

TECHNICAL RECOMMENDATIONS FOR HIGHWAYS

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**GUIDELINES FOR THE HYDRAULIC  
DESIGN AND MAINTENANCE OF  
RIVER CROSSINGS**

**VOLUME II :       STRUCTURAL ASPECTS,  
BRIDGE CONFIGURATIONS  
AND FOUNDATIONS**

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## PREFACE

At the rate at which new information is generated and made available it is becoming increasingly difficult for the practising civil engineer to decide on the appropriate norms and analytical methods to be used in designs. Although there will always be cases necessitating a comprehensive independent literature study to ascertain the best suited norms and methods to achieve a sound solution, it is recognised that they tend to be the exception rather than the rule. The designer cannot be expected to undertake such detailed studies for each case as this would become impractical. Consequently the need for practical guidelines.

The main aims of these guidelines are to make recommendations on methods of calculation, design norms as well as legal and other issues which need to be taken into consideration in the pursuit of providing safe, economical and viable river crossings. The intention is not to stifle original thinking and new development, and thus designers are expected to deviate from the general recommendations where optimum solutions clearly fall outside the general applicable norms. The guidelines are furthermore intended to serve as a basis for governing bodies to formulate their policies on design standards with due consideration of legal and other risks.

These guidelines comprise seven volumes each dealing with a particular subject or related subjects.

## SYNOPSIS

During the 1970s and 1980s a number of major floods caused serious damage to the road and rail infrastructure in southern Africa. These events prompted the re-evaluation of design norms to secure cost effective and safer designs to withstand the expected imposed forces.

This volume deals with structural problems which have arisen, suggested bridge configurations, foundation investigations, foundation design and structural design.

Whilst this document should not be considered as comprehensive and all-inclusive, it presents basic issues which need to be considered when judgement is made.

**Keywords :** Bridge configuration, structural design, flood damage, foundation design, foundation investigation, investigation techniques.

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## NOTATION

Symbols not included or varying from those given in this list are of minor importance or are used in special cases only. Where they occur, they are adequately explained in the text.

		SI UNITS
$B_t$	Width of contact (diameter of pile)	m
$C_u$	Undrained shear strength	kN/m <sup>2</sup>
$D$	Base diameter	m
$f_{su}$	Ultimate bond stress between wall of pile and rock	MPa
$K_h$	Modulus of horizontal subgrade reaction (cohesionless soils)	MPa/m
$K_{hs}$	Modulus of horizontal subgrade reaction (cohesive soils)	MPa/m
$N$	Standard penetration readings (blows per 300 mm)	No
$N_h$	Constant of horizontal subgrade reaction	-
$q_a$	Unconfined compressive strength	MPa
$q_c$	Point resistance of Dutch probe	kN/m <sup>2</sup>
ucs	Unconfined compressive strength	MPa
$z$	Depth below ground	m

## 1. STRUCTURAL PROBLEMS WHICH HAVE RESULTED FROM FLOODS

### 1.1 Introduction

At times of high flow, several effects come into play and in the event of failure, it is seldom possible to attribute failure to a single cause.

Research done during 1990 on 58 bridge failures in southern Africa since 1902, mainly rail carrying, revealed causes and modes of failure as listed in Sections 1.2 and 1.3 below. Ten failures occurred during 1959, eight occurred during the 1984 Domoina cyclone and five occurred during 1987.

### 1.2 Causes of failure

Inadequate bridge openings and inadequate foundations were respectively the most and second most common direct or contributory causes of failure.

A further common direct cause of failure, noticeably in Natal, was the location of bridges to take advantage of shallow founding material at the expense of good hydraulic alignment. In such cases flood waters impinged directly on an approach embankment washing it away and in some cases portions of the bridge as well.

A common contributory cause of failure was debris collection. To a lesser extent contributory causes of failure were found to be structural design and construction errors.

### 1.3 Modes of failure

The most common mode of failure was found to be pier damage or loss (with associated effects on decks), followed by embankment washaways, abutment damage or loss, and displacement of decks.

### 1.4 Structural shortcomings

#### 1.4.1 General

Certain failures could have been averted, had structural shortcomings as listed below been correctly designed and constructed.

- Shallow pile founding due to under-estimation of scour depths and severe limitations in construction equipment in the earlier years.
- Inadequate bending strength of piles and caissons due to inadequate cognizance taken of scour depth and/or hydrodynamic forces aggravated by debris collection.
- Inadequate restraint and/or lack of air escape vents which resulted in dislodging of decks. (Beam and slab decks particularly prone).
- Inadequate concreting in a caisson due to construction difficulties.

- Inadequate reinforcing steel in a caisson due to a design shortcoming.
- Incorrectly positioned caisson due to obstructions encountered during construction.
- Inadequate vertical steel in multiple column piers with capping beams which caused bending failure at connections between columns and capping beams and between columns and bases.
- Foundations founded on boulders thought to be bedrock.
- Uneven founding of a large caisson which resulted in tilting of the caisson.

## 2. BRIDGE CONFIGURATIONS

### 2.1 General

Guidelines on the required hydraulic opening, the location of the bridge, protection works, channel training works, siting relative to embankments, effects of skew, etc. are given in Section 5 Volume I "Hydraulics, Hydrology and Ecology". This section deals exclusively with components of the bridge.

### 2.2 Piers

Pier alignment should correspond as closely as possible with the direction of river flow at the maximum design flow. Supports comprising multiple columns should be used with caution and with the approval of the client as debris collects therein, reducing waterway opening and increasing scour and hydrodynamic forces. Wall type supports are advocated for this application unless such a configuration adversely affects the flow pattern. If multiple columns are used the minimum clear spacing should be 2,5 m.

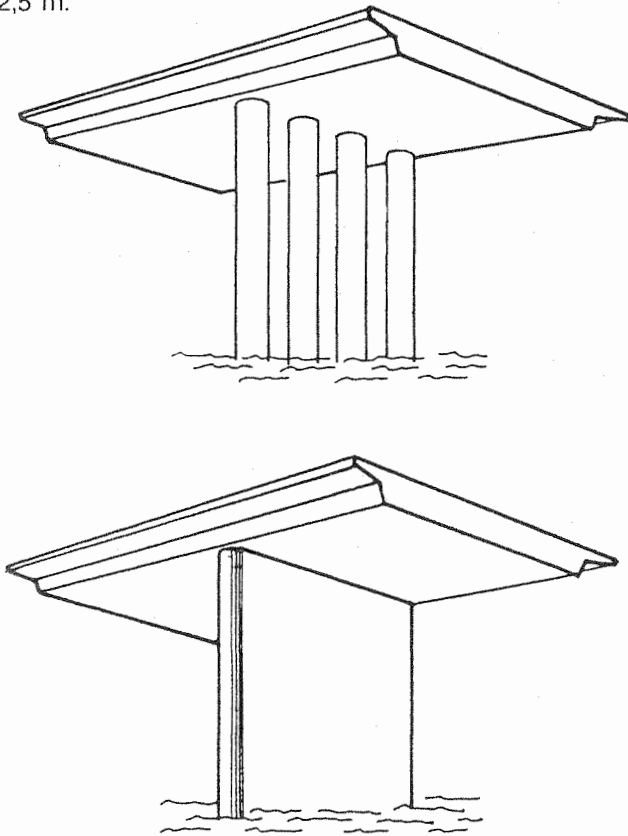


FIGURE 2.1 : RIVER PIER CONFIGURATIONS

It should be borne in mind that increase in pilecap width will increase scour depth. For this reason the width selected must be considered together with scour depth to arrive at the most economical solution. (Section 4 of Volume I "Hydraulics, Hydrology and Ecology")

Floataction or buoyancy forces arising from hollow piers must be taken into account when considering overall bridge stability.

The pier ends should be rounded to reduce drag forces and reduce tendency for debris to collect. Drag coefficients for various edge treatments are given in Section 6 of Volume I, "Hydraulics, Hydrology and Ecology".

## 2.3 Abutments

### 2.3.1 General

Too many independent and individual requirements influence the selection of abutments to allow meaningful directives to be given. However, some guidelines are given below:

- Spillthrough abutments are undesirable for river structures, unless it can be shown that minimal erosion of the spillthrough material will occur and that the embankment settlements and movements will be within acceptable limits.
- Perched abutments founded in fill should be used only when there is no danger of the embankment being washed away, i.e., where the flood level is low. This condition may arise when the grade line is dictated by the road geometry. When perched abutments on piles are selected, the system must be stable under flood action, without the surrounding embankment and without live load. The drag-down effect of settling embankments must be taken into account in the design of the raked piles.
- Wingwalls are preferred to return walls unless it can be shown that return walls provide a more economical solution and that the approach embankments will be safe under the action of design flood.

## 2.4 Decks

### 2.4.1 General

Bridge decks commonly used for river bridges fall into two broad categories, namely continuous and simply supported. If adequate support is assured by spread footings on rock or adequate piles socketed into rock, continuous decks are preferred. If there should be doubt about the adequacy of support such as suspect piles, uncertainty about founding stratum, etc. simply supported decks should be considered to facilitate reinstatement in the event of a washaway.

### 2.4.2 Advantages of continuous decks

The advantages of continuous decks are the following :

- Superior aesthetics.
- Improved rideability.

- Reduced maintenance of joints.
- Reduced risk during construction where construction options such as incremental launching and balanced cantilever construction methods (which do not require support from the ground or reduce the available hydraulic opening during construction) are used.
- Cost savings in most cases.

#### 2.4.3 Advantages of simply supported decks

The advantages of simply supported decks are the following :

- Shorter delays to effect reinstatement if loss of support leads to structural failure of the deck during or after construction.
- Decreased risk if decks are supported from ground during construction.
- Convenient configuration for use of precast beams.

#### 2.4.4 Floatation

Decks should be designed to minimise the effects of floatation if liable to be inundated. Beam and slab decks and other deck configurations which will trap air under flood conditions shall be provided with air escape openings (vents) to all compartments. As a general guide the area of the vertical escape openings should be not less than 0,01% of the plan area of air trapping compartments. Air escape openings must be placed at high points in compartments to ensure that all air may escape. Holes need not all be through the deck as continuity of air escape paths may be attained by providing horizontal openings through webs and/or diaphragms directly below the deck slab. Drainage downspouts located in air trapping compartments will serve a dual function by allowing air escape as well.

### 3. FOUNDATION INVESTIGATIONS AND DESIGN

#### 3.1 Foundation investigations

##### 3.1.1 Introduction

Section 3.1 sets out the minimum requirements for foundation investigations for river bridges and is primarily aimed at:

- designers who stipulate the requirements for site investigations,
- geotechnical engineers and engineering geologists who undertake the investigations, and
- those responsible for the drawing up of contract documents.

Chapter 3 does not provide details of investigation methods or laboratory tests. However, references are provided where appropriate.

##### 3.1.2 Responsibility for investigations

Foundation investigations should be carried out by geotechnical engineers (registered professional engineers) or engineering geologists (normally registered natural scientists) who are experienced in the type of work envisaged.

The individual or company appointed to undertake the work should be covered by a Professional Indemnity insurance policy, the value of which should be commensurate with the scope of the work.

This document provides guidelines only and does not attempt to prescribe how the investigation should be undertaken.

##### 3.1.3 Purpose of investigation

Foundation investigations for river bridges have two major purposes:

- The investigations will assist the designer in the selection of foundation types, the preparation of foundation designs and the design of approach fills.
- The investigations will provide guidance to the contractor in the preparation of his tender and for the planning and execution of the work.

The consequences of a poor site investigation are the inappropriate selection of foundation types, inadequate foundation designs, settlement problems and claims arising from unforeseen ground conditions.

### 3.1.4 Information required by the designer

The foundation investigation report should provide clear guidance for the design engineer, enabling him to select the most appropriate foundation types for the bridge. The investigation should quantify the parameters required for the design of the proposed foundations and approach embankments.

The more important requirements are given below:

#### (i) General

- Recommendation as to the appropriate foundation type.
- Any aspects of the site conditions requiring special design consideration.

#### (ii) Spread footings

- Expected depth of founding and nature of founding material.
- Nature of overburden and stability of foundation excavations.
- Depth of water table and date measured.
- Allowable bearing pressures at likely alternative levels.
- Compressibility of founding material for calculation of foundation settlements.
- Heaving characteristics.
- Depth of general scour as deduced from material characteristics.
- Requirements for the inspection and approval of founding material.

#### (iii) Piled foundations

- Recommendations as to most appropriate pile type(s).
- Depth of water table and date measured.
- Expected pile founding depth, nature of founding material and required configuration at pile toe i.e. end bearing, socketed or underreamed.
- Design parameters for the assessment of pile capacity.
- Consistency/compressibility of material through which the piles are installed for assessment of subgrade reaction.
- Compressibility of founding stratum for assessment of pile group settlement.
- Extent of any expansive material and heave forces likely to be exerted on pile shafts.
- Depth of general scour as deduced from material characteristics.
- Presence and type of obstructions.
- Aggressiveness of groundwater.
- Requirements for the inspection and approval of founding material.
- Rock classifications in accordance with Table 6113/1 of the CSRA Standard Specifications in order that quantities may be scheduled accurately and to enable the resident engineer to assess the conditions for payment purposes.

## (iv) Caisson foundations

- Recommendations as to the most appropriate caisson type(s).
- Depth of water table and date measured.
- Allowable bearing pressure.
- Requirements for the inspection and approval of founding material.
- Expected caisson founding depth, nature of founding material and required configuration at the base i.e. end bearing or socketed.
- Design parameters for the assessment of caisson capacity.
- Consistency/compressibility of material through which the caissons are installed.
- Compressibility of founding stratum for assessment of settlement.
- Extent of any expansive material and heave forces likely to be exerted on caissons.
- Depth of general scour as deduced from material characteristics.
- Presence and type of obstructions.
- Rock classifications in accordance with Table 6113/1 of the CSRA Standard Specifications in order that quantities may be scheduled accurately and to enable the resident engineer to assess the conditions for payment proposes.

## (v) Approach embankments

- Shear strength of founding soils for assessment of safe side-slopes.
- Compressibility of founding soils for assessment of settlement embankment and need for pre-loading.
- Restrictions on rate of construction to allow dissipation of excess pore water pressure.
- Magnitude of downdrag forces on piles due to settlement of the embankment and founding soils.

## 3.1.5 Information required by contractor

The foundation investigation report, together with an inspection of the site, should provide the contractor with sufficient information to reasonably anticipate any problems which may occur during the execution of the works in accordance with the specified requirements. This will enable the contractor to submit a realistic price for the execution of the work and to select the most appropriate equipment and construction techniques.

