Related papers:

Rowe, R.K. and Armitage, H.H. (1987). “Theoretical solutions for axial deformation of drilled shafts in rock”, *Canadian Geotechnical Journal*, 24(1):114_125.

Rowe, R.K. and Armitage, H.H. (1987). “A design method for drilled piers in soft rock”, *Canadian Geotechnical Journal*, 24(1):126_142.

Rowe, R.K. and **Armitage**, H.H. (1987). “A new design method for drilled piers in soft rock: Implications relating to three published case histories”, Proceedings of the 6th International Congress on Rock Mechanics, Montreal, pp. 497_502.

Rowe, R.K. and Armitage, H.H. (1984). “The design of piles socketed into weak rock”, Research Report GEOT_11_84, Faculty of Engineering Science, University of Western Ontario. (Major report submitted to the National Research Council, Canada, 368p.).

THE DESIGN OF PILES SOCKETED INTO WEAK ROCK

DSS File Number: 24SU.31944-3-0017

Contract Serial Number: 1SU83-00082

Project Manager: Mr. I.C. MacFarlane

Project Scientist: Dr. M. Bozozuk

Prepared for:

National Research Council Canada
Ottawa, Ontario
Canada

Prepared by:

R.K. Rowe B.Sc.,B.E.,Ph.D.

and

H.H. Armitage B.E.Sc.

Faculty of Engineering Science
University of Western Ontario
London, Canada

Geotechnical Research Report GEOT-11-84

THE DESIGN OF PILES SOCKETED INTO WEAK ROCK
by R.K. Rowe and H.H. Armitage
GEOT-11-84

TABLE OF CONTENTS

GLOSSARY

Complete Socketed Pile: a pile which derives the majority of its load carrying capacity from a combination of side-shear and endbearing resistance developed in the rock socket.

Endbearing Socketed Pile: a pile which carries the applied load entirely in endbearing resistance developed at the base of the rock socket.

Full Slip: condition where relative displacement occurs along the entire pile-rock interface, once the shear strength of the interface is reached.

Recessed Pile: a section of concrete pile which is directly above the socketed pile, but contributes little or no resistance to the load-carrying capacity of the socketed pile.

Rock Socketed Pile: a drilled, cast-in-place concrete pile which is constructed in a rock socket (other common names include, rock socketed pier or caisson)

Rock Socket: a cylindrical hole drilled, churned or augered into rock. Concrete (and reinforcing steel) is placed in the hole to form a rock socketed pile.

Sideshear Resistance (bond): cohesive and/or frictional resistance developed at the interface between the concrete pile and surrounding rock, under an applied load.

Sideshear Resistance pile: a socketed pile which is intended to carry the applied load entirely by shear resistance developed at the pile-rock interface.

NOTATION

a	radius of a rock socketed pile (m)
c_p	peak pile-rock interface adhesion (MPa)
c_r	residual pile-rock interface adhesion (MPa)
D	diameter of a rock socketed pile (m)
E_b	rock mass modulus below the base of the rock socket (MPa)
E_d, E_d^*	design values of rock mass modulus (MPa) (* indicates modified value for seams, see section 6.4.2)
E_{eq}	equivalent modulus for a rock mass which contains seams (MPa)
E_p	Young's modulus of the concrete (with or without steel reinforcement) pile (MPa)
$\bar{E}_r$	expected value of rock mass modulus (MPa)
E_r	mass modulus of rock surrounding the rock socketed pile (MPa)
E_s	modulus of seam material within a rock mass (MPa)
E_t	mass modulus of rock surrounding the recessed pile (MPa)
I	settlement influence factor
I_n	net settlement influence factor for a recessed socketed pile
I_{no}	settlement influence factor when the socketed pile is not recessed
I_d, I_d^*, I_s	design settlement influence factor (* indicates a modified value, see section 6.4.2)
I_g	gross settlement influence factor relative to the top of a recessed pile
I_h	"fictitious" homogeneous settlement influence factor
L	length of the socketed pile (m)
L_{dmax}, L_{dmax}^*	maximum socket length assuming all load carried by socket shear (m). (*indicates modified value for seams, see section 6.4.2)

L_s	length of seams along the rock socket (m)
L_e	length of a recessed pile (m)
L_i	length of rock socket (m)
L_s	length of seams along rock socket (m)
L/D	length to diameter ratio
$(L/D)_d$	design length to diameter ratio
$(L/D)_{dmax}$, $(L/D)^*_{dmax}$	maximum length to diameter ratio assuming all load carried by socket shear (* indicates modified value to account for seams)
M	total number of Monte Carlo simulations
$N\{\rho\}_e > \rho_d$	number of generated settlements which exceed the design settlement
P_b	load carried in endbearing (MN)
P_b/P_t	proportion of load carried in endbearing at the pile base
$(P_b/P_t)_d$	design value of the proportion of load carried in endbearing at the pile base
$(P_b/P_t)_{fs}$	proportion of load carried in endbearing at the pile base at full slip
P_s	load carried by sideshear resistance at the pile-rock interface (MN)
P_t	total axial load applied to the socketed pile (MN)
P_{ts}	total axial load applied to socketed pile at full slip (MN)
P.D.F.	probability density function
$PR(\rho > \rho_d)$	probability of exceeding the design settlement (%)
q	vertical applied pressure (MPa)
q_{ave}	average applied pressure at the surface of a circular footing (MPa)
q_b	base pressure at the base of the socketed pile (MPa)
q_{ba}	allowable bearing pressure at the base of the socketed pile (MPa)
q_m	maximum base pressure at the base of the socketed pile (MPa)

q_{ma}	maximum bearing pressure at the base of the socketed pile (MPa)
q_t	total average applied pressure on the socketed pile (MPa)
q_u	unconfined compressive strength of rock (MPa)
$(RF)_{E_r}$	rock mass modulus reduction factor
$(RF)_s$	reduction factor for ultimate endbearing conditions
$(RF)_\rho$	settlement reduction factor (for a recessed socketed pile)
$(RF)_\tau$	average sideshear resistance reduction factor
r	radial distance from the centreline of a socketed pile (m)
S	proportion of seams within a rock mass
z	vertical depth below the ground or rock surface (m)
α	normalized average sideshear resistance (i.e., τ_f/q_u)
δ_d	limiting normal displacement at the pile-rock interface
δ_r	relative normal displacement at the pile-rock interface
ε_r	radial strain
ε_z	vertical strain
ε_θ	circumferential strain
ν_p	Poisson's ratio for (concrete) pile
ν_r	Poisson's ratio for rock material
ρ_{avg}	average vertical displacement of a rock socket pile (i.e., average of centreline and edge displacements) (m)
ρ_d	design settlement for a rock socketed pile (m)
$\sigma_z(r)$	vertical stress at a radial distance r from centreline (MPa)
σ_z	vertical stress (MPa)
$\sigma_{\ell nE_r}$	standard deviation of $\ell n\overline{E}_r$
$\sigma_{\ell n\tau}$	standard deviation of $\ell n\overline{\tau}$

$\bar{\tau}, \bar{\tau}^*$	expected value of available sideshear resistance (MPa) (* indicates a modified value for seams, see section 6.4.2)
τ_{cr}	critical (no slip) value of average sideshear resistance (MPa)
τ_d, τ_d^*	design values of available sideshear resistance (MPa) (*indicates a modified value for seams, see section 6.4.2)
τ_f, τ_r	average available sideshear resistance at the pile-rock interface (MPa)
τ_{peak}	peak average sideshear resistance at the pile-rock interface (MPa)
$\tau_{residual}, \tau_{re}$	residual average sideshear resistance at the pile-rock interface (MPa)
τ_s	average sideshear resistance at the pile-seam interface (MPa)
ϕ	friction angle (degree)
ϕ_p	peak friction angle at the pile-rock interface (degree)
ϕ_r	residual friction angle at the pile-rock interface (degree)
ψ	interface dilatancy angle
$\{E_r\}_e$	statistically determined point estimate of rock mass modulus (MPa)
$\{\rho\}_e$	predicted estimate of pile settlement (m)
$\{\tau\}_e$	statistically determined point estimate of average sideshear resistance (MPa)
℄	centreline

EXECUTIVE SUMMARY

Rapid growth in many metropolitan areas has resulted in the construction of taller and heavier structures. The large foundations loads associated with these structures are often supported by piles socketed into rock.

At present, these piles are generally designed using empirical values of allowable stresses at the base and along the shaft. Various researchers have suggested that socketed piles be designed using higher allowable values of sideshear resistance along with the use of elastic theory. While evidence exists to support the adequacy and safety of both approaches, they also may lead to an overly conservative and uneconomical pile design.

A number of recent, well documented field case histories have produced a sizeable quantity of data concerning socketed pile behaviour. The objective of this study has been to gather and review the available field data and in conjunction with an appropriate theoretical analysis, develop a rational procedure for the design of rock socketed piles. The results of this investigation have shown that:

(a) The magnitude of the average sideshear resistance depends not only on strength parameters, but also upon dilatancy at interface, the length to diameter ratio and the relative modulus of the pile and rock.

(b) Following a careful review of the information contained in available case histories, field data was used to establish correlations

between a common index test (i.e., unconfined compressive strength) and the 1) average available sideshear resistance and 2) rock mass modulus. Simple empirical equations based on these correlations have been suggested as an aid to the selection of preliminary design parameters.

(c) A statistical study showed that the distribution of socketed pile settlement was strongly influenced by the variability of rock mass modulus and sideshear resistance particularly near load levels which cause slip at the pile-rock interface. Also, the analysis indicated that the probability of exceeding a particular design settlement was affected by 1) the socket geometry and 2) the value of reduction factors applied to the expected values of sideshear resistance and rock mass modulus obtained from the empirical correlations.

(d) By incorporating the results of the above analyses, a relatively simple procedure for the design of rock socketed piles has been developed. The design method is based upon 1) satisfying a specified design settlement criterion and 2) ensuring that an adequate factor of safety exists against the possibility of collapse.

(e) The method has been illustrated by comparing the results of a series of detailed design calculations with actual field sockets from several documented case histories. It was found that piles designed to have the same geometries as the tested piles would have adequately satisfied both settlement and collapse criteria.

It is anticipated that the information contained in this report will contribute to the design capabilities of consultants who are involved with the design of rock socketed piles.

CHAPTER 1
INTRODUCTION

1.1 GENERAL

Large foundation loads are often supported by piles socketed into rock. Historically, the design of these piles has been carried out using empirical values of allowable stresses on the base and the shaft. More recently, various researchers have advocated the use of higher allowable shear resistance in conjunction with the use of elastic theory for socketed pile design (eg. Pells and Turner, 1979; Horvath, 1980; Williams and Pells, 1981 and others).

There is considerable evidence to suggest that both approaches are safe. However, they may be too safe. As design column loads and the cost of forming rock sockets both increase, there is considerable motivation for the development of new design approaches which will avoid the increasing costs of unnecessary overdesign whilst guarding against the consequences of underdesign.

This need has been recognized by a number of authorities around the world who have funded large scale field tests (eg. in Canada DSS/NRC funded work reported by Horvath (1980); in Australia the Commonwealth Department of Construction funded work reported by Pells, Rowe and Turner (1980) and the Country Roads of Victoria funded work reported by Williams (1980)). These investigations have provided a wealth of field data which indicates that socketed piles may be safely designed to carry loads well in excess of those commonly adopted. However, these field investigations have not been adequately complemented by theoretical analysis and upto this point in time there is still no soundly based design procedure which takes full advantage of the increased pile capacity indicated by these extensive, and expensive, field testing programmes.

1.2 SCOPE

The objectives of this research project are:

(a) To delineate the mechanisms governing the response of piles socketed into weak rock;

(b) To develop an appropriate theoretically based design procedure supported by simple design charts, which will allow a more efficient, but safe, design of socketed piles.

(c) To the extent possible, to verify this design procedure by comparison with observed field behaviour.

The current state-of-the-art with respect to the design of concrete piles socketed into rock is discussed in Chapter 2. Chapter 3 describes the results of an extensive parametric study which was conducted to both delineate the mechanisms governing the response of socketed piles and to develop a series of design charts. The theoretical technique adopted (Rowe and Pells, 1980) makes provision for plastic failure within the soft rock, slip at the pile/rock interface, and dilatancy at the interface (dilatancy is related to the socket roughness). The approach also permits consideration of different construction procedures such as endbearing only, socket shear only or full endbearing and socket shear sockets.

Chapter 4 summarizes the results of available socketed pile case

histories. Based on the available data, empirical correlations are developed between unconfined compressive strength of rock and (1) the measured (average) sideshear resistance and (2) the backfigured rock mass modulus. These correlations are presented in the form of simple equations which provide some guidance in the selection of preliminary design parameters. The existing field data is also used to obtain an estimate of the variability of sideshear resistance and modulus for rock with a given unconfined strength q_u.

The implications of statistical variation in design parameters are discussed in Chapter 5. Using a Monte Carlo simulation technique, estimates of the probability of exceeding the design settlement are obtained. The effect of applying "partial factors" or "reduction factors" to the expected available sideshear resistance and rock mass modulus upon the probability of exceeding the design settlement is then demonstrated.

Chapter 6 draws together the work described in the earlier chapters and describes a limit state design procedure for complete rock socketed piles (i.e., piles which derive their load carrying capacity from both sideshear and endbearing). This procedure takes account of the possibility of slip along the shaft of the socketed pile. The design procedure is illustrated by typical calculations. Comparisons between designs based on this technique and observed field behaviour are then discussed.

Chapter 7 provides a more detailed discussion of some specific or unusual aspects of three well documented series of instrumented field tests. Finally, Chapter 8 summarizes the findings from this study and Chapter 9 presents a series of recommendations based on these findings.

This report presents guidelines for the design of piles socketed into rock. Since no guidelines can cover all possible field situations, these guidelines should only be used by qualified geotechnical engineers who can assess the suitability of the information provided in this report for any specific application.

CHAPTER 2

LITERATURE REVIEW

2.1 THEORETICAL ANALYSIS OF SOCKETED PILE BEHAVIOUR

In principle, the behaviour of axially loaded socketed piles is amenable to analysis using both finite element (eg. Zienkiewicz, 1977) and boundary element (eg. Poulos and Davis, 1980) techniques. However, the simplicity of boundary element solutions may be lost when considering the behaviour of piles socketed into soft rock where radial compatibility, slip at the pile-rock interface, failure in the rock and rock non-homogeneity become important factors governing the pile response. As a consequence, the majority of theoretical solutions for piles socketed into rock have been developed using axisymmetric finite element analyses.

Coates and Yu (1970) and Osterberg and Gill (1973) were among the first to examine the elastic load transfer behaviour of a pile socketed into strong rock. These analyses assumed full bonding between the pile and rock (i.e., no slip) along the socket interface. Their results for complete sockets were presented in a series of charts. These charts show the distribution of load along the length of the embedded socket. Practical application of their analyses is limited by the small range of socket geometries (i.e. length to diameter ratio) and pile-rock stiffness ratios (E_p/E_r) considered.

Pells and Turner (1979) extended this work, again assuming an elastic fully bonded interface, to cover a more representative range of socket dimensions and stiffnesses. They provided solutions for the average settlement response (at the top of the socket) for sideshear-only and complete sockets embedded in a homogeneous rock mass. Additional elastic solutions for socketed piles were presented by Donald et al. (1980). While their design charts for a full socket in a homogeneous rock mass were similar to Pells and Turner's (1979), they also provided solutions where the modulus of the rock mass above and below the base of the socket is different.

At very low load levels, the use of elastic design charts for estimating the load distribution and settlement of a socketed pile may be expected to provide reasonable results. However, these same solutions suggest that high shear stresses may be expected at the pile-rock interface. Once these shear stresses reach the shear strength of the interface, slip will occur.

A technique for readily analyzing socketed pile behaviour upto and including the time at which full slip occurs along the pile-rock interface was developed by Rowe and Pells (1980). This technique is described in Appendix B and will be the method used for analyzing socketed pile behaviour in this report.

2.2 EXPERIMENTAL INVESTIGATIONS OF ROCK SOCKETED PILE BEHAVIOUR

Load tests on socketed piles are generally performed:

(i) to determine design parameters,

(ii) as a proof load test to check whether specific design criteria are satisfied, or

(iii) as a research oriented test conducted to investigate various aspects of socket behaviour.

From a review of the literature, it appears that most published results fall into categories (i) and (iii), although some studies represent a combination of both.

A recent survey of Canadian consultants (see Appendix A) indicated that the majority of socketed pile load tests which fall into category (ii), often remain in the files of the consultants. Sideshear and end-bearing values deduced from proof load tests on piles which perform satisfactorily are, by their nature, lower bound estimates of the available strength. Experience developed in this manner appears to be the basis of much socketed pile design.

A large number of published test results for socketed piles belonging to category (i) were performed specifically to determine the sideshear resistance developed at the pile-rock interface (eg. Spencer, 1944; Moore, 1964; Matich and Kozicki, 1967; Seychuk, 1970; Gibson and Devenny, 1973; Rosenberg and Journeaux, 1976; Nemec, 1979; Spanovich and Garvin, 1979; Bertok and Berezowski, 1983 etc.). As shown in Table 2.1, the mobilized sideshear resistance is often found to be much greater than the design values originally presumed.

Field testing of instrumented socketed piles has been carried out (Jackson et al., 1974; Buttling, 1976; Aurora and Reese, 1977; Gupton et al., 1982; Glos and Briggs, 1983; Horvath et al., 1983 etc.) to examine the load transfer experienced by complete sockets. A number of field studies have shown reasonable agreement between theoretical (elastic) solutions and the observed load transfer behaviour (eg. Jackson et al., 1974; Bauer, 1980; Koutsoftas, 1981; Bushell and Baker, 1982 etc.). Many of the published case histories have reported difficulties in the performance of instrumentation. Often, the problems arose from damage to equipment during construction, malfunction of equipment during the test and uncertainties regarding interpretation of data.

The influence of socket roughness on interface sideshear resistance has been investigated by Pells et al. (1980), Williams (1980), and Horvath et al. (1983). Results indicate that there is little or no decrease in the average peak sideshear resistance, provided "adequate" sidewall roughness is ensured. However, in situations where the socket sidewall is relatively smooth, a "brittle" failure at the pile-rock interface has been observed (Pells et al., 1980), resulting in a lower post-peak (residual) sideshear resistance. The results of Horvath et al.'s (1983) field tests suggest that an increase in the average sideshear resistance may be expected as the interface roughness is increased.

A reduction in the measured peak average sideshear resistance has been observed in cases where the interface has been contaminated during construction, (eg. bentonite, rock or seam smear) (Buttling, 1976; Pells et al., 1980).

To distinguish between the behaviour associated with "smooth" and "rough" sockets, several recent attempts have been made to categorize the degree of socket roughness. Pells et al. (1980) have suggested four roughness categories, each of which depends on the depth, width and spacing of grooves or indentations of the excavated socket. Similarly, Horvath et al. (1983) have developed a mechanical device which traces the

profile of the socket sidewall prior to concreting. Using the data obtained from the profile, the authors proposed a simple equation which yields a numerical value for the socket roughness (i.e., "Roughness Factor"). From a practical standpoint, it seems that the classification given by Pells et al. (1980) provides an experienced inspector with a more rapid method of assessing socket roughness in field situations.

The concept of using tracings of the socket profile to estimate the load-displacement behaviour of sideshear-only sockets has been presented by Chiu and Dight (1983). Using a numerical method described by Dight and Chiu (1981), a comparison between existing load-settlement curves (see William, 1980) appears promising; however, no attempts have been made to use the analysis to predict the response of a socketed pile.

In terms of current design and construction practice, the majority of consultants surveyed (Appendix A), indicated they do not require that the sockets be intentionally roughened during socket excavation. Perhaps doubts concerning the economic advantages (Hartley, 1981) of such an operation may exist, although Horvath et al. (1983) note that this procedure is common for socketed piles constructed in Denver, Colorado.

On the basis of field load tests in sandstone, Pells et al. (1980) indicated that the influence of socket geometry on the interface shear resistance may not be overly significant. However, Webb and Davies (1980) suggest a trend of increasing shear resistance for length/diameter ratios less than about 3. At present, no other reported field case histories tend to support either hypothesis, although small scale model tests in sandstone (Pells et al., 1980) also indicated this influence to be minor.

Stroud (1977) and Tomlinson (1977) have expressed concern whether any differences may exist between the sideshear resistance obtained from uplift (tension) sockets and values determined from compression tests. The previous lack of sufficient field results is the probable reason why no attempts have been made to address this matter.

In making a comparison between the sideshear resistances reported in the literature, it should be noted that the results do not necessarily represent an upper limit, since many of the load tests were not carried to failure.

Results of laboratory tests on small scale sockets have been given by Kenney (1977), Horvath and Kenney (1979), Pells et al. (1980) and Ladanyi and Domingue (1980). A number of conclusions based on these investigations can be briefly summarized as follows:

i) socket diameter and length/diameter ratio have little influence on shear displacement behaviour.

ii) roughened sockets tend to increase the mobilized sideshear resistance.

iii) a layer of contaminated material at the interface tends to inhibit the development of sideshear resistance, particularly for smooth sockets.

iv) an optimum socket roughness may exist, where the mobilized sidewall friction is a maximum (this assumes a "post-failure" condition, where all adhesion is lost).

v) difficulties in providing model sockets with the lateral confinement existing in the field may lead to an inaccurate assessment of in-situ socket behaviour.

Several of the above points have been confirmed by field investigations on full size sockets. However, it appears unlikely that many practical aspects of socketed pile behaviour can readily be assessed by tests on small scale laboratory models.

2.3 INVESTIGATION OF ENDBEARING CAPACITY OF ROCK

Design values for endbearing capacity on rock have generally been selected on the basis of:

i) building code requirements or

ii) consultant/company experience

The wide range of recommended, allowable, endbearing pressures on rock which are found in various codes of practice (see D'Appolonia et al., 1975; Peck et al., 1974) and those currently used in practical designs (see Appendix A), suggests that the factors governing their selection are neither well known nor thoroughly understood.

2.3.1 Bearing Capacity Theories for Rock

Comprehensive reviews of the literature pertaining to bearing capacity theories applicable to rock have been presented by Couetdic and Barron (1975), Pells and Turner (1980) and Williams (1980) and the reader is referred to these articles for details.

A number of these authors have suggested that there are three possible categories into which bearing capacity theories for rock fall. They are: (1) failure of rock is assumed to be plastic and follows a Mohr-Coulomb failure criterion, (2) a failure mechanism which allows for brittle behaviour (and the transition from elastic state to a brittle or plastic phase), (3) prediction of the point of incipient failure, after which total collapse occurs. A summary of the equations associated with these bearing capacity theories can be found in the above noted papers.

Pells and Turner (1980) question why bearing capacity theory used for soils should not necessarily be the same for rock. The two reasons they suggest are, (1) most rocks are brittle whereas soils tend to behave plastically, and (2) the in-situ behaviour of rock is a function of rock substance and discontinuity properties whereas the latter are usually absent in soils. In particular, they regard modified formulas for rock (see Coates, 1981; Coates and Gyenge, 1966; Rehnman and Broms, 1971; Ladanyi and Roy, 1971 etc.), as incorrect solutions for the bearing capacity of an ideal Mohr-Coulomb material.

The bearing capacity theories currently proposed for rock implicitly assume that the rock mass is intact. There has been some attention given to the development of bearing capacity theories for jointed rock (eg. Meyerhof, 1953; Ladanyi and Roy, 1971; Davis, 1980); however, there is little published field evidence to support their validity.

2.3.2 Field and Laboratory Investigations of Bearing Capacity

The results of laboratory and field investigation on small scale steel plates/indentors bearing on or embedded into relatively intact rock

have been reported in the literature (eg. Ladanyi and Roy, 1971; Rehnman and Broms, 1971; Wagner and Schumann, 1971; Couetdic and Barron, 1975; Pells and Turner, 1980). Also, tests which considered the bearing capacity on jointed rock have been given by Brown and Trollope (1971), Burman and Hammett (1975), Pells and Turner (1980). A number of their conclusions and observations can be summarized as follows:

i) in some cases (eg. Rehnman and Broms, 1971; Pells and Turner, 1980) the observed bearing capacity of model tests compared favourably to the Terzaghi (1943) bearing capacity theory.

ii) other cases indicated behaviour similar to that given by Ladanyi's (1966) expanding sphere theory (eg. Couetdic and Barron, 1975; Pells and Turner, 1980).

iii) increased bearing capacity was observed for model tests for an increasing embedment into rock.

iv) ultimate bearing capacity for sandstone and carbonatite rocks are mobilized at very large displacements (eg. 20% of footing diameter) (Pells and Turner, 1980; Ladanyi and Roy, 1971).

v) allowable bearing capacity of jointed rocks should not exceed the tensile strength of the rock elements (Burman and Hammett, 1975).

vi) tight joints do not appear to cause a major reduction in the ultimate bearing capacity (Pells and Turner, 1980).

An examination of a large number of endbearing-only footing/socket field tests on intact Melbourne mudstone have been reported by Williams (1980). These tests considered a wide range of diameters and embedment depths. Williams found that surface piles tended to produce a failure mechanism similar to that postulated by Prandtl. The observed failure mechanism for the embedded sockets consisted of a conical plug of rock being formed beneath the base of the socket as failure progressed. This behaviour was associated with a work strengthening load-settlement response. On the basis of his tests, Williams concluded that; (1) the brittleness of Melbourne mudstone may preclude the application of plasticity analyses to the bearing capacity of even weak rocks, (2) observed failure mechanism was not compatible with Ladanyi's expanding sphere theory, (3) fair agreement between test results (and others) and the incipient (initial yield) failure theory was observed.

Unfortunately, the results of the above investigations highlight the uncertainties regarding the selection of an appropriate bearing capacity theory for rock.

In Chapter 6, further attention will be given to this issue, along with recommendations concerning the selection of appropriate values which can be used in socketed pile design.

2.4 CORRELATION OF DATA FROM FIELD INVESTIGATIONS

Establishing correlations between results obtained from a specific index test with design parameters backfigured from field load tests has long been of interest to geotechnical engineers. The value of these correlations lies in the presumption that the correlation developed at one site may be used to infer design parameters from the results of index tests for subsequent design projects.

2.4.1 Sideshear Resistance

In the literature, it is quite common to find a correlation between

the skin resistance (or adhesion) exhibited by piles in clay to the undrained shear strength of the surrounding clay soil (eg. Vesic, 1969). Consequently, design charts have been produced (Tomlinson, 1971) which relate the undrained shear strength of the soil to the average available sideshear resistance. It seems reasonable to investigate whether a similar approach can be developed for socketed piles in rock.

An early attempt to establish a relationship between the unconfined compressive strength of rock and the socket sideshear resistance measured in the field, was given by Rosenberg and Journeaux (1976). This preliminary correlation was rather vague because of the limited data available at the time. Horvath (1978) conducted a comprehensive search for load test results on socketed piles and consequently, was able to produce a much clearer correlation. Similarly, Horvath and Kenney (1979) produced separate correlations for small (dia < 400 mm) and large (dia $\geq$ 400 mm) diameter sockets. Based on best-fit curves, they proposed a simple equation relating the unconfined compressive strength of the weaker material (concrete or rock) and the expected sideshear resistance of the socketed pile. However, the authors indicated that some data used in defining their correlation required a certain degree of interpretation (eg. estimates of q_u). Similar correlations were developed specifically for load tests conducted at a particular site (eg. Pells et al., 1980; Williams, 1980). This information was subsequently included in a correlation given by Williams and Pells (1981).

In addition to providing an updated correlation for conventionally constructed, large diameter sockets, Horvath et al. (1983) have suggested a relationship for the normalized shaft resistance (α) and their proposed

"Roughness Factor" (see section 2.3). This correlation should be regarded as tentative, since it is only based on limited field and laboratory test results. A parallel relationship, using an empirical correlation of field data, has been developed for the design of grouted rock anchors (Ballivy and Dupuis, 1980; Ballivy et al., 1981).

The data used to establish these correlations have been obtained from numerous sources and have involved various testing techniques, socket geometries and rock types. Preliminary examination of published socketed pile case histories have indicated that this correlation may be influenced by the socket's diameter and roughness. However, the possibility of other factors influencing this correlation have not been investigated in the literature.

2.4.2 Rock Mass Modulus

A number of laboratory and field methods are available to determine the modulus of rock. Unconfined compression tests can be used to define the intact rock modulus, while plate load and pressuremeter tests etc. have been used to estimate the mass or in-situ modulus.

Deere (1968) has presented a series of charts correlating the uniaxial compressive strength with the tangent modulus for a variety of intact rock groups. A similar relationship has been developed by Hobbs (1975) specifically for chalk and other Triassic rocks.

Coon and Merritt (1970) have attempted to define a relationship between the rock quality designation (RQD) and the modulus reduction factor (E_{mass}/E_{intact}). However, Bieniawski (1978) noted the

difficulties in estimating the rock deformability for RQD less than 70% from their correlation. Consequently, Bieniawski (1978) proposed that a correlation between the in-situ modulus of deformation and the geomechanics rock mass rating (RMR) (Bieniawski, 1976) appeared to be more reliable.

