensate for lower standard of workmanship). They can readily be lengthened by welding.	Main contractors do not generally have the experience of specialists in avoiding installation problems. Bottom driven cased piles with their steel casings are not suitable if they are too long since casings can break in tension during driving (lack of movement of top of casing gives appearance of refusal) and/or hard upper strata can grip casing and prevent penetration of soft stratum at depth. Vibration can cause damage to neighbouring buildings.

Comments on bored piles

Comments/advantages	Warnings/disadvantages
Bored piles are sometimes chosen for the questionable reason that the designer feels confident that if the pile is installed in accordance with drawings he can be sure of its performance, and not liable to as many claims as driven piles. However bored piles are not appropriate for all ground conditions and difficulties in installation still lead to claims and expensive delays. Surface of site and adjacent piles are not raised by displacement.	**Not usually suitable for bearing in granular soil.** Concrete strength is not so important: the **need to prevent voids forming is paramount.** Borings must be cased or constructed under bentonite if ground is unstable. Casing must be clean to ease extraction. **It is usually impossible to drive casing ahead of bore in granular soils and overdig occurs.** **Voids, necks or 'growers'**, are likely to occur on lifting of casing, but can usually be avoided by careful contractor. To prevent voids developing near top an excess head of concrete is maintained, but sometimes this is not practical. If there is a risk of necks forming in piles in soft ground the designer is advised to pay for permanent casings. Unless the level of the reinforcement has changed it is impossible to tell afterwards from above ground (even the 3-tube method of integrity testing cannot detect slumping of concrete outside reinforcement). Driving casing is a slow expensive process: the contractor will always try to avoid it and is likely to try to qualify in the tender that depth of casing of so much has been assumed on basis of a site investigation. Waterfilled bores in weak rocks and stony ground are likely to have **heavy sediment at bottom** so that only shaft should be relied on for load transfer. Sealing the casing into rock is likely to be difficult and silt may flow in. Some piles have been constructed with a precast base driven out of the casing into rock to expel silt from the bottom. When placing concrete under water the **water must have reached equilibrium** and a **tremie** must be used. Segregation and cavities have been reported even when water is not under artesian pressure. Shaft adhesion is affected by carelessness of gang. Water added to ease boring reduces adhesion, particularly in short term (ie for test). **Reinforcement must not congest flow of concrete.** Minimum bar spacing 150 mm. Make reinforcement same length in all piles and not full length: changing lengths to suit bores causes delays and softening of ground; lapping causes congestion. Bobs should be avoided during installation or made tangential: inwards gets in way of concrete, outwards in way of casing. **75 mm cover is usually needed,** not 40 mm. Concrete must have high workability. At cutting back care should be taken not to fracture pile, particularly when concrete is green.

Comments on bored piles (contd)

Comments/advantages	Warnings/disadvantages
Small diameter:	
In general groups of small diameter piles are cheaper than large diameter for the same load, particularly for friction piles because of their greater perimeter/area ratio. But the pile cap for a large number of small piles can be more expensive.	**Serious disadvantages that inspection of shaft and base not possible.**
Not so likely to need casing as large diameter piles because the soil is better able to arch around a small hole.	
Can be installed with limited headroom.	Actual sizes can be significantly different from nominal sizes specified. If design specifies 500 mm a tripod contractor could offer a 19 in, which below level of casing is 17 in. An auger-rig could only do 18 in which is called 450 mm even though it is 20 in around casing. Designers need to check that what they want is practical and specify it precisely.
	500 mm diameter is not a standard for augered piles which have diameters in increments of 150 mm from 450 mm.
	Proper supervision of large number of tripod rigs can be impractical.
Larger diameter:	
Soil can be inspected and probed in-situ, except when constructed with bentonite.	Uneconomic for small structures because of high mobilisation costs. Heavy rigs require good access and large clearance.
The larger bores are more economic. A 750 mm diameter hole can be bored as quickly as 450 mm holes.	Difficult to install in poor ground because casings are installed full length and not segmentally, and have to be pulled out and replaced by longer lengths every 3 m. However use of bentonite is becoming more common.
Speed of construction reduces ground relaxation and so enables greater bearing pressures and shaft adhesions to be utilised.	**Underreams must be cleaned and inspected.** Defective equipment or incompetent driver can result in 7 or 8 skips of debris instead of 2.
Capacities of up to and exceeding 2 000 tons can be achieved.	Pile caps must be thicker to carry high concentrated loads.
Underreams can be made on piles with 750 mm shafts and larger (but **not** smaller).	If pilot holes are bored prior to main boring to establish base and casing levels these should be beside and not through main bore or underream area, so that water encountered does not flood or soften main boring.