The more important requirements are given below:

## (i) Spread footings

- Nature and depth of founding stratum.
- Depth of water table, date measured, and expected rate of inflow.
- Nature and consistency of material to be excavated.
- Opinion should also be expressed on the stability of open excavations and the need for temporary lateral support.

## (ii) Piled and caisson foundations

- Nature, depth and hardness of founding stratum.
- Nature and consistency of the material through which piles or caissons must be installed with particular reference to boulders or other obstructions.
- In the case of augured piles, the expected stability of the sidewalls of the auger hole and need for casing.
- Depth of water table, date measured and expected rate of water inflow.
- Special requirements for cleaning of pile holes eg. hand cleaning of bottom of hole or removal of sidewall smear.
- Nature and hardness of socket material.
- Trafficability of site and access for piling and caisson installation equipment.
- Clear description of equipment used for site investigation, drilling rates and diameter of hole.

## (iii) Approach embankments

- Source and properties of borrow material.
- Trafficability of site for construction equipment.

## 3.1.6 Investigation techniques

The investigation for river bridges normally takes place in a number of phases. These phases include a desk study and site visit, route/site selection, preliminary field work, detailed investigation and verification of conditions during construction. In certain instances, the process may be iterative prior to final site selection and commencement of detailed investigation.

The investigation techniques available fall broadly into four categories, namely remote sensing, geophysical methods, direct investigation by means of boreholes/trialholes and field/laboratory testing. These techniques are described below.

## (i) Remote sensing

The most commonly employed form of remote sensing is the study of aerial photographs and photo mosaics. Such photo interpretation normally forms an important part of the route/site selection phase of the investigation.

Aerial photographs, photo mosaics and ortho-photos complement topographical and geological maps of the area and provide planners and designers with an overview of the route or site being investigated. Overlapping aerial photographs studied through a stereoscope can provide detailed information with regard to geological features (faults, dykes, geological boundaries, etc.), soil types and drainage often not apparent even in the field.

This information is valuable not only for site selection but also in the planning of a geotechnical investigation.

Other forms of remote sensing include infrared thermal scanning which can be of assistance in dolomitic areas and satellite imagery.

(ii) Geophysical methods

Geophysical methods in common use include seismic, electrical resistivity, electro-magnetic and gravimetric surveys. Of these, the seismic survey is the most common.

Seismic surveys are capable of detecting the location of interfaces at which changes in seismic wave velocity occur and of measuring the seismic wave velocity in the various strata. These surveys provide information on the rockhead topography and an indication of the hardness and hence excavatability of the rock. Geophones may be placed under water on the river bed permitting the determination of rockhead topography across the river. The results of a seismic survey should be viewed as an indication or approximation of the subterranean topography and must always be confirmed by direct investigation.

Electromagnetic and electrical resistivity surveys are generally used to detect dykes or boundaries between materials with different electric/magnetic properties. Gravimetric surveys are generally used only in dolomitic areas.

(iii) Boreholes and trialholes

(a) Purpose

The purpose of boreholes and trialholes is to permit visual examination and testing of the in situ material or of samples recovered from such holes.

(b) Definitions

The term borehole commonly refers to small diameter holes drilled using rotary core drilling or percussion drilling techniques from which cores or chip samples are recovered. Trial holes generally include shallow backactor test pits or large diameter auger holes, both of which may be profiled in situ.

(c) Choice of equipment

In most instances, there are definite advantages to be gained from using similar equipment, both in terms of type and capacity, for the investigation as will be used for the construction of the foundations. For example, where augured piles are to be installed, it is preferable to include the use of a large diameter auger rig similar to that which will be used for the installation of piles, in the site investigation programme where circumstances permit. This will provide a direct indication of the problems likely to be experienced during construction. Conversely, the use of a backactor for the excavation of test pits holds distinct advantages on a site where shallow spread footings are envisaged. Rotary core drilling is often the most suitable method of investigation below water level.

Other important factors dictating the selection of the type of equipment to be used for a site investigation include:

- Depth of water table and presence of surface water.
- Expected depth of founding.
- Presence of obstructions such as boulders.
- Type of samples required.
- In situ tests envisaged.
- Cost implications.

(d) Safety of personnel

In the selection of investigation techniques, due account must be taken of the safety of personnel undertaking the investigation.

All profiling and inspection of auger holes and test pits shall be carried out in accordance with SAIEG, SAICE and AEG (1992) "A code of practice on the safety of persons working in small diameter shafts and test pits for civil engineering purposes".

(e) Spacing and depth

The foundation investigation should cover the full length and width of the structure including the approach fills. Occasionally additional holes may be required beyond the extent of the structure to establish the geological discontinuities or other such features.

In general, at least one hole is required below each abutment or pier position. Holes should be positioned on both sides of the bridge centreline. Additional holes may be required to investigate variations in material properties or geological discontinuities.

The depth of investigation should be sufficient to provide information on all materials whose properties may influence the performance of the foundations and scour behaviour. In soils, this may include all materials to a depth equivalent to twice times the width of the foundation measured from the underside of the foundation. In rocks, the investigation should be continued to a depth below the founding level at which the presence of weak or compressible strata within the rockmass is unlikely or will not influence the foundation performance. In most cases this will require proving the properties of the rock for a minimum depth of 3m below the rockhead or below founding level. In boulder beds, dolomites or other situations where random variations are possible, the depth to which the rock is proved may have to be increased.

(f) Records

It is essential that full records are provided of all drilling operations and excavation of trial holes undertaken on site. Such records should include the date of excavation/drilling, type of plant used, size of hole, depth of casing, date of profiling and name of profiler. This information should be included in any summary provided to the contractor for tender purposes.

(g) Profile descriptions

Soil and rock profiles must be described in accordance with guides compiled by SAIEG, SAICE and AEG (1992). The use of non-standard terminology can be misleading and give rise to misunderstandings and claims.

Specifications regarding core drilling, percussion drilling, test pitting and augering are published in CSRA (1992) Standard Specifications for Drilling Investigations.

Wherever possible, the origin of the material should be clearly indicated. In particular, a clear distinction should be made between transported and residual soils. The pebble marker at the base of the transported profile should be identified where present. As an initial assessment it may be assumed that general scour is unlikely to occur in the residual material below the pebble marker unless the flow of the river is significantly altered by the construction of the bridge (Section 4 of Volume I "Hydraulics, Hydrology and Ecology"). Any evidence of recent erosion and deposition of material should also be noted to assist in the assessment of the likely depth of scour.

Where auger holes are drilled to provide information likely to be used for an augured pile contract, the geotechnical engineer or engineering geologist can, with minimal additional effort, provide much information which will be of value to the piling contractor in the preparation of his tender. This information includes an accurate description of the type of auger rig used, the type of teeth fitted to the flight, the rate of drilling particularly near the bottom of the hole, the total drilling time, reasons for stopping the hole (water inflow, refusal, etc.) and the nature of refusal encountered (whether abrupt or gradual). Additional observations to be made during profiling include collapse of sidewalls, depth over which water inflow occurs, thickness of sidewall smear and sidewall roughness.

An accurate description of the soil profile using standard terminology is often the most valuable part of a site investigation report.

(iv) Field and laboratory testing

(a) General

The most important design parameters are the shear strength and compressibility of the founding material. These two parameters facilitate the computation of bearing capacity and settlement respectively. The properties of the material above founding level are also important in so far as they affect the shaft friction of piled foundations and the stability of excavations during construction. In certain instances, the corrosivity of the ground and ground water may also be of importance.

Refer to Tables 6.1.1 and 6.1.2 in Appendix 6.1 which provide details of laboratory tests on soils and rocks and field testing methods. References are provided in Tables 6.1.1 and 6.1.2 for further information.

## (b) Shear strength

With few exceptions, the direct determination of shear strength tests are carried out in the laboratory.

Where the material being tested is rock, such tests include unconfined compressive strength or point load tests conducted on intact specimens of rock core. However, it must be remembered that the weaker (and hence more critical) layers in the material may not be recovered intact, giving rise to non-representative sampling in which only the stronger portions of the material are tested.

In the case of soils, shear box and triaxial tests are the most common methods of shear strength determination. For the determination of bearing capacity or the long term stability of approach fills, effective (drained) strength parameters are required. In the case of soft clays, the undrained unconsolidated shear strength will govern the stability of fills and excavations during construction.

The most common field test used for the direct determination of shear strength of mainly soft cohesive soils is the shear vane. Such tests may be carried out from the surface using a tripod mounted vane and torque head or in a trial hole using a hand-held shear vane.

In situ tests also provide an indirect measure of the shear strength or bearing capacity of the material, usually based on empirical correlations. Such tests include standard penetration tests (SPT's), dynamic or static cone penetration tests (CPT's) and pressure meter tests.

## (c) Compressibility tests

Compressibility of the founding soils may be determined either in situ or in the laboratory.

In the case of rocks, field tests include Goodman jack or pressuremeter tests in NX size rotary core boreholes or plate load tests in auger holes or test pits. Laboratory tests on rocks normally comprise unconfined compressive strength tests in which the vertical and horizontal strain of the specimen is measured by means of strain gauges.

Soil compressibility may be determined in the field by means of horizontal plate load tests in auger holes or pressuremeter testing in rotary core boreholes. Laboratory tests include one dimensional consolidation testing in an oedometer or a triaxial consolidation. In both these tests, the rate of consolidation can also be determined. Larger test specimens can be accommodated in the Rowe cell.

As with the determination of shear strength, in situ tests can provide an indirect measurement of compressibility based mainly on empirical correlations. Such tests include SPT's and CPT's.

## (d) Heave and collapse movements

Where expansive soils are encountered, heave of the overburden can exert significant uplift forces on pile shafts. Grading and indicator tests or double oedometer tests are frequently used to identify expansive materials and to quantify the amount of heave expected. The magnitude of the uplift forces will depend on the shear strength of the material at natural moisture content rather than in a saturated condition. Such shear strength determination may be carried out either in the triaxial or shear box test.

Where expansive soils are investigated, the minimum information to be provided should include grading and indicator test results, in situ density and moisture content. These should be supplemented with oedometer test results and shear strength test results as necessary.

In some instances, the soils above the water table may exhibit a marked change in stiffness as the moisture content increases. A typical example is soils with a collapsible grain structure which will settle significantly on saturation. In such cases, the results of in situ tests at natural moisture content may produce misleading results. Oedometer tests are normally used in the identification of collapsible soils. In situ densities and moisture contents may also be of assistance but are not reliable indicators.

(e) Depth of historical scour

Apart from the identification of the origin of soils as discussed in Section 3.1.6 (iii)(g), there are few reliable indicators of the likely depth to which general scour had occurred. Nevertheless, careful logging of soil profiles and certain field tests such as cone penetration tests may assist in identifying features which indicate the base of depositional episodes in the transported soil profile. These features include layers of gravel or coarse sand often accompanied by abrupt changes in permeability. Such coarse layers can often be detected by penetrometer refusal on gravel or from samples retrieved from standard penetration tests. Changes in permeability can be detected by piezo-cone penetration tests in which the pore water pressure generated by advancing the cone into the soil is measured.

(f) Frequency of testing

The frequency of testing is left to the discretion of the geotechnical engineer or the engineering geologist.

It is recommended that sufficient tests be undertaken to confirm the relevant properties of each soil horizon which could have a material effect on the performance of the foundations except where the properties can be ascertained by visual observation or experienced judgement. Nevertheless, where a particular property of the material (such as shear strength or compressibility) is critical to the performance of the foundations, it is recommended that at least three tests to determine this property be undertaken on each material type concerned in order to ensure that a representative result is obtained.

### 3.1.7 Foundation investigation reports and contract documents

(i) Foundation investigation report

The foundation investigation report should accurately reflect the conditions on the site taking due account of the information required by both the designer and the contractor given in Sections 3.1.4 and 3.1.5. In certain instances, two separate reports may be issued for the designer and contractor.

The foundation investigation report must include the following essential information:

- The terms or reference and purpose for which the investigation was conducted (eg whether for planning purposes, foundation design, etc.)
- The precise location of the site.

- A description of the site including regional geology, vegetation, topography and other general features.
- A description of the field work giving details of the type of equipment used, problems experienced and any other information which may be relevant.
- A description of the typical soil/rock profile in each soil/rock zone identifying potential problems which may arise from this profile.
- Concise recommendations for the design and construction of foundations and approach fills in particular with regard to scour.
- Detailed profile record sheets and the full results from any laboratory and field tests.
- Scale drawings showing the location of all testholes and physical features of the site including setting out points. Wherever possible, the co-ordinates and collar elevations of all testholes should be given.
- Reference to the method of classification of materials used.

Where information from the site investigation report is summarised on the drawings, all relevant information should be given, including the date of drilling, type of equipment used and any notes given on the original profile log sheets. Particular care should be taken not to omit information regarding the water table. Even where such a summary is given, the full site investigation report should be made available to the contractor at the time of tender.

(ii) Contract documents

It is recommended that the site investigation report be made an integral part of the contract documents. It is undesirable to limit the validity of the report by means of a disclaimer in the contract documents. Such disclaimers negate the very purpose for which the investigation was conducted, namely to provide reliable information on which the design would be based and the work be priced and executed. It is preferable to provide in the contract documents a means whereby any deviation from the conditions described in the report can be measured and paid for in a fair and equitable manner.

Insistence that each tenderer verify the findings of the investigation during the tender period results in unnecessary duplication of effort and is often unreasonable and impractical. Where the contractor is required to take full responsibility for the ground conditions and their effect on the works as may be the case in certain design and construct contracts, an item should be provided in the contract documents for the carrying out of a further site investigation by the contractor after award of the contract. Provision should be made for full payment of this contingency in the contract documents.

In the case of design and construct contracts or where alternative foundation designs are permitted, full details must be given of the design standards to be met, the loads exerted on the foundations, permissible settlements and the like. Failure to provide such information may lead to the receipt of tender designs based on widely varying assumptions severely complicating the task of adjudicating tenders.

### 3.1.8 Summary

The purpose of a site investigation is to provide an accurate assessment of the conditions on the site to facilitate the design, pricing and execution of the work. In order to achieve this objective, it is essential that the work is carried out in a competent manner, soil profiles are described using standard terminology, the results are reported in full and the validity of the information is not disclaimed.