Using a numerical analysis, Kulhawy (1978) suggested a correlation between discontinuity spacing and the modulus reduction factor. A similar chart based on limited field data has been suggested by Williams (1980). Using data from a comprehensive socketed pile test program, Williams (1980) has also produced tentative correlations between (1) the sideshear resistance and modulus reduction factor, and (2) backfigured modulus from pile load tests and in-situ pressuremeter modulus values.

Several observations concerning the above correlations should be noted:

(1) correlations using strictly the laboratory defined modulus do not appear to be reasonable for design purposes.

(2) the detailed in-situ rock data required to use the rock mass rating proposed by Bieniawski (1976), is not generally available from literature concerning socketed pile load tests. Also, in practical socketed pile design cases, the modulus of rock is rarely determined (Appendix A).

(3) Verification of the proposed correlations to practical design situations are difficult to find in the literature.

2.5 CURRENT DESIGN METHODS

Current design procedures used for socketed piles in rock appear to vary considerably among consultants. In a recent survey (Appendix A), most designers stated that they relied on personal or company experience as a guide for subsequent socket designs. Simplified procedures such as those given by the Canadian Foundation Engineering Manual (1978) or the National Building Code of Canada (1980) were rarely used as the sole basis for design.

More rational design approaches have been published by Rosenberg and Journeaux (1976), Ladanyi (1977), Pells and Turner (1979), Williams (1980), Pells and Rowe (1981). There are several examples where these methods have been used (eg. Sales, 1983; Stocker, 1981), but in most cases, designers have not adopted them in practice. The lack of acceptance may be partially due to the hesitation of consultants to depart from their current design strategies which have given satisfactory (if possibly conservative) results.

Many of the consultants questioned indicated that bearing capacity and construction constraints control their designs. As noted in section 2.3, procedures for determining the bearing capacity of socketed piles are very crude and generally quite conservative, as evidenced by the fact that bearing capacity failures are rare even in tests intended to determine bearing capacity. Because of the difficulty in causing collapse, "bearing capacity" failure is often associated with the displacement attaining a limiting value. Consequently, piles designed on the basis of this "bearing capacity" are really being designed on the basis of a

de facto limited settlement criterion. (A similar situation exists when considering the bearing capacity of cohesionless sands, eg. see Peck, 1976).

Although most designers recognize that socketed piles generally carry load both in sideshear and endbearing, there is considerable uncertainty regarding the contribution of the two components to the total load-carrying capacity even though information concerning the load transfer behaviour exhibited by socketed piles is available from a number of sources (see sections 2.1 and 2.2).

Design parameters for the allowable bearing pressures on rock can be determined from building codes or correlations (eg. Peck et al., 1974) with rock quality; but appear to be primarily selected on the basis of the designers' own experience. Again, values for allowable sideshear resistance are the result of the individual's experience, however parameters given by the proposed empirical correlations (eg. Horvath and Kenney, 1979; Williams and Pells, 1981 etc.) were frequently used.

The survey of Canadian consultants revealed that in many cases different allowable design parameters were used for sockets which were constructed in the same rock formations. While this discrepancy may be partly explained by differing interpretations of in-situ conditions, it also appears to be due largely to different perceived "factors of safety" (varying from 2 to 6) which have been incorporated into the design parameters.

2.6 PROBABILISTIC METHODS RELEVANT TO SOCKETED PILE DESIGN

Currently, most socketed pile designs are based on design parameters that include a "Factor of Safety", which is intended to take account of uncertainty concerning the actual parameters. While designers recognize the importance of addressing such uncertainties, there is little indication of the level of reliability they hope to achieve when selecting a specific "factor of safety".

Meyerhof (1970) states that the main factors influencing the margin of safety for earthworks and foundation engineering are the inherent variabilities in the applied loads and soil resistance. Meyerhof (1982, 1984) also suggests that a more uniform margin of safety could be achieved by using partial safety factors. These partial safety factors refer to load factors (generally greater than unity) and resistance factors (less than unity) which in turn correspond to an overall factor of safety.

A number of authors (eg. Kay, 1976; Baecher and Rackwitz, 1982; Olson and Dennis, 1983; Jaeger and Bakht, 1983) have carried out studies using statistical and probabilistic approaches to examine the relationship between reliability and factors of safety for the axial capacity of piles in soils. Formulation of such a relationship generally requires that a sufficient amount of field load test data be available. Until recently, the lack of adequate field information from socketed pile load tests has limited the use of probabilistic methods for conducting similar analyses.

While it may be argued that capacity is the main criterion for piles founded in soils, as was mentioned earlier (section 2.5), a settlement criterion may be appropriate for design of foundations in rock.

An example where a probabilistic approach has been used to examine the settlement of a surface footing on a jointed rock mass has been given by Madhav and Krishna (1980). The model chosen for their investigation has been previously described by Kulhawy (1978). The authors illustrated the variation between the probability of failure (defined as exceeding an allowable settlement) with a number of rock/joint parameters. However, their analysis assumes that mean values of rock modulus, spacing and normal stiffness of the discontinuities can be adequately determined. The expense associated with such a requirement may prove overly restrictive for many practical applications.

It has been noted that variability of soil/rock properties is one of the main factors influencing the margin of safety. Investigations concerning statistical variations of soils have been well discussed in the literature (eg. Lumb, 1966, 1970, 1971; Lee et al., 1983; Bowles and Ko, 1984 etc.), whereas, papers involving rock properties are more difficult to find (eg. McMahon and McMahon, 1980; Weir-Jones and Lauga, 1973). In the absence of adequate geotechnical data, several authors (eg. Singh, 1971; Pariseau, 1973) have used a "Monte Carlo" technique (see Hammersley and Handscomb, 1964; Sobol, 1974 etc.) to investigate the variability of soil/rock properties. From their analysis, they were able to select design parameters as well as have an indication of the reliability associated with a particular factor of safety. This approach might also be expected to be useful for piles socketed into rock.

2.7 SUMMARY

(1) Design charts for piles socketed into rock have been given in the literature. While these theoretical solutions are useful for examining the elastic load transfer and settlement behaviour, further extensions which consider interface slip may be of significant practical use.

(2) The increasing number of experimental investigations dealing with socketed piles have largely involved estimating the mobilized sideshear resistance using field and laboratory (model) load tests. Other aspects such as load transfer, socket geometry and socket roughness have also been investigated. Little attention has been given to a comprehensive comparison and analysis of published results.

(3) Results from either model or field tests have failed to confirm the validity of any one of the three bearing capacity theories currently proposed for rock. Such a confused situation may partly explain why consultants currently use an allowable endbearing value based on personal/company experience for their socket designs. It appears that a more rational approach may be developed specifically for design of rock socketed piles.

(4) A correlation between measured sideshear resistance from socket load tests and a common index test (unconfined compressive strength of rock) has been proposed by a number of authors. These correlations may prove useful for estimating preliminary design parameters.

(5) Procedures currently used in practice to design rock socketed piles are largely based on the designer's/company's previous experience. Although more rational approaches using elastic theory have been proposed, it is thought that consultants may be reluctant to adopt this alternative over design methods which have proven satisfactory in the past. It also appears that the current practice is to use allowable endbearing pressures to implicitly satisfy a limited settlement criteria. However, an explicit settlement based design may be more appropriate.

(6) The use of probabilistic methods has been proposed to assess the level of reliability associated with "factors of safety" for piles in soils. Until recently, the lack of sufficient field data has hindered the development of similar approaches relating to piles socketed into rock. The use of statistical techniques to estimate engineering design parameters has been used for a number of situations, although not specifically for socketed pile design.

TABLE 2.1 DESIGN AND OBSERVED VALUES OF THE AVERAGE INTERFACE SHEAR RESISTANCE FROM VARIOUS ROCK SOCKETED PILE CASE HISTORIES

Case	Rock Type	Design Value (MPa)	Observed Value (MPa)	Comments
Moore (1964)	fractured & weathered shale & sandstone	.24	>.96	- not tested to failure
Jackson et al. (1974)	Limestone	.86	>1.63	- not tested to failure
Koutsoftas (1981)	Mica Schist	.48	>1.51	- not tested to failure
Glos and Briggs (1983)	Interbedded Sandstone and Shale	.24	2.15	- approaching maximum
Bertok and Berezowski (1983)	Siltstone	.35	>.31	- close to failure

CHAPTER 3

THEORETICAL CONSIDERATIONS FOR PILES SOCKETED INTO ROCK

3.1 GENERAL

This chapter describes a theoretical examination of a number of factors affecting the behaviour of socketed piles. First, consideration is given to the effect of a number of fundamental parameters upon the average peak and residual sideshear values mobilized along the pile. Secondly, a series of solutions are developed for estimating the settlement of a pile socketed into a homogeneous layer when the pile bears on a stiffer or weaker layer. Thirdly, the effect of weaker horizontal seams upon pile response will be considered. Finally, the effects of recessing a socketed pile below the rock surface and its influence on the pile response will be examined.

3.2 FINITE ELEMENT ANALYSIS

3.2.1 Theoretical Development

The rock-pile interaction analyses described in this chapter were performed using the general method of analysing rock (soil)-structure interaction problems described by Rowe et al. (1978). This method of analysis makes provision for plastic failure within the rock as well as for slip at a cohesive-frictional dilatant rock-pile interface. Details regarding the method of analysis are given in Appendix B.

3.2.2 Principal Assumptions and Problem Idealization

This chapter is concerned with the prediction of the final equilibrium settlements (neglecting creep) and all parameters are drained parameters. It is assumed that the pile is an isotropic, homogeneous elastic solid with a Young's modulus E_p and Poisson's ratio $\nu = 0.15$. The rock is considered to be isotropic and layered. The moduli of the rock adjacent to and beneath the socketed pile are E_r and E_b respectively. In situations where the socketed pile is recessed below the surface of the rock mass (discussed in section 3.6), the modulus is given by E_t. The modulus of soft horizontal seams (discussed in section 3.5) encountered in the rock mass is E_s. The Poisson's ratio for all rock material is 0.3.

It will be assumed that slip may occur at the interface between the pile and the rock and, in some cases, strain softening and/or dilatancy of this interface material may occur. Provision is also made for plastic failure (governed by the strength parameters: peak cohesion - c_p and peak friction angle - ϕ_p) within the rock mass, but strain softening or brittle failure within the rock mass are not considered.

The results reported in this chapter were obtained using an elasto-plastic, axisymmetric finite element program (ROSOC) developed at The University of Western Ontario. A typical finite element mesh using eight noded isoparametric elements is shown in Figure 3.1. The inset to Figure 3.1 illustrates the finite number of dual nodes which are located along the pile-rock interface. All finite element formulations involve discretizing the continuum into independent elements while enforcing

compatibility and equilibrium at the nodal points. Thus, slip (or rupture) must be initiated at discrete nodal points and so the details of the response will vary slightly with the location of the dual nodes. Consequently, where there is an otherwise plastic response, a slight drop off in load with the rupture of the last dual node pair may be a numerical consequence of discretization rather than real.

3.2.3 Numerical Checks of the Finite Element Analysis

To check the suitability of the finite element mesh, a comparison was made with other solutions. The two approaches used to provide comparisons are:

(1) analyses using several different finite element mesh arrangements where the degree of mesh refinement was varied.

(2) using published numerical solutions (eg. Mattes and Poulos, 1969; Pells and Turner, 1979; Donald et al., 1980).

On the basis of these checks, it is considered that the basic finite element mesh is adequate and that the solutions presented in this chapter (and Appendix C) are accurate to within 5-10%.

3.3 FACTORS INFLUENCING SIDESHEAR RESISTANCE

3.3.1 Introduction

Most field load tests provide information regarding the average sideshear resistance developed along the pile-rock interface (eg. see Figure 3.2), but do not provide much insight regarding the factors which govern the magnitude of this sideshear resistance. Consequently, in this section, theoretical consideration will be given to the effect of a number of fundamental parameters upon the development of average sideshear. Although these parameters are usually not available for the design of actual sockets, this fundamental analysis provides valuable guidance in the interpretation and application of field test data to the design of socketed piles.

3.3.2 Methods Used to Evaluate Sideshear Resistance

Field tests to determine average sideshear resistance take a number of forms in terms of both the socket arrangement and the method of loading. Frequently, endbearing resistance is eliminated by casting the concrete socket above the base of the drilled hole. Thus, the average sideshear resistance is simply the applied load divided by the surface area of the concrete socket and this socket is referred to herein as a "sideshear socket". Alternatively, endbearing is permitted and instrumentation is provided so that the proportions of load carried in endbearing and sideshear can be determined. Loading conditions may range from a displacement defined test in which only sufficient load is applied to ensure a given displacement (i.e., the total load may increase or decrease) and a load defined test in which the applied load is increased, or held constant, while the displacement and stress distribution within the pile are monitored.

A displacement defined test will commonly be used in conjunction with a test on sockets where endbearing has been eliminated, since this then allows convenient determination of both the peak and residual

sideshear values. A load defined test is frequently used in conjunction with endbearing sockets and here the peak and residual sideshear values may be deduced from the distribution of load (rather than the total applied load).

Displacement defined tests on sideshear sockets tend to be cheaper and easier to perform (since lower loads are required) than load defined tests on complete sockets. However, for many (but not all) practical situations, the latter test more truly represents the real situations and so it is important to note that there are fundamental differences between the nature of the two approaches and that this may significantly affect the deduced sideshear values, particularly for relatively smooth sockets. For a given mobilized average sideshear, the applied load on a complete socket will be greater (and hence Poisson's effects will be greater) than for a sideshear socket. Furthermore, the boundary conditions of an unloaded region beneath the socket as compared with an applied endbearing stress will alter the stress distribution along the pile. Thus, even with the same loading method, some differences would be expected between sideshear and complete socketed piles. The difference between load techniques commonly adopted in these cases will accentuate the difference in observed behaviour, especially for sockets exhibiting strain softening. In the displacement defined test on a relatively smooth isolated socket, the applied load will decrease as softening occurs at the pile-rock interface (i.e., as the cement bond is broken). The reduction in applied load reduces the normal stress along the interface which, in turn, also reduces the frictional component of the shear resistance, resulting in a very brittle response. In a load defined analysis on a relatively smooth endbearing socket, the applied load remains constant or increases as softening occurs at the pile-rock interface, with the additional load being carried in endbearing. This maintenance or increase in applied load maintains, or increases, the normal stress along the interface thereby increasing the frictional component of the available shear resistance and reducing the brittleness of the response. The greatest discrepancy between the results from different tests will occur for very smooth sockets. Increasing roughness of the socket (reducing brittleness) will reduce this discrepancy.

A theoretical study of the effects of a number of fundamental parameters upon the response of sideshear sockets subjected to a displacement defined loading has been reported by Rowe and Pells (1980). In subsequent sections, attention will be directed to the behaviour of complete socketed piles in a load defined test.

3.3.3 Parametric Study of Sideshear Resistance

The peak and residual average sideshear resistance of a socketed pile depends upon the combined influence of the pile modulus (E_p) to rock modulus (E_r); the length (L) to diameter (D) of the socket; initial stress; the rock and interface strength properties; and the interface roughness. To illustrate the relative effects of these parameters, a comparison will be made using experimental data obtained from two case histories.

3.3.3.1 Model Description and Explanation of Terms

If it is assumed that the pile-rock interface behaviour is governed by a Mohr-Coulomb failure criterion, then slip will occur when the mobilized shear stress τ reaches the strength of the material i.e.,

$$\tau = c_p + \sigma_n \tan\phi_p \qquad (3.1)$$

where c_p is the peak interface "adhesion"

σ_n is the normal stress at the pile-rock interface

ϕ_p is the peak angle of friction of the pile-rock interface

A simple model of the interface behaviour would involve the degradation of the pile-rock interface parameters after the peak stress has been reached, thus

$$\tau = c + \sigma_n \tan\phi \qquad (3.2)$$

where c and ϕ are a post-peak interface adhesion and friction angle respectively. In general, the degradation from peak to residual strength at a point will occur with increasing relative displacement between the two sides of the rupture. For convenience, it is assumed here that c and $\tan\phi$ degrade linearly with relative displacement and that the residual parameters (c_r, ϕ_r) are attained after a relative displacement δ_r.

If an interface has a dilatancy angle ψ (Davis, 1968) at a point when slip occurs, then increments in the normal and tangential displacement may be related by this dilatancy angle. Experimental evidence (eg. Williams, 1980; Pells and Rowe, 1981) indicates that the pile-rock interface will dilate until a limiting normal displacement (which depends on rock stiffness) is reached, and will then deform at constant volume. To sufficient accuracy, the experimentally observed behaviour may be idealized as a dilation with a constant dilatancy angle ψ until the limiting normal displacement (dilation) δ_d is attained after which the dilatancy angle at that point is reduced to zero (ψ = 0 implies constant volume deformation).

For further details regarding the application of this approach to the numerical analysis, the reader is referred to Appendix B.

3.3.3.2 Effects of Increasing Interface Roughness

Consider firstly, a concrete pile socketed into a homogeneous rock mass with an unconfined compressive strength of 16.8 MPa. This rock bears some similarity to the Hawkesbury sandstone examined by Pells et al (1980) and suggests interface strength parameters, c_p = 4 MPa and ϕ_p = 39°.

Initially consider a perfectly smooth socket which does not dilate at failure and has residual interface parameters δ_r = 0.003D, ϕ_r = 36° and c_r between 0 and 2 MPa (c_r is a cohesion intercept resulting from the fact that the failure envelope is curved at low stress levels). The variation in average shear with length to diameter ratio (L/D) and modulus ratio is shown in Figures 3.3a and 3.4a for c_r = 0 and 2 MPa respectively. The response of these piles is quite brittle and the residual sideshear values are considerably lower than the peak values. The ratio of peak shear to unconfined compressive strength typically lies

between 0.1 and 0.25 and although this ratio does vary with c_r, the general effect of c_r upon peak shear is not large. As might be expected, increasing the residual cohesion intercept, c_r, increases the magnitude of the average residual sideshear by approximately an amount equal to the change in c_r.

The peak and residual sideshear resistance both tend to decrease with increasing length to diameter ratio (L/D) for piles in moderately strong rock (eg. E_p/E_r = 10 or 25). The effect of length to diameter ratio is relatively small for piles in rock with a low mass modulus (E_p/E_r = 100). The variation in the peak shear is the most complex since this is highly dependent upon the shape of the normal and shear stress distribution along the pile during loading. This in turn is a complicated function of geometry and relative stiffness of the pile and rock (E_p/E_r). The residual sideshear resistance decreases monotonically with increasing length to diameter ratio and increasing modulus ratio (E_p/E_r).

Roughness of the socket may be simulated in terms of an initial dilation angle ψ and a maximum dilatancy δ_d as described in the previous section. Assuming initially, that these parameters are independent of length to diameter ratio and rock modulus, and adopting values of ψ = 20° and δ_d = 0.001D, a variation in peak and residual sideshear with L/D may be obtained as shown in Figures 3.3b and 3.4b for c_r = 0 and 2 MPa respectively.

Increasing the socket roughness increases both the peak and residual sideshear values and reduces the brittleness of the response (i.e., $\tau_{residual}/\tau_{peak}$ tends to unity with increasing roughness). The sideshear values tend to decrease with increasing length to diameter ratio, particularly for stronger rock (E_p/E_r = 10 and 25). However, the assumption that roughness is independent of socket length may not be valid for field situations where the probability of fractured or roughened zones occurring along the socket will generally increase with length. Hence, longer sockets may be rougher than short sockets, thereby masking the length to diameter effect predicted for sockets of equivalent roughness.

The parameters selected for this comparison were based on the available experimental data relating to Hawkesbury sandstone, and it is noted that the range of peak shear resistances predicted for both smooth and rough sockets is very similar to that observed in field tests for the two cases (eg. see Figure 3.2 for q_u = 16.8 MPa).

The peak shear resistance is greatly dependent on the progressive failure of the points along the interface. This progression depends on the normal stresses developed across the interface as well as the distribution of shear stress along the socket. For a given maximum dilation, δ_d, the highest normal stresses will be developed near the top of the socket where the modulus of rock is closest to that of concrete (i.e., for lower values of E_p/E_r) and accordingly the effect of roughness on the peak shear resistance will be highest for this case (see Figures 3.3 and 3.4). However, the distribution of shear stress also varies with the length to diameter ratio of the socket. Generally, the more uniform the shear stress distribution, the higher will be the peak shear resistance.

The more uniform shear distributions are usually found for long sockets where the modulus ratio is high (eg. E_p/E_r = 100). The interaction of the effects of normal stress at the interface and shear stress distribution give rise to the higher values of peak shear resistance observed for E_p/E_r = 100 and L/D = 10 than for E_p/E_r = 100 and L/D = 5 as shown in Figure 3.4.

3.3.3.3 Effect of Variations in Rock Mass Stiffness and Interface Roughness

In general, relatively little is known about the relationship between rock mass modulus, roughness and dilatancy at the interface. However, the work by Williams (1980) does provide some guidance regarding the parameter for very weak rock (silurian mudstone). Considering a rock with an unconfined compressive strength of 0.72 MPa, c_p = 0.2 MPa and ϕ_p = 32°, analyses were performed for a range of modulus ratios (E_p/E_r) corresponding to different levels of weathering and jointing in weak rock. The interface strength parameters (c_p = 0.2 MPa, ϕ_p = 32°, c_r = 0, ϕ_r = 21°) were taken to be independent of rock stiffness. The roughness (ψ, δ_r, δ_d) parameters were selected from consideration of the experimental data and varied with rock stiffness as indicated in Figure 3.5. The results from these analyses given in Figure 3.5 indicate a decrease in available shear resistance with L/D. A similar trend was noted previously for the stronger rock (q_u = 16.8 MPa) and the same comment regarding the independence of roughness and length may be made. The ratio of average peak shear to unconfined compressive strength (τ_f/q_u) ranges between 0.3 and 1.05 and is much higher than that obtained for stronger rock. This trend of increasing τ_f/q_u with decreasing rock strength and indeed the range of values predicted bears some similarity with field observations (eg. see Figure 3.2).

In the case of the strong rock where roughness parameters were independent of the rock mass modulus, it was noted that the average residual shear decreased with decreasing rock modulus and that relationship between peak shear and modulus was not as clearly defined. In the case of the weaker rock where (more realistically) the roughness parameters were assumed to be a function of rock stiffness, both the peak and residual shear decrease with decreasing rock mass modulus. This finding is of practical significance since empirical data on shear resistance is often related directly to the unconfined strength of the rock (eg. Figure 3.2). For a rock with a given unconfined compressive strength of typical (intact) samples, the actual mass modulus of the rock may vary appreciably depending upon the degree of jointing. Figure 3.6 illustrates the effect of the mass modulus (relative to an assumed intact modulus) upon the ratio of the average peak sideshear. It is also noted that decreasing rock mass modulus tends to increase the brittleness of the interface behaviour for smooth and slightly roughened sockets.

3.3.3.4 The Influence of High Horizontal Rock Stress

In the foregoing analyses, it was assumed that the initial horizontal stresses at the pile-rock interface were negligible relative to the normal stresses due to Poisson's effect and dilation of the interface. The assumption is likely to be valid for most practical cases, however stress measurements around the world have indicated that high horizontal

stresses may be encountered near the surface of a rock mass (eg. Palmer and Lo, 1976, and others). Under certain circumstances, involving a time-dependent rock mass subjected to high horizontal stresses, one may expect the development, with time, of appreciable horizontal stresses between the pile and the rock after casting of the socket. Analyses performed for this case indicate that, as might be expected, increasing horizontal stress increases the magnitude of the peak and residual mobilized shear and decreases the brittleness of the response of relatively smooth sockets.

3.3.4 <u>Summary</u>

The elasto-plastic analyses reported in this section were performed using rock strength parameters equivalent to the peak interface parameters. Some analyses were also performed for higher friction angles and lower cohesions in the rock mass. During loading upto, and including full slip along the pile, significant plastic failure within the rock mass only occurred near the top of the pile and it was found that the results obtained assuming the rock mass remained elastic were not significantly different from the full elasto-plastic analysis results. Clearly, plastic failure at the interface governs the behaviour of the piles studied. However, it is noted that plastic failure within the rock mass would be significant if there were very high levels of dilation at the interface.

3.4 SETTLEMENT FACTORS AND LOAD DISTRIBUTION

3.4.1 <u>Assumptions Made in the Analysis</u>

Elastic solutions (eg. Pells and Turner, 1979; Donald et al., 1979) currently used to estimate socket settlement and the proportion of load carried to the base of the socketed pile were developed assuming full bond between the pile and the rock (i.e., no slip). Consideration of the elastic stress distribution would suggest that in practice considerable slip may be anticipated at the interface (although this might often be undetected).

Recognizing the likelihood that some slip will occur at the pile-rock interface, it would seem reasonable to take account of this slip during the design procedure. Accordingly, finite element solutions will be presented which permit the determination of:

- the pile head settlement and load carried to the base under elastic (no slip) conditions;
- the load at which full slip occurs and the corresponding pile head settlement; and
- the pile head displacement for any given load following full slip.

The behaviour of a socketed pile upto and including collapse could be predicted using the approach described in the previous sections. However, in practice, there will generally be insufficient data to justify such a detailed analysis. Thus, for purposes of design, it is convenient to adopt a simplified approach in which it is assumed that:

- all failure occurs at the pile-rock interface;
- the available shear resistance τ_f at the pile-rock interface is constant along the pile length and is taken to be the average sideshear resistance determined either from a fundamental analysis (see section 3.3.3) or, more probably, from field load test results and/or empirical correlations (see Chapter 4), with possible adjustments for the effect of mass modulus as indicated by Figure 3.6;
- the rock adjacent to the pile is homogeneous with a mass modulus E_r and $\nu = 0.3$ (seams will be considered in section 3.5);
- the rock beneath the pile base is homogeneous with a secant mass modulus E_b and $\nu = 0.3$ (nonlinearity of this rock may be approximately considered by use of an appropriate secant modulus).

3.4.2 Results of the Finite Element Analysis

The results in this section were obtained for a range of pile to rock modulus (E_p/E_r) values and base to side rock modulus (E_b/E_r) values. For specific values of E_p/E_r, E_b/E_r and L/D, the results are presented in terms of the dimensionless displacement, $I = \rho E_r D/P_t$ corresponding to a given ratio of load P_b/P_t being carried to the base of the socketed pile where

P_t is the applied load on the socketed pile at the rock surface;

P_b is the load carried to the base of the socketed pile;

D is the socketed pile diameter;

L is the socketed pile length;

ρ is the <u>average</u> displacement (an average of the centreline and edge displacements) at the head of the socketed pile.

The sketch given in Figure 3.7 illustrates the terminology used in this section. (The terminology for socketed piles recessed beneath the rock surface will be addressed in section 3.6.)

Typical solutions for $E_b/E_r = 1.0$ and E_p/E_r of 1 and 100 are presented in Figures 3.8 and 3.9 (a complete set of design charts is given in Appendix C).

The elastic solution shown as the lower dotted contour on Figures 3.8 and 3.9 represents the limiting case of minimum displacement and minimum load carried to the base of the pile. Increasing slip along the pile results in increased displacements and an increased proportion of load carried to the base of the pile. The upper dotted line corresponding to $\tau_{ave}/\tau_f = 1$ represents the case of full slip. Thus, for known values of E_p/E_r, E_b/E_r and L/D, the ratio of load carried to the base $(P_b/P_t)^*$ and dimensionless displacement $I^* = \rho E_r D/P_t$ at full slip may be taken directly from the design charts. At full slip, the load carried in sideshear, P_s, is known viz.

$$P_s = \pi D L \tau_f \tag{3.3}$$

and since

$$\left(\frac{P_b^*}{P_t}\right) = \frac{P_t - P_s}{P_t} = 1 - P_s/P_t^* \tag{3.4}$$

the load P_t^* required to cause full slip may be calculated. The corresponding average pile load displacement is then given by the expression

$$\rho^* = \frac{P_t^*}{E_r D} I^* \tag{3.5}$$

where P_t^*, E_r, D and I^* are now all known.

<u>After full slip</u>, it is assumed that the load carried in sideshear remains constant at the value given by equation (3.3) and hence the value of P_b/P_t for a given applied load P_t, is

$$P_b/P_t = 1 - \frac{\pi D L \tau_f}{P_t} \tag{3.6}$$

(provided $P_t \geq P_t^*$)

and hence the dimensionless displacement I may be read from the design charts for the specified value of P_b/P_t and L/D and hence

$$\rho = \frac{P_t}{E_r D} I \tag{3.7}$$

Thus the results given in these figures provide a means of determining the load displacement behaviour for a given pile.

In the design of socketed piles, it may be argued that it is necessary to

(a) limit settlements to some specified allowable value ρ_s; and

(b) ensure an adequate factor of safety against endbearing failure.

Provided that the conditions are satisfied, there would appear to be no fundamental reason for not allowing full slip along the socket. (Indeed, as previously noted, slip is likely to occur irrespective of whether or not it is considered in design.) Assuming that the possibility of full slip is acceptable and that the pile and rock properties E_p, E_r, E_b, τ_f and D can be estimated, then the optimum length of pile required to limit deflection to the prescribed value ρ_s for a given load P_t may be conveniently determined from the appropriate design chart as follows:

From equation (3.6)

$$\frac{P_b}{P_t} = 1 - \frac{\pi D L \tau_f}{P_t} = 1 - 4(\frac{L}{D})\frac{\tau_f}{q_t} \tag{3.8}$$

where $q_t = \frac{P_t}{(\pi D^2/4)}$ is the average applied pressure on the pile at the rock surface.