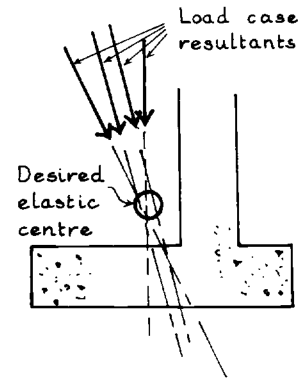
APPENDIX C ARRANGEMENT OF PILES IN GROUP

The following procedure is followed by some designers to determine the optimum arrangement of piles in a group to resist inclined loads.

It is assumed that the rows of piles are in two sets with different rakes (including vertical). At least one set has two or more rows. Consequently there must be at least three rows in all.

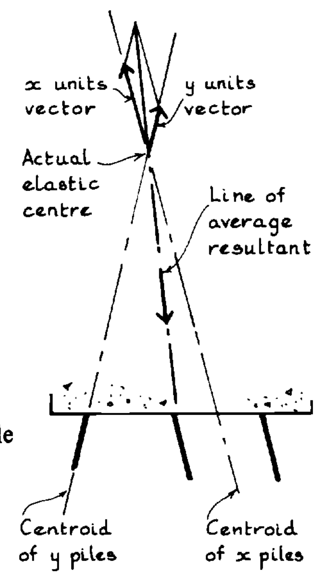
1 Dimension the pile cap as if it were a spread footing. The maximum concentration of load under a pile cap is often about 400 kN/m^2.

2 Draw the lines of action of all the load case resultants on a cross-section of the pile cap. The point at which they are most nearly coincident is the optimum point for the elastic centre. The elastic centre of a group with two sets of piles at different rakes is at the intersection of the raking centroid lines of the two sets. (Resultant loads through the elastic centre induce only thrusts in the piles in each raking set: the relative magnitude of the load in each set depends on the inclination of the resultant. The moment on the group due to eccentricity of a resultant from the elastic centre is opposed by variations in axial loads across the rows of piles at each rake: at least one set of piles must have two or more rows to provide a lever arm.)



3 By inspection a line is drawn representing the average of the load case resultants and passing through the preferred elastic centre position. Some construction load cases may not fit the general pattern and these should be ignored at this stage, though checked in later analysis.

4 A pile group is now sought which has the above elastic centre and which would have equal loads in all piles when subjected to the average load case resultant. Lines are drawn through the preferred elastic centre at the two rakes to represent possible centroid lines. The relative numbers of piles at each rake are proportional to the relative lengths of component vectors along the centroid lines that are required to provide a resultant vector along the average load case resultant line. The total number of piles is found by dividing the total load by the maximum permissible load per pile. The piles at each rake are then arranged about their centroid with an adequate spacing for transferring the loads to the ground. If it is not possible to design a pile layout so that the elastic centre is as low as the intersection of load case resultants, then piles are inclined as steeply as possible, subject to the maximum practical rake, in order to bring the elastic centre as low as possible on the average load case resultant line.



4 If the extreme load case resultants differ markedly from the average the number of piles might need slight modification. If the load case giving maximum loads causes a variation in the loads in parallel rows of piles, then the variation can be effectively removed by moving the whole group laterally by an amount approximately equal to the moment in the parallel rows (due to the inequality of pile loads) divided by the resultant applied load. From comparisons of maximum pile loads for two positions of the group the designer can interpolate the optimum position.

5 For abutments it is often convenient to design all the piles except the back row with the same slope as the average load case resultant, and then design the back row to provide equilibrium for the load case with minimum horizontal component.

APPENDIX D DEPARTMENT OF TRANSPORT MEMORANDA RELATED TO THIS REPORT

Memoranda current in March 1978:

Number	Title	Date	Division
H11/70	Site investigation procedure (Amendment 7/11/75) (Amendment 11/8/77)	19/10/70	E Int
H5/72	Notes on the treatment of old filled mine shafts and disused shallow coal workings	30/6/72	E Int
H3/76	Model contract document for site investigation (Amendment 16/8/76)	Feb 1976	E Int
IM4	Pulverised fuel ash backfilling to structures	19/12/69	BES
BE1/73	Reinforced concrete for highway structures (1st Revision 9/8/73)	30/1/73	BES
BE7/74	Lateral loading on piled foundations	17/12/74	BES
BE5/75	Rules for the design and use of freyssinet concrete hinges in highway structures	7/3/75	BES
BE7/76	Suite of bridge design and analysis programs Program HECB/B/8 (RETWAL)	20/12/76	HECB
BE1/77	Standard highway loadings	14/2/77	BES
BE4/77	The inspection of highway structures	1/4/77	BET
BE5/77	Suite of bridge design and analysis programs Program HECB/B/7 (PGROUP)	1/4/77	HECB
BE3/78	Reinforced earth retaining walls and bridge abutments for embankments.	27/4/78	BES

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