## 3.2 Foundation design

### 3.2.1 General

Foundations for bridges fall in three broad categories viz. spread footings, piles and caissons. Spread footings are preferred where they are capable of providing adequate support and can be installed more economically than piled foundations or caissons. Caissons were used extensively in the early years of this century but have lost favour as piling methods and equipment have improved. Caissons should however always be considered as they provide suitable and economical founding solutions under certain circumstances.

An important consideration in designing river bridges is that of scour. There is ample evidence to suggest that certain bridge failures in South Africa were due to deep scour around pier foundations combined with inadequate founding. This occurred on some of the older structures which were founded too shallow due to the inability of the piling equipment or caissons used to penetrate deeper.

It is thus extremely important to determine the likely scour depth around the pier foundations. The assessment of scour depth is covered in Volume 1 "Hydraulics, Hydrology and Ecology".

It is essential that foundations are located below expected scour depths or on stable rock. Where foundations are located below the scour depth on material other than stable rock, foundation movements must be within limits that can be tolerated by the structure without distress and impairing its functions.

In the rare case of approach embankments not being in any danger from flood action, spread footings may be placed within the embankment, provided that allowance is made for settlements.

### 3.2.2 Design methods

Ultimate limit state methods for design are generally accepted for the design of bridges, excluding the foundations. The most recent codes generally used in South Africa for foundation design, BS 8004 and SABS 0160 are based on the working load concept.

The Transportation Research Board Publication 343 (1991) incorporates manuals for the design of various foundation types based on limit state methods and serves as a useful introduction to this method of design.

It is recommended that the working load concept for the design of foundations is used until the British and/or South African codes incorporate the limit state methods.

### 3.2.3 Spread footings

Careful considerations must be given to the nature of the material at the founding level as well as underlying layers. In particular, the following aspects must be borne in mind:

- Erodability of material at founding level and below.
- Nature and extent of rock fractures and bedding planes.
- Soft layers below proposed founding level.
- Possibility that boulders may be mistaken for bedrock.
- Dip of inclined or jointed strata toward exposed faces and discontinuities in strata.

It is not possible to give universally applicable allowable bearing pressures due to the vast variation in conditions and circumstances which apply at different sites. Presumed bearing values which may be used for different categories of foundation materials for initially proportioning foundations are given in BS5004 (1986) Refer Table 6.2.1. in Appendix 6.2.

Spread footings on rock should preferably render the structure stable without the use of stressed ground anchors or dowels. If stressed ground anchors can not be avoided, they should be designed to permit checking of residual load in the future. Corrosion resistant dowels may be used provided that the necessary anchorage is assured in the bedrock. Abutments and piers anchored into rock are known to have overturned together with the attached rock, the thickness of which was approximately equal to the dowel embedment length.

### 3.2.4 Pile foundations

#### (i) Global behaviour

The global behaviour of piled foundations with their environment should be considered before starting detail design and calculations.

An 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, in which case alternative assessments and calculations for favourable and unfavourable assumptions such as degrees of fixity, subgrade reaction, inaccuracy of installation and scour depths are catered for.

The whole foundation system should be sketched before the design is commenced. Pile groups are sometimes designed with consideration being given only to the arrangement of the piles at the cap, with lengths selected independently thereof 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 as illustrated in Figure 3.1.

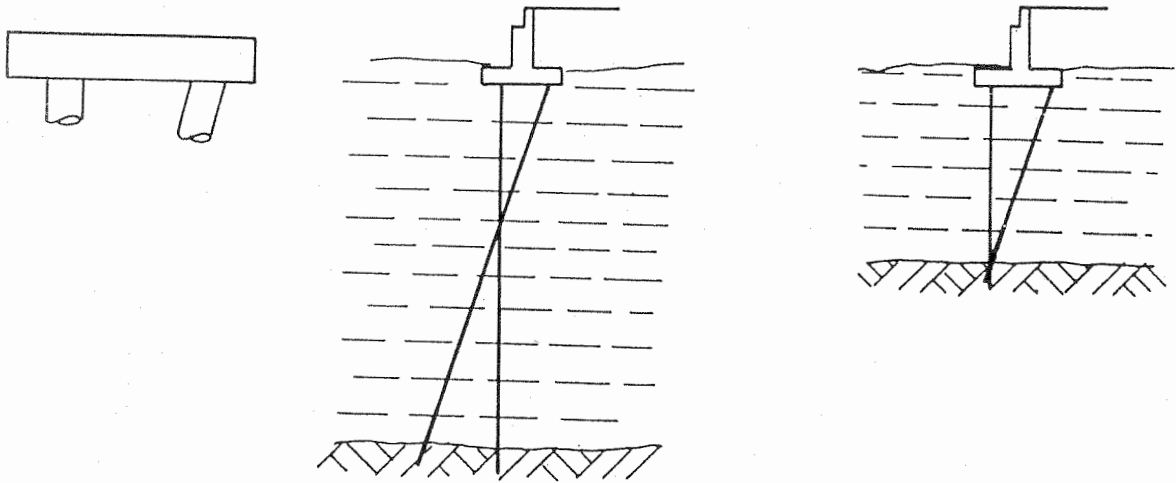


FIGURE 3.1 : PILE LAYOUT

The piles can be spaced at the cap and appear likely to distribute the load to the substratum; while in fact they unnecessarily concentrate the loads at depth. Where piles are not socketed in rock a bridge support may be more efficiently supported by a fan arrangement of piles than by a group with all central piles vertical and edge ones raked as illustrated in Figure 3.2.

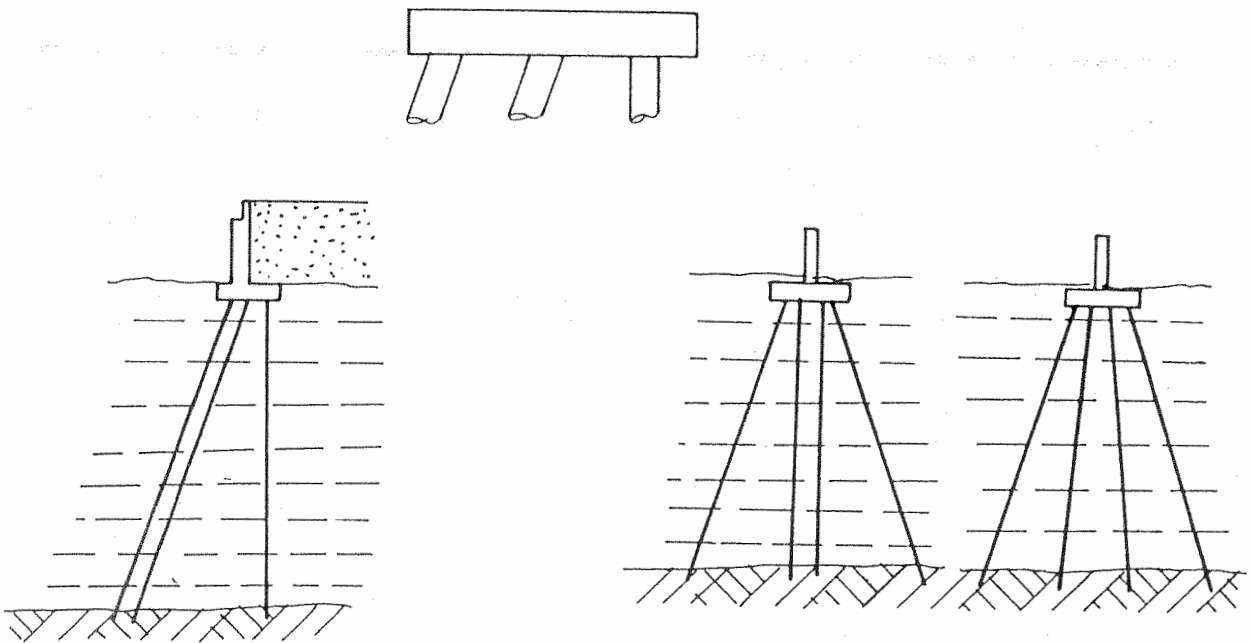


FIGURE 3.2 : PILE LAYOUTS

The pattern of group displacement needs to be considered carefully, particularly if the group includes piles of different rakes as shown in Figure 3.3. A pile group under a vertical load may cause differential settlement in the substratum and the outer piles within a pile group may bear in ground that settles less than piles beneath the centre and thus be subjected to much greater axial and bending loads. Failure of such piles has been known to take place.

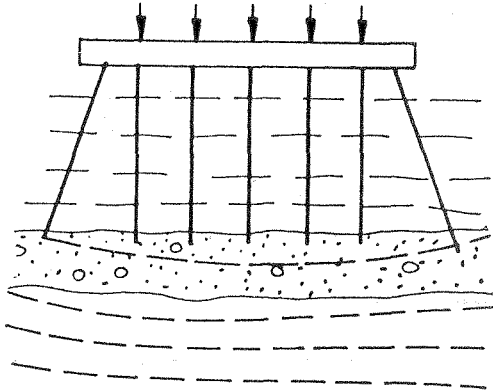


FIGURE 3.3 : DIFFERENTIAL SETTLEMENT

Settlement of an embankment behind an abutment with piles raked backwards can subject these piles to severe bending (Figure 3.4). For this reason back raking piles 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.

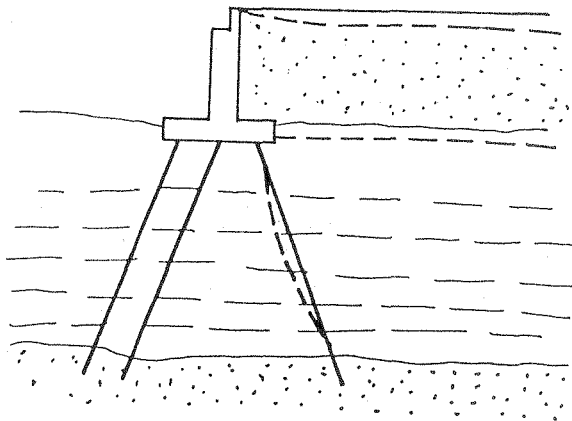


FIGURE 3.4 : BACK RAKING PILES ON ABUTMENTS

The most serious deficiency of many methods of pile group analyses for abutments is that they do not cater for the influence of lateral loading on the piles caused by the embankment behind. It may in certain instances be reasonable to assume that a dense upper stratum could resist horizontal forces on the fronts of piles, but if the upper stratum is soft, then unfavourable active pressures may be applied on the backs of the piles (Figure 3.5). Even when the ground is firm the strain under the embankment in the region of the piles may cause the ground to displace forwards relative to a stiff pile group. It is usually only possible during design 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.

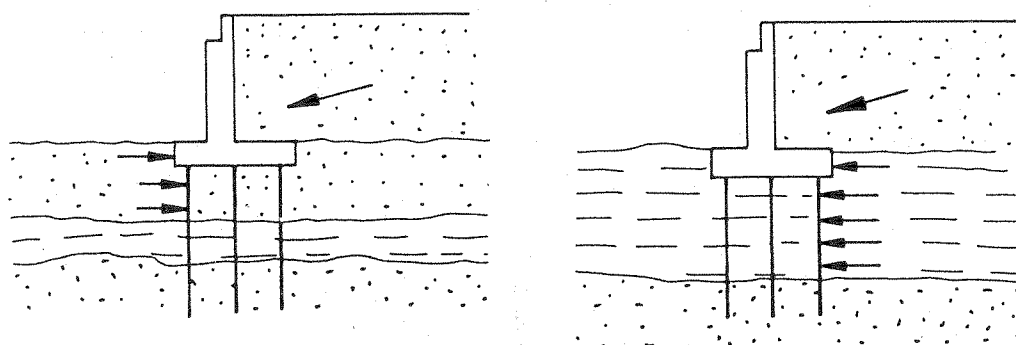


FIGURE 3.5 : HORIZONTAL LOADING ON ABUTMENT PILES

It is advisable to study global behaviour when selecting the factors of safety appropriate for pile design. In a large pile 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 as shown in Figure 3.6 could lead to progressive failure of the bridge then a higher factor of safety should be adopted.

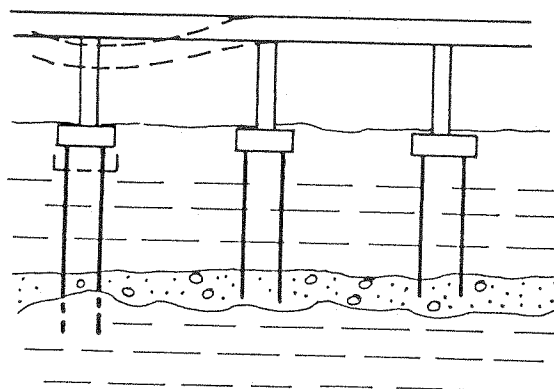


FIGURE 3.6 : FAILURE OF PILES UNDER AN ISOLATED FOUNDATION

If there is an underlying deposit of weak soil such as soft clay, which will not support, with a reasonable margin of safety, the weight of the soil behind a wall without exceeding the shearing strength of the formation on which it is placed, a deep-seated or base failure may occur.

A deep-seated failure, in which the failure arc passes below the tips of foundation piles and thereby makes them ineffective, is shown in Figure 3.7(a). A deep-seated failure, where piles are driven through a soft deposit into a firm deposit to secure adequate bearing capacity, is illustrated in Figure 3.7(b). In this case, the load due to the retained soil causes shearing stresses in the soft deposit which exceeds its shearing strength. It is therefore unstable and failure will occur along an arc through the soft deposit unless the pile foundation can resist the lateral thrust it receives from the soft deposit, as shown by the arrow in the figure, in addition to the lateral thrust introduced by the retaining wall. This thrust would commonly exceed the flexural strength of the number of piles it is feasible to provide.

When the conditions shown in Figure 3.7(a) exist, some solution other than constructing a retaining wall must be found. A spill-through abutment might provide a solution or the bridge might be lengthened to attain a more favourable location for the abutment. If parallel retaining walls are to be constructed to retain a fill between them for an elevated highway or railway, this form of construction might be abandoned in favour of a viaduct. The above comments concerning the foundation in Figure 3.7(a) may apply to that in Figure 3.7(b), except that in the latter case it may be economically feasible to excavate through the weak soil and found the wall on the firm deposit. If this is done, the lateral earth pressure against the wall, due to the thrust of the weak soil, will be large. However, a retaining wall can be designed to carry a thrust which piles could not resist.