Since the average sideshear τ_f and applied pressure q_t are known, the ratio of base load to top load P_b/P_t is a linear function of L/D. The optimal pile length is that which satisfies equation (3.8) and gives a dimensionless displacement

$$I_s = \frac{\rho_s E_r D}{P_t} \tag{3.9}$$

This may be determined graphically by plotting the straight line given by equation (3.8) on the appropriate design chart and finding the intersection between this line and the contour corresponding to the dimensionless displacement I; this then gives the required pile length. This pile automatically satisfies (a). The value of (P_b/P_t) for this pile is read from the chart and the base pressure q_b, can then be calculated, viz.

$$q_b = \frac{(P_b/P_t)P_t}{(\pi D^2/4)} \tag{3.10}$$

compared with the allowable endbearing pressure. Provided that condition (b) is satisfied, the optimal pile length has been determined.

The effect of increasing the pile length beyond that required to give the allowable settlement ρ_s depends upon the relative stiffness of

the pile and rock. For given applied load and pile and rock parameters, equation (3.8) remains valid for all loads above the full slip load (i.e., above the dotted line $\tau_{ave}/\tau_f = 1$). Thus, for low values of E_p/E_r (eg. Figure 3.8), increasing the pile length will decrease the load carried to the pile base but may in fact increase the pile head settlement if the load is sufficient to cause full slip. (This situation arises because of compression of the pile. It is noted that for rock with a modulus similar to, or greater than, that of the pile, the response of a rock continuum will be stiffer than that of a pile socketed into the rock if there is any slip along the pile-rock interface.) Increasing the pile length for piles with a modulus ratio E_p/E_r significantly greater than unity (eg. Figure 3.9) results in a decrease in both the base load and the pile head settlement.

When considering the results given in Figures 3.8 and 3.9 (and Appendix C), it should be recognized that the equilibrium conditions for full slip are defined by equation (3.8). Thus, for any specified applied load and rock-pile parameters, there is a unique (or no) value of $I = \rho E_r D/P_t$ corresponding to this particular pile geometry and load. Thus, the contours of the influence factor I <u>do not</u> correspond to a variation in displacement for a given load but rather correspond to a variation in both load <u>and</u> displacement where the actual load at any point along the contour must be determined using equation (3.8). These contours reflect both the compressibility of the pile and rock, and the distribution of load along the pile. Consequently, the shape of the contours varies appreciably with modulus ratio E_p/E_r as may be appreciated by comparing the results given in Figures 3.8 and 3.9.

Figures 3.10 and 3.11 show the variability in load distribution (P_b/P_t) as a function of modular ratio (E_p/E_r) and base to side modulus ratio (E_b/E_r), respectively, for a range of influence factors I. The load carried to the base of the pile for a given value of L/D and I increases significantly with increasing pile modulus (E_p/E_r) and increasing base modulus (E_b/E_r). These charts may be used to estimate the value of the dimensionless displacement I (for given P_b/P_t) when the modulus ratios E_p/E_r or E_b/E_r do not correspond directly to the results given by the full set of design charts in Appendix C.

The results presented in this section (and Appendix C) were obtained for the case where there is no strain softening at the interface (i.e., $\tau_f = \tau_{peak} = \tau_{residual}$). The charts may be approximately used for the case where $\tau_{peak}/\tau_{residual}$ is greater than one by taking $\tau_f = \tau_{residual}$ in the full slip range. This approach should be conservative since full slip may not actually occur if the mobilized shear is less than the peak value. (This would generally lead to reduced settlement and a reduced proportion of load carried to the base of the pile.)

3.5 EFFECT OF HORIZONTAL SEAMS ON SOCKETED PILE BEHAVIOUR

3.5.1 Assumptions Made in the Analysis

The behaviour of piles socketed into rock which is tightly jointed may be approximately predicted using the theory of the previous section in conjunction with an appropriate mass modulus. However, if the joints

are filled with weathered material, particular consideration must be given to the effect of these weaker zones upon the pile response.

In this section, the effect of contiguous horizontal seams extending to a distance of 25D will be examined. It is assumed that these seams have a modulus E_s and a shear resistance τ_s at the pile-seam interface. The effect of the seam upon pile head settlement and load distribution might be expected to be a function of the percentage of seams and their distribution along the pile, the relative seam modulus E_s/E_r and relative shear resistance τ_s/τ_r (where τ_r is the average shear resistance of the pile-rock interface). Seams may be evenly distributed along the pile or may be concentrated over some portion of the pile. However, it is generally conservative to restrict attention to the situation where the seam is concentrated just above the base of the pile and the results presented herein are for this case.

3.5.2 Results of the Finite Element Analysis

The proportion of base load (P_b/P_t) and the settlement factor I will both increase due to the presence of soft seams adjacent to the pile. One approximate approach for estimating I and P_b/P_t for this case is to determine an "equivalent" modulus $E_{eq} = S\,E_s + (1-S)E_r$ (where S is the proportion of seam along the socket) and then use the appropriate charts (described in section 3.4) to determine $I = \rho E_r D/P_t$ and P_b/P_t for $E_p/E_r = E_p/E_{eq}$. This procedure will typically underestimate I and P_b/P_t by 10% and 20% respectively for elastic conditions and moderate proportions of seam. However, once slip occurs, it is far more convenient to adopt an alternative procedure wherein the settlement factor I_s for the actual case is related to a fictitious "homogeneous" settlement factor I_h which corresponds to the same load distribution P_b/P_t. This then permits convenient and more accurate use of the solutions presented in section 3.4 for determining socket behaviour in the presence of seams. Figure 3.12a illustrates such a relationship where the displacement factor for a pile in a rock with 10, 20 and 40 percent seam (E_s/E_r and τ_s/τ_r are both equal to 0.1 and 0.25) is related to the displacement factor for a homogeneous mass such that the proportion of the load carried the pile base (P_b/P_t) is the same in both cases.

Provided similar relationships can be obtained for different pile lengths (L/D) and modulus ratio (E_p/E_r), this would allow the adoption of an approach similar to that described in the previous section. In designing for full slip conditions in the presence of seams:

- the pile length must be estimated;
- the equilibrium equation is then given by equation (3.8) where τ_f is an average sideshear resistance over the entire pile length: i.e.,

$$\tau_f = S\ \tau_s + (1-S)\tau_r$$

  and S is the proportion of seam along the pile length;
- the allowable settlement factor I_s is calculated from equation (3.9) where E_r is the rock mass modulus (neglecting seams);
- knowing the relative proportion of seam and its properties (E_s/E_r, τ_s/τ_r) for the assumed pile length (L/D) the target displacement I_s may be converted to a (fictitious) homogeneous displacement factor I_h using Figures 3.12-3.16;

- use the equilibrium equation and the fictitious displacement factor I_h with the design charts given in Appendix C to determine an improved estimate of the pile length required;
- repeat above procedure for improved estimates of L/D if necessary.

Figure 3.12a provides a means of estimating the homogeneous displacement factor I_h for a range of seam properties (E_s/E_r = τ_s/τ_r = 0.25 and 0.1; S = .1, .2 and .4). Values of I_h for intermediate seam properties may be obtained by interpolation between the results given for the required I_s as illustrated for I_s = 0.8 in Figures 3.12b and 3.12c. It is noted that the presence of even a small proportion of seam may have a significant effect upon the pile behaviour and that the difference in effect between 0 and 10% seam is greater than that between 10% and 40%. If more than 40% of the socket length is "seam", it would be more appropriate to use the homogeneous results of section 3.4 in conjunction with a relevant mass modulus rather than consider the effect of seams separately.

An alternative means of representing the relationship between the displacement factor I_s and the fictitious homogeneous factor I_h given in Figure 3.12a, is to plot I_s versus the displacement ratio I_h/I_s as shown in Figures 3.13. For a truly homogeneous rock I_h/I_s = 1, and hence the departure from the value for a given I_s illustrates the effect of seam while the close bracketing of the results for S = .1, .2 and .4 allows convenient interpolation.

The results presented in Figures 3.12 and 3.13b,d assume the same reduction in seam modulus and seam-pile resistance. In fact, for a given load distribution (P_b/P_t), reducing the seam modulus and reducing the seam-pile shear resistance have opposite effects upon the displacement ratio I_h/I_s as illustrated in Figures 3.13a and c. If E_s/E_r remains constant and τ_s is reduced, there is an increase in the displacement ratio ($I_h/I_s > 1$) with increasing proportion of seam. When both E_s/E_r and τ_s/τ_r are reduced by the same proportion there is an interaction between the two differing effects. For L/D = 2, E_p/E_r = 10, the effect of decreasing E_s dominates the effect of τ_s and I_h/I_s is less than unity for all load levels. This is not the case for L/D = 10 and E_p/E_r = 10. Here the effect of reducing the shear resistance (τ_s/τ_r) dominates the behaviour at low load levels while the reduction in modulus (E_s/E_r) dominates the behaviour at high load levels. The interaction between the two effects gives rise to the complicated relationship between the displacement factor I_h/I_s and the proportion of seams, shown in Figures 3.14a,c.

The effect of seams upon pile response is greatest for long piles since these piles normally carry the greatest proportion of the load in sideshear. Thus, the effect of decreasing τ_s is greatest for long piles in relatively strong rock (low E_p/E_r, see Figures 3.14) while the effect of decreasing E_s is greatest for long piles in relatively weak rock (high E_p/E_s, see Figure 3.15). The displacement ratio (I_h/I_s) for piles in relatively weak rock (E_p/E_r = 100), τ_s/τ_r and L/D = 2, 10 is given in Figure 3.16. For piles in weak rock, the effects of seam modulus reduction dominates and I_h/I_s is less than unity for all load levels.

Although results are only given for a limited number of cases, it should be possible to obtain an estimate of the effect of seams for a much wider range of situations by interpolating between the results given in Figures 3.12 to 3.16.

An illustrative example using these charts will be given in Chapter 6.

3.6 BEHAVIOUR OF SOCKETED PILES RECESSED BELOW THE ROCK SURFACE

3.6.1 Introduction

In many practical design situations, it may be necessary to ensure that no load is transferred to the rock above a certain depth (eg. to avoid interaction with the foundations of an adjacent building). This can be achieved by recessing the socket below the surface of the rock mass. Recessing the socket may be expected to reduce the pile settlement and it is of some interest to determine the likely magnitude of this reduction.

A number of authors (eg. Fox, 1948; Janbu et al., 1955; Burland, 1970; Christian and Carrier III, 1978; Pells, 1983, etc.) have considered the settlement response associated with a circular footing recessed below the ground surface. Consequently, design charts giving the settlement reduction factor for recessed footings have been produced. An estimate of the settlement for the recessed footing is obtained by multiplying the appropriate reduction factor by the corresponding settlement of the footing if it were at the surface. However, these solutions are not directly applicable for analyzing the behaviour of a recessed pile since they do not consider the influence of the embedded portion of the socketed pile. In the subsequent sections, attention will be directed towards the following aspects of socketed pile behaviour:

(1) discussion of existing solutions which can be used to estimate the decrease in settlement due to a socketed pile being recessed below the rock surface.

(2) presentation and discussion of finite element solutions which consider the settlement behaviour of recessed full socketed piles.

(3) a re-examination of existing solutions for recessed sideshear sockets.

As illustrated in Figure 3.17, a recessed socketed pile will be taken to consist of two portions; (1) a recessed pile length (L_e) where the concrete pile is essentially a free-standing column (i.e., no bond is developed at the pile-rock interface) and (2) the socketed pile length (L) which is embedded into the rock such that there is full bonding at the pile-rock interface. As a simplification, the unbonded length (L_e), measured from the rock surface to the head of the socketed pile, will be referred to as the "recessed pile". Similarly, the bonded length (L), measured from the head of the socketed pile to the socket base, will be denoted as the "socketed pile".

Depending on the method of construction used, the diameter of the socketed pile (D) may not be the same as the diameter of the recessed pile (D_1).

3.6.2 <u>Existing Solutions for Recessed Socketed Pile Behaviour</u>

Elastic finite element solutions for the settlement response of a recessed, sideshear only pile socketed into a homogeneous rock mass have been given by Pells and Turner (1979). Their design charts allow a rapid estimate to be made of the reduction in settlement for a particular recessed pile geometry (L_e/D), socketed pile geometry (L/D) and modulus ratio (E_p/E_r). Further reference will be made to these solutions in section 3.6.4.

For recessed endbearing-only socketed piles, an estimate of the settlement reduction can be made using finite element solutions for a rigid circular area (Donald et al., 1980). Pells and Turner (1979) suggest that the rigid solution is valid where the modulus ratio (E_p/E_r) is greater than 50. When E_p/E_r is less than 50, the solution for a recessed flexible circular footing (eg. Pells and Turner, 1979; Donald et al., 1980) may be applicable.

Currently, there appears to be no appropriate solutions which consider the reduction of settlement which occurs for a recessed, complete socketed pile.

3.6.3 <u>Settlement Reduction for Recessed Complete Socketed Piles</u>

3.6.3.1 Assumption Made in the Analysis

In the present analysis, the terminology concerning a recessed complete socketed pile will follow that given in section 3.6.1 (eg. see Figure 3.17). The solutions are for an elastic, fully bonded interface along the socketed pile.

Finite element solutions for a range of socketed pile geometries (L/D), recessed pile geometries (L_e/D) and pile-rock modulus ratios (E_p/E_r) are given in the subsequent sections.

Since the recessed pile will experience only elastic compression under the applied load, the displacements will be referenced to the head of the socketed pile (see Figure 3.17). The settlement at this location will be designated the net average displacement. The associated settlement influence factor is given by

$$I_n = I_g - \frac{E_r}{E_p} \frac{L_e D_1}{A} \tag{3.11}$$

where

I_n is the net influence factor for the average displacement at the head of the socketed pile;

I_g is the gross influence factor for the average displacement of the pile at the rock surface;

$\frac{E_r}{E_p} \frac{L_e D_1}{A}$ is the elastic compression of the recessed pile (with length L_e).

As in section 3.4, the average displacement represents the average of the displacements determined at the centreline and edge of the socketed pile.

3.6.3.2 Settlement Influence Factors

Typical results for the net settlement influence factors for recessed socketed piles are shown in Figures 3.18 and 3.19. These figures indicate that the variation in settlement is a function of the recessed pile geometry (L_e/D), socketed pile geometry (L/D) and the pile-rock modulus ratio (E_p/E_r). Both figures illustrate that the largest changes in settlement occur for socketed pile geometries (L/D) less than 4, irrespective of the recessed pile geometry (L_e/D). Alternatively, the reduction in settlement for a particular socketed pile geometry (L/D) is shown in both figures to be influenced by variations in the recessed pile geometry (L_e/D). However, from Figures 3.18 and 3.19, it is clear that as the recessed pile geometry increases, there is progressively less relative change in the settlement (eg. I_n) for a particular socket geometry (L/D).

Additional figures for the net settlement influence factors for recessed socketed piles can be found in Appendix C.

3.6.3.3 Settlement Reduction Factor

In certain situations, it may be more useful in design calculations to use a dimensionless parameter to express the reduction in settlement which occurs for a recessed socketed pile.

The solutions given in the previous section (and Appendix C) will be modified using a settlement reduction factor $(RF)_\rho$ rather than the net influence factor (I_n) for a recessed socketed pile. This reduction factor is defined as,

$$(RF)_\rho = \frac{I_n}{I_{no}} \tag{3.12}$$

where

I_n is the net influence factor for a recessed socketed pile (described in section 3.6.3.2);

I_{no} is the settlement influence factor when the socketed pile is not recessed (I, from design charts described in section 3.4).

Therefore, the settlement reduction factor simply expresses the relative decrease in net average settlement between two similar socketed piles (i.e., with the same L/D ratio), where one is recessed below the surface of the rock, the other is not.

Solutions giving the settlement reduction factors for a complete socketed pile recessed in a homogeneous rock mass are presented in Appendix C. A typical design chart is shown in Figure 3.20a. This figure illustrates the variation in $(RF)_\rho$ with different socketed pile and recessed pile geometries (eg. L/D, L_e/D, respectively) for a particular pile-rock modulus ratio (E_p/E_r). An alternative form of this chart is given in Figure 3.20b. The two different formats allow for rapid interpolation of intermediate values of L_e/D and L/D.

From the finite element analysis, it was found that variations in the modulus ratio tended to affect the value of $(RF)_\rho$ for a particular socket and recessed pile geometry.

Figure 3.21 shows the effects of changes in E_p/E_r on the reduction factor for different socketed pile length/diameter ratios. The

figures suggest that a significant variation in the reduction factor occurs for modulus ratios less than 100. The contours for L_e/D shown in Figure 3.21a (L/D = 0) have been derived using an alternative form of equation (3.11) viz.

$$(RF)_{\rho_{L/D=0}} = \frac{I_n}{I_n(\text{surface})} \qquad (3.13)$$

where

$I_n(\text{surface})$ = .929 is the influence factor for a flexible surface footing (obtained from the finite element analysis);

I_n is the influence factor for an endbearing only pile at a depth L_e/D (see Appendix C, Figure C23).

In most cases, the charts given in Figure 3.21 can be used directly in design calculations for recessed socketed piles.

Often, site conditions may require that a socketed pile be recessed in order to avoid endbearing on poorer quality rock (eg. a weathered zone of rock). This rock will generally possess a modulus which is less than that of the rock surrounding the socketed pile. In order to assess the effects of varying rock quality on the settlement response of the recessed socketed pile, three simple field conditions, illustrated in Figure 3.22, will be considered.

Case A represents the situation where the quality of rock is such that the modulus ratio, E_t/E_r approaches zero. Thus, no reduction in settlement may be expected to occur (eg. $(RF)_\rho = 1$).

Case B represents the field condition where the socketed pile has been recessed in a homogeneous rock mass (eg. $E_t/E_r = 1$). Reduction factors for such cases have been discussed previously (eg. see Figures 3.20, 3.21 and Appendix C).

Finally, Case C represents the intermediate case where $E_t/E_r = 0.5$. Solutions for this situation are shown in Figure 3.21e,f. While these charts bear a resemblance to those in Figure 3.21a-d, the reduction factor for any particular combination of L/D, L_e/D and E_p/E_r will, of course, be smaller.

The variation in the settlement reduction factor for the three situations considered are presented in Figure 3.23. These solutions indicate that a conservative design estimate of $(RF)_\rho$ could be made by linear interpolation for the appropriate value of E_t/E_r given the reduction factor for $E_t/E_r = 1$.

3.6.4 Settlement Reduction Factors for Recessed Sideshear Sockets

As mentioned earlier (section 3.6.2), Pells and Turner (1979) have given reduction factors for recessed sideshear sockets. These charts have been reproduced in Figures 3.24 and 3.25. Using their solution, it was possible to construct tentative design charts (Figure 3.26) which show the variation in $(RF)_\rho$ with E_t/E_r.

3.7 SUMMARY

A theoretical approach for the prediction of the behaviour of piles socketed into weak rock has been described. This approach allows consideration of plastic failure within the rock, as well as slip, strain softening and dilatancy at the pile-rock interface.

The fundamental difference between tests commonly used for determining average sideshear resistance has been discussed. It was suggested that there will generally be a discrepancy between the average sideshear obtained from displacement controlled tests on sideshear sockets and that obtained from load defined tests on endbearing piles. This discrepancy is small for moderately long and rough sockets however the results obtained from sideshear sockets may underestimate the available residual sideshear resistance of endbearing piles. The magnitude of this discrepancy will depend upon the relative stiffness of the pile and rock as well as the length and roughness of the socket. In theory, the discrepancy may be more than an order of magnitude for typical parameters and a perfectly smooth socket although in practice, this variation would not be expected because of the finite roughness of even "smooth" sockets.

The effect of a number of fundamental parameters upon the average peak and residual sideshear values was examined for endbearing piles subjected to a load defined test. It was shown that the magnitude of the average sideshear depends not only upon the strength parameters, but also upon dilatancy at the interface, the length to diameter ratio and the relative modulus of the pile and rock. The average sideshear deduced using typical values of the fundamental parameters for two types of rock were of similar magnitude to those observed in field tests. These results suggest that the apparent scatter of results obtained from different field tests on rock with apparently the same strength characteristics, may be partly due to the effect of parameters such as pile length and modulus ratio.

A series of solutions were presented for a pile socketed into a homogeneous rock and for a pile bearing on either stiffer or weaker rock. In addition to the elastic response, these solutions provide a means of estimating the load at which full slip occurs and the corresponding pile head settlement; as well as the pile head displacement for any given load following full slip. Nonlinearity of the bearing strata may be approximated by the selection of an appropriate secant modulus.

The effect of weak horizontal seams along the pile shaft was examined and theoretical solutions for assessing the significance of these seams upon the pile response were presented.

Finally, a theoretical analysis which illustrates the settlement behaviour of complete sockets recessed below the rock surface has been presented. The results of this analysis are given as a series of design charts which will provide an estimate of the reduction in settlement experienced by a socketed pile recessed in either a homogeneous or non-homogeneous rock mass.

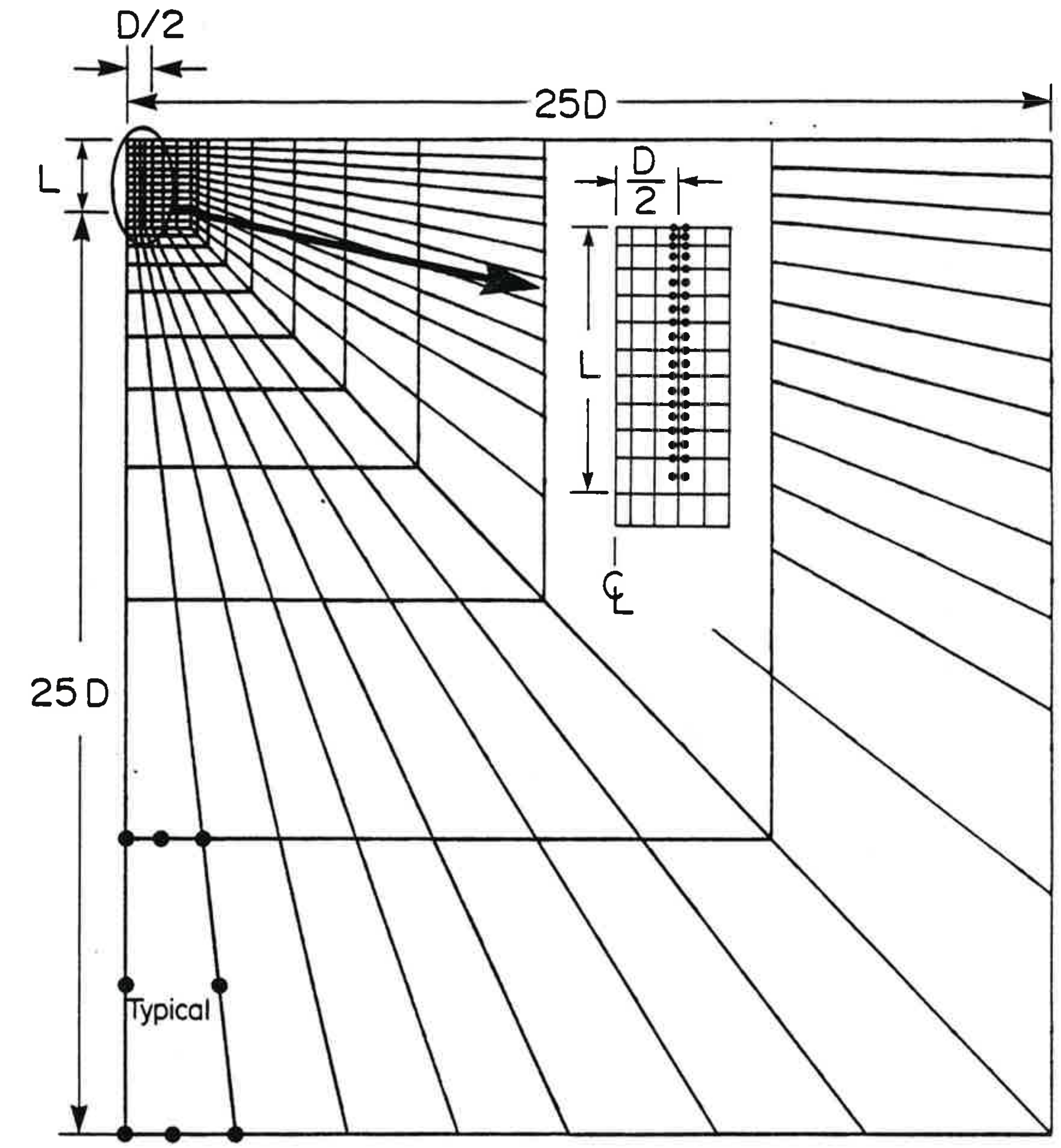


FIGURE 3.1 FINITE ELEMENT MESH

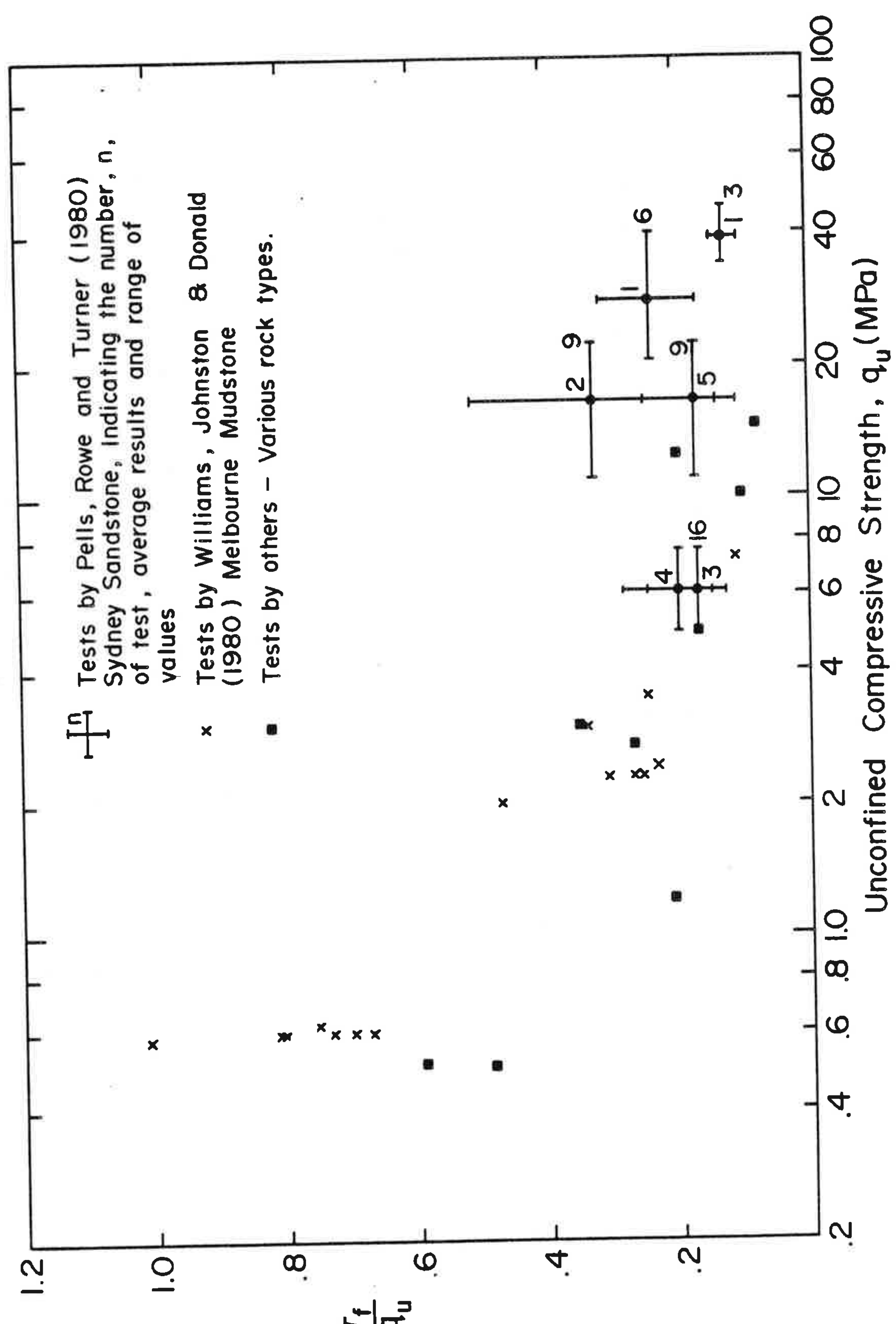


FIGURE 3.2 VARIATION IN AVERAGE SHEAR WITH UNCONFINED COMPRESSIVE STRENGTH

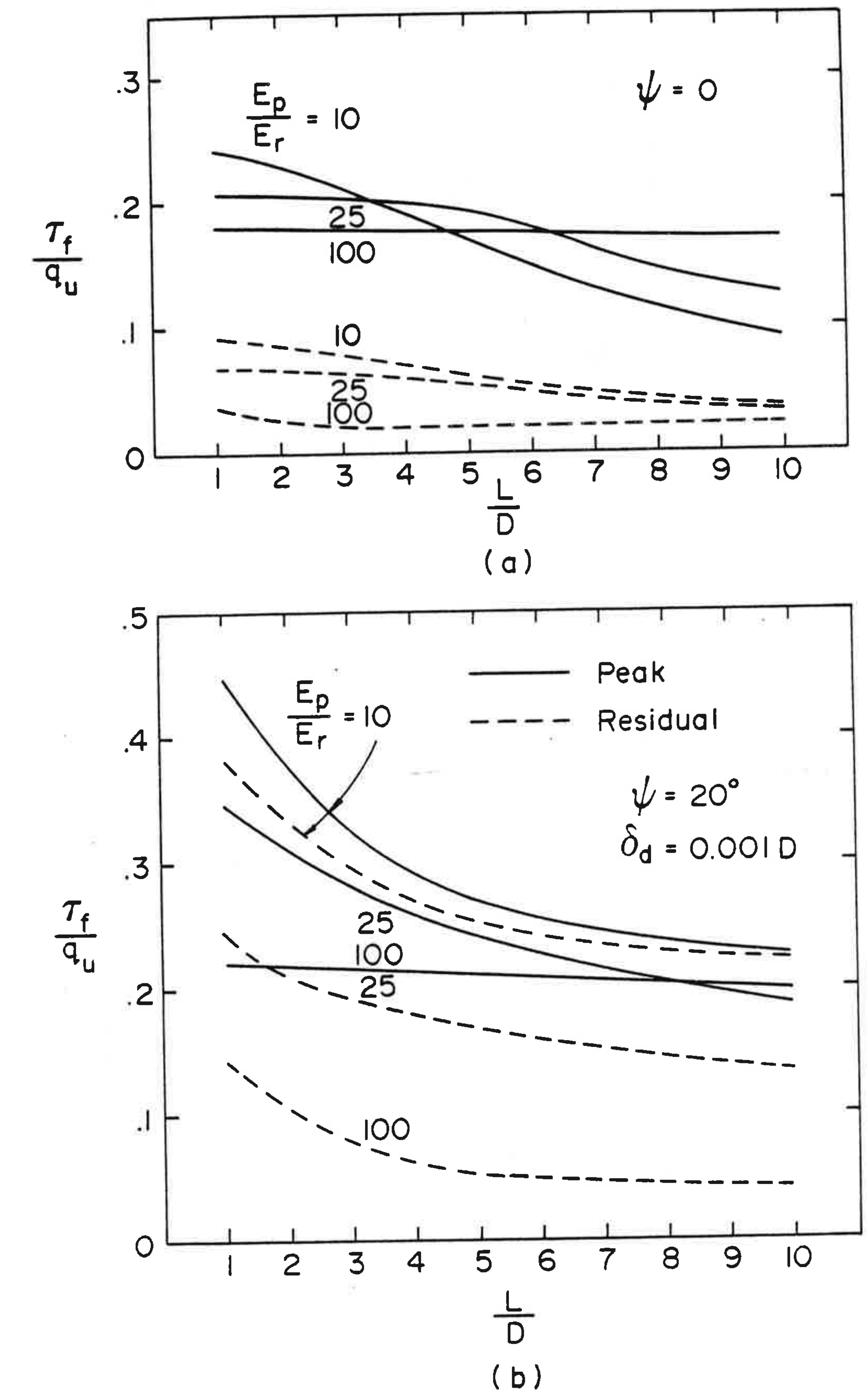


FIGURE 3.3 VARIATION IN AVERAGE SHEAR WITH L/D
(c_p=4MPa, ϕ_p=39°, c_r=0 MPa, ϕ_r=36°, q_u=16.8 MPa)

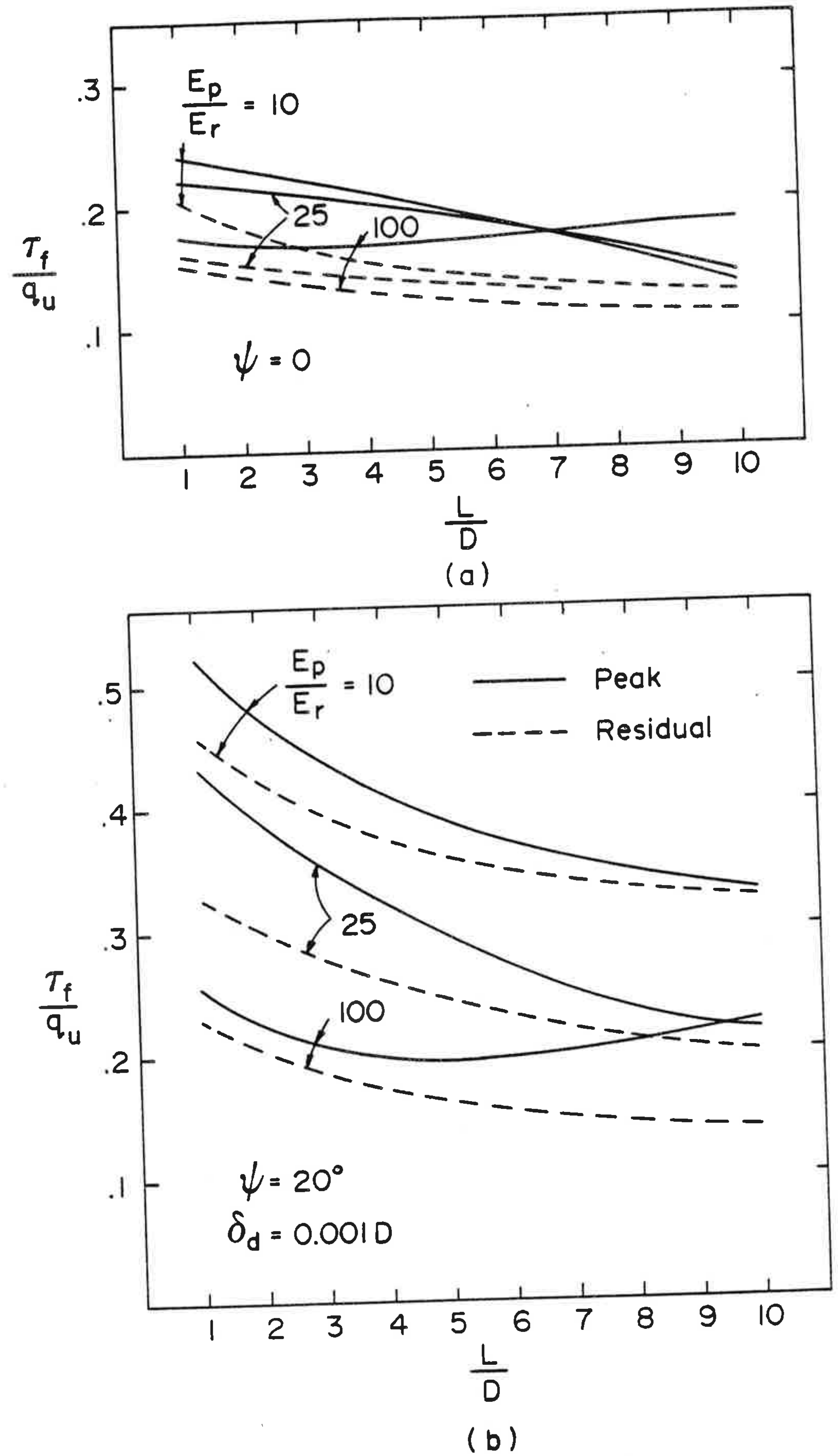


FIGURE 3.4 VARIATION IN AVERAGE SHEAR WITH L/D
(c_p = 4 MPa, ϕ_p = 39°, c_r = 2 MPa, ϕ_r = 36°, q_u = 16.8 MPa)

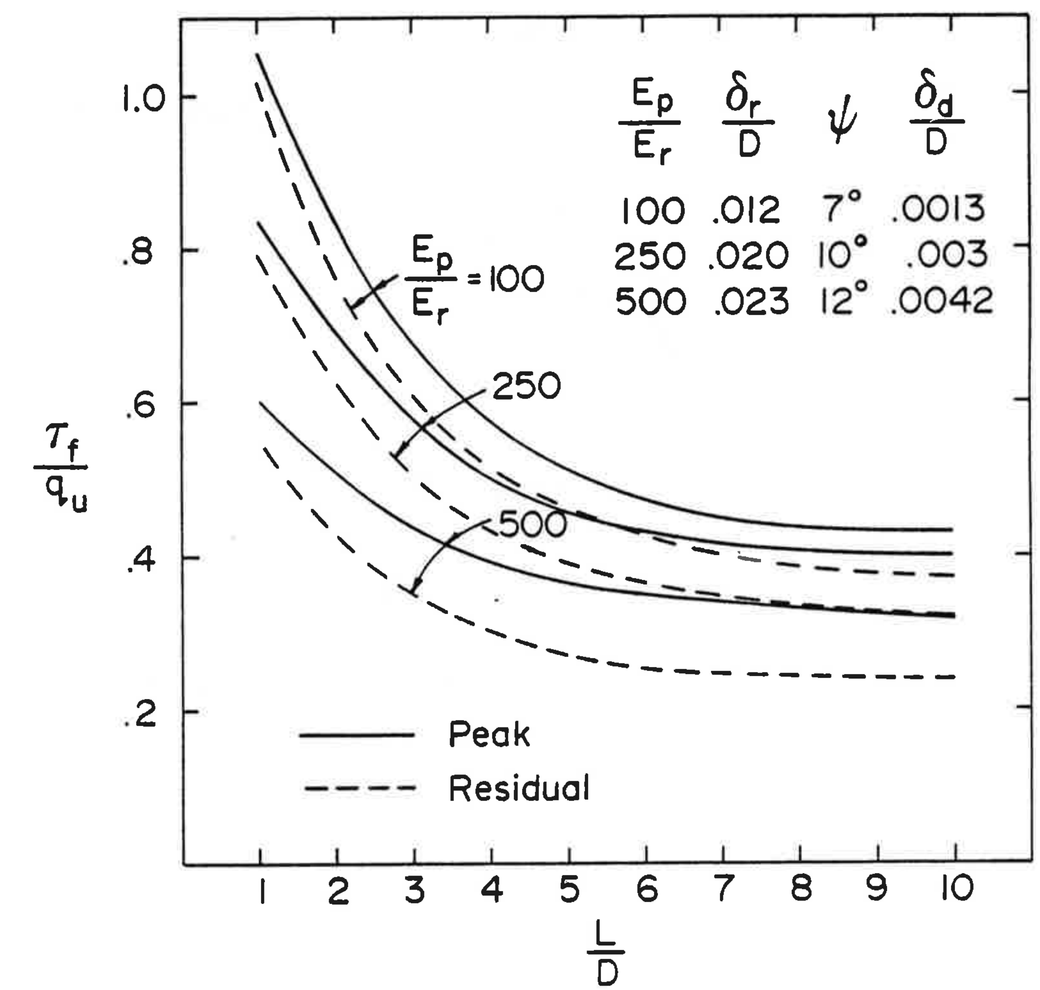


FIGURE 3.5 VARIATION IN AVERAGE SHEAR WITH L/D
(c_p = .2 MPa, ϕ_p = 32°, c_r = 0, ϕ_r = 21°, q_u = 0.72 MPa)

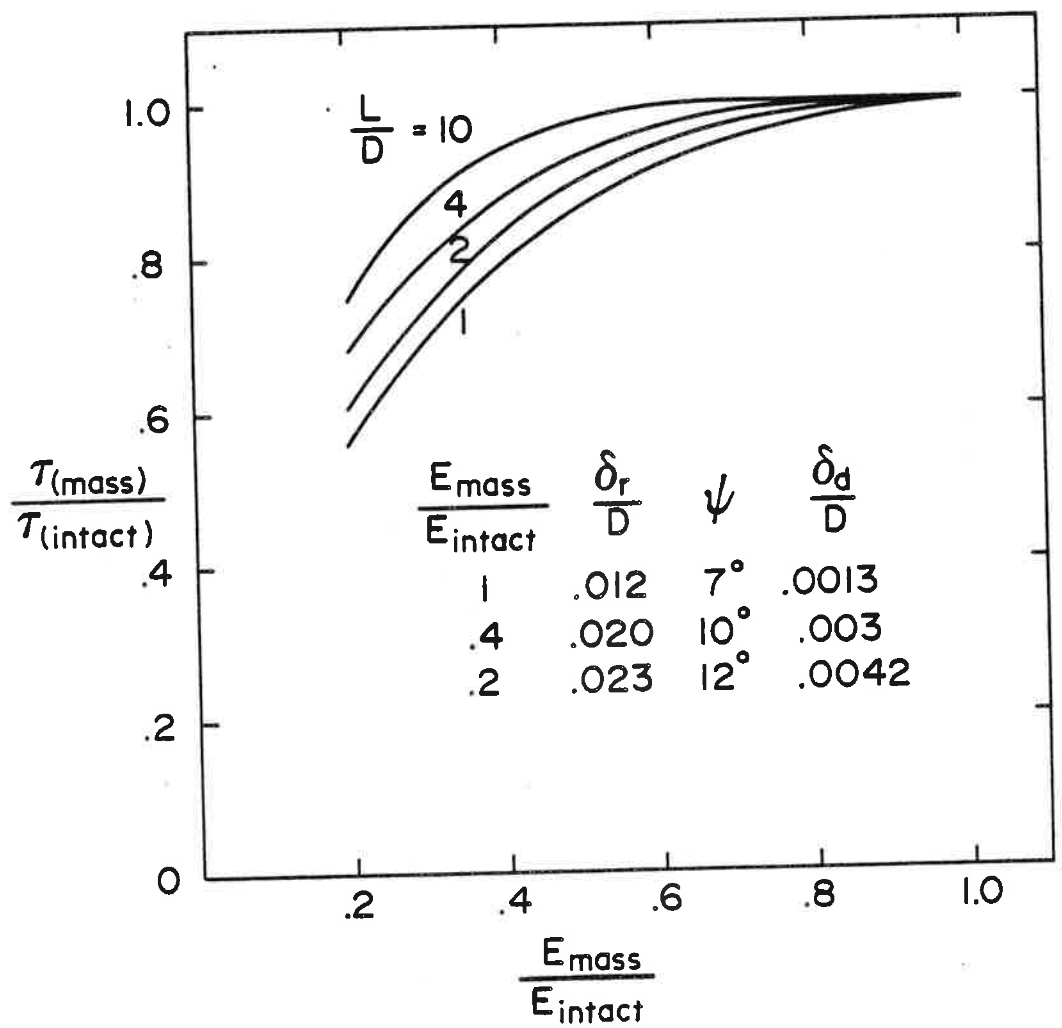


FIGURE 3.6 REDUCTION IN PEAK SHEAR DUE TO DECREASING MASS MODULUS (c_p = .2 MPa, ϕ_p = 32°, c_r = 0, ϕ_r = 21°, q_u = 0.72 MPa)

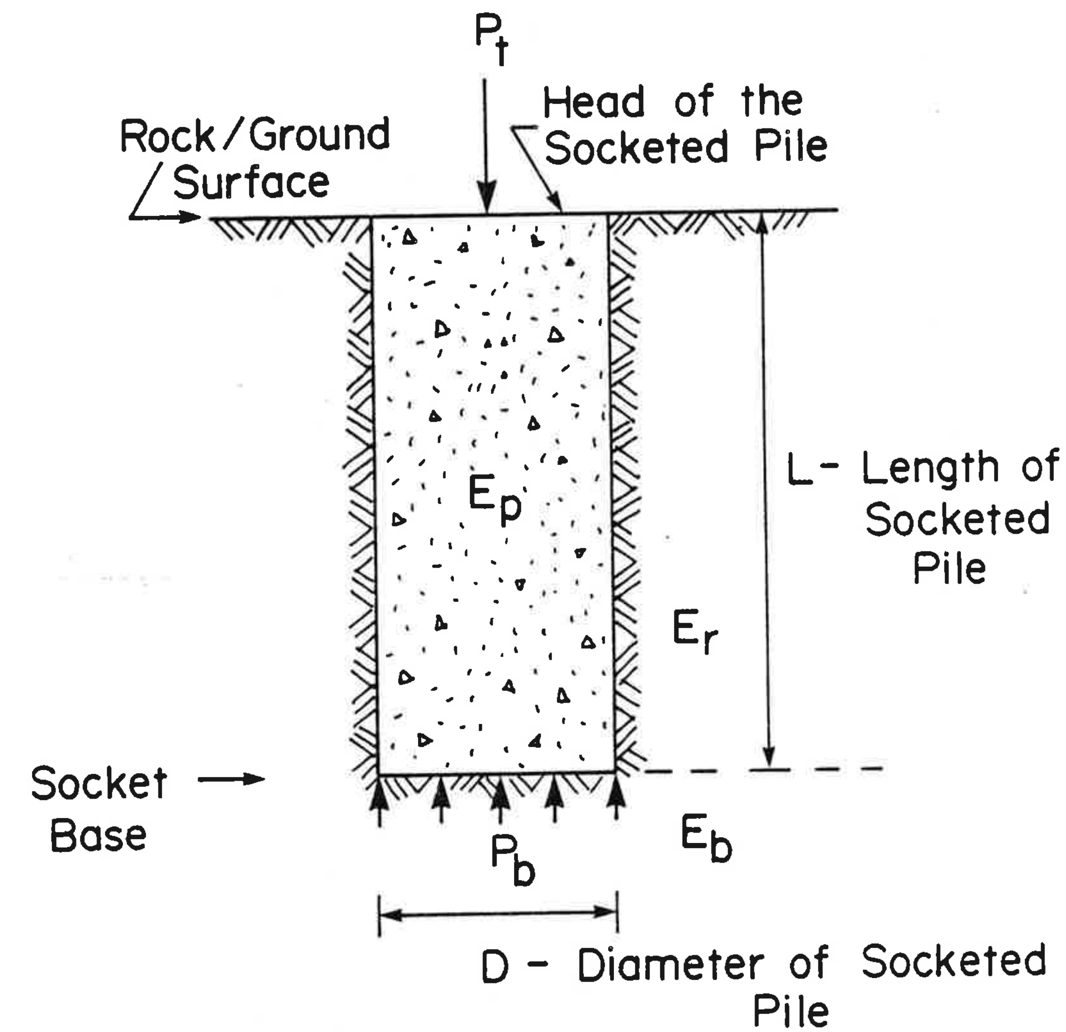


FIGURE 3.7 NOTATION FOR A COMPLETE ROCK SOCKETED PILE

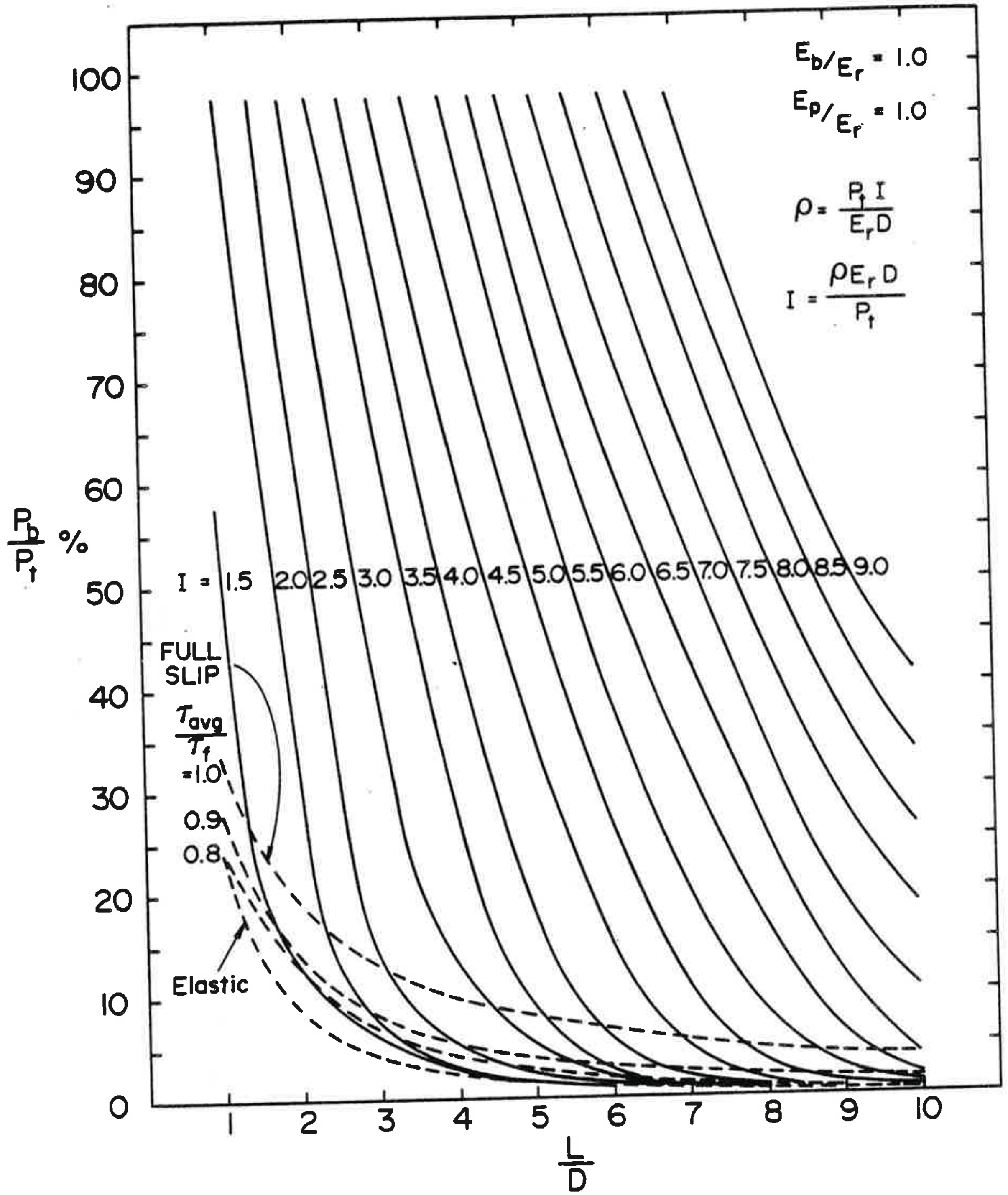


FIGURE 3.8 DESIGN CHART FOR A COMPLETE SOCKETED PILE (E_b/E_r = 1.0 and E_p/E_r = 1.0)

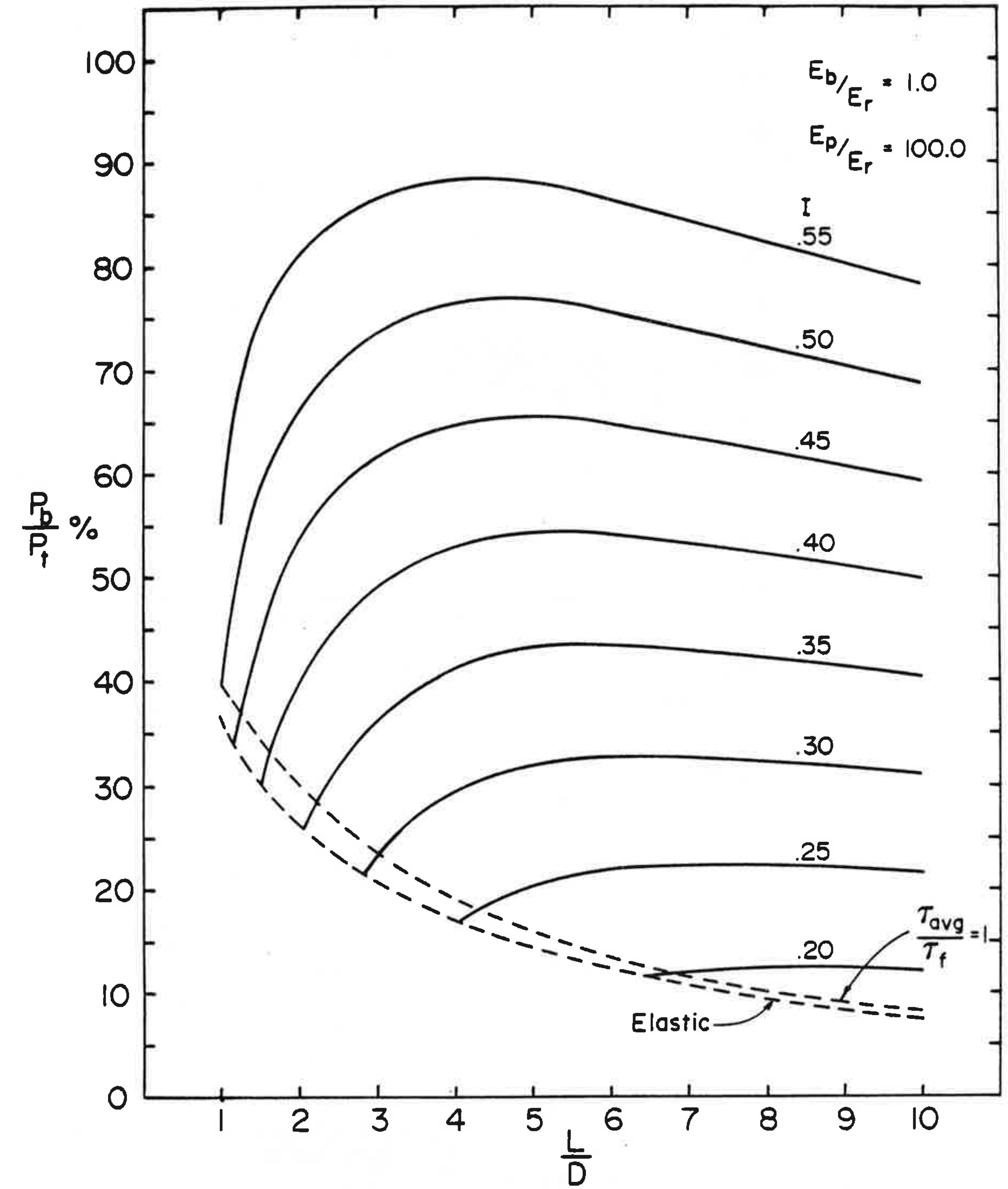


FIGURE 3.9 DESIGN CHART FOR A COMPLETE SOCKETED PILE (E_b/E_r = 1.0 and E_p/E_r = 100.0)