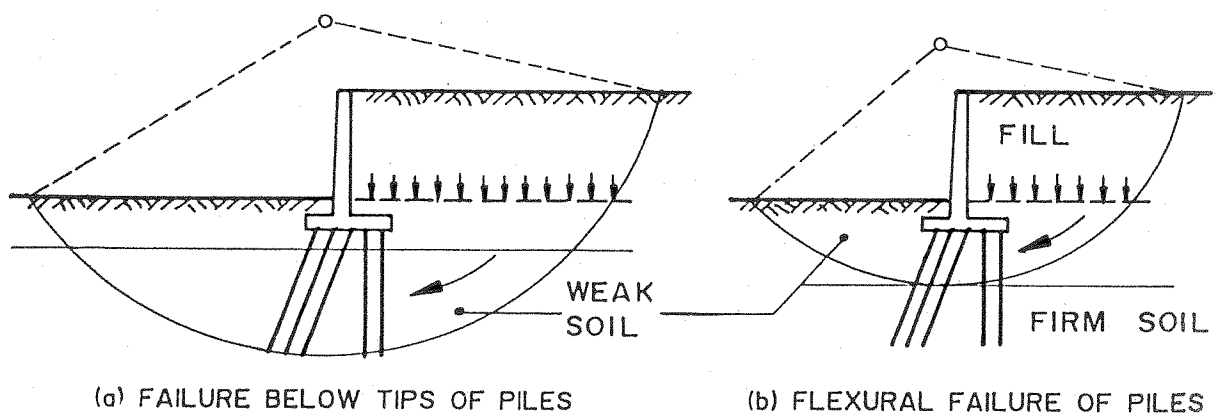


FIGURE 3.7 : DEEP SEATED FAILURES OF PILE FOUNDATIONS

Piling contractors are the greatest source of experience on the installation of their types of pile and designers should seek their advice early on in the design process. 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 solution. It will be advantageous to have recourse to an independent expert during the tender period, to assist in the selection of alternative pile designs and verification of associated specifications.

(ii) Selection of pile type

Selection of a pile type is an aspect of bridge design for which most designers find it necessary to consult a specialist. Extensive experience is necessary to assess the suitability of the various types of pile for installation in different ground conditions. The choice is influenced not only by cost but also by the nature of the soil and groundwater conditions, the type of plant used, the competence of the contractor and access to the site. The installation process can increase the strength of some soils and decrease the strength of others. The selection may even not be straightforward to an expert, if he does not have experience in the use of identical piles under similar conditions, and he may well request a trial installation to confirm his choice.

There are at present eight pile types that are regularly used in South Africa. These are listed hereunder in the order of their general suitability, for bridge foundations. In addition to these eight, the *Caisson pile* (or permanently cased large diameter bored piles) has been used on a number of river bridges in the past but is not widely used at present because of the high cost of the permanent casing. As it has its uses it has been included for completeness. Particular sub-surface conditions may rule out the suitability of one or more of the pile types. A detailed description of these pile types is given by Braatvedt (1986).

■ Oscillator Piles	Bored cast in situ large diameter
■ Driven Tube Piles	Driven cast in situ medium diameter
■ Precast Piles	Driven pre-formed small diameter
■ Underslurry Piles	Bored or augered cast in situ large diameter
■ Auger Piles	Augered cast in situ full range
■ Forum Bored Piles	Bored cast in situ small diameter
■ Driven displacement (Franki type) piles	Driven cast in situ medium diameter
■ Augercast Piles	Augered cast in situ medium diameter
■ Caisson pile	Bored cast in situ large diameter.

All of these pile types could theoretically be used for river bridges. However, in practice the subsurface material typically found on river bridge sites will limit the choice considerably. Client bodies furthermore also have their own requirements which limit the choice of pile type. In the following examination of the various systems the positive and negative features of each system with regard to river bridge foundations are summarised.

(a) Oscillator piles

This is the pile type most often used in South Africa for major river bridges which are prone to scour problems.

The positive features are:

- The diameters available are in the range 900 to 1500 mm. These sizes can accommodate the large moments and shear forces.
- The system permits penetration through most sub-strata obstructions such as boulder and cobble layers.
- The system is suitable for the forming of a rock socket for increasing the bearing, shear and tension capacity.
- The installation of these piles can be accomplished under dry and underwater conditions.
- A thin walled permanent casing can be accommodated if required.
- Raking piles of up to 1 in 4 can be achieved with the system.
- The depth capacity of the system is considerable (piles up to 65 m deep have been installed).

The negative features are:

- The cost of these piles is relatively high.
- The system is not suited for founding in cohesionless soils below the water table as the associated installation disturbance of the founding material will result in a drastic reduction in the end bearing component of the pile's capacity.

The pile diameters that are available with the oscillator system are related to the size of the temporary casing. These are 1080, 1200 and 1500 mm outside diameter. As the temporary casing has a wall thickness of 50 mm the resulting pile diameter is 100 mm less than the outside diameter. If a permanent casing is not used, the pile diameter will be equal to the outside diameter of the temporary casing.

The range of permanent casings in use are:

- 900 or 950 mm dia for the 1080 mm temporary casing size.
- 1000 or 1050 mm dia for the 1200 mm temporary casing size.
- 1300 or 1350 mm dia for the 1500 mm temporary casing size.

It shall be noted that casings with diameters smaller than the lower limit will be inadequately restrained within the temporary casing while casings with diameters larger than the upper limit will have insufficient clearance to the temporary casing.

The oscillator pile is capable of penetrating all soil types and most boulder obstructions. The larger the boulders and the harder they are the more difficult and slow is the penetration rate. In certain cases it may be beneficial to pre-split the boulders using explosives.

Penetration into the bedrock depends on the hardness of the rock and the amount of fracturing and weathering of the formation. Rock with an unconfined compression strength (ucs) of less than 10 MPa which is classified as a very soft rock (R1) or a soft rock (R2) can be socketed into fairly readily

irrespective of the amount of fracturing. Socket lengths in these grades of rock of up to 4 to 5 pile diameters can be achieved.

Penetration into medium hard rock (R3) with a ucs of 10 to 25 MPa also presents little difficulty with the more fractured rock being easier to penetrate. Socket lengths should be limited to 1,5 pile diameters or 2 metres whichever is the lesser if the rock is massive, and 2,5 pile diameters or 4 m if the rock is fractured.

When the rock is classified as hard rock (R4) with a ucs of between 25 and 70 MPa, socket penetration will depend to an even greater extent on the fracturing of the rock. Rock types such as sandstone, dolerite, granite and quartzite are difficult to penetrate if they are in a massive state. In these circumstances the socket length should be limited to 1,0 pile diameters or 1,5 m whichever is the lesser. If the rock is heavily fractured, penetration is achieved much more readily and a longer socket length of say 1,5 to 2,0 pile diameters with a maximum of 3 m can be used.

In very hard rock (R5) with a ucs in excess of 70 MPa which has little if any fracturing, penetration will prove very difficult. Socket lengths should not exceed 0,5 pile diameters and the alternative method of using dowels as described hereunder should be considered in these circumstances. However, if the rock is fractured a socket length of 1,0 pile diameter or 1,5 m, whichever is the lesser, can be achieved without using special techniques.

With the alternative dowel anchoring, the surface of the rock shall be levelled off within the temporary casing area and whereafter the pileshaft shall be constructed with drilling sleeves cast into the concrete for the subsequent drilling of the dowel holes. Once the concrete has hardened the dowel holes shall be drilled into the bedrock and the dowel bars grouted into the holes. The dowels shall be designed to resist the applied loads.

Explosives can be used to facilitate the penetration of a pile socket into very hard rock. Severe shattering can occur from blasting and therefore the use of explosives should be used with caution.

Normally a self compacting concrete is used and because of the high slump, the maximum 28 day cube strength that can be achieved is limited to 35 MPa. Concrete cover is normally 75 mm.

The reinforcing comprises a cylindrical cage with main bars of 25 or 32 mm diameter. The transverse bars vary between 8 to 12 mm in diameter and are either in the form of conventional circular hoops or spirals. Sufficient circular hoops shall be provided to stiffen the cage to facilitate lifting at one end into the vertical position without permanent distortion of the cage.

#### (b) Driven tube piles

There are a number of major bridges in South Africa that are founded on this pile type which has since the introduction of the oscillator pile, been overlooked. It can provide an excellent foundation for a river bridge and is often less expensive than the oscillator pile.

The positive features are:

- The pile can be installed to considerable depths. Depths in excess of 50 m have been achieved.
- The internal bore of the pile can be inspected prior to placing the concrete.
- The concrete in the pileshaft can be placed in the dry and thus higher strength concrete can be used. There is generally better control of the concreting with a resulting better quality of pileshaft.
- The pile is driven to a "set" calculated to provide the required capacity. This avoids the problem of assessing the adequacy of the founding conditions as is the case with bored piles.
- As the pile is a driven displacement type, the shaft friction generated contributes appreciably to the pile's bearing capacity.
- This pile type provides a good solution where piles are to be founded in sand below the water table. In these conditions bored piles suffer from reduced friction and end bearing capacity caused by disturbance of the sand.
- Raking piles of up to 1 in 4 can be achieved with the system.

The negative features are:

- The ability of the pile to penetrate a particular soil stratum is often difficult to assess at the design stage and the presence of boulders and cobbles makes the assessment even more difficult. However, experience has shown that where the matrix between the boulders and cobbles is not dense the pile will penetrate such layers. In sandy materials penetration over the upper 15 m can be improved by water jetting.
- The diameter of the pile is limited to a maximum of 600 mm.
- As the system utilizes a thick wall permanent casing the cost per meter tends to be high.

The diameter of the pile can vary between 250 and 600 mm. Where possible the diameters specified should be based on the standard tube sizes available. The minimum wall thickness of the tube should be 6 mm with the onus on the contractor to adopt the thickness to suit the conditions.

The toe end of the tube is closed off with a steel plate of which the thickness thereof should not be less than 4 percent of the pile diameter, and the outside diameter thereof should be 20 mm larger than the outside diameter of the tube. A rock penetrating point with locating gusset plates can be welded to the plate to assist in penetrating boulder and cobble layers. A rock point is essential when founding on steeply sloping rock faces.

The length of the casing can be extended once the first section has been driven. Casings can be butt welded but experience has shown that the use of an outer collar splice is preferable. This collar is welded to the leading casing before it is pitched into the piling rig. The subsequent casing is then lowered into

the collar, lined up and a full penetration fillet weld made between the collar and the casing to ensure continuity of axial and bending strength.

The quality and workmanship in the manufacture of the casing is extremely important and shall be in accordance with a recognised specification. Random ultrasonic and/or X-ray tests shall be carried out on the welds. Experience has shown that the barrel type tube construction has been more reliable than the spiral welded type. Barrels of up to 9 m in length can be handled by the installation rig thus limiting the number of circumferential welds.

The reinforcing cages are fabricated as for those in oscillator piles. Main bars are typically either 25 mm diameter or 32 mm diameter with the lateral tie being an R8 spiral with half pile diameter pitch. As the cage is small in diameter, stiffening rings will only be necessary for the heavier cages.

40 MPa concrete can be accomplished in the shafts of these piles. The concrete cover over the reinforcement tie bars varies generally between 50 and 75 mm.

(c) Precast concrete piles

Because of the slenderness of these piles they are normally only suited to river bridges where scour is not likely to be very significant. In such cases the precast pile can provide a very economical foundation solution.

The positive features are:

- Relatively low cost.
- High rate of production which results in shorter construction times.
- High quality pile shaft provided it is not damaged during driving.
- Piles fitted with pointed rock shoes can key into sloping faces where the rock classification is R2 or softer.
- Jointed piles can be installed to considerable depth. (Depths of 50 to 60 m have been achieved).
- Precast piles have a good penetrating ability provided there are no large boulders or other large obstructions present. Cobble layers can be penetrated provided the matrix is not dense or cemented.
- Loads are resisted by friction and end bearing. Precast piles need not be founded on rock and are often founded in a dense sand layer.
- Precast piles can be installed to a rake of up to 1 in 4.

The negative features are:

- Non-jointed piles are limited in length to about 15 m.
- The piles are relatively slender with the result that limited shear and moment can be resisted.
- There may be concern about corrosion resistance of the pile joints especially if the piles are designed to resist tensile forces.
- The pile lengths have to be determined prior to installation. Unforeseen changes in the founding levels can result in excessive waste of pile shafts or alternatively pile shafts of inadequate length.
- Pile shafts can fracture during driving.

Size and cross-section (square, hexagonal or round) preferences vary between contractors. The most common available cross sectional areas are 60 000, 90 000 and 120 000 square millimetres with the most common shape being square.

The longitudinal steel reinforcing in precast piles is normally in excess of 1 percent of the cross-sectional area for handling and driving purposes. The main bar size is 16, 20 or 25 mm and high tensile steel is used. A R6 spiral at 100 to 150 mm centres reducing to 50 mm centres at the section ends is typical.

The shaft stresses in precast piles under working loads, is generally 16,5 MPa when used in building foundations. For river bridges there are grounds to suggest that this should be limited to 12 MPa. The concrete in the piles should have a minimum 28-day cube strength of 50 MPa.

(d) Underslurry piles

This pile type was introduced to South Africa in 1975 and has not been used on many bridges mainly because of the fact that raking piles cannot be achieved with this system.

While a detailed description of this system is given in Braatvedt (1986), it is important to note that these piles are split into two groups. The *Auger underslurry* piles which are excavated using conventional auger rigs whereas the *Barrette* is excavated using a special grab. With the former the pile shafts are circular and the latter rectangular or special shapes comprising rectangular components.

The positive features are:

- Vertical pile shafts of large cross sectional area can be installed to considerable depth of up to 42 m.
- The cost of supporting the excavation using bentonite slurry is generally much less than that of temporary casings.
- The physical dimensions of the Barrette type of pile are suited to resist large horizontal loads.

- Rotary rock drilling can be used down a bentonite slurry supported excavation for forming rock sockets. Considerable depths of penetration into rock can be achieved.

The negative features are:

- Only vertical piles can be installed.
- The system is regarded as having a poor penetration ability without the assistance of drilling equipment. It is thus not suited to sub-strata which have a proliferation of boulders although cobbles in a loose matrix can be penetrated.
- The formation of a rock socket is not achieved easily and is not normally attempted unless the rock is soft enough so that it can be drilled using an auger rig. Harder rocks can be drilled with rotary rock drilling equipment. The formation of rock sockets on Barette type piles is even more difficult and should only be attempted in exceptional circumstances.

With the auger underslurry pile the sizes are normally those used with conventional auger piles with a minimum size of 900 mm diameter and a maximum of 1500 mm. Larger sizes can be achieved in ideal soil conditions. The depth of these piles is limited to a maximum of 42 m with the equipment presently available in the country.

Barette piles are rectangular in shape and can vary in size from 2200 by 600 mm to 2200 by 1200 mm for a single pass of the grab. Longer panels and shaped panels can be constructed in a manner similar to that used in the construction of a diaphragm wall. The depth of Barette piles is not limited as the grab is cable suspended.

Reinforcing cages for the circular piles are as for the oscillator pile. For Barette piles the cage is rectangular with longitudinal and distribution steel on both of the longer sides. To stiffen the cage, diagonal bracing bars similar in shape to shear steel are used. These bars should be kept well clear of the centre of the cage so as to allow unrestricted passage for the tremie pipe.

The concrete used generally attains a minimum 28-day cube strength of 35 MPa.

#### (e) Auger piles

This is one of the most widely used pile types in South Africa as it provides an economical solution especially in the interior of the country where the soils are generally cohesive and where there is an absence of a water table. The pileshafts are excavated with an auger rig and under the abovementioned circumstances lateral support to the excavation need normally not be provided. Auger piles are normally not suitable for use in river beds due to the frequent presence of shallow water tables and the cohesionless nature of many river substrata. In exceptional cases where the substrata contains some clay and the water table is very deep, auger piles could be considered.

The positive features are:

- A fast and economical piling system.
- The system has a great flexibility in the size of pile which can vary from 150 to 2000 mm in diameter.
- Piles with a rake of up to 1 in 4 can be installed.
- Auger rigs can penetrate the softer rocks (classification R2 and softer) so good end bearing in the form of a rock socket can be achieved in most cases.
- In piles of diameters greater than 750 mm loose material at the toe of the pile can be removed by hand in order that the end bearing of the pile is not impaired.
- The concrete can be placed in the dry with the resulting better control of this operation.

The negative features are:

- The system is most suited to soil conditions where the pile excavation will not collapse. Temporary casings can be used where local collapse occurs but this will tend to render the system less economical. It is sometimes difficult to forecast beforehand whether the pile excavations will collapse or not.
- Auger rigs cannot drill through the hard boulders larger than about half the pile diameter or with a least dimension of 300 mm, whichever is the lesser dimension. These have to be removed using special techniques which are best executed in the dry. If boulders are present and there is a risk of the hole collapsing, the oscillator pile is preferred.

One of the major advantages of the Auger system is the flexibility with the size of the pile. The diameters can range from 150 to 2000 mm. For river bridges, however, the normal sizes are 750, 900, 1000, 1200, 1350 and 1500 mm. Most truck mounted auger rigs can drill to 36 m depth but there are crane mounted auger rigs in the country that can drill up to 42 m deep. Auger piles with a diameter of less than 750 mm are not recommended for river bridges as they cannot be cleaned out adequately prior to concreting.

Auger piles can be installed to a maximum rake of one in four. It is advisable, however, to limit the depth of raked auger piles to about 15 m as it is not possible to prevent the auger tending to dip over the longer lengths and more inclined rakes. Long raking auger piles tend to have a degree of curvature in the shaft caused by dipping of the auger.

The installation of temporary casings on a rake is also difficult especially if the rake is more inclined than one in eight. Under conditions of collapsing soils in conjunction with raking piles it is advisable to use an oscillator type of pile rather than the auger pile.

The larger auger rigs are capable of drilling into the softer rocks i.e. classification R2 and softer. It is common practice to design the end bearing component of the pile capacity at a working stress of up to 5 MPa if refusal of a Williams Digger is achieved. Penetration into harder rock can be achieved in cases

where the rock has natural planes of weakness or fractures. Friction on the sides of a socket can greatly increase the pile's bearing capacity. (Section 3.2.4(iii)).

The reinforcing steel cages are made up as for the cages for the oscillator piles. Concrete strengths in excess of 40 MPa at 28 days can be attained if the concrete is placed in the dry.

(f) Forum bored piles

This is a smaller diameter (410 and 600 mm) bored cast in situ type of pile which can either be socketed into soft rock (R2) or can be founded in soil by forming an enlarged base. The system is suitable for use on smaller river bridges.

The positive features are:

- The equipment is light, so establishment costs are low. It is thus suited to small river bridges in remote areas.
- The small rig can be manoeuvred into difficult access and low headroom situations. The system has thus been used in the past to underpin existing footings or to extend existing piled foundation on bridge widenings.
- A rock socket or enlarged base can be formed to resist uplift forces.
- As the system makes use of a temporary casing, the presence of a water table is normally not a problem in cohesive materials. However, the presence of clean sand below the water table can complicate the installation as it is difficult to seal the casing against the inflow of water and running sand. The pile can normally be installed successfully provided that there is a final cohesive layer in which the casing can form a seal.
- The reinforcing cage can be extended into the enlarged base and thus these piles can resist considerable tensile forces.

The negative features are:

- The diameter of the pile is limited to 600 mm.
- The system has a depth limitation of about 12 m.
- For reasons stated above, the system is not suited to substrata comprising clean sand below the water table directly overlying hard rock.
- Some authorities are not in favour of the shaft concrete coming into contact with the saturated soil on extraction of the temporary piling tube as is the case with this pile type.

There are only two pile sizes available which are 410 and 600 mm diameter but the former is the more common size. The working load for the two sizes is 600 and 1000 kN respectively.

## (g) Driven displacement (Franki type) piles

This is one of the most widely used piling systems in the country and there are a number of bridges founded on this pile type. Its use on river bridges is not very common for reasons stated below under negative features. The Franki pile system is the most common of this type used in South Africa.

The positive features are:

- The pile is formed with an enlarged base. The action of enlarging the base in the ground results in the end bearing capacity being far superior to that of most other pile types not founded on rock. The system is thus ideally suited to shallow founding in softer ground especially sandy material.
- The reinforcing cage can be extended into the enlarged base and thus these piles can resist considerable tensile forces.
- The piles can be raked at up to 1 in 4.

The negative features are:

- Some authorities are not in favour of the unset shaft concrete coming into contact with the saturated soil on extraction of the temporary piling tube as is the case with this pile type.
- The system has a depth limitation of 15 m.

The size range is 410, 520 and 610 mm diameter for working loads of 750, 1200 and 1600 kN respectively.

As this is a driven pile, the substrata must be suited to the driving of piles. In non-collapsing ground conditions the pile can be predrilled to achieve the founding depth and to speed up the installation process. In dense sands water and air jetting can be used to facilitate penetration.

This pile type can also be installed in what is known as the Franki precast composite pile in which a precast shaft is used in combination with an enlarged base. This technique has been used in cases where the client or engineer is concerned about unset concrete coming into contact with saturated soil in which there could be ground water movement. These piles perform very well in compression but the connection between the precast shaft and the enlarged base cannot resist tension.

## (h) Augercast piles

This piling system is least suited to river bridge work. The system is very operator-prone and there is limited control on aspects such as cover to the reinforcement. Decompression of saturated cohesionless soils occurs during the drilling operation which reduces the friction bearing component in these substrata. Where possible these piles should be founded on rock where end bearing capacity is not affected by the drilling operation.

At present the range of pile sizes is 300 to 750 mm with 300, 400, 500, 600, 750 mm being the most common sizes. Depths of up to 20 m can be achieved. The pile rakes should be limited to one in six.

A sand/cement grout is used in the pileshafts for these piles in lieu of concrete. The main reasons for this are that grout can more readily be pumped through the hollow stem auger than concrete and it is easier to place the reinforcing cage into grout than into concrete. The working stress on these pile shafts is normally limited to 6 MPa.

(i) Caisson piles (or permanently cased bored piles)

This name can be confusing but the term is used to refer to large diameter piles with a permanent casing. The system and the method of forming these piles is described by Braatvedt (1986).

The positive features are:

- Large pile diameters can be achieved.
- In certain substrata such as impermeable clay a seal against water inflow can be formed between the permanent casing and the founding substratum. Such a seal will allow the casing to be de-watered and the socket in the founding substratum to be inspected prior to concreting the shaft.
- The permanent steel casing can be utilized to contribute to the overall strength of the pile.
- By using a telescopic casing configuration great pile depths of the order of 80 m can be achieved.
- Bending moment and shear force resistance are higher than those possible with other piles of equivalent size due to the permanent casing and the higher concrete strengths which are attained when casting the concrete in dry conditions.
- The piles can be raked up to 1 in 4.

The negative features are:

- The high cost of this pile when a thick walled permanent casing is used.
- The permanent casing can be damaged in the process of driving it through boulders layers and other hard obstructions. If there is a predominance of boulders in the substrata then an oscillator type of pile is most likely the better choice.

Pile diameters 900, 1000, 1200, 1350 and 1500 mm are the most common. The 28 day concrete cube strength in the pileshafts is normally a 35 MPa or lower but 40 MPa can be attained if the concrete is placed in the dry.

Reinforcement is provided by the casing and/or by longitudinal reinforcing bars tied together with helical reinforcement bars and welded hoops formed from 35 x 6 mm mild steel strips spaced at 2 m centres. Under the most favourable subsurface conditions from a corrosion and abrasion point of view (non-aggressive soil below the level of oxygen replenishment over zones where the river flow velocity is less than 5 m/sec at maximum design flood) the outside diameter of the casing shall be assumed to be 3 mm

less when calculating the stresses in the pile. An assessment shall be made of the corrosion and abrasion conditions and a suitable sacrificial layer allowed for in the design to account for loss of cross-section.

(iii) Supervision of installation

The importance of vigilant supervision of installation to ensure that the designer's requirements are met cannot be stressed too strongly.

(iv) Pile capacity design

Most piles used on river bridges are founded on rock and are therefore predominantly end bearing. Frictional resistance capacity can however be a substantial component in certain cases and there are no grounds for ignoring it, except in cases of scour. In some cases it may prove economical to found a bridge on friction piles which have little or no end bearing capacity.

To enable the designer to make a rough assessment of a pile's bearing capacity, some design parameters are given in Tables 3.1, 3.3 and 3.4. These are ultimate friction and end bearing values to which a factor of safety of between 2,5 and 3,0 should be applied to arrive at allowable working loads. Refer to Braadvedt (1986) page 148 for a bearing capacity formula.

(a) Ultimate friction design parameters

(1) Cohesionless soils

Design parameters for ultimate skin friction stresses are given in Table 3.1.

Type of Pile	Ultimate Friction Stress (kPa)
Oscillator Piles (uncased)	$q_c/180$ or 3 N
(cased)	Nil
Driven Tube Piles	$q_c/120$ or 4 N
Precast Piles	$q_c/120$ or 4 N
Underslurry Piles	$q_c/180$ or 3 N with a maximum of 70 kPa
Auger Piles	$q_c/180$ or 3 N
Forum Bored Piles	$q_c/100$ or 5 N
Driven Displacement Piles	$q_c/180$ or 3 N
Augercast Piles	$q_c/180$ or 3 N
Caisson Piles	
N = Standard Penetration readings (blows per 300 mm)	
$q_c$ = Point resistance of Dutch Probe (kPa)	

TABLE 3.1 : ULTIMATE FRICTION STRESSES FOR DIFFERENT PILE TYPES IN COHESIONLESS SOILS.

## (2) Cohesive soils

Ultimate skin friction stresses are normally related to the undrained shear strength of the soil ( $C_u$ ). If the undrained shear strength has been determined as part of the foundation investigation then the factors from Table 3.3 below can be applied to the measured values to arrive at the skin friction values. If the undrained shear strength has not been measured then very approximate values can be obtained from Table 3.2.

Cohesive soil classification and SPT range	Undrained Shear Strength $C_u$ (kPa)
Soft normally consolidated clays SPT N values 1 to 5	5 N to 6 N
Stiff normally consolidated clays SPT N values 6 to 12	6 N to 8 N
Very stiff overconsolidated clays SPT N values 15 to 50	10 N to 12 N

TABLE 3.2 : UNDRAINED SHEAR STRENGTH DERIVED FROM SPT N VALUES

Using the simple method, according to Tomlinson (1987) of applying an alpha reduction factor to the undrained shear strengths the ultimate friction stresses may be obtained from Table 3.3. For more detailed information on alpha factors refer to Tomlinson (1987).

Type of Pile	Ultimate Friction Stress (kPa)
Oscillator Piles (uncased)	0,4 $C_u$
(cased)	Nil
Driven Tube Piles	0,3 $C_u$
Precast Piles	0,3 $C_u$
Underslurry Piles	0,4 $C_u$
Auger Piles	0,4 $C_u$
Forum Bored Piles	0,4 $C_u$
Driven displacement Piles	0,4 $C_u$
Augercast Piles	0,4 $C_u$
Caisson Piles	0,4 $C_u$

TABLE 3.3 : ULTIMATE FRICTION STRESSES FOR DIFFERENT PILE TYPES IN COHESIVE SOILS.

It is advisable to limit the ultimate friction in cohesive soils to 100 kPa.

(b) Ultimate end bearing in soil

When deciding on end bearing design parameters for piles it is important to consider the strength of the soil not only at the founding level but also at least four base diameters above and below the base. For soils that are improving in strength with depth the rule of averaging the soil strength parameters by 4D above the base and 1D below the base has been found to give a more realistic assessment of end bearing capacity. On the other hand, if the soil strength decreases below the base of the pile, then the averaging should be based on 4D below and 1D above.

The concept of averaging is illustrated for the last example mentioned above:

Take the average value of the soil strength parameter under consideration (SS) between the foundation level and a depth of 1D and multiply by four giving  $4SS_1$ . Take the average value of SS between depths 1D and 2D and multiply by three giving  $3SS_2$ . Take the average value of SS between depths 2D and 3D and multiply by two giving  $2SS_3$ . Take the average value of SS between depths 3D and 4D and multiply by one giving  $SS_4$ . Take the average value of SS between the foundation level and 1D above the foundation level and multiply by 2 giving  $2SS_5$ , then

$$SS = \frac{4SS_1 + 3SS_2 + 2SS_3 + SS_4 + 2SS_5}{12}$$

(1) Cohesionless soils

Bored piles for river bridges are very seldom founded in cohesionless material because of the decompression that can occur when excavating below the water table. For this reason only the ultimate design stress for *driven piles* are given in Table 3.4.