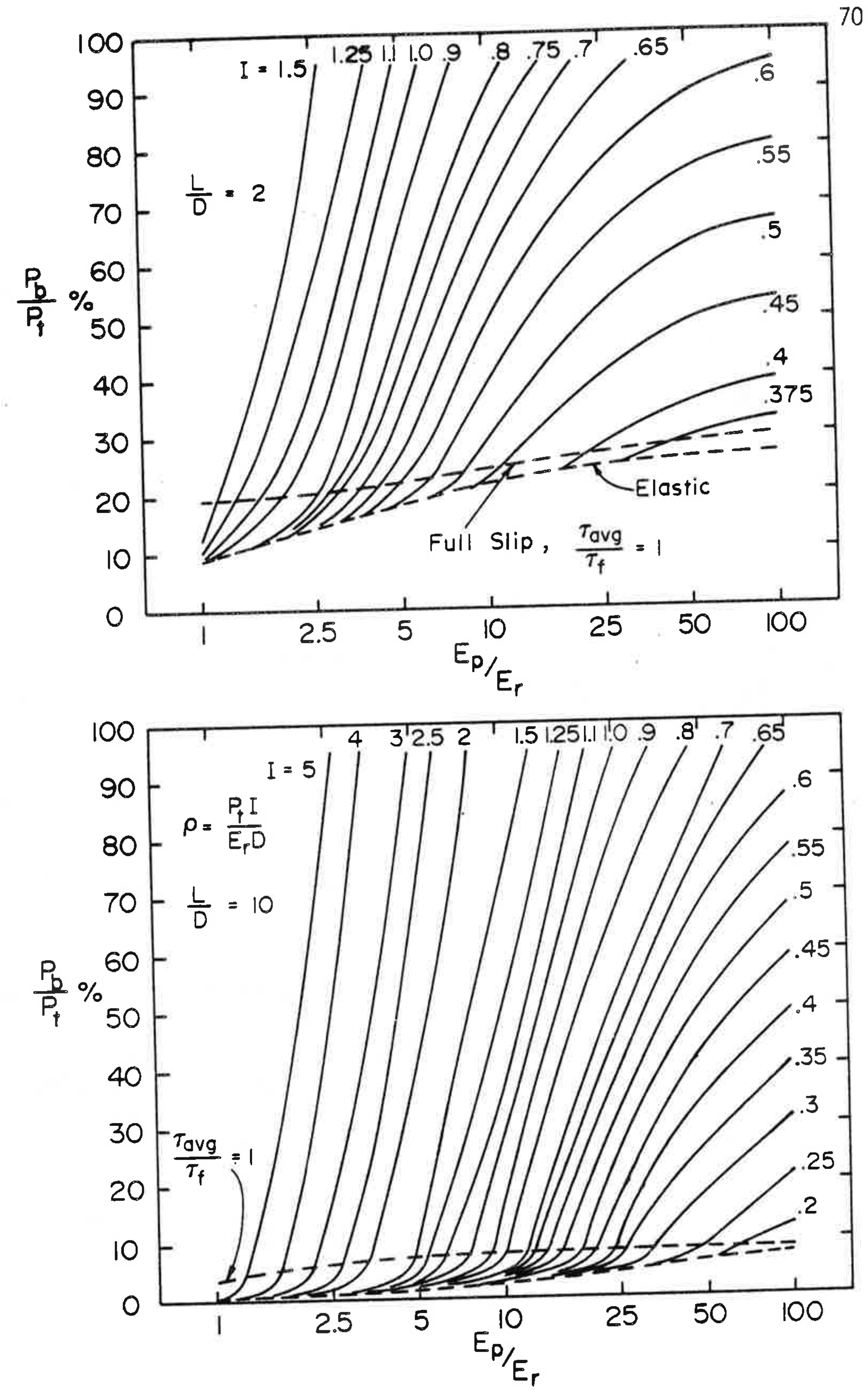


FIGURE 3.10 VARIATION IN PILE RESPONSE WITH STIFFNESS RATIO E_p/E_r; E_b/E_r = 1

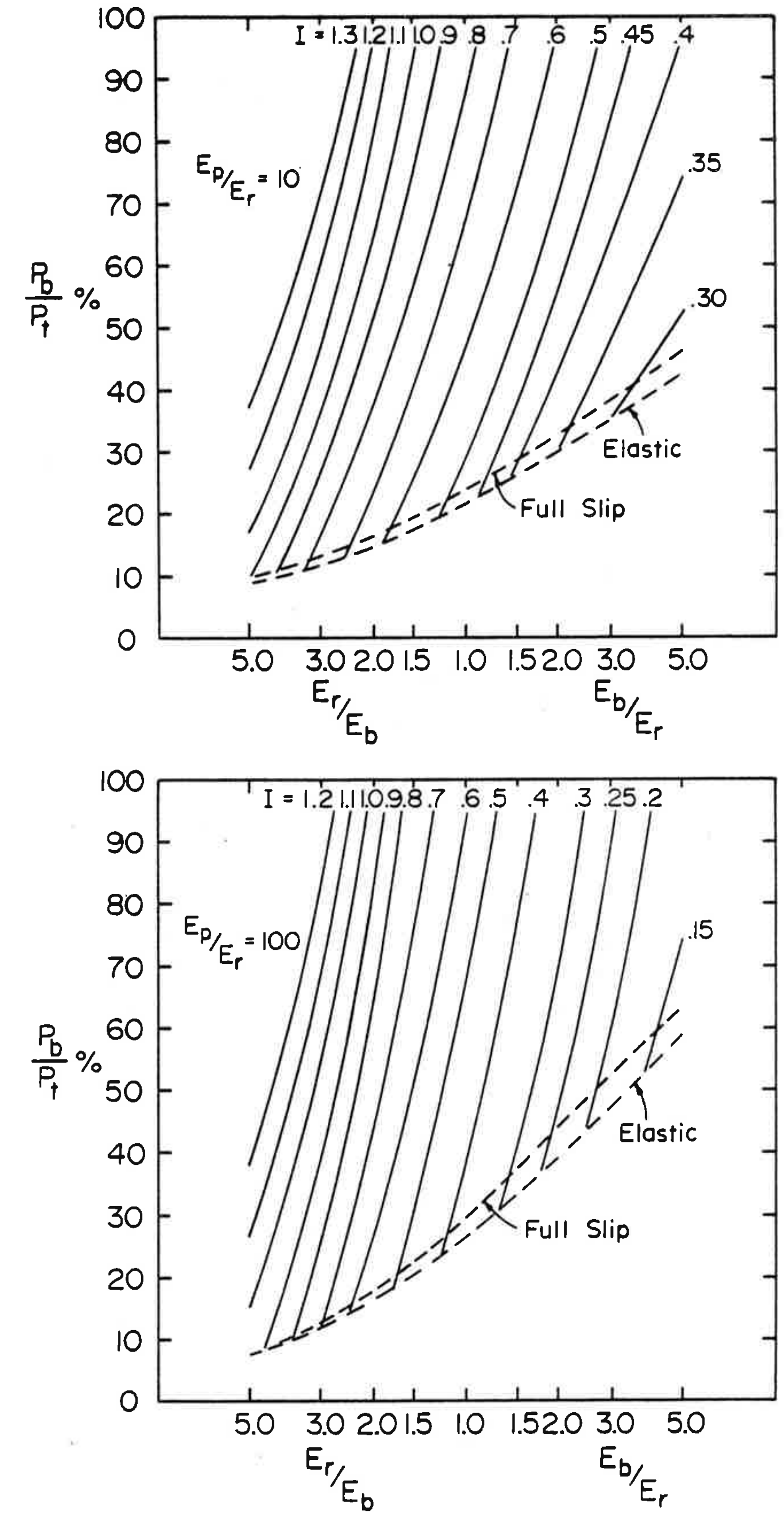


FIGURE 3.11a VARIATION IN PILE RESPONSE WITH BASE STIFFNESS E_b/E_r (L/D = 2)

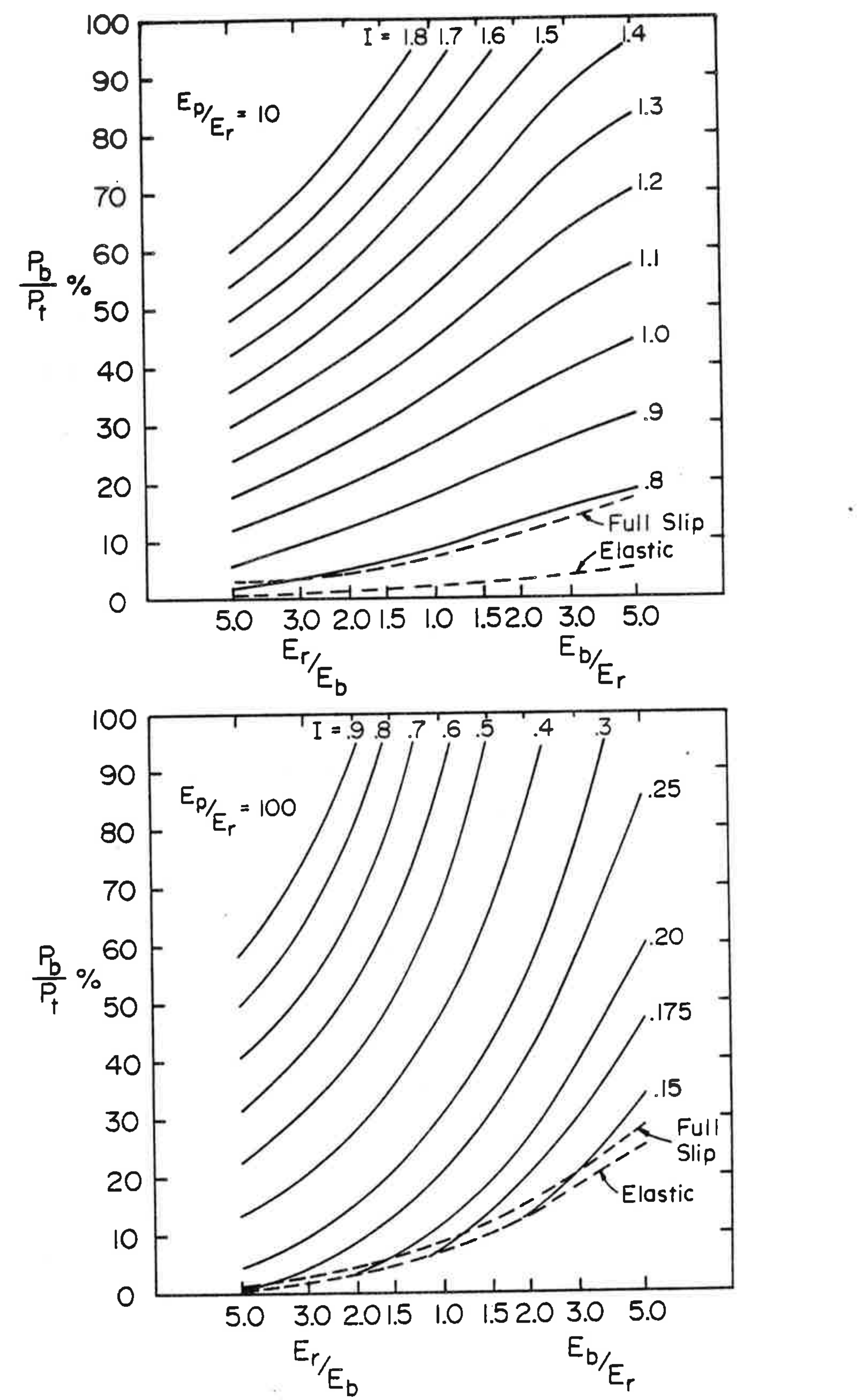


FIGURE 3.11b VARIATION IN PILE RESPONSE WITH BASE STIFFNESS E_b/E_r (L/D = 10)

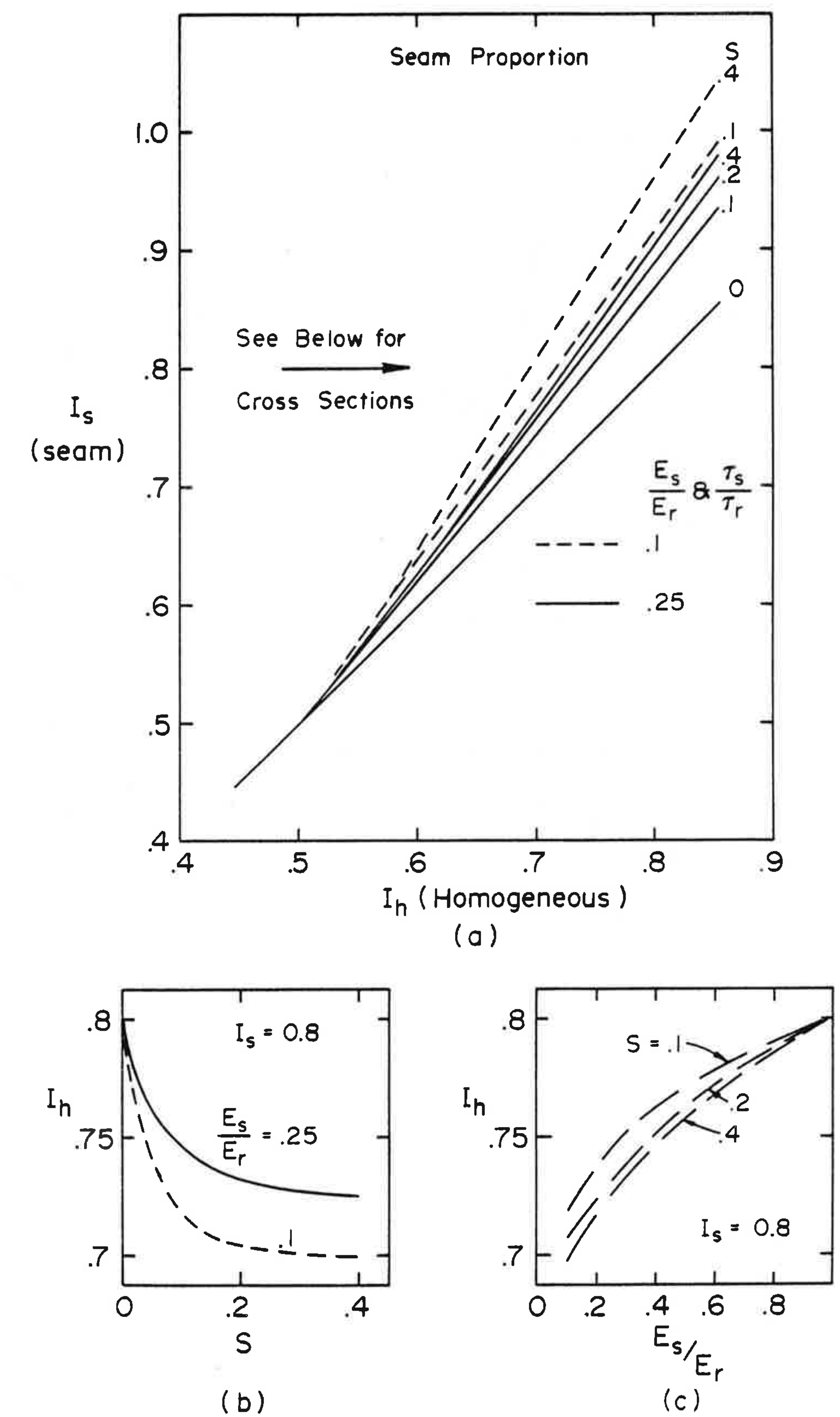


FIGURE 3.12 EFFECT OF SEAMS: L/D = 2, E_p/E_r = 10, E_b/E_r = 1 (FULL SLIP)

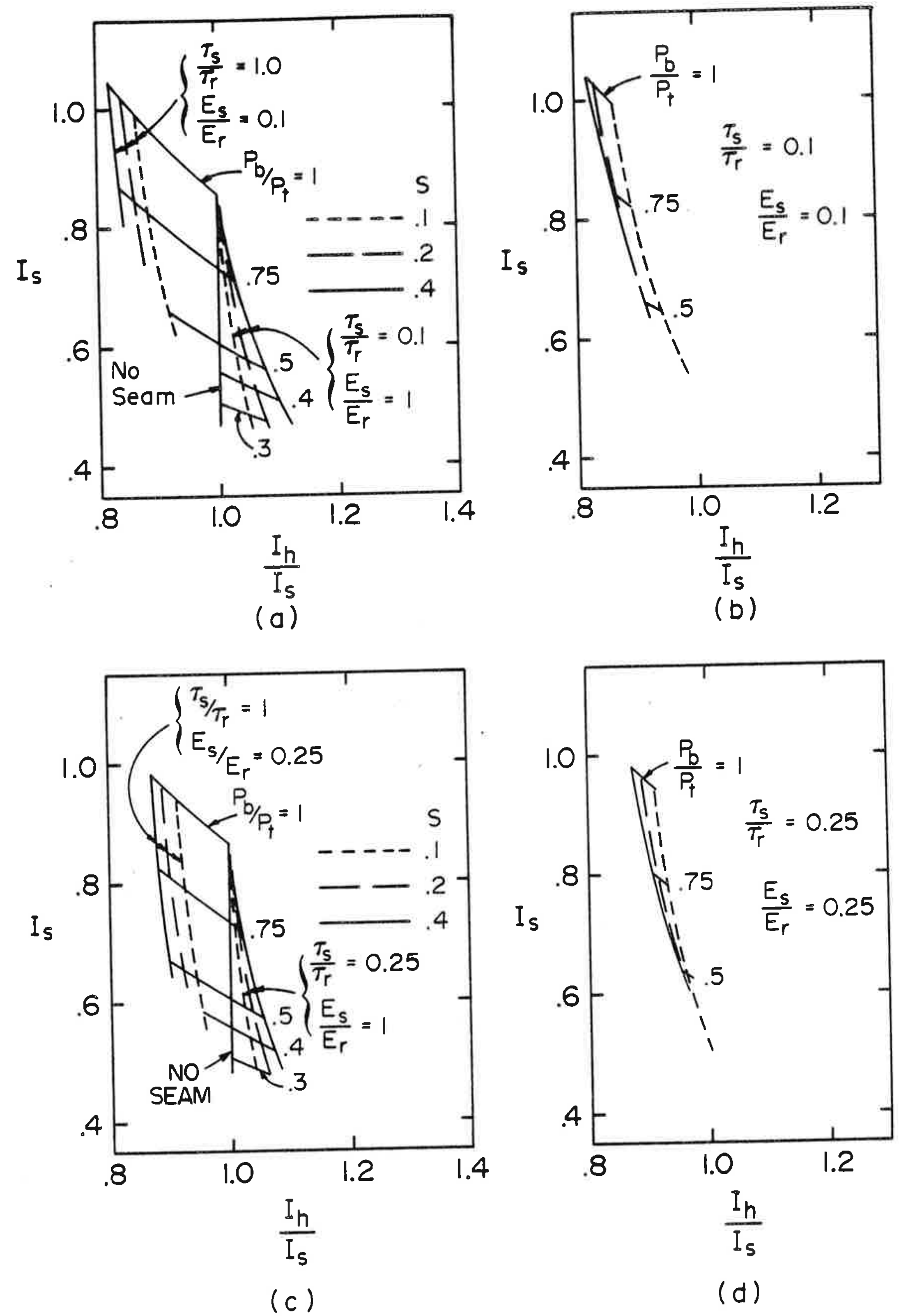


FIGURE 3.13 EFFECT OF SEAM PROPERTIES UPON DISPLACEMENT RATIO FOR GIVEN P_b/P_t (FULL SLIP) ($L/D = 2$, $E_p/E_r = 10$)

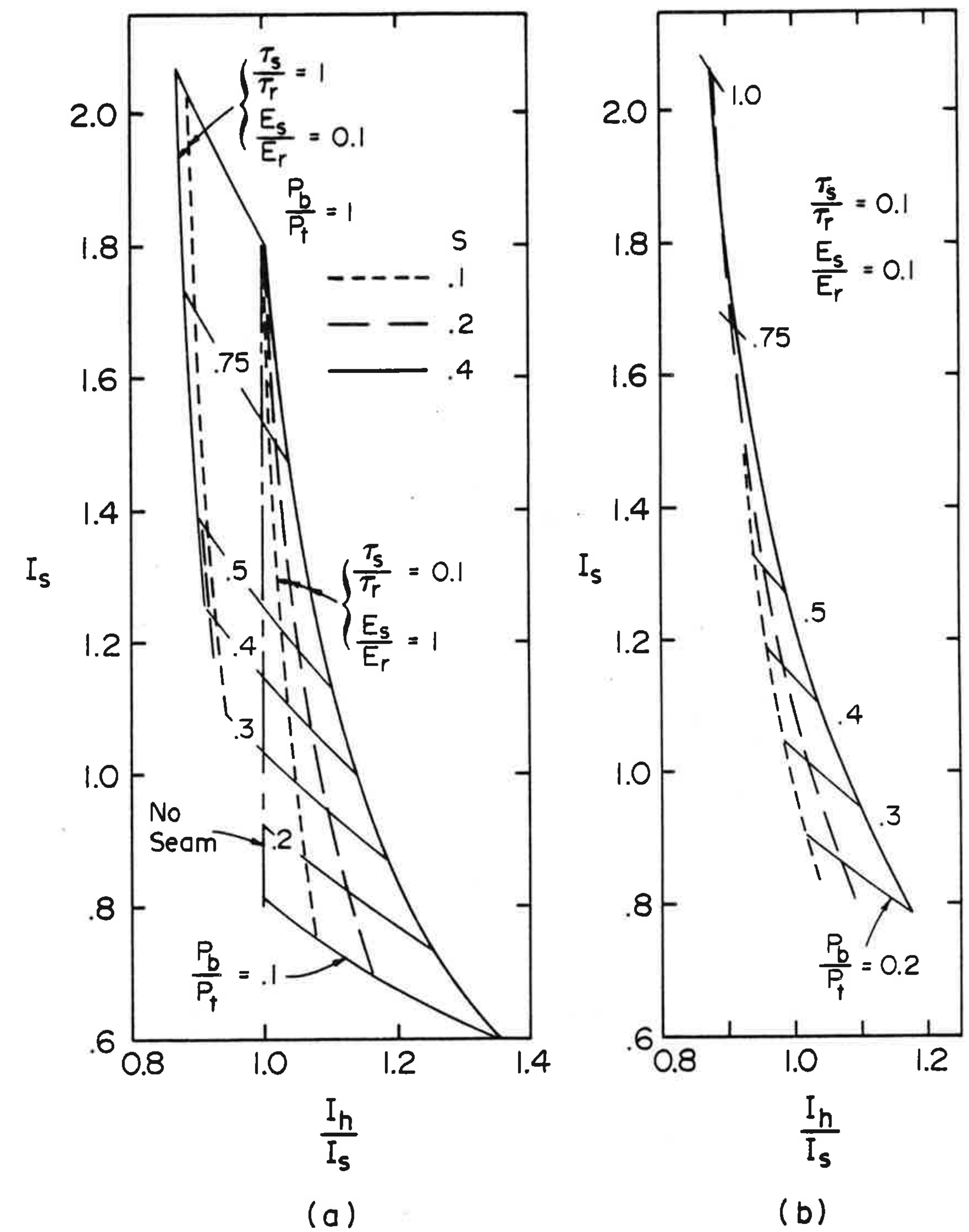


FIGURE 3.14a,b EFFECT OF SEAM PROPERTIES UPON DISPLACEMENT RATIO FOR GIVEN P_b/P_t ($L/D = 10$, $E_p/E_r = 10$)

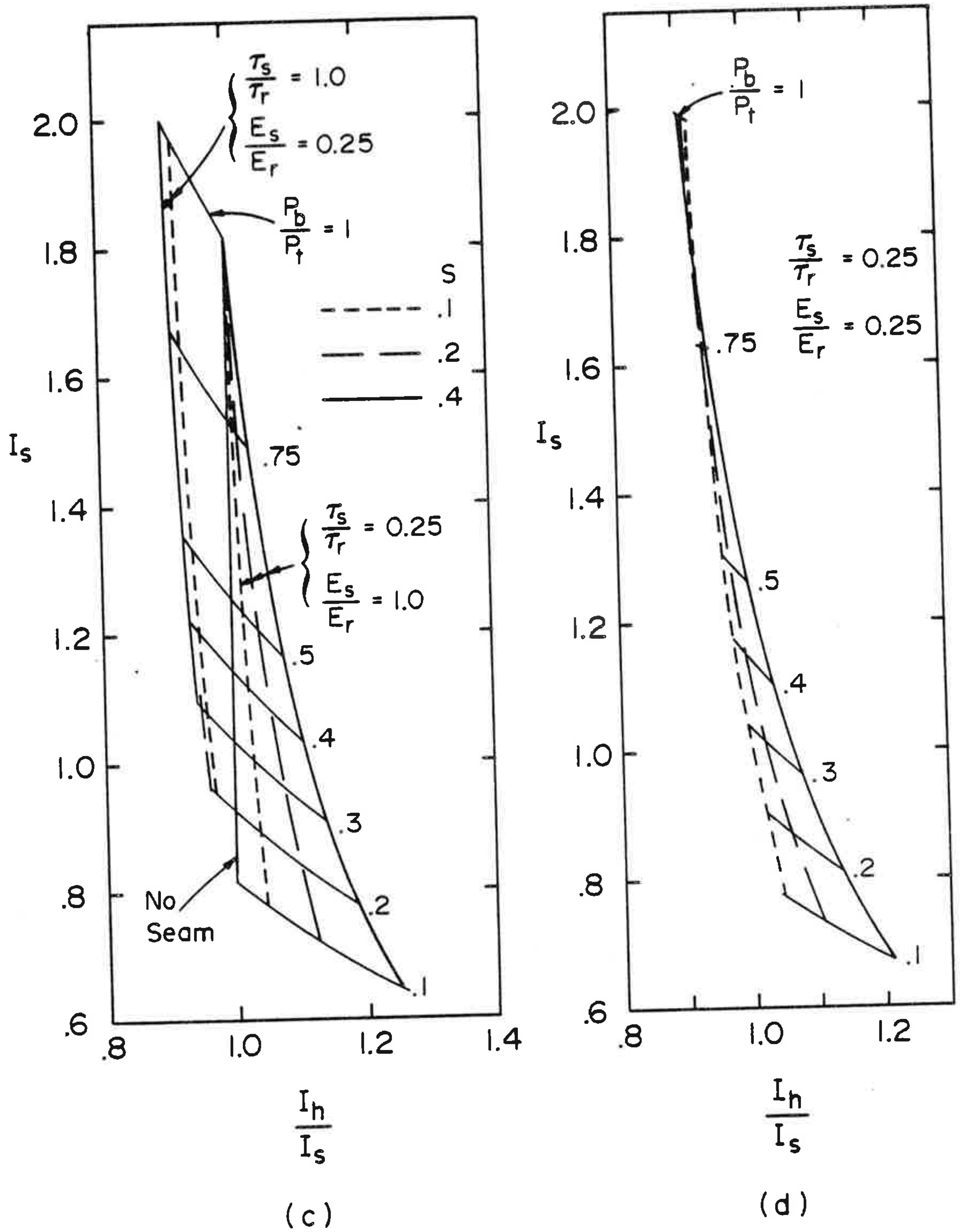


FIGURE 3.14c,d EFFECT OF SEAM PROPERTIES UPON DISPLACEMENT RATIO FOR GIVEN P_b/P_t ($L/D = 10$, $E_p/E_r = 10$)

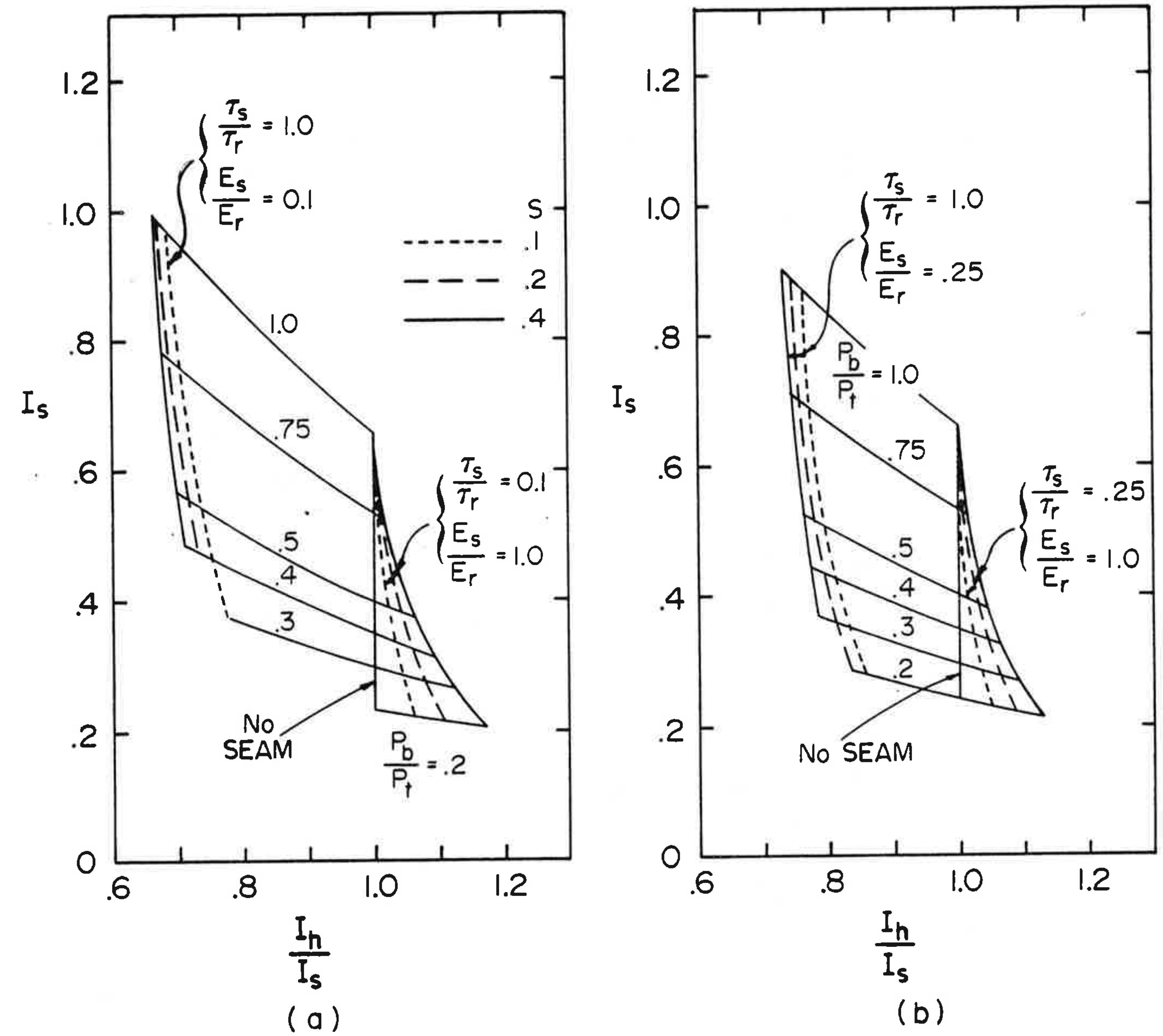


FIGURE 3.15 EFFECT OF SEAM PROPERTIES UPON DISPLACEMENT RATIO FOR GIVEN P_b/P_t ($L/D = 10$, $E_p/E_r = 100$)

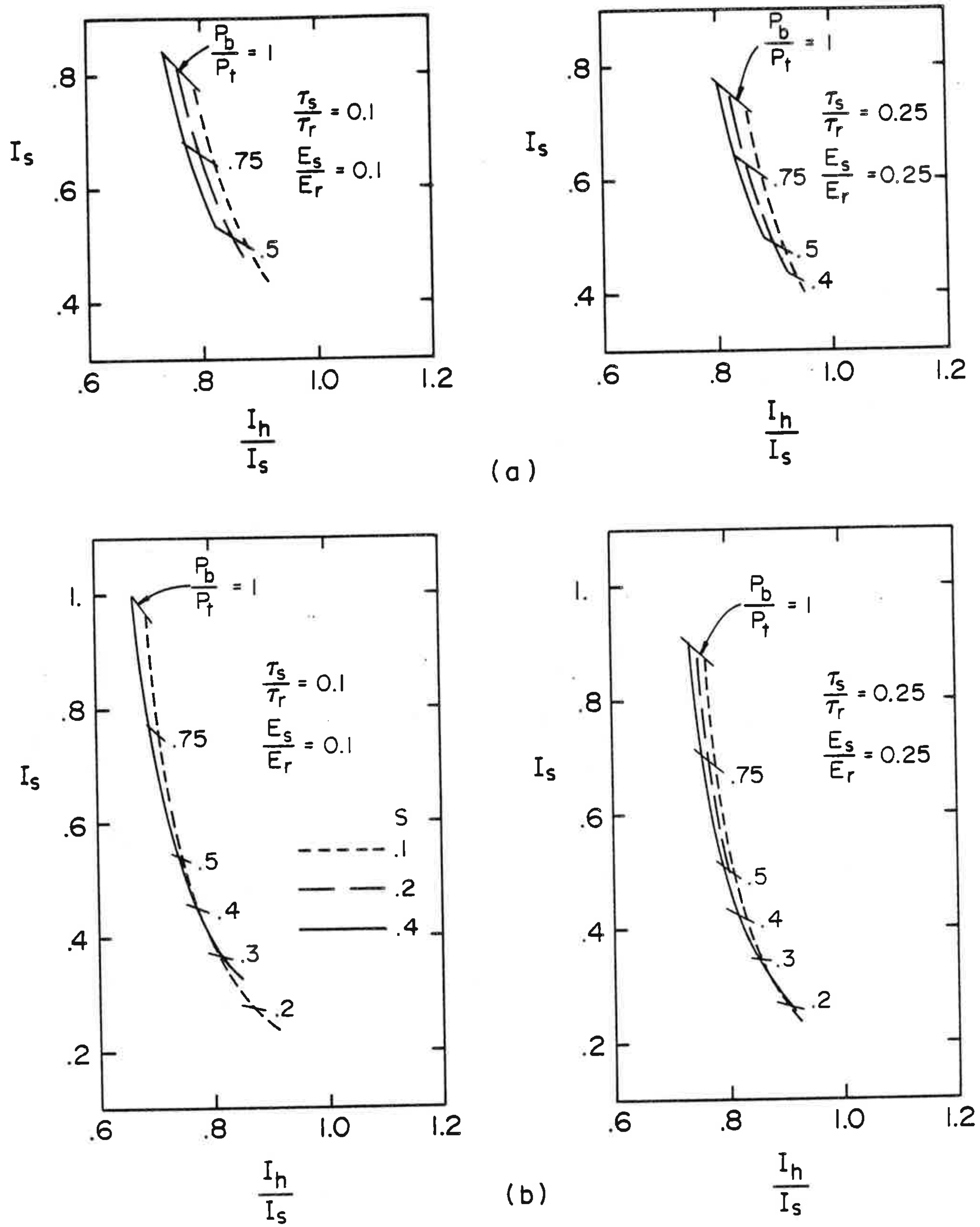


FIGURE 3.16 EFFECT OF SEAM PROPERTIES UPON DISPLACEMENT RATIO E_p/E_r = 100 a) L/D = 2 b) L/D = 10

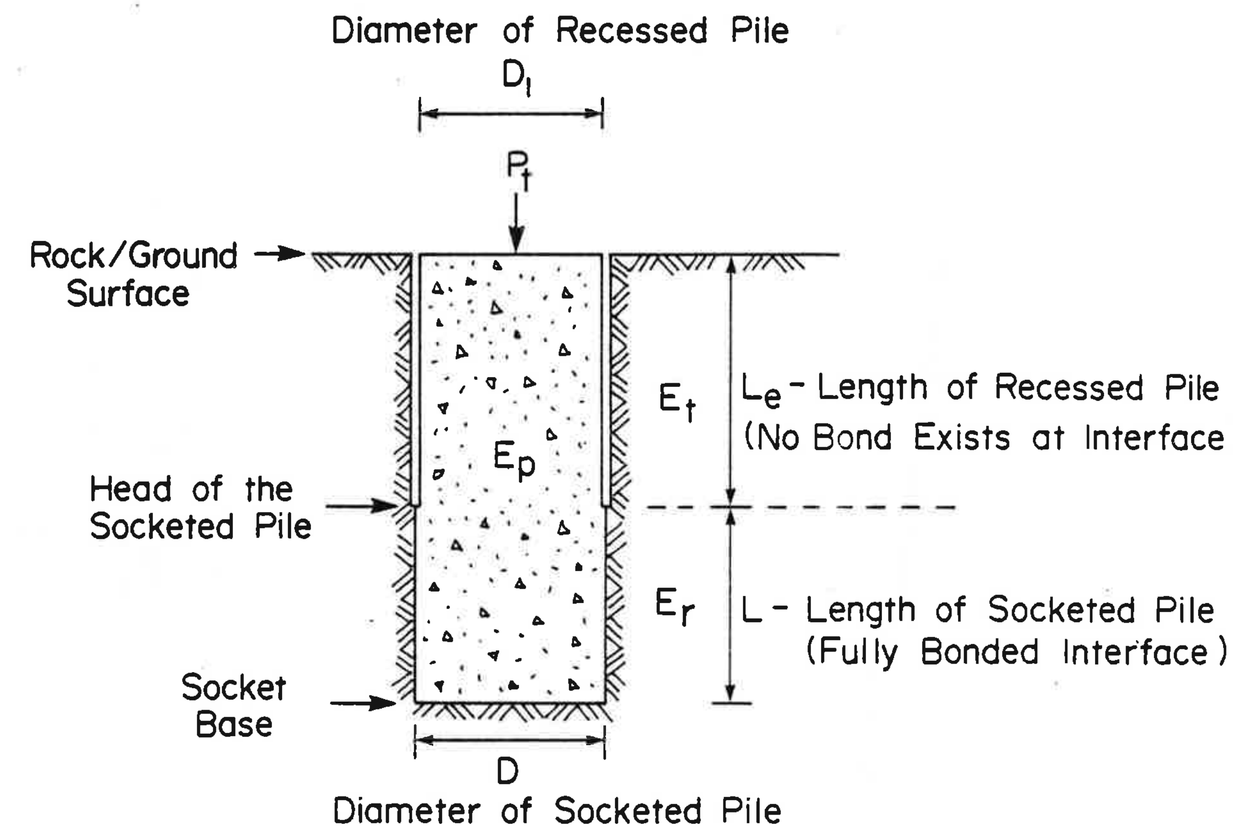


FIGURE 3.17 NOTATION FOR A RECESSED COMPLETE SOCKETED PILE IN ROCK

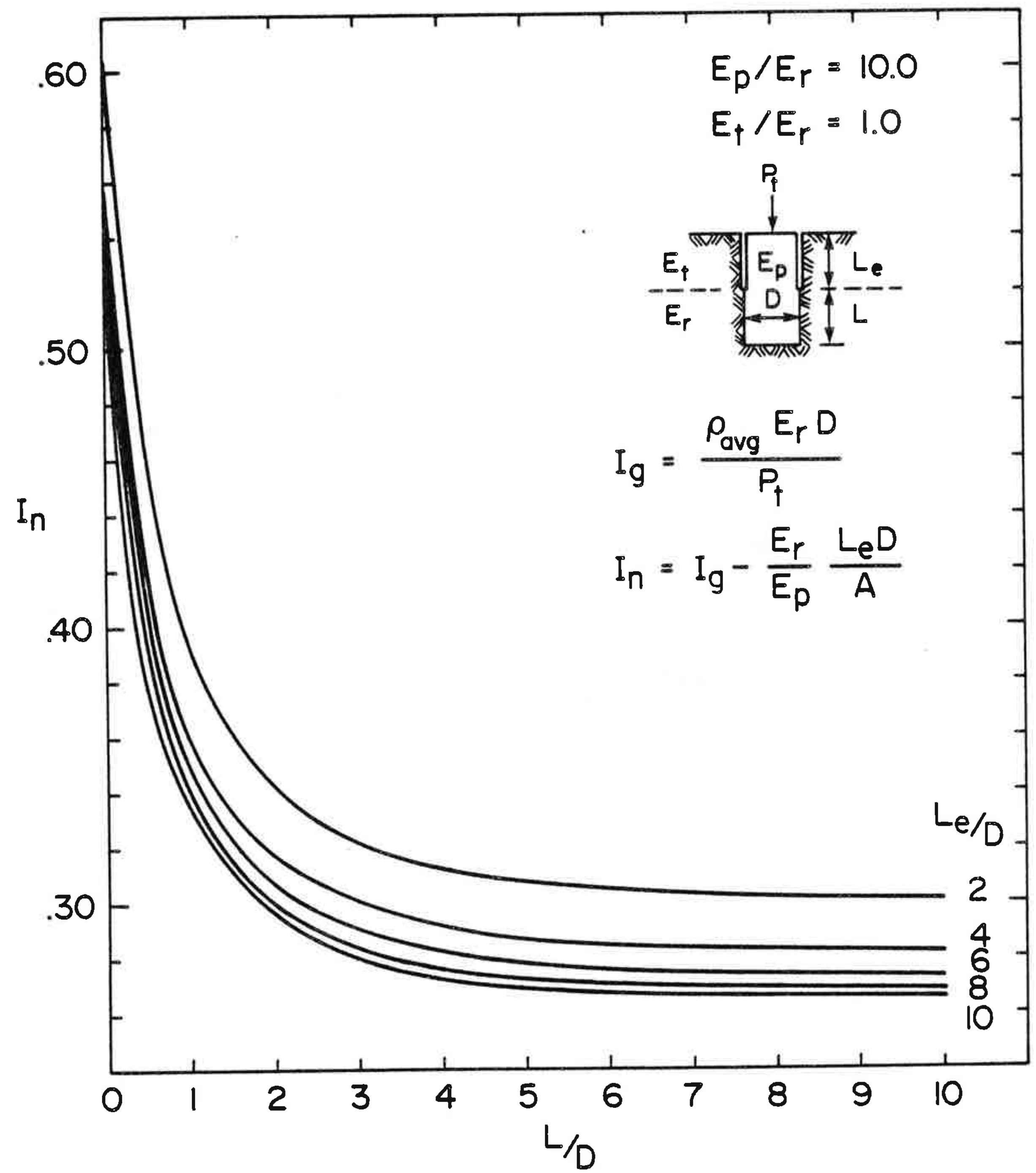


FIGURE 3.18 NET INFLUENCE FACTOR FOR A RECESSED, COMPLETE SOCKETED PILE (E_p/E_r = 10.0, E_t/E_r = 1.0)

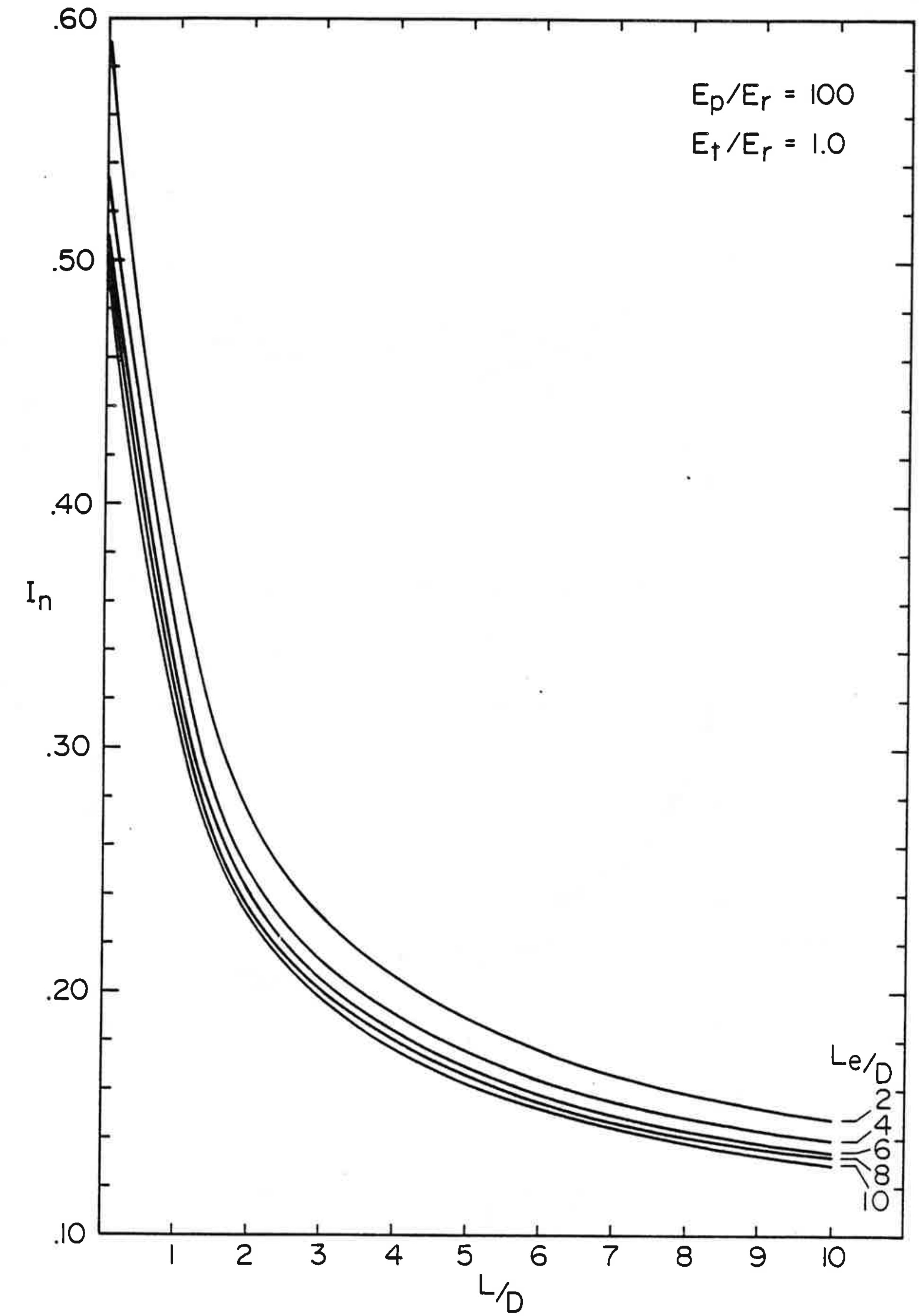
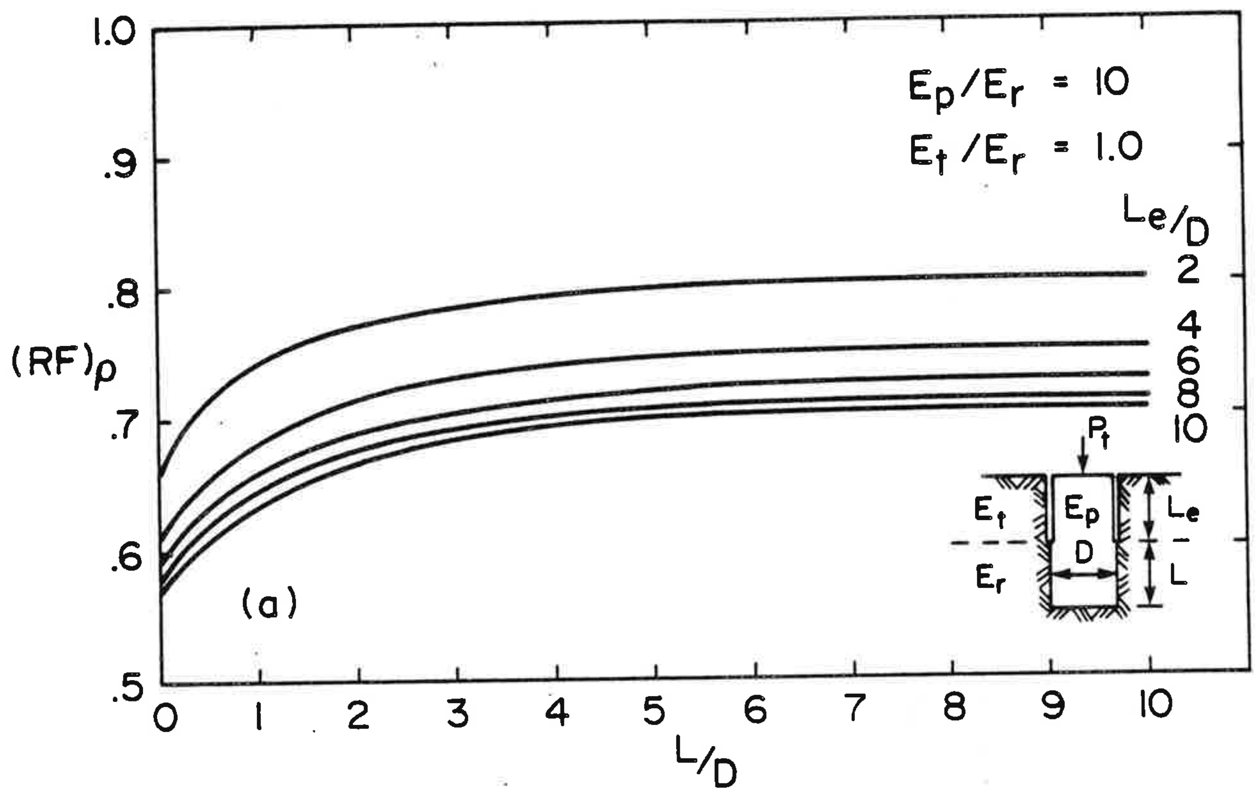
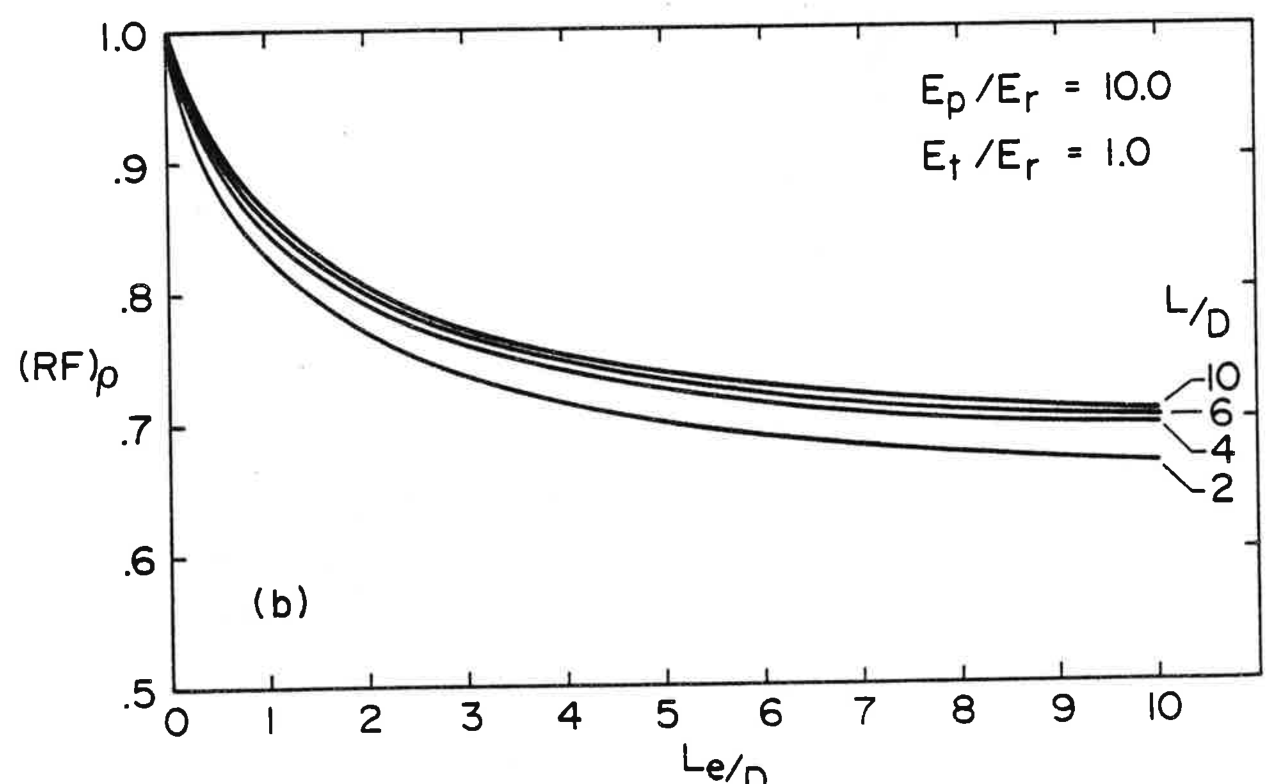


FIGURE 3.19 NET INFLUENCE FACTOR FOR A RECESSED, COMPLETE SOCKETED PILE (E_p/E_r = 100, E_t/E_r = 1.0)



(a)



(b)

FIGURE 3.20 SETTLEMENT REDUCTION FACTOR FOR A COMPLETE SOCKETED PILE (E_p/E_r = 10, E_t/E_r = 1.0)