Type of Pile	Ultimate End Bearing Stresses (kPa)
Driven Tube Piles	$q_c$ or 400 N
Precast Piles	$q_c$ or 400 N
Forum Bored Piles (with expanded base)	$1,25 q_c$ or 500 N
Driven displacement Piles (with expanded base)	$1,25 q_c$ or 500 N

TABLE 3.4 : ULTIMATE END BEARING STRESSES FOR DRIVEN PILES

(2) Cohesive soils

With driven piles it is unlikely that there would be any loss of end bearing capacity in cohesive soils caused by the remoulding of the soil displaced in the driving process. Certain checks such as driving to obtain a second set 48 hours after completion of the installation will normally indicate whether any reduction of capacity has occurred.

Because of the cohesive nature of the soil, it is assumed that, with bored piles, the material of the founding substratum will not be disturbed if due care is exercised. To achieve good end bearing the surface must also be thoroughly cleaned and loose material removed.

Under these circumstances the ultimate end bearing stress can be taken as 4,5 times the unconfined compression strength (ucs) of the soil or 9 times the undrained shear strength ( $C_u$ ).

(3) Rock end bearing and socket capabilities

Provided the contact with the rock is clean and the rock is sound the ultimate end bearing for a pile base can be calculated using an ultimate stress of 4,5 times the unconfined compression strength of the rock. If the quality of the rock is not sound due to fracturing, weathering and the presence of bedding planes then the above figure may have to be reduced to allow for the effects these discontinuities may cause.

The majority of augered and bored piles for river bridges are not founded on the surface of the rock but are socketed into the rock. Where the strength of the rock is such as to allow sockets to be formed economically, the use of sockets is recommended. The side friction in the socket can increase the bearing capacity of the pile considerably.

The ultimate end bearing design stress for a rock socket is the same as that for a pile founded on rock as given above. The curve given in Figure 6.3.1 in Appendix 6.3 taken from Williams, Johnston and Donald can be used to determine the ultimate side friction stress from the unconfined compression strength of the rock. These values may have to be reduced if the rock is not intact and the sidewalls are not rough. Guidance can be obtained from Rock-socketed piles: G Davies. Figure 6.3.1 has been used to calculate the safe socket capacities (allowing for end bearing as well as side friction) given in the tables in Appendix 6.5. A factor of 3 has been used to calculate these safe socket capacities which should be used as initial assessment only.

(v) Pile layout or arrangement

An important consideration is the intersection of piles either above or below the cut-off level. Below the cut-off level the shafts of the piles should be kept a reasonable distance apart so that piles do not intersect within the same vertical plane. Intersection above the ground must also be considered when the cut-off level is well below the level from which the piles are installed.

Site accessibility and working space for piling equipment must be assessed when deciding on pile layouts. When using raking piles, it must be borne in mind that with most piling systems the piling rig must be positioned on the line of the axis of the pile with the pile raking away from the rig. The required width of a piling platform will thus double if there are both forward and backward raking piles.

(vi) Negative skin friction

In many cases involving river bridges the approach fills are of considerable height and the material over which the fill is placed is of a compressible nature. In these cases the abutment piles can be subjected to negative skin friction (downdrag). On long piles these forces can be large and should be taken into account in the design of the piles. Negative skin friction can be taken as 70 percent of the ultimate

positive friction over the depth in which consolidation will take place, as a rough indication. It takes less than 3 mm differential movement to generate the ultimate negative skin friction in clays and silts and about 20 to 25 mm differential movement in sands.

(vii) Scour and scour depths

Where the scour depth is greater than 12 pile diameters below the pilecap soffit level, the slenderness of the pile must be considered in the structural design of the pileshaft. It thus follows that larger diameter piles must be used for the cases with the largest scour depths.

Using the caisson pile described earlier, larger pile diameters are feasible and, in fact, two major bridges are founded on 1500 mm diameter caisson type piles. The thick wall permanent casing also contributes considerably to the overall strength of the section. The cost of the caisson pile is unfortunately high due to the cost of the permanent casing.

(viii) Modulus of horizontal subgrade reaction

Most computer programs for designing pile groups such as those supporting river piers which are subject to large horizontal loading require a modulus of horizontal subgrade reaction as part of the input data. This parameter is covered widely in texts on soil mechanics of which Evaluation of coefficients of subgrade reaction: Terzaghi K (1955) is the most important.

(a) Cohesive soils

In cohesive soils the modulus of horizontal subgrade reaction can be regarded as constant with depth for a homogeneous soil. The metricated values quoted in Terzaghi K (1955) for clays for a pile 1000 mm in diameter are given in Table 3.5.

Cohesive soil classification	Undrained Shear strength $C_u$ (kPa)	Modulus of Horizontal Subgrade Reaction ( $kN/m^3$ )
Firm to stiff	50 to 100	3200 to 6400
Stiff to very stiff	100 to 200	6400 to 12800
Hard	200	12800

TABLE 3.5 : MODULUS OF HORIZONTAL SUBGRADE REACTION IN COHESIVE SOILS.

From this table it can be seen that the modulus of horizontal subgrade reaction in  $kN/m^3$  is approximately equal to  $64 C_u$ .  $C_u$  is approximately equal to 5 N to 6N for soft to firm normally consolidated clays increasing to 10 N to 12N for very stiff overconsolidated clays, where N is the SPT result for the material.

## (b) Cohesionless soils

According to Terzaghi K (1955) the modulus of horizontal subgrade reaction can be regarded as increasing linearly with depth in a homogeneous sand, and accordingly introduced the concept of a constant of horizontal subgrade reaction as follows:

$$K_h = \frac{N_h \times z}{B_1}$$

where

$K_h$	=	Modulus of horizontal subgrade reaction ( $\text{kN/m}^3$ )
$N_h$	=	Constant of horizontal subgrade reaction
$z$	=	depth below ground level (m)
$B_1$	=	width of contact = diameter of the pile (m)

Figure 6.4.1 in Appendix 6.4 gives two curves for the constant of horizontal subgrade reaction  $N_h$  for sands of different relative densities. The lower one is from Terzaghi K (1955) and the higher one from Reese et al (1974). The Terzaghi K (1955) values are regarded as conservative and those of Reese et al are preferred. These are both for a pile 1000 mm in diameter. By using the constants of subgrade reaction from this diagram and the above formula the coefficient of horizontal subgrade reaction can be calculated for different pile diameters, depths and sand densities.

Soil surrounding a pileshaft can have its properties altered during the installation process and this could also change the modulus. The modulus could also be changed by scour action. Fortunately designs are not too sensitive to changes in the modulus. The axial load on the pile is hardly affected at all and the moment increases only slightly by halving of the modulus. The pilecap deflection on the other hand, is significantly affected by changes in modulus and a sensitivity analysis should be carried out if this method of analysis is used. Computer programmes are available that are well suited to estimate pile group deflections and these should be used where deflections are to be estimated accurately.

## (ix) Load and integrity testing of piles

At present there are four acceptable methods to establish the adequacy of individual piles, viz

- Load test.
- Nuclear back-scatter integrity test.
- Sonic integrity test.
- Diamond cores of shaft and rock.

The first mentioned method tests the load carrying capacity and the remainder establish the quality of the concrete. The last mentioned also establishes the quality of the rock and the quality of the material at the concrete and rock interface.

(a) Load test

On major river bridges, where the pile diameter as well as the loads on the piles are large, a load test is an expensive and in some cases an impractical form of testing. However, it is feasible to load test a pile to 10 MN using available equipment. Loads larger than this will need specially manufactured equipment and will thus render the cost even higher.

A further disadvantage is that dissipation of the skin friction component occasioned by scour over appreciable depths cannot be simulated and load tests will exhibit load capacities which are higher than those applicable under scour conditions.

For small piles a load test is more feasible and less costly and thus this type of test is relatively common. Refer CSRA (1987) Standard Specifications for Road and Bridge Works, clause 6113(a) for load test requirements.

The reaction for the load test can be provided by either kentledge, anchor piles or cable anchors. In the normal course of events, there is no reason why either of these systems should be preferred over the others. The choice should be left to the contractor to select the most economical method. If there is likely to be more than one load test, then the kentledge method may be cheaper as the establishment costs can be spread over the number of tests. The use of cable anchors is generally the most expensive form of providing reaction.

The load is applied using a hydraulic jack and the monitoring of the pressure in the jack normally forms the sole method of measuring the applied load. For this reason the jack should be calibrated prior to carrying out the test. Where more than one jack is used, they should be fed from a common hydraulic line and all the jacks should be calibrated before testing commences. Great care should be taken to ensure, as far as possible, that an axial load is applied to the pile.

Measurement of the deflection of the head of the pile is achieved using either deflectometers or precise level readings but preferably both. The deflectometers should be fixed to a rigid frame supported at points well away from the pile. A timber frame is recommended as it is less prone to temperature variations. The precise level readings are normally taken on millimetre scale rules fixed on either side of the pile. These level readings are a good back-up to the deflectometers in case the latter are damaged.

Testing shall be in accordance with the requirements of the specification applicable to the particular project.

(b) Nuclear back-scatter integrity testing

This form of test is available in South Africa and has been used on a number of recently constructed bridges. The system uses a nuclear isotope which is lowered down sleeves cast into the shaft of the pile. The recording device registers the amount of back-scatter from the surrounding concrete and is thus able to indicate the density of the concrete. This is compared to base data derived from a mathematical model or to that measured in a special reference pile which is cast with specific imperfections and thus the integrity of a piles shaft can be checked.

## (c) Diamond coring of the shaft and rock

Diamond coring of the pileshaft is not normally carried out as a routine method of integrity testing. It is sometimes resorted to when there is some doubt about the integrity of a particular pile or where the diamond coring of the contact between the pileshaft and the rock is carried out as a routine check. A sleeve is cast into the pile such that the toe of the sleeve is about a metre off the bottom of the pile to reduce on the drilling cost to ensure that the drill toe does not wander outside the diameter of the pile and to speed up the process. The drill rig can then be set up at a later stage to drill through the one metre of shaft into the rock up to the desired depth to check the contact and the quality of the rock beneath the toe of the pile.

## (d) Sonic integrity testing

There are two different systems available in South Africa for carrying out sonic integrity testing.

The one is known as sonic logging and involves lowering a transmitter and a receiver down two separate holes cast into the pile shaft. The transmitter emits a signal which passes through the concrete and is received by the receiver. If four pipes are cast into the pile then a total of six tests can be made. Only the concrete on the path between transmitter and receiver will be examined for defects. The instrument has to be calibrated in a similar way to that of the nuclear back-scatter system. Sonic logging is one of the more reliable systems for integrity testing of large diameter piles.

The other system involves the striking of the head of the pile with a hammer and measuring the reflected waves using a sensing device which is hand-held against the surface of the concrete. The signal is recorded in the memory of the instrument and may be plotted when required.

The test is simple to perform and can be carried out on any pile after the concrete has aged at least five days. The only preparation needed is exposing of sound concrete at the head of the pile.

The method has the following limitations:

- The reflected wave from the toe of the pile can only be recorded if the pile length is less than about 40 to 60 pile diameters.
- Only major defects will be indicated.

This test can be a useful additional source of integrity information. It is an inexpensive test and in excess of 100 piles can be tested in a day with a single set of equipment. Piles need very little preparation for testing. The majority of bridge piles should be within the depth confidence range of the system:

## 3.2.5 Caissons

## (i) General

Caissons have lost favour in this country as piling equipment and techniques improved, especially after the introduction of oscillator bored piles.

Advantage of caissons over piled foundations:

- Large obstructions such as boulders and buried concrete are more easily removed.

Caissons can be used to good effect in deep water for the construction of spread footings where the provision of working platforms and associated other work is not possible or very expensive.

Disadvantages of caissons over piled foundations:

- Installation procedure is normally slower.
- Higher risk involved due to possibility of larger, more expensive components sticking and/or tilting during sinking operations.
- Sealing at the cutting edge against ingress of water is difficult in all materials except impermeable clays.
- Depth of operation is less than that for piles and is generally limited to about 30 m.

Careful considerations must be given to the nature of the material at the founding level as well as underlying layers. In particular, the following aspects must be borne in mind:

- Erodability of material at founding level and below.
- Nature and extent of rock fractures and bedding planes.
- Soft layers below proposed founding level.
- Possibility that boulders may be mistaken for bedrock.
- Dip of inclined or jointed strata toward exposed faces and discontinuities in strata.

It is not possible to give universally applicable allowable bearing pressures due to the vast variation in conditions and circumstances which apply at different sites. Presumed bearing values which may be used for different categories of foundation materials for initially proportioning foundations are given in BS 8004 (1986). Refer Table 6.2.1 in Appendix 6.2.

Caissons on rock should preferably render the structure stable without the use of stressed ground anchors or dowels. If stressed ground anchors can not be avoided, they should be corrosion resistant and designed to permit checking of residual load in the future.

Corrosion resistant dowels may be used provided that the necessary anchorage is assured in the bedrock.

#### (ii) Types of caissons

Caissons fall into four broad categories viz pneumatic caissons, box caissons, cylinders and open caissons.

## (a) Pneumatic caissons

The essential feature of the pneumatic caisson is the air filled chamber in which workmen may carry out excavation procedures. The air pressure approximately equals the corresponding hydrostatic pressure of the water in the ground. A roof is provided near the lower end of the caisson to form a working chamber and air locks are provided above the roof to the surface to allow access for workmen, materials, equipment and mucking operations.

Advantages of the system are:

- Work under submerged conditions is eliminated which permits unhindered viewing and improved conditions for the removal of obstructions as well as accurate assessment of founding conditions.
- Air pressures may be adjusted to assist with the sinking process.

The disadvantages of the system are:

- Extremely high costs. Specialised equipment is required and workmen have to work under stringently controlled regulations which require reduced working hours and recovery times after working under enhanced pressures. Also, additional service personnel such as lock tenders, gauge tenders, compressor operators, specially trained medical attendants, fitters etc are required.
- Depth of operation is limited by the air pressure in the chamber under which workmen may operate, usually about 30 m below water level.

This method is used only when all other methods are not feasible and due to its rare use does not warrant further description here. Foundation Engineering: Leonards (1962) should be consulted for details and additional information.

## (b) Box caissons

Box caissons are provided with floors at the lower ends, thus creating buoyant structures which may be floated into place. The system has limited application as pre-prepared bearing surfaces are required which are extremely difficult to prepare under water.

## (c) Cylinders

Cylinders are open ended single thin walled tubes with singular openings and are generally sunk by the addition of kentledge, by jacking down or by impact. The combination of the relatively light cylinder and the kentledge placed on top of the cylinder result in a high centre of gravity. Cylinders are therefore difficult to keep plumb while sinking and staging is normally necessary to ensure successful sinking.

(d) Open caissons

Open caissons are open ended double walled or thick walled structures with single or multiple openings and are generally sunk under their own weight or by the addition of concrete or other permanent filling. The centre of gravity is lower than that of kentledge/cylinder combinations and are therefore easier to keep plumb. This system has been the most popular in South Africa.

## 4. STRUCTURAL DESIGN

### 4.1 Introduction

Structural design shall be carried out in accordance with the requirements of TMH 7 Parts 1, 2 and 3. Hydraulic forces and associated load factors are discussed in Section 6 of Volume I, "Hydraulics, Hydrology and Ecology". It will be noted that a range of possible coefficients applicable to the load formulae result in appreciable variations in forces. Users of these guidelines should consult later literature to obtain more refined information which will become available from experimental work presently under way.

### 4.2 Overturning effects

When considering overturning effects on piled structures the depth of scour must be taken into account to ensure that the full overturning effect of the hydraulic forces acting on the deck, piers, abutments and exposed piles or caissons will be resisted. Refer, Section 6 of Volume I, "Hydraulics, Hydrology and Ecology".

Particular attention should be paid to:

- The combined effects of bending and axial loads together with vibration of the unsupported pile lengths.
- The additional bearing stresses at the undersides of the pile/caisson resulting from hydraulic loads.
- Resistance to pull out of piles.
- Floatation forces resulting from hollow piers and air trapped in or under decks.

### 4.3 Displacement of decks

Decks may be subject to the combined effects of horizontal and vertical load due to the action of floodwater.

Resistance to horizontal forces should be provided by mechanical means such as uni-directional bearings, dowels, keys between the deck undersides and substructures or upstand walls on the substructures provided on the downstream side of the substructures.

Beam and slab decks and other deck configurations which could trap air under flood conditions should be provided with air escape openings (vents) to all compartments. As a general guide the area of the vertical escape openings should not be less than 0,01% of the plan area of air trapping compartments. Air escape openings shall be placed at the high point of each compartment. Holes need not all be through the deck as continuity of air escape paths may be attained by providing horizontal openings through webs and/or diaphragms directly below the deck slabs. Drainage downspouts located in air trapping compartments will serve a dual function by allowing air escape as well.

Appropriate measures should be taken with voided decks if the floodwater level is likely to rise above the soffit of the deck. Air vents should be considered together with openings to the voids to allow the ingress of water and rapid release when floodwaters subside to prevent excessive loads on the deck.

If adequate and reliable measures are not taken to prevent uplift, the decks shall be held down by mechanical means.

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## 6. APPENDICES

## 6.1 Tests

TEST	MATERIAL	SAMPLE TYPE	REMARKS	REFERENCES
Grading Analysis (a) Sieving	Granular soils and gravels	D	Usually carried out in conjunction with Atterberg limit tests to give an indication of the soil behaviour and to classify the soil.	TMH 1 (1986)
(b) Sedimentation	Cohesive and fine grained soils	D		BS 1377 (1975)
Atterberg Limits	Cohesive and fine grained soils	D	Plastic limit, liquid limit, plasticity index and linear shrinkage. Give indication of soil behaviour.	TMH 1 (1986) TMH 1 (1979)
Moisture Content	Soils or rocks	D/U	Frequently carried out as part of other tests. Required for determination of degree of saturation.	TMH 1 (1986) BS 1377 (1975)
Specific Gravity	Soils or rocks	D	Used in conjunction with other tests such as density, moisture content and sedimentation.	TMH 1 (1986) BS 1377 (1975)
Bulk Density	Soils	D/I	May be carried out on undisturbed samples in the lab. Cohesionless soils must be tested in situ. Used with above two tests to determine degree of saturation and void ratio.	TMH 1 (1986) BS 1377 (1975)
Triaxial Compression (a) Undrained unconsolidated	Saturated, normally consolidated clays	U	Undrained shear strength ( $\Phi=0$ ). Short term stability. In fissured clays, sample size has a significant effect.  Effective strength parameters ( $c'$ , $\Phi'$ ).  Effective strength parameters ( $c'$ , $\Phi'$ ) Long term stability.	Bishop & Henkel (1962) Marsland (1971)
(b) Consolidated undrained with p.w.p measurements	Saturated normally consolidated clays. Partially saturated clays (soaked)	U		Bishop & Henkel (1962) Akroyd (1957)
(c) Consolidated drained	Clayey sands, sandy clays, silts. Partially saturated clays (soaked)	U U		Bishop & Henkel (1962) Akroyd (1957)
Direct Shear Box: (a) Immediate	Saturated clayey sands, silts and clays	U	Undrained shear strength. Triaxial tests generally preferred.  Effective strength parameters ( $c'$ , $\Phi'$ ).  Angle of shearing resistance ( $c'=0$ ).	Akroyd (1957)
(b) Drained	Clayey sands, sandy clays and silt	U		Akroyd (1957)
	Dry or saturated sands	R		Akroyd (1957) Bishop (1948)
Unconfined Compressive Strength	Saturated intact clays	U	Simple and rapid substitute for undrained triaxial test.	Akroyd (1957)
One dimensional Consolidation	Cohesive and fine grained soils	U	Gives measure of compressibility, pre-consolidation pressure and coefficient of consolidation.	Akroyd (1957)
Triaxial Consolidation	Cohesive and fine grained soils. Recompacted sands	U R	Triaxial consolidation gives measure of elastic modulus.	Akroyd (1957) Bishop & Henkel (1962)
Rowe Cell Consolidation	Cohesive and fine grained soils	U/R	Rowe cell uses larger samples and confining pressure may be varied.	Rowe & Barden (1966)

U = undisturbed

D = Disturbed

R = Remoulded

I = Insitu

TABLE 6.1.1 : LABORATORY TESTS ON SOILS

TEST	MATERIAL	SAMPLE TYPE	REMARKS	REFERENCES
Moisture Content, Bulk Density, Porosity	All rocks	C/L	Gives indication of strength, modulus of elasticity and degree of weathering.	Int Soc Rock Mech (1979)
Swelling Test	Mainly argillaceous rocks	C/L	Indicates moisture sensitivity of rock and possible volume changes.	Duncan et al (1968)
Point Load Test	Isotropic rocks	C/L	Quick and cheap indicator of rock strength. Useful aid to core logging.	Hoek (1977)
Uniaxial Compression Test	Most rocks which can be cored	C	Strength of intact rock. Upper limit for jointed rock mass strength. Widely used for predicting bearing capacity and skin friction. Gives elastic properties of "intact" rock core. This will over estimate modulus of jointed rock.	Hoek (1977) Hawkes & Mellor (1970) Clark (1966)
Triaxial Compression Test	Very soft/soft rock "intact" weathered rock	C	As above. Only weak rocks may be tested with commonly available equipment.	Hoek (1977)
Direct Shear Box Test	Usually applied to rock discontinuities or intact rock	L/I	Gives shear strength along discontinuities or of intact soft rocks.	Hoek (1977)

C = Core Sample

L = Lump Sample

I = In situ

TABLE 6.1.1 continued : LABORATORY TESTS ON ROCK

TEST	MATERIAL	REMARKS	REFERENCES
Standard Penetration Test (SPT)	Mainly sands and weak rocks. Also used on other soils	Performed generally at 1,5 m intervals in boreholes. In rocks, the penetration may be recorded for a given number of blows (say 4 sets of 20 blows). Disturbed sample obtained for identification. Gives indication of consistency/relative density of soil. Results correlated to many soil properties and empirical design methods.	Webb (1976) SAICE & NITRR (1978) Sanglerat (1972) Ervin (1983)
Dynamic Cone or Dynamic Spoon Test	Most soils	Performed by driving cone or spoon with no intermediate drilling or reaming of hole. Not as widely accepted as SPT test but cheaper to perform. Disturbed sample obtained from spoon test. Results correlated with SPT results.	Webb (1976) SAICE & NITRR (1978) Sanglerat (1972) Ervin (1983)
Static Cone Test (Dutch Probe)	Loose granular soils and soft to firm clays	Gives point resistance (cone) and total resistance (cone and friction sleeve). Results correlated with bearing capacity and compressibility.	TMH6 (1984) Webb (1976) SAICE & NITRR (1978) Sanglerat (1972) Ervin (1983)
Piezocone	Saturated, loose, granular soils and soft to firm clays	Tip resistance, sleeve resistance and dynamic pore pressure are measured while continuously driving the probe into the ground.	Clemence (ED) (1986)
Pressuremeter	Soils and weak rocks	Pressuremeters may either be inserted into boreholes, be driven into the ground in a slotted casing or be self boring. Results give indication of elastic modulus and soil strength.	SAICE & NITRR (1978) Menard (1965) Windle & Wroth (1977) Ervin (1983)
Goodman Jack	Soft to hard rocks	Jack inserted into NX sized borehole. Rock loaded by curved plates and deformations measured. May be installed in horizontally drilled holes and rotated to determine degree of anisotropy. Results give indication of elastic modulus of in situ rock mass.	Goodman et al (1968)
Piezometer	All soils and rocks	Used to determine ground water pressure at various depths in the ground. In permeable ground, standpipe piezometers are used but in impermeable conditions or where rapid response is require, hydraulic, pneumatic or electric peizometers are used.	SAICE & NITRR (1978) BS 5930 (1981) Penman (1960)
Vane Shear Test	Saturated cohesive soils	Normally restricted to saturated clays with an undrained shear strength of less than 100 kPa. This method can give peak and residual undrained shear strengths.	SAICE & NITRR (1978) BS 5930 (1981) Ervin (1983)
Plate Bearing Test	Moist soils and soft rocks. Generally above water table	Test performed in trench or auger hole by jacking circular plates against the soil/rock. May be carried out horizontally (across width of hole) or vertically (jacking against a kentledge). Size of plate depends on hole size and stiffness of material generally 75 - 300 mm for horizontal tests and 200 - 1000 mm to vertical test.	Ervin (1983) Wrench (1984)

TABLE 6.1.2: FIELD TESTS

## 6.2 Allowable bearing pressures (from BS 8004)

Presumed allowable bearing values under static loading (see 1.2.3 and 1.2.4)			
NOTE: These values are for preliminary design purposes only, and may need alteration upwards or downwards. No addition has been made for the depth of embedment of the foundation (see 2.1.2.3.2 and 2.1.2.3.3).			
Category	Types of rocks and soils	Presumed allowable bearing value (kPa)	Remarks
Rocks	Strong igneous and gneissic rocks in sound condition	10 000	These values are based on the assumption that the foundations are taken down to unweathered rock. For weak, weathered and broken rock, see 2.2.2.3.1.12
	Strong limestones and strong sandstones	4 000	
	Schists and slates	3 000	
	Strong shales, strong mudstones and strong siltstones	2 000	
Non-cohesive soils	Dense gravel, or dense sand and gravel	> 600	Width of foundation not less than 1 m. Groundwater level assumed to be a depth not less than below the base of the foundation. For effect of relative density and groundwater level, see 2.2.2.3.2
	Medium dense gravel, or medium dense sand and gravel	< 200 to 600	
	Loose gravel, or loose sand and gravel	< 200	
	Compact sand	> 300	
	Medium dense sand	100 to 300	
	Loose sand	< 100	
Cohesive soils	Very stiff boulder clays and hard clays	300 to 600	Group 3 is susceptible to long-term consolidation settlement (see 2.1.2.3.3). For consistencies of clays, see table 5
	Stiff clays	150 to 300	
	Firm clays	75 to 150	
	Soft clays and silts	< 75	
	Very soft clays and silts	Not applicable	
Peat and organic soils		Not applicable	See 2.2.2.3.4
Made ground or fill		Not applicable	See 2.2.2.3.5