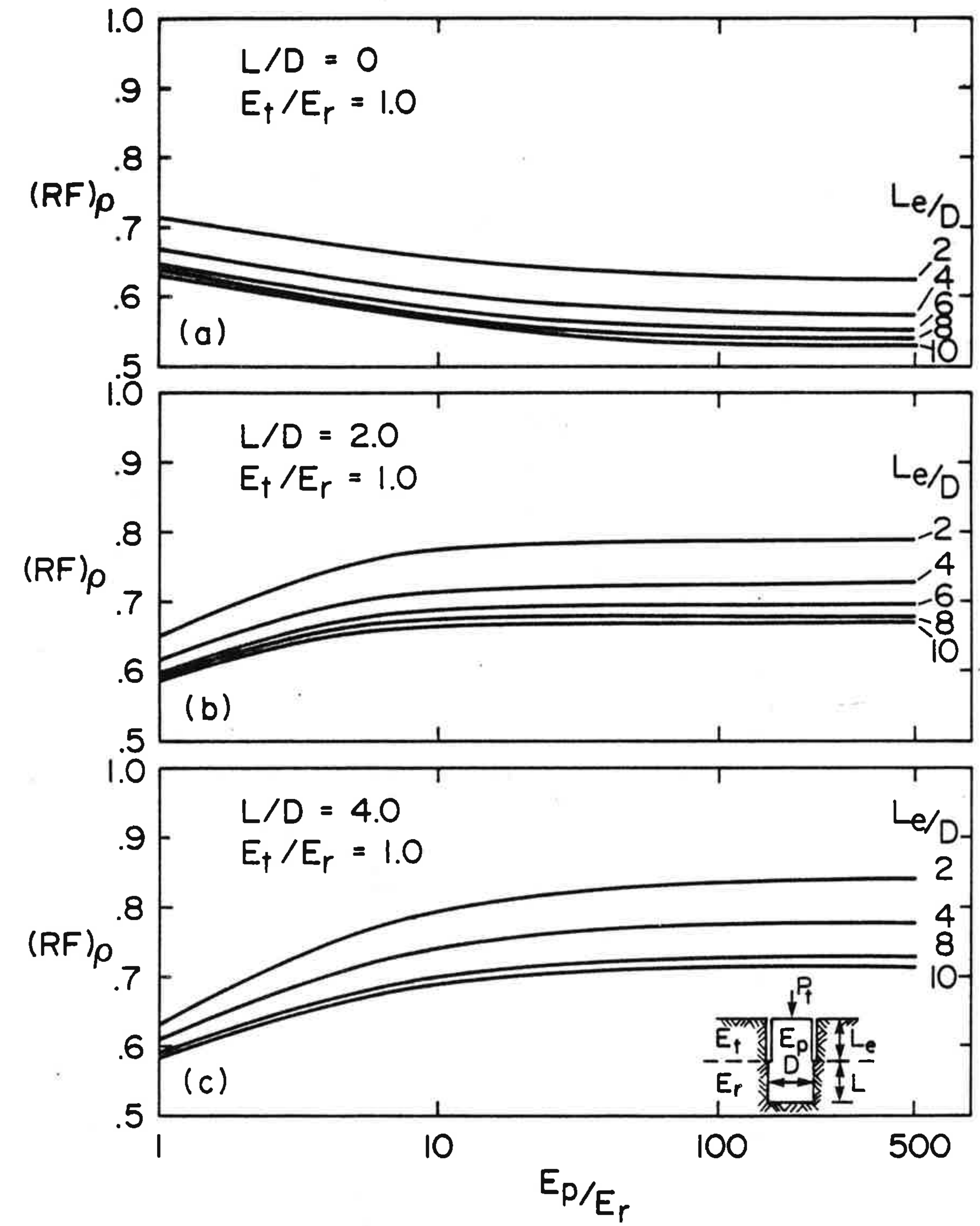


FIGURE 3.21a,b,c SETTLEMENT REDUCTION FACTORS FOR VARIATION IN E_p/E_r RATIO

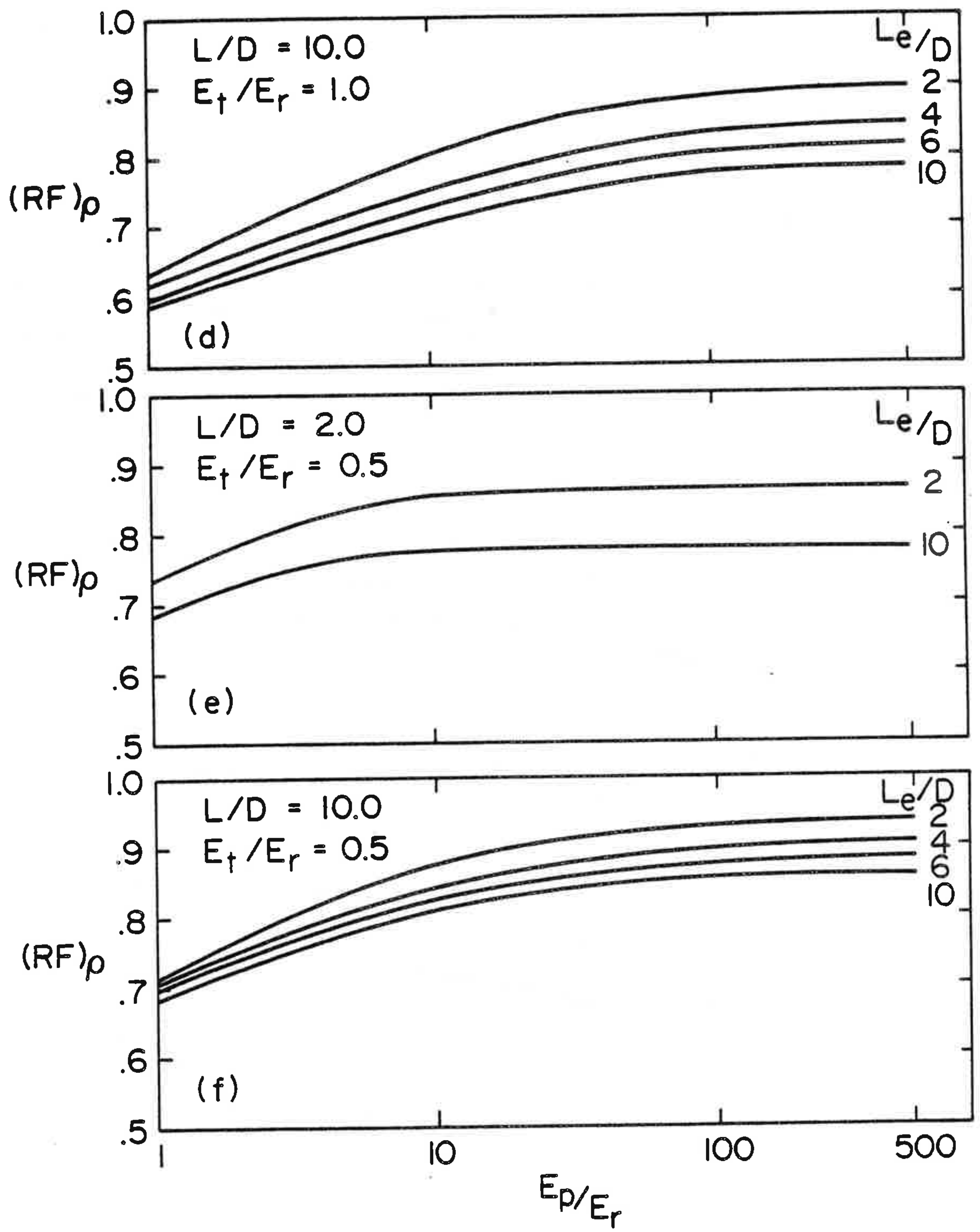


FIGURE 3.21d,e,f SETTLEMENT REDUCTION FACTORS FOR VARIATION IN E_p/E_r RATIO

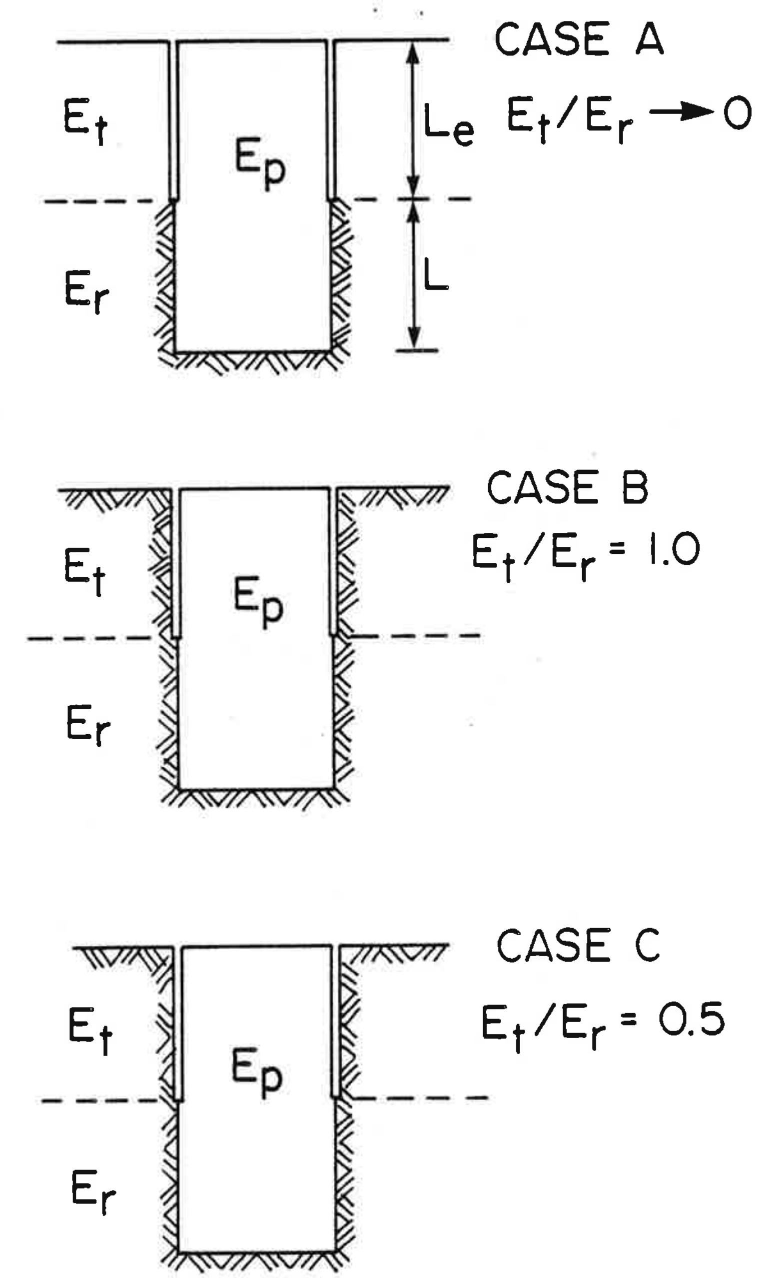


FIGURE 3.22 ILLUSTRATION OF VARIOUS CASES WHICH INFLUENCE THE VALUE OF $(RF)_\rho$

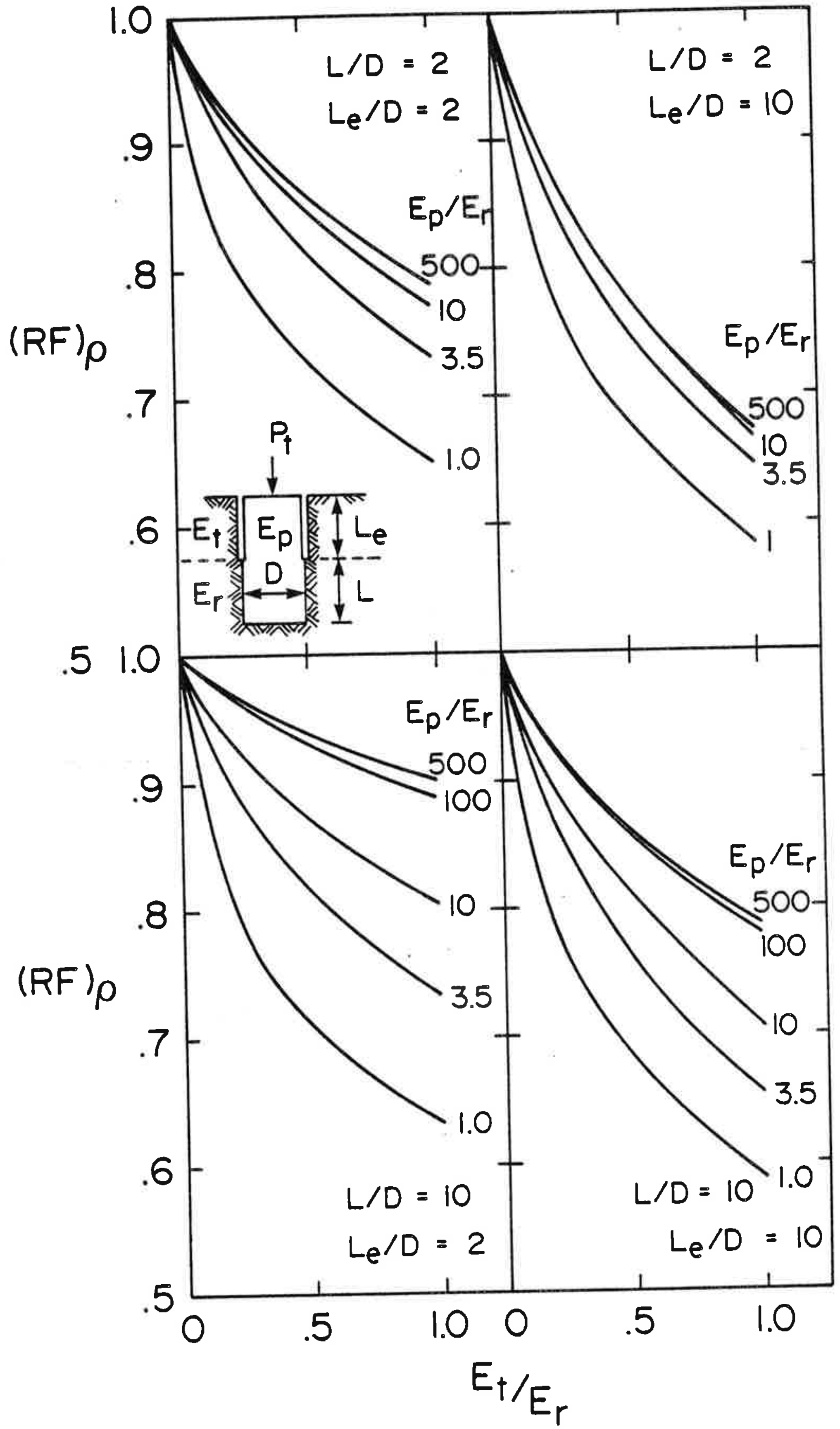


FIGURE 3.23 VARIATION IN SETTLEMENT REDUCTION FACTOR WITH MODULUS RATIO E_t/E_r

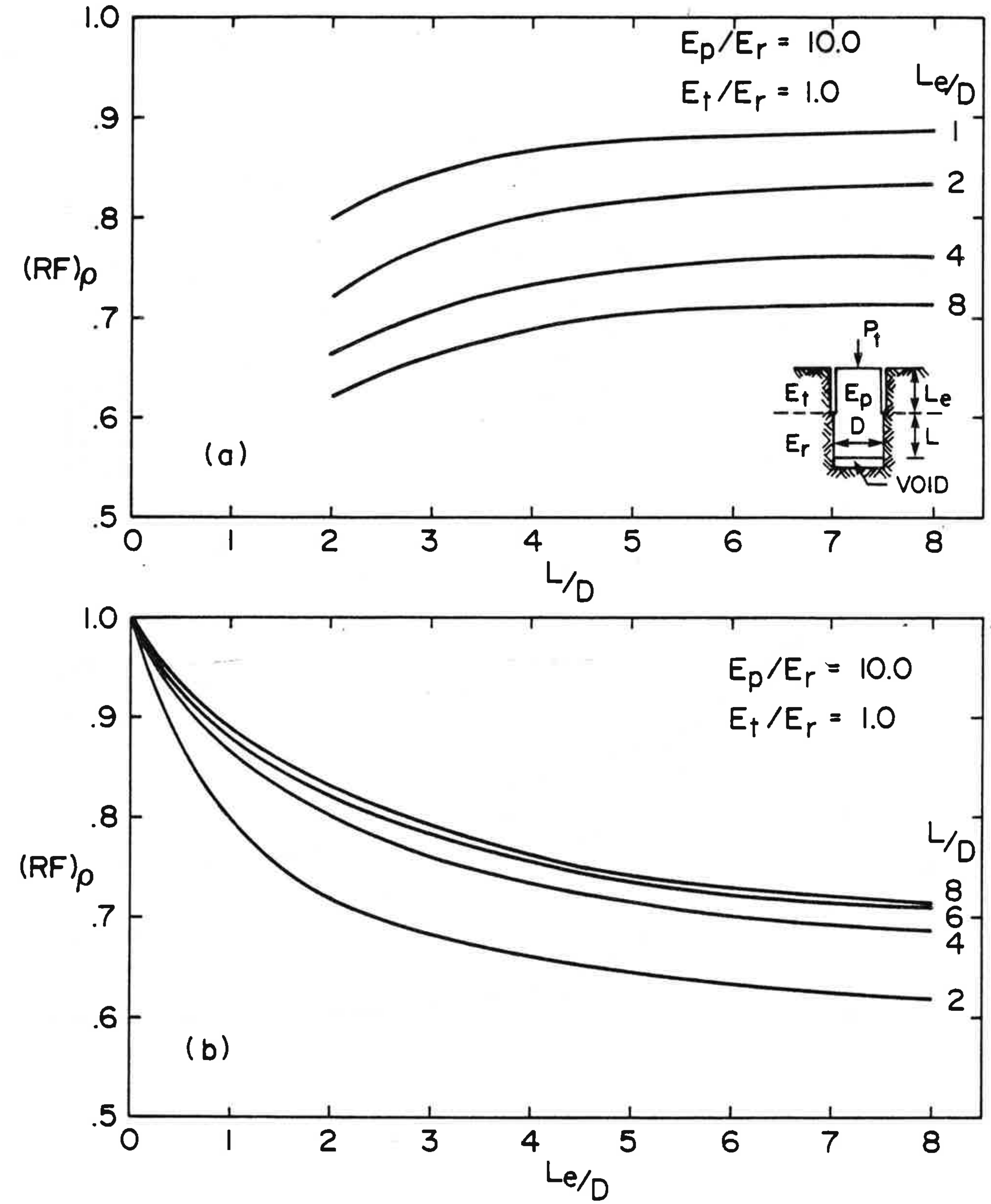
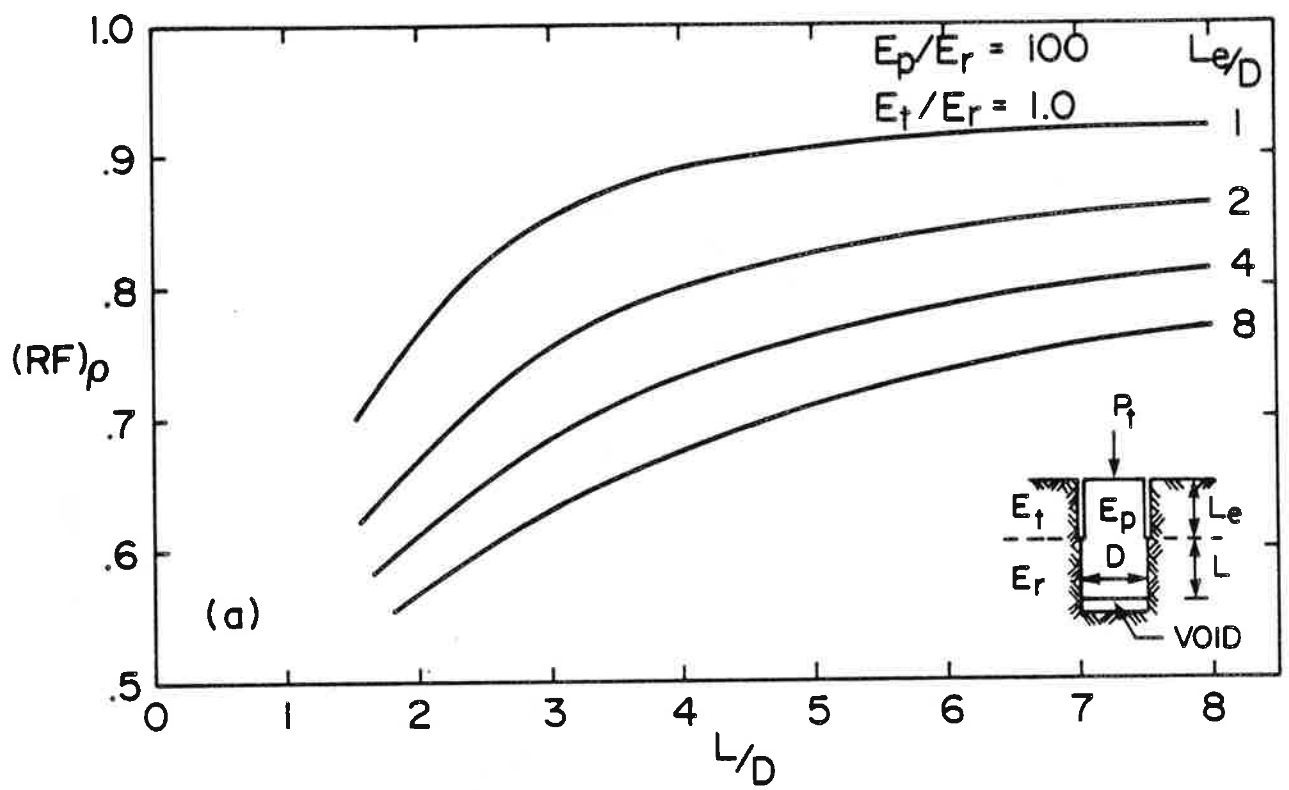


FIGURE 3.24 SETTLEMENT REDUCTION FACTOR FOR A SIDESHEAR SOCKETED PILE (E_p/E_r = 10.0, E_t/E_r = 1.0, MODIFIED FROM PELLS AND TURNER, 1979)



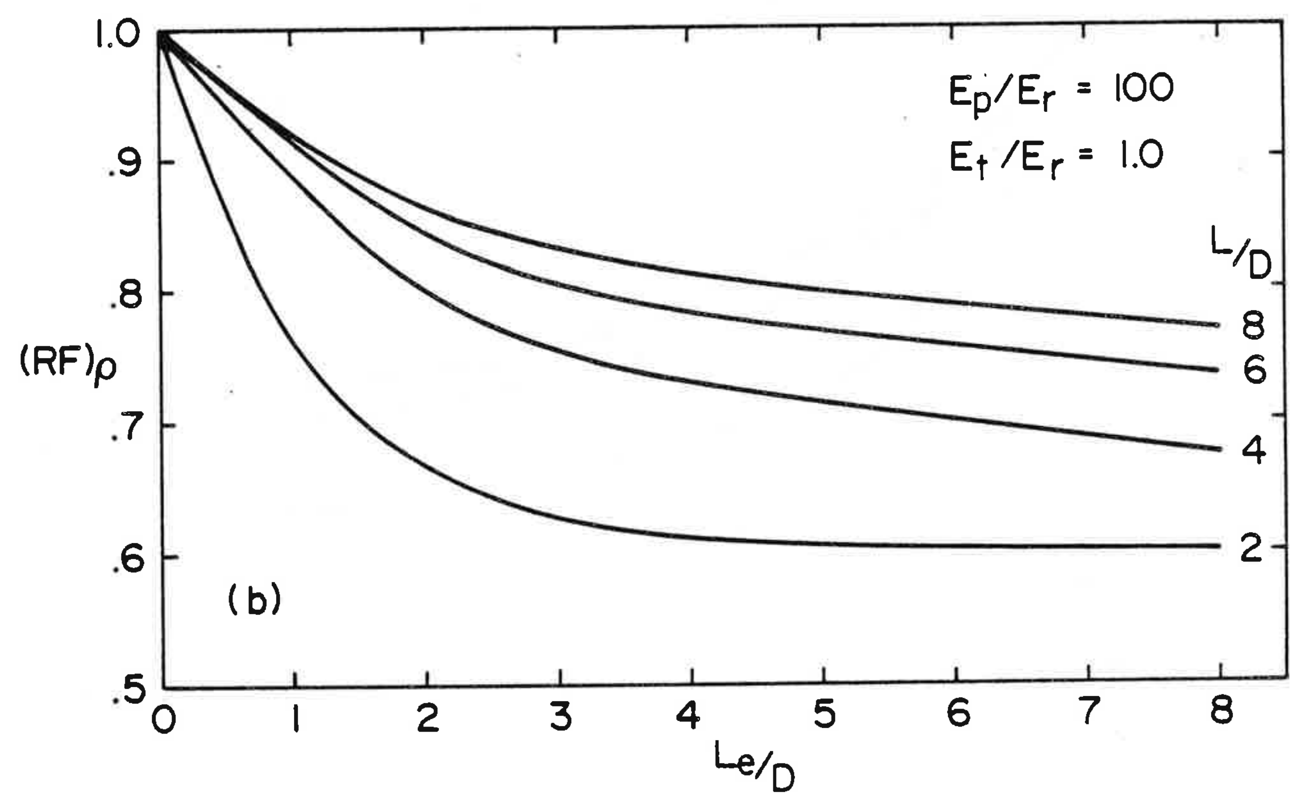


FIGURE 3.25 SETTLEMENT REDUCTION FACTOR FOR A SIDESHEAR SOCKETED PILE (E_p/E_r = 100.0, E_t/E_r = 1.0, MODIFIED FROM PELLS AND TURNER, 1979)

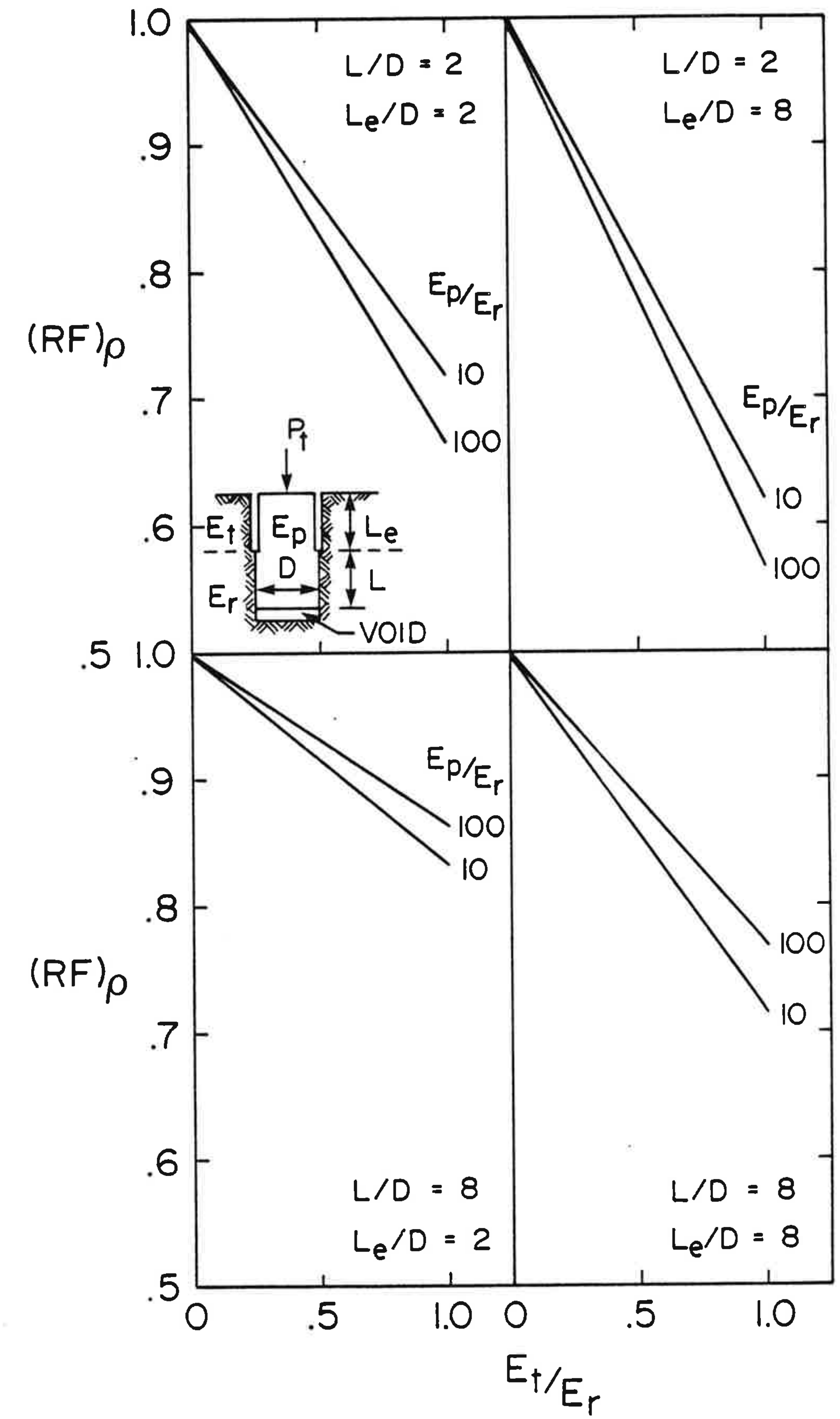


FIGURE 3.26 VARIATION IN SETTLEMENT REDUCTION FACTOR WITH MODULUS RATIO E_t/E_r, (DEDUCED FROM PELLS AND TURNER, 1979)

CHAPTER 4

EMPIRICAL CORRELATIONS AND PRELIMINARY DESIGN PARAMETERS BASED ON SOCKETED PILE TEST DATA

4.1 GENERAL

The settlement of piles socketed into rock will depend on the geometry of the socket, the applied load, the available sideshear resistance and the mass modulus of the rock. Several investigators have suggested empirical correlations between the average unconfined compressive strength of rock and the peak average sideshear resistance determined from field tests on selected piles. It seems likely that a correlation will also exist between the average unconfined compressive strength and the mass modulus of rock provided that the rock mass does not contain open joints or joints filled with a significant thickness of gouge material. Consequently, in this report, the concept of empirical correlations between unconfined compressive strength and (a) available sideshear resistance, and (b) rock mass modulus will be examined.

4.2 RELATIONSHIP BETWEEN SIDESHEAR RESISTANCE AND UNCONFINED COMPRESSIVE STRENGTH

Early attempts at establishing a relationship between measured average sideshear resistance and unconfined compressive strength of the rock were presented by Rosenberg and Journeaux (1976). While their correlation illustrated a general trend, it lacked sufficient data to draw any firm conclusions.

As more reported cases of field tests on socketed piles became available, further attempts were made to establish this correlation. Horvath (1978) compiled an extensive list of field data concerning sideshear resistance for drilled pile foundations socketed into rock. Using this accumulated data from full size sockets, small diameter plugs and laboratory model test data he proposed the relationship

$$\tau(\text{psi}) = 4\sqrt{q_u} \qquad (4.1a)$$

where τ = the available sideshear resistance; and

q_u = the unconfined compressive strength of the rock in psi

This can be rewritten

$$\tau(\text{MPa}) = 0.58\sqrt{q_u} \qquad (4.1b)$$

where q_u = the unconfined compressive strength of the rock in MPa

Horvath (1982) and Horvath et al. (1983) subsequently revised this correlation and suggested the approximation,

$$\tau(\text{MPa}) = b(q_u)^a \qquad (4.2a)$$

The parameters a,b determined by Horvath from a least squares fit of the data from both small and large diameter piers are given in Table 4.1. As noted by Horvath, these correlations are based on his interpretation of the available data for many sockets where incomplete data regarding the concrete and rock properties were reported. After considering the

correlation given in Table 4.1, Horvath suggested the approximate relationship

$$\tau(\text{MPa}) = b\sqrt{q_u} \qquad (4.2b)$$

where b = 0.2 to 0.3 and q_u is in MPa.

4.2.1 <u>Examination of Currently Available Data</u>

The validity of any empirical correlation will depend upon the quality of the data used to establish the correlation. In attempting to establish a relationship between the unconfined compressive strength of the in-situ rock and the measured value of sideshear resistance developed in a rock socket, it seems reasonable to ensure that the test data conform to some basic criteria before it is routinely incorporated into the correlation.

Due to the diverse nature in which socketed pile load test results are both conducted and reported, it becomes necessary to adopt some method to define what may be reliable and unreliable data.

The criteria adopted in this report involved separating the published data into the following categories:

(A) **Direct Data**

(1) typical values of the unconfined compressive strength of rock samples obtained from the strata in which the pile is founded are reported; and

(2) the load test was conducted in such a manner that the sideshear resistance could be determined directly (i.e., endbearing was eliminated) or the endbearing load could be determined either from strain gauges located near the base of the socket or from a load cell at the base of the socket.

(B) **Indirect Data**

(1) where either conditions (1) and/or (2) above are not satisfied; or

(2) where the method of testing the socketed pile or preliminary site investigation is suspect.

Within each of these categories a further subdivision was made:

I) Compression Sockets (diameters > 350 mm)

II) Compression Sockets (diameters < 350 mm)

III) Socket Pull-Out (Tension) Tests (diameters > 350 mm)

IV) Socket Pull-Out (Tension) Tests (diameters 150 mm - 350 mm)

V) Anchors (diameters < 150 mm).

These subdivisions are necessary to determine if there is any significant difference in behaviour among the individual groupings. The selection of the >350 mm, <350 mm diameter categories was arbitrary and seemed to be a reasonable value to distinguish large and small diameter sockets. Again, an arbitrary selection of <150 mm was used to denote rock anchors.

A careful review of the published load test data yielded Tables D1 and D2, which are given in Appendix D and are categorized into direct and indirect data respectively.

For many of the cases classified as indirect data (Table D2), the investigators did not provide values for the unconfined compressive strength of the rock strata in which the pile was founded. Some authors (eg. Rosenberg and Journeaux, 1976; Horvath, 1982) have used their own estimates of q_u in developing their correlations. For example, Horvath (1982) used the engineering classification system for rock (Deere, 1968) to estimate q_u when values were not reported. While this is a reasonable engineering approach, the writers consider that these values of q_u should be viewed with some caution and should not be given the same weight as direct data when developing a correlation.

In other cases, strength properties of the rock have been determined by methods other than unconfined compression tests on rock cores, for which interpretation and their use in the correlation (for τ_f vs q_u) remains a matter of judgement. Finally, in some of the published case histories the test procedure or the method of determining sideshear resistance of the socketed pile (eg. see Wilson, 1976; Spanovich and Garvin, 1979) raised doubts as to whether inclusion in the correlation of τ vs q_u was valid.

The data used to develop the correlations discussed in the following sections have been selected from Table D1. Test results where the reported unconfined compressive strength of rock is greater than 40 MPa have been excluded from the analysis. Several reasons are offered for their exclusion,

(1) often reported values of sideshear resistance do not represent the peak value since the load tests in strong rock (eg. $q_u > 40$ MPa) have not been carried to failure.

(2) in situations where the unconfined compressive strength of the rock exceeds that of the pile concrete, it will be the strength of the weaker material (eg. concrete) which will control the maximum value of shaft resistance that can be mobilized (Horvath and Kenney, 1982). Therefore, when the strength of the concrete (f'_c) is not explicitly given, it is difficult to justify inclusion of the data in the correlation.

4.2.2 Correlations Between τ and q_u Based on Direct Data

Assuming a functional relationship similar to that adopted by Horvath (1982),

$$\tau = b\ q_u^a \tag{4.3a}$$

the normalized sideshear resistance $\alpha = \tau/q_u$ is given by,

$$\alpha = \tau/q_u = b\ q_u^{a-1} \tag{4.3b}$$

This relationship can be rectified by a log transformation, viz.

$$\ell n\ \alpha = \ell n\ b + (a-1)\ell n\ q_u \tag{4.3c}$$

This will plot as a straight line on log-log plot having an intercept

at $q_u = 1$ ($\ell n\ q_u = 0$) of log b and a gradient (a-1). The values of these coefficients can be determined by linear regression of the variables $\ell n\ \alpha$, $\ell n\ q_u$.

Five groupings of data were examined as summarized in Table 4.2. Figure 4.1 shows the data for large (D > 350 mm) compression sockets, excluding sockets which had a roughness category R4 as defined by Table 4.3. The values of b and (a-1) from linear regression are 0.43 and -0.535. The corresponding correlation coefficient of 0.82 indicates that the correlation between α and q_u is statistically significant. Equation 4.3b assumes a particularly convenient form when (a-1) = -0.5 (i.e., a = 1/2), viz.

$$\alpha = b\ q_u^{-1/2} \tag{4.4a}$$

which gives the same form of relationship between τ and q_u as that proposed by Horvath viz.

$$\tau(\text{MPa}) = b\sqrt{q_u} \tag{4.4b}$$

Assuming a = 0.5, equation 4.3a reduces to

$$\ell n\ \alpha = \ell n\ b - 0.5\ \ell n\ q_u \tag{4.4c}$$

where only ℓn b must be determined. This is referred to as the forced equation and b is determined, by the method of least squares, to be 0.41. The simplification of taking a = 0.5 does not result in a significant difference in the correlation.