The clause numbers in this table refer to BS 8004 and not to these guidelines

TABLE 6.2.1: PRESUMED ALLOWABLE BEARING VALUES UNDER STATIC LOADING

6.3 Side resistance reduction factor

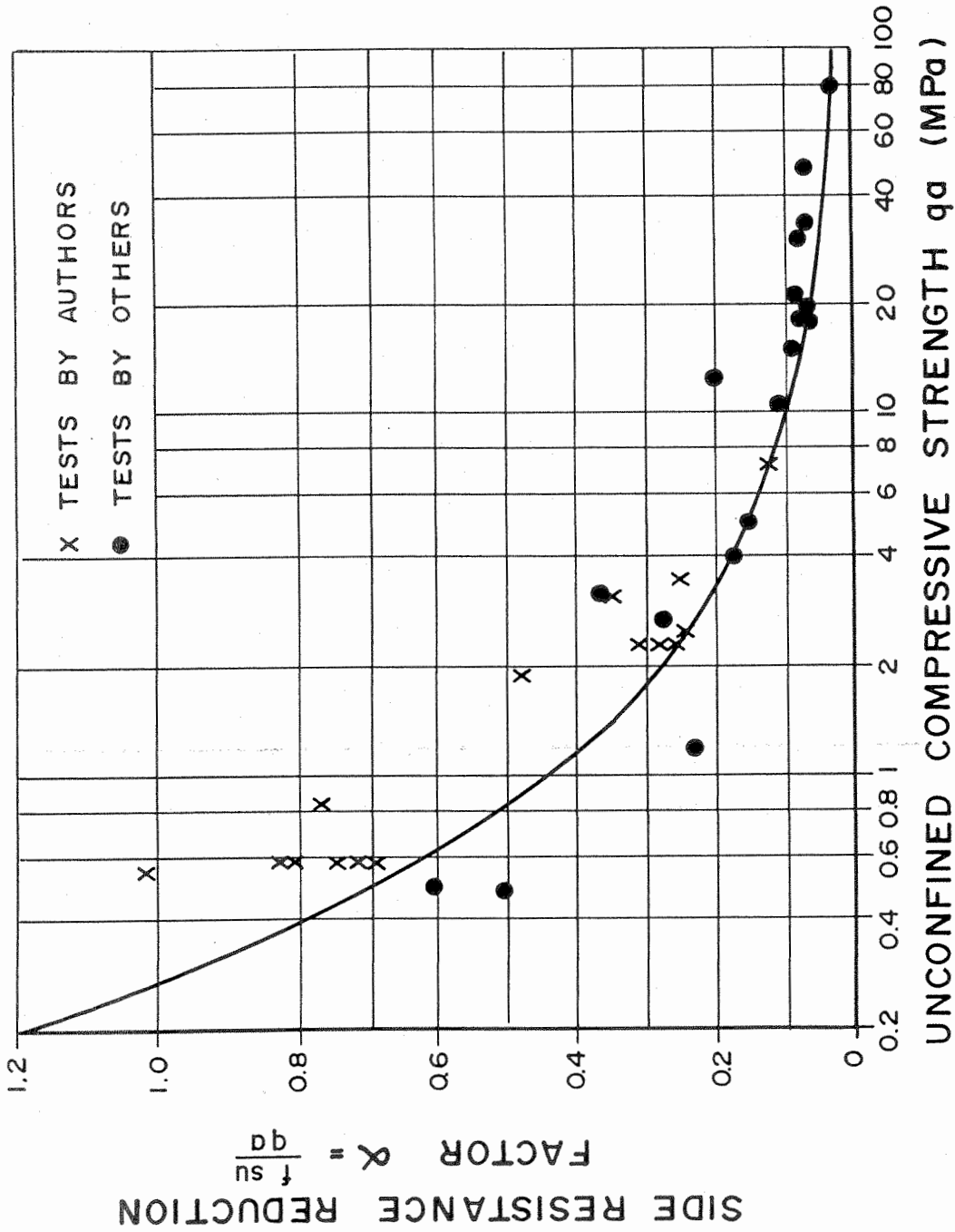


FIGURE 6.3.1 : GRAPH OF SIDE RESISTANCE REDUCTION FACTOR vs UNCONFINED COMPRESSIVE STRENGTH

6.4 Horizontal subgrade reaction

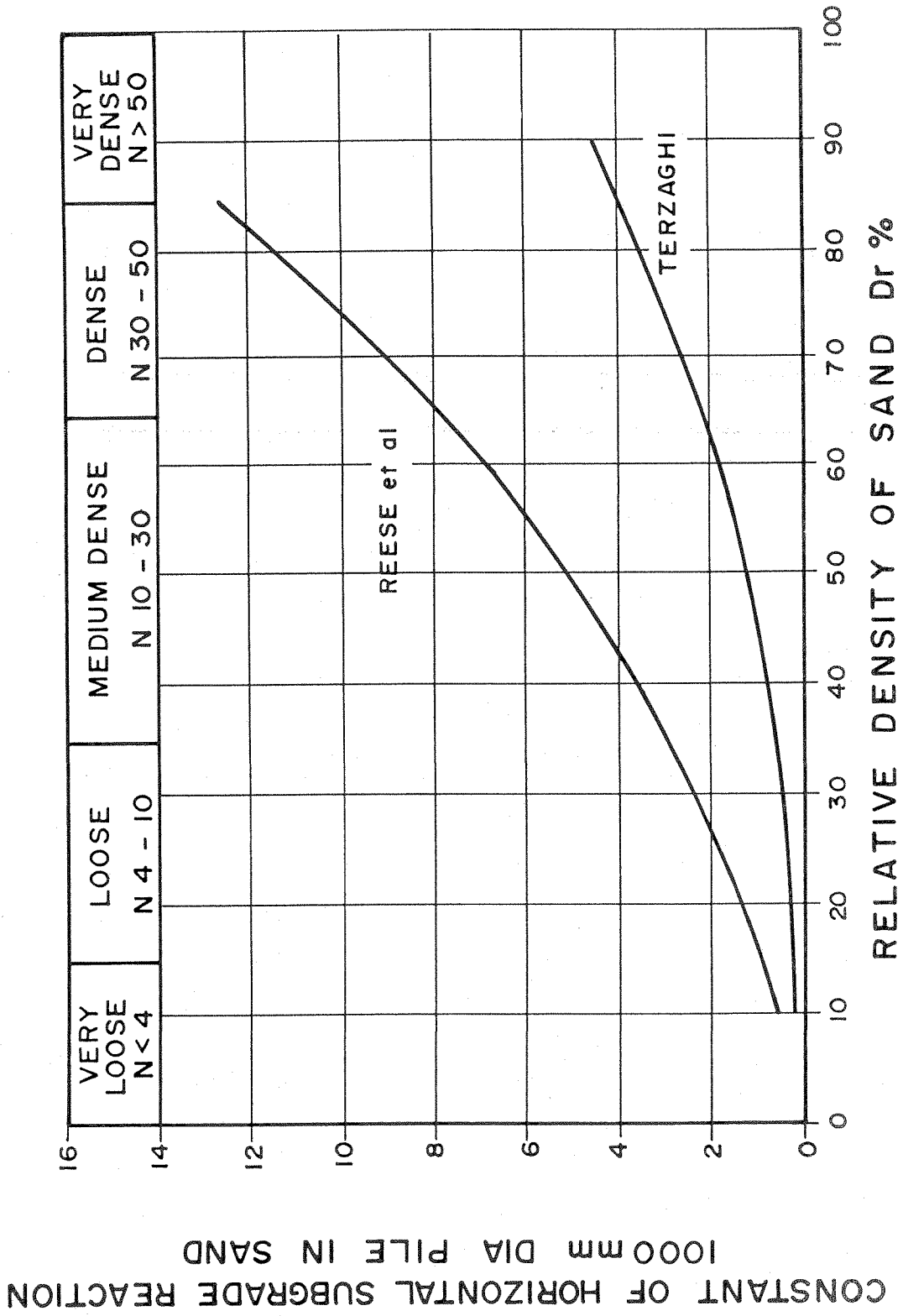


FIGURE 6.3.1 : GRAPH OF CONSTANT OF HORIZONTAL SUBGRADE REACTION FOR 1000 mm DIA. PIPE IN SAND vs RELATIVE DENSITY OF SAND

## 6.5 Rock socket design for piles

UCS of the Rock (kPa)		1000	2000	3000	4000	5000	6000	7000	8000	9000
*Side resistance reduction factor		0,43	0,27	0,22	0,175	0,15	0,13	0,11	0,1	0,1
End Bearing Factor		4,5	4,5	4,5	4,5	4,5	4,5	4,5	4,5	4,5
Socket Depth (m)	Diams	Safe Socket Capacity kN								
0	0	424	848	1272	1696	2121				
0,3	0,5	505	950	1397	1828	2262				
0,6	1	586	1052	1521	1960	2403				
0,9	1,5	667	1154	1646	2092	2545				
1,2	2	748	1255	1770	2224					
1,5	2,5	829	1357	1894	2356					
1,8	3	910	1459	2019	2488					
2,1	3,5	991	1561	2143						
2,4	4	1073	1663	2268						
2,7	4,5	1154	1764	2392						
3	5	1235	1866							
3,3	5,5	1316	1968							
3,6	6	1397	2070							

Shaft stress assumed to be limited to 8 MPa  
Max Pile Load 2262 kN at 8 MPa shaft stress

TABLE 6.5.1 : ROCK SOCKET DESIGN FOR 600 mm DIA PILE

UCS of the Rock (kPa)		1000	2000	3000	4000	5000	6000	7000	8000	9000
*Side resistance reduction factor		0,43	0,27	0,22	0,175	0,15	0,13	0,11	0,1	0,1
End Bearing Factor		4,5	4,5	4,5	4,5	4,5	4,5	4,5	4,5	4,5
Socket Depth (m)	Diams	Safe Socket Capacity kN								
0	0	663	1325	1988	2651	3313	3976			
0,375	0,5	789	1484	2182	2857	3534				
0,75	1	916	1643	2377	3063	3755				
1,125	1,5	1043	1802	2571	3269	3976				
1,5	2	1169	1962	2766	3475					
1,875	2,5	1296	2121	2960	3682					
2,25	3	1423	2380	3154	3888					
2,625	3,5	1549	2439	3349						
3	4	1676	2598	3543						
3,375	4,5	1802	2757	3738						
3,75	5	1929	2916							
4,125	5,5	2056	3075							
4,5	6	2182	3234							

Shaft stress assumed to be limited to 8 MPa  
Max Pile Load 3534 kN at 8 MPa shaft stress

TABLE 6.5.2: ROCK SOCKET DESIGN FOR 750 mm DIA PILE

UCS of the Rock (kPa)		1000	2000	3000	4000	5000	6000	7000	8000	9000
*Side resistance reduction factor		0,43	0,27	0,22	0,175	0,15	0,13	0,11	0,1	0,1
End Bearing Factor		4,5	4,5	4,5	4,5	4,5	4,5	4,5	4,5	4,5
Socket Depth (m)	Diams	Safe Socket Capacity kN								
0	0	954	1909	2863	3817	4771	5726			
0,45	0,5	1137	2138	3134	4114	5089				
0,9	1	1319	2367	3423	4411	5407				
1,35	1,5	1501	2596	3703	4708	5726				
1,8	2	1684	2825	3982	5005					
2,25	2,5	1866	3054	4262	5301					
2,7	3	2048	3283	4542	5598					
3,15	3,5	2231	3512	4822						
3,6	4	2413	3741	5102						
4,05	4,5	2596	3970	5382						
4,5	5	2778	4199							
4,95	5,5	2960	4428							
5,4	6	3143	4657							
Shaft stress assumed to be limited to 8 MPa.										
Max Pile Load 5089 kN at 8 MPa shaft stress										

TABLE 6.5.3 : ROCK SOCKET DESIGN FOR 900 mm DIA PILE

UCS of the Rock (kPa)		1000	2000	3000	4000	5000	6000	7000	8000	9000
*Side resistance reduction factor		0,43	0,27	0,22	0,175	0,15	0,13	0,11	0,1	0,1
End Bearing Factor		4,5	4,5	4,5	4,5	4,5	4,5	4,5	4,5	4,5
Socket Depth (m)	Diams	Safe Socket Capacity kN								
0	0	1178	2356	3534	4712	5890	7069			
0,5	0,5	1403	2639	3880	5079	6283				
1	1	1628	2922	4225	5445	6676				
1,5	1,5	1854	3204	4571	5812	7069				
2	2	2079	3487	4917	6178					
2,5	2,5	2304	3770	5262	6545					
3	3	2529	4053	5608	6912					
3,5	3,5	2754	4335	5953						
4	4	2979	4618	6299						
4,5	4,5	3204	4901	6644						
5	5	3430	5184							
5,5	5,5	3655	5466							
6	6	3880	5749							
Shaft stress assumed to be limited to 8 MPa										
Max Pile Load 6283 kN at 8 MPa shaft stress										

TABLE 6.5.4 : ROCK SOCKET DESIGN FOR 1000 mm DIA PILE

UCS of the Rock (kPa)		1000	2000	3000	4000	5000	6000	7000	8000	9000
*Side resistance reduction factor		0,43	0,27	0,22	0,175	0,15	0,13	0,11	0,1	0,1
End Bearing Factor		4,5	4,5	4,5	4,5	4,5	4,5	4,5	4,5	4,5
Socket Depth (m)	Diams	Safe Socket Capacity kN								
0	0	1696	3393	5089	6786	8482	10179			
0,6	0,5	2021	3800	5587	7314	9048				
1,2	1	2345	4207	6085	7841	9613				
1,8	1,5	2669	4614	6582	8369	10179				
2,4	2	2993	5022	7080	8897					
3	2,5	3318	5429	7578	9425					
3,6	3	3642	5836	8075	9953					
4,2	3,5	3966	6243	8573						
4,8	4	4290	6650	9070						
5,4	4,5	4614	7057	9568						
6	5	4939	7464							
6,6	5,5	5263	7872							
7,2	6	5587	8279							
Shaft stress assumed to be limited to 8 MPa										
Max Pile Load 9048 kN at 8 MPa shaft stress										

TABLE 6.5.5 : ROCK SOCKET DESIGN FOR 1200 mm DIA PILE

UCS of the Rock (kPa)		1000	2000	3000	4000	5000	6000	7000	8000	9000
*Side resistance reduction factor		0,43	0,27	0,22	0,175	0,15	0,13	0,11	0,1	0,1
End Bearing Factor		4,5	4,5	4,5	4,5	4,5	4,5	4,5	4,5	4,5
Socket Depth (m)	Diams	Safe Socket Capacity kN								
0	0	2147	4294	6441	8588	10735	12882			
0,67	0,5	2557	4809	7071	9256	11451				
1,35	1	2968	5325	7701	9924	12167				
2,025	1,5	3378	5840	8331	10592	12882				
2,7	2	3788	6355	8960	11260					
3,375	2,5	4199	6871	9590	11928					
4,05	3	4609	7386	10220	12596					
4,725	3,5	5019	7901	10850						
5,4	4	5430	8417	11480						
6,075	4,5	5840	8932	12110						
6,75	5	6250	9447							
7,425	5,5	6661	9962							
8,1	6	7071	10478							
Shaft stress assumed to be limited to 8 MPa										
Max Pile Load 11451 kN at 8 MPa shaft stress										

TABLE 6.5.6 : ROCK SOCKET DESIGN FOR 1350 mm DIA PILE

UCS of the Rock (kPa)		1000	2000	3000	4000	5000	6000	7000	8000	9000
									0	0
*Side resistance reduction factor		0,43	0,27	0,22	0,175	0,15	0,13	0,11	0,1	0,1
End Bearing Factor		4,5	4,5	4,5	4,5	4,5	4,5	4,5	4,5	4,5
Socket Depth (m)	Diams	Safe Socket Capacity kN								
0	0	2651	5301	7952	10603	13254	15904			
0,75	0,5	3157	5938	8730	11428	14137				
1,5	1	3664	6574	9507	12252	15021				
2,25	1,5	4170	7210	10285	13077	15904				
2,3	2	4677	7846	11062	13902					
3,75	2,5	5184	8482	11840	14726					
4,5	3	5690	9118	12617	15551					
5,25	3,5	6197	9755	13395						
6	4	6703	10391	14173						
6,75	4,5	7210	11027	14950						
7,5	5	7717	11663							
8,25	5,5	8223	12299							
9	6	8730	12936							
Shaft stress assumed to be limited to 8 MPa.										
Max Pile Load 14137 kN at 8 MPa shaft stress										

TABLE 6.5.7 : ROCK SOCKET DESIGN FOR 1500 mm DIA PILE

\* Refer Figure 6.3.1

#### Illustrative example to calculate rock socket length

Calculate socket depth required to resist a working load of 12000 kN. Unconfirmed compressive strength of rock is 3000 kPa.

Initially select a 1350 mm dia pile. Read down column headed UCS 3000 kPa until 12000 kN is exceeded and read horizontally to 4,5 m or 3 pile diameters. Check

whether a 1200 mm diameter pile will suffice. By following the same procedure it will be found that the line indicating a shaft stress in excess of 8 MPa will be crossed and a 1200 mm diameter pile will therefore not be suitable.