The correlation proposed by Horvath (see Eq. 4.2b) is also shown on Figure 4.1. Horvath's (1982) lower limit for b = 0.2 represents the lower bound estimate of τ for a given value of q_u. This approximation appears to provide a conservative estimate of τ for full scale compressive sockets. However, it will not provide a good indication of the expected behaviour of a socketed pile since, on average, it will underestimate the available sideshear resistance by up to a factor of two. [It should be noted that 50% of the data used in the correlation was from test which were not carried to failure and the available sideshear is in excess of that used in the correlation.]

Figure 4.2 shows the data points for sockets assessed to be of roughness R4 (see Table 4.3) and includes results from artificially roughened sockets. The correlations obtained using linear regression on Eq. 4.3c and the forced equation (Eq. 4.4) are also shown in Figure 4.2. A statistical examination of the data for the rough (R4) and regular (roughness R1, R2 or R3) sockets indicates:

(i) For the rock with 6 MPa < q_u < 10 MPa, the available sideshear for rough sockets is significantly greater than that for regular sockets at the 0.5% level (i.e., the probability that this statement is incorrect is less than 0.5%)

(ii) For rock with 0.4 MPa < q_u < 0.7 MPa, the available sideshear for the "rough" sockets does not appear to be significantly greater than that for regular sockets at the 5% level (i.e., a statement claiming that roughened sockets do have a higher available sideshear than regular sockets for this rock has a greater than 5% chance of being incorrect)

(iii) For rock with $q_u \simeq 2.3$ MPa the difference between the two data points shown in Figure 4.2 and the expected value obtained from the correlation is not statistically significant at the 5% level (i.e., we have no reason to believe that this sample does not conform with our hypothetical distribution).

Roughening of the socket has a much greater effect for higher values of q_u (> 1 MPa) than for lower values of q_u (1 MPa). This is because forming a regular socket in very soft rock (q_u < 1 MPa) generates considerable roughness and hence many of the regular sockets had a roughness, R3, which is close to that of the "rough" (R4) sockets. In stronger rock, the regular socket is generally much smoother (typically R1-R2) than both a regular socket in soft rock and a rough (R4) socket in the stronger rock. It would be desirable to develop correlations for each roughness class but at present there is insufficient data to do so.

The forced equation provides a reasonable approximation to the observed data while retaining the same form of equation as used for the regular sockets. This approximation will be somewhat conservative for q_u > 5 MPa.

Figures 4.3, 4.4 and 4.5 show both the regression correlations and the forced fit approximation for the three remaining categories of data. In each case, the simplified forced fit curve (a = 0.5) provides a good approximation to the observed behaviour. Consideration of compression and tension sockets and both small and large sockets does not greatly affect the correlation although the approximation for large regular compression sockets (Figure 4.1 and Table 4.2) would provide estimates of α or τ, 10% less than would be obtained using the approximation based on all regular sockets (Figure 4.5 and Table 4.2). There is insufficient data to determine whether this difference is real or simply represents the difference between different samples from the same population. In any event, the difference is small and since the approximation in Figure 4.5 is based on a much larger data set (64 points versus 28), this will be adopted for all regular sockets.

On the basis of this study, the expected value of the available sideshear resistance τ (in MPa) for a given average value of q_u (in MPa) is given by

$$\bar{\tau}(\text{MPa}) = 0.45\sqrt{q_u} \quad \text{for regular sockets (roughness less than R4)} \tag{4.5a}$$

$$\bar{\tau}(\text{MPa}) = 0.6\sqrt{q_u} \quad \text{for rough sockets (R4)} \tag{4.5b}$$

If there is any doubt as to whether the socket is rough, then Eq. 4.5a should be adopted.

4.2.3 Suggested Design Values for Sideshear Resistance (τ_d)

For the purposes of designing for allowable settlement of foundations and structures (the serviceability limit), Meyerhof (1984) has recommended the use of a partial factor of 0.7 to ensure an adequate reliability for serviceability estimates. If this partial factor is applied to the expected sideshear resistance $\bar{\tau}$ deduced from Eq. 4.5, then the value of sideshear resistance used in the settlement calculation, τ_d, would be,

$$\tau_d = 0.32\sqrt{q_u} \quad \text{for regular sockets (roughness less than R4)} \tag{4.6a}$$

$$\tau_d = 0.42\sqrt{q_u} \quad \text{for rough sockets (R4)} \tag{4.6b}$$

where τ_d and q_u are both in MPa.

If the sideshear resistance τ is assumed to be log-normally distributed about the expected values defined by Eqs. 4.5a and 4.5b, then the standard deviation in $\ell n\ \tau$ is 0.42 and 0.29 for regular and rough sockets respectively. Under these conditions, the values of τ_d determined from Eq. 4.6 may be expected to be conservative in approximately 80% of cases for regular sockets and 90% of cases for rough sockets. It is considered that these probabilities would be considered acceptable for calculation of the serviceability limit state.

For the purpose of calculating the ultimate limit state, Meyerhof (1984) recommends a partial factor of 0.5 for soil cohesion. If this partial factor is applied to the expected sideshear resistance given by Eqs. 4.5a or 4.5b, then the resulting values of τ used in estimating the ultimate limit state may be expected to be conservative in at least 95% and 99% of cases for regular and rough sockets respectively.

The value of expected available sideshear resistance $\bar{\tau}$ given by

Eqs. 4.5a and 4.5b are plotted together with the field data on Figures 4.6 and 4.7 respectively. Also shown are the values of τ obtained by applying a partial factor of 0.7 and 0.5 to the expected available side-shear resistance.

4.3 RELATIONSHIP BETWEEN ROCK MASS MODULUS AND UNCONFINED COMPRESSIVE STRENGTH

It may be expected that there will be a relationship between the mass modulus and unconfined compressive strength of weak rock provided that the rock mass behaviour is not governed by open joints or joints filled with very compressible gouge material. The development of such a relationship requires a knowledge of the average unconfined compressive strength at a site and the mass modulus mobilized in an actual pile load test. Many of the case records classified as providing "direct data" in the previous section have included a load-displacement curve for the individual load tests. The rock mass modulus was backfigured from these case records as described in the following section.

4.3.1 Analysis of Available Data

4.3.1.1 Method of Calculating the Backfigured Rock Modulus (E_r)

The method used to calculate the backfigured rock modulus from field load tests was similar to the approach adopted by Pells and Turner (1979). A brief outline of the method is summarized as follows:

(1) From the load-displacement curve for the particular field load test, identify the region over which the socket displacements show a

linear relationship with the applied axial load. Select a load P and corresponding displacement ρ from this portion of the load-deflection curve. Often this linear region is observed up to 50% of the total applied load (eg. Pells et al., 1980; Williams, 1980).

(2) Adjust the measured displacement ρ, to take account of any elastic compression/extension which occurs within portions of the pile which are not part of the actual socketed test section.

Accounting for elastic compression/extension requires a knowledge of the composite pile modulus E_p which may be deduced from a knowledge of the concrete modulus, the steel modulus and the percentage of steel in the pile cross-section.

(3) Estimate the ratio of pile modulus to rock modulus (referred to as the modulus ratio; E_p/E_r); this generally involves making an initial guess regarding the rock mass modulus, E_r.

(4) Select the appropriate elastic solution for the socket type (endbearing and sideshear or sideshear only) and modulus ratio (E_p/E_r) and determine the displacement influence factor for the given geometry (i.e., given length/diameter ratio; L/D).

(5) Calculate the backfigured modulus from the equation,

$$E_r = \frac{PI}{\rho D}\,(RF)_\rho \tag{4.7}$$

in which

E_r = rock mass modulus

ρ = displacement at head of socketed pile, at load P

P = applied load giving rise to displacement ρ

a = radius of pile

$D = 2a$ = diameter of the socketed pile

$(RF)_\rho$ = a settlement reduction factor, to account for recessment of the socket below the top surface of the rock (see Chapter 3, section 6)

I_ρ = displacement influence factor determined from appropriate elastic solutions.

(6) Calculate the modulus ratio E_p/E_r and compare with that assumed in Step 3. Repeat Steps 3 to 6 if necessary.

While the above steps outline the general procedure used in this report, a number of points require further comment.

4.3.1.2 Calculation of the Pile Modulus (E_p)

In some cases, the concrete modulus and reinforcing details were reported and so a direct estimate of the pile modulus E_p could be made from

$$E_p = \frac{A_{steel}E_{steel} + A_{concrete}E_{concrete}}{A_{steel} + A_{concrete}} \tag{4.8}$$

in which

$A_{steel}, A_{concrete}$ are the cross-sectional area of steel and concrete respectively

$E_{steel}, E_{concrete}$ are the Young's modulus values for steel and concrete respectively, where the modulus of the concrete is a secant value over the expected strain range.

In cases where $E_{concrete}$ or A_{steel} were not reported, an estimate of the quantities was made. Table 4.4 gives typical reported values of concrete compressive strengths and modulus. On the basis of this data, a "typical" value for $E_{concrete}$ of 35 GPa was adopted when data was not provided.

In sockets where only minimal reinforcing was used to support instrumentation, its influence on the overall stiffness of the pile may be minor (Williams, 1980) and therefore has been neglected in the calculations. In other cases, substantial reinforcing steel (or a rolled steel section) has been used in the test pile and this contribution to the pile modulus will be significant. For cases where details regarding reinforcement have not been explicitly given, an estimate of the area of steel has been made. Using available information (eg. Glos, personal communication, 1983; Matich and Kozicki, 1967 etc.), an area of steel equivalent to approximately 6% of the total cross-sectional area of the concrete pile was selected as being representative.

4.3.1.3 Sources of Error in Modulus Calculations

The following are thought to be the major potential sources of error in the backfigured modulus values:

1) Difficulties in scaling values from the load-displacement curve. These difficulties may arise from distortion of the graph during reduction for publication. Also, errors resulting from scaling values from the curve. In order to minimize the latter, several values of P and ρ were obtained (where possible) and a representative value of E_r was then backfigured.

2) Errors due to redefining the origin of the load-displacement curve when the pile exhibits some initial nonlinearity due to "bedding in" of the test section (Pells, Rowe and Turner, 1980).

3) Errors involved in selecting a value of pile modulus and determining the elastic compression of any freestanding (or cased) portions above the socketed test section.

4) Errors involved using elastic solutions, viz.

   i) inaccuracies due to scaling values from charts

   ii) interpolation (or extrapolation) of values which may not be explicitly defined on these charts.

In discussing these potential sources of error, it has been implicitly assumed that the load-displacement curves given in the reported case histories have been accurately plotted.

4.3.2 Comments on Backfigured Rock Modulus Values

Values of the backfigured rock modulus for sockets where load-displacement curves were available are given in Appendix D (Table D3). This table also contains the backfigured modulus calculated by the original investigators. Although reasonable agreement exists between the two independently calculated modulus values, some cases indicated significant differences. It is believed that these differences may be explained in part by the factors given in the previous section concerning errors. For example, the modulus calculated for socket B4 (Pells et al., 1980) has been rechecked, but still differs from the original investigators' value by a factor of 2. It is considered that the original calculation is incorrect. The difference between modulus values deduced for Horvath's

(1982) and Horvath et al.'s (1983) field tests appear to be largely due to a difference in the values of I used in Eq. 4.7.

4.3.3 Correlation Between E_r and q_u

Table D4 summarizes the backfigured modulus (E_r) and other relevant data from the socketed pile load tests. The data in this table represents the mean and standard deviation for the various rock types and site locations where load tests have been conducted.

The relationship between the mean values of backfigured modulus (E_r) and unconfined compressive strength of the rock (q_u) have been plotted as illustrated in Figures 4.8 and 4.9. In some cases the deviation about the mean value of backfigured modulus for a particular site is quite significant as shown in Figure 4.8. This may reflect the local variations in geological conditions but in some instances (eg. Williams, 1980) it may also be due to the differences in the testing procedures used.

When the mean values for the backfigured modulus (E_r) and unconfined compressive strength are plotted using logarithmic axes, a near linear relationship appears to exist between the data (see Figure 4.9). Assuming that E_r can be related to q_u by an equation of the form,

$$E_r = bq_u^a \qquad (4.9a)$$

this can then be rectified by a log transformation, i.e.,

$$\ell n\ E_r = \ell n\ b + a\ \ell n\ q_u \qquad (4.9b)$$

When the data shown in Figure 4.9 is used in a linear regression analysis, the coefficients a,b were found to be 0.48, 221 respectively. For a value of a = 0.5, the line of best fit using a least squares analysis gives a value of b = 215. Thus, a reasonable approximation between the expected value of rock mass modulus ($\bar{E}_r$) and unconfined compressive strength (q_u) is given by,

$$\bar{E}_r = 215\sqrt{q_u} \qquad (4.10)$$

provided that q_u is representative of the rock mass. These correlations should not be used if open joints are present. Table 4.5 summarizes the results of this statistical analysis.

4.3.4 Suggested Design Value for Rock Mass Modulus (E_d)

As mentioned in section 4.3.2, Meyerhof (1984) has recommended the use of a partial factor of 0.7 for the serviceability limit state. He has noted that such a factor is necessary due to uncertainty and variability of in-situ soil-structure stiffness. In order to design for the allowable settlement of a socketed pile foundation, this partial factor may be applied to the empirical correlation for rock mass modulus given by Eq. 4.10.

Therefore, a design value which could be used for settlement calculations can be deduced from,

$$E_d = 150\sqrt{q_u} \qquad (4.11)$$

where E_d and q_u are both in MPa.

If it is assumed that the expected rock mass modulus given by Eq. 4.10 is log-normally distributed, then the standard deviation in $\ell n\ E_r$ is 0.557. As such, the design values of E_d obtained from Eq. 4.11 would be expected to be conservative in approximately 63% of cases.

The expected modulus obtained from Eq. 4.10 has been plotted on Figure 4.9. The suggested relationship given by Eq. 4.11 is shown on this figure.

4.4 INVESTIGATION OF OTHER CORRELATIONS

As mentioned in Chapter 2, there have been numerous attempts to establish correlations between particular rock properties (eg. index properties) and other in-situ rock properties. Table 4.6 summarizes some of the more common relationships which have been presented in the literature.

The majority of correlations shown in this table involve correlations developed between rock modulus and other rock properties.

Relationships developed specifically from data obtained from socketed pile load test programmes (eg. Douglas, 1980; Johnston et al., 1980; Williams, 1980 etc.) appear promising however, they are based on results from a limited number of load tests and rock types. At present, it is difficult to attempt further improvements to such correlations since many of the reported socketed pile case histories do not provide sufficient detail regarding in-situ rock properties at the test site.

4.5 SUMMARY

Test results obtained from available socketed pile case histories have been presented. The information given in these cases histories was carefully reviewed and subsequently placed into one of two distinct categories. Consequently, data from the category which conformed to certain basic requirements concerning its reliability was later used to develop the empirical correlations. Within each category, a further subdivision was made between the type of test and socket diameter so that any differences in behaviour among these groupings could be examined.

Using the appropriate data, empirical correlations between the unconfined strength of intact rock and (1) the measured (average) sideshear resistance and (2) the backfigured rock mass modulus have been developed.

On the basis of a least squares fit of the correlated data, several simple equations have been suggested which can be used to select preliminary values of sideshear resistance and rock mass modulus for socketed pile design. Any design based on empirical correlations should be validated by proof loading.

TABLE 4.1 COEFFICIENTS a AND b FOR CORRELATION OF SHAFT RESISTANCE AND COMPRESSIVE STRENGTH (after Horvath, 1982)

DESCRIPTION	TOTAL DATA POINTS	a	b
Large Diameter Piers	83	0.45	0.33
Small Diameter Piers and Anchors	119	0.52	0.35
All Data	202	0.52	0.31

τ and q_u in MPa units

TABLE 4.2 VALUE OF CORRELATION COEFFICIENT FROM LINEAR REGRESSION FOR $\ell n\ q_u$ VS $\ell n\ \alpha$ FOR VARIOUS CASES

Case Description	Equation From Linear Regression	Equation From Forced Curve	Correlation Coefficient (for linear regression) r	No. of Data Points Considered
	$\alpha = bq_u^{a-1}$			
1 Compression Sockets (Dia. > 350 mm) Excl. Rough Sockets	$\alpha = .43q_u^{-.535}$	$\alpha = .41q_u^{-\frac{1}{2}}$	-.824	28
2 All Rough Sockets (R4 - Roughness Classification)	$\alpha = .55q_u^{-.389}$	$\alpha = .60q_u^{-\frac{1}{2}}$	-.897	12
3 Compression and Tension Sockets (Dia. > 350 mm) Incl. Rough Sockets	$\alpha = .45q_u^{-.494}$	$\alpha = .46q_u^{-\frac{1}{2}}$	-.812	49
4 All Direct Data (Dia. > & < 350 mm) Incl. Rough Sockets	$\alpha = .41q_u^{-.461}$	$\alpha = .47q_u^{-\frac{1}{2}}$	-.785	76
5 All Direct Data (Dia. > & < 350 mm) Excl. Rough Sockets	$\alpha = .40q_u^{-.43}$	$\alpha = .45q_u^{-\frac{1}{2}}$	-.738	64

TABLE 4.3 ROUGHNESS CLASSIFICATION (after Pells, Rowe & Turner, 1980)

ROUGHNESS CLASS	DESCRIPTION
R1	Straight, smooth sided socket, grooves or indentations less than 1.00 mm deep.
R2	Grooves of depth 1-4 mm, width greater than 2 mm, at spacing 50 mm to 200 mm.
R3	Grooves of depth 4-10 mm, width greater than 5 mm, at spacing 50 mm to 200 mm.
R4	Grooves or undulations of depth greater than 10 mm, width greater than 10 mm at spacing 50 mm to 200 mm.

TABLE 4.4 REPORTED VALUES OF COMPRESSIVE STRENGTH (f_c') AND YOUNG'S MODULUS (E_c) OF CONCRETE

DATA	$E_{concrete}$ (GPa)	f_c' (MPa)	REMARKS
Glos and Briggs (1983)	26.8	41.5	- tests on concrete cylinder obtained at time of pouring sockets (tested at 28 days)
Horvath (1982)	37.1	53.7	- E_c is the secant Young's modulus at .45 f_c'
Williams (1980)	35	40.5	- concrete cylinders obtained at time of pouring sockets (S1,S3,S5)
Webb and Davies (1980)	35	53	- cube strength of the concrete from which the test piles were constructed

TABLE 4.5 RESULTS OF THE STATISTICAL ANALYSIS FOR $\ell n\ q_u$ AND $\ell n\ E_r$

CORRELATION	EQUATION FROM LINEAR REGRESSION	EQUATION FROM FORCED CURVE	CORRELATION COEFFICIENT (FOR LINEAR REGRESSION) r	STANDARD DEVIATION IN $\ell n\ E_r$	NO. OF DATA POINTS CONSIDERED
Using mean values of q_u and E_r for the test sites included	$E_r=221\ q_u^{.48}$	$E_r=215\ q_u^{.5}$	.77	.557	19

TABLE 4.6 PUBLISHED CORRELATIONS BETWEEN ROCK PROPERTIES AND ROCK MODULUS

Author	Type of Correlation Developed	Comments
	ROCK MODULUS	
1) Deere (1968)	$E_{t_{50}}$ vs q_u	- $E_{t_{50}}$ = laboratory tangent modulus at 50% ultimate strength - correlations developed for various intact rock types
2) Coon & Merritt (1970)	(a) E_m vs RQD (b) $E_m/E_{t_{50}}$ vs RQD (c) $E_m/E_{t_{50}}$ vs Seismic Velocity Index	- E_m from in-situ jack tests - Bieniawski (1978) suggests these correlations unreliable due to uncertainties in RQD and Velocity Index
3) Hobbs (1975)	E vs q_u	- various methods used to determine E - correlation developed for sedimentary rocks in U.K.
4) Bieniawski (1978)	(a) $E_m/E_{t_{50}}$ vs RMR (b) E_m vs RMR	- correlation (b) was found to give less scatter of results than (a) - RMR defined by 6 in-situ rock parameters - Chappell and Maurice (1980) question the adequacy of correlation (b)
5) Kulhawy (1978)	(a) E_{mass}/E_{intact} vs Discontinuity spacing (b) E_{mass}/E_{intact} vs RQD	- based on a theoretical study - based on a theoretical study
6) Douglas (1980)	E vs moisture content	- E determined by field and laboratory methods
7) Johnston et al. (1980)	E_{mass} vs moisture content	- E_{mass} backfigured from pile load tests
8) Chappell & Maurice(1980)	E_{mass}/E_{intact} vs no. of joints	- based on a theoretical study
9) Williams (1980)	E_{mass}/E_{intact} vs joint frequency	- based on both field and theoretical studies
10) Williams & Pells (1981)	E_{mass} vs $E_{press.}$	- E_{mass} - backfigured from pile load tests; $E_{press.}$ from pressuremeter results
	ENDBEARING PRESSURE	
11) Peck et al. (1974)	$q_{b_{allow.}}$ vs RQD	
12) Pells et al. (1978)	$q_{b_{measured}}/q_{b_{allow.}}$ vs RQD	

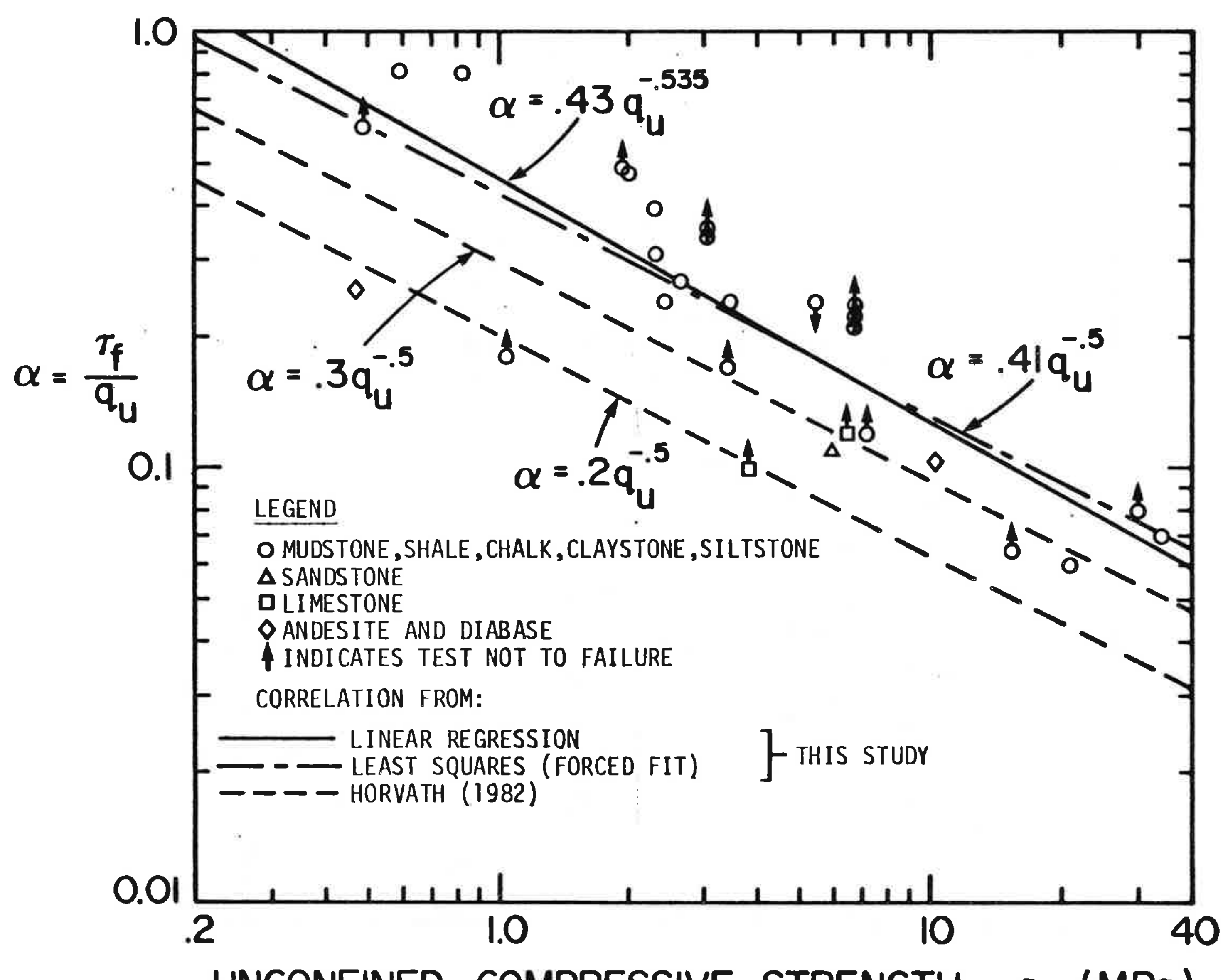


FIGURE 4.1 CORRELATIONS FOR COMPRESSION SOCKETS (DIA. > 350 mm, EXCLUDING ROUGH SOCKETS)

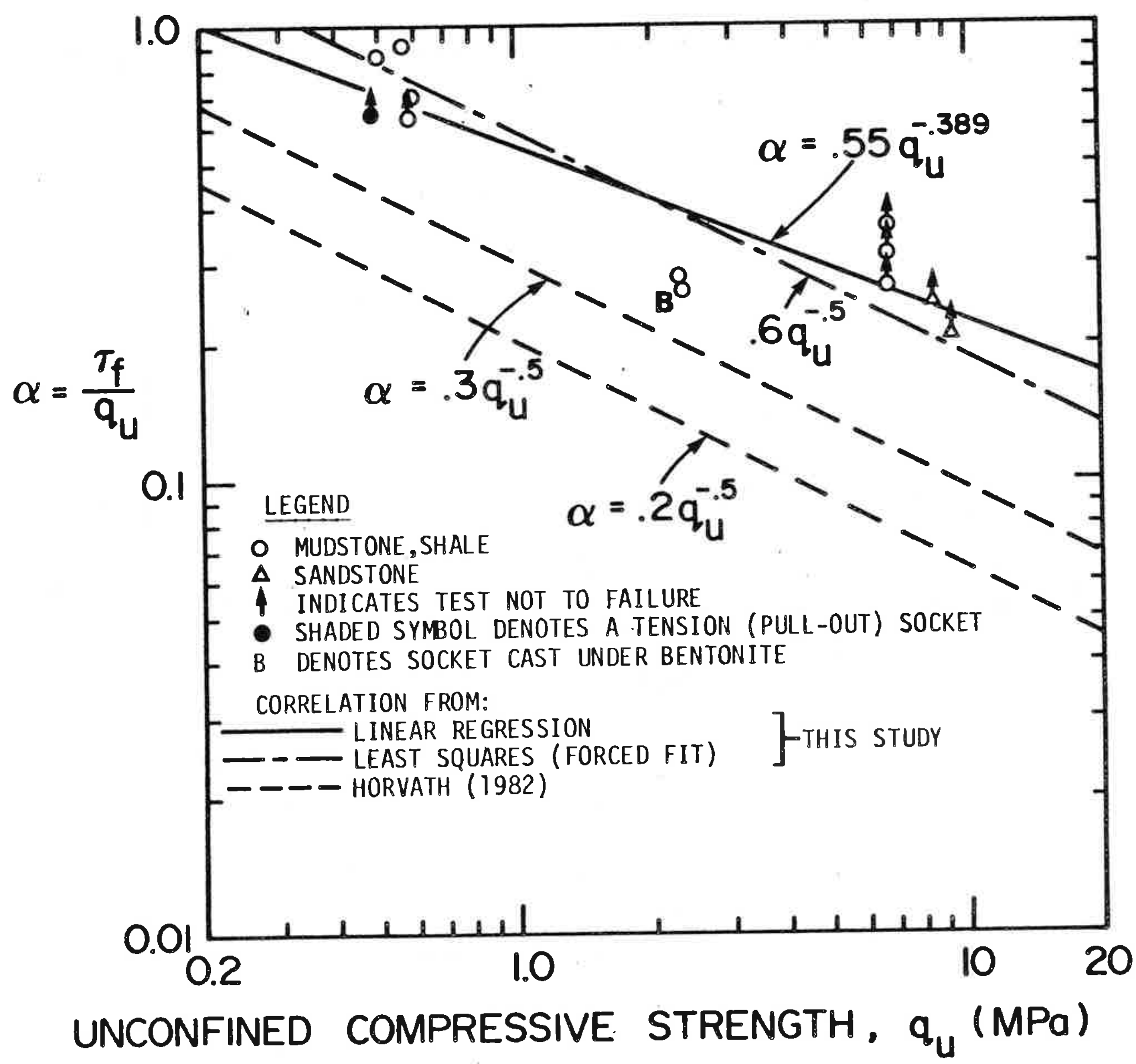


FIGURE 4.2 CORRELATIONS FOR ROUGH COMPRESSION SOCKETS ONLY (R4 - ROUGHNESS CLASSIFICATION, DIA > 350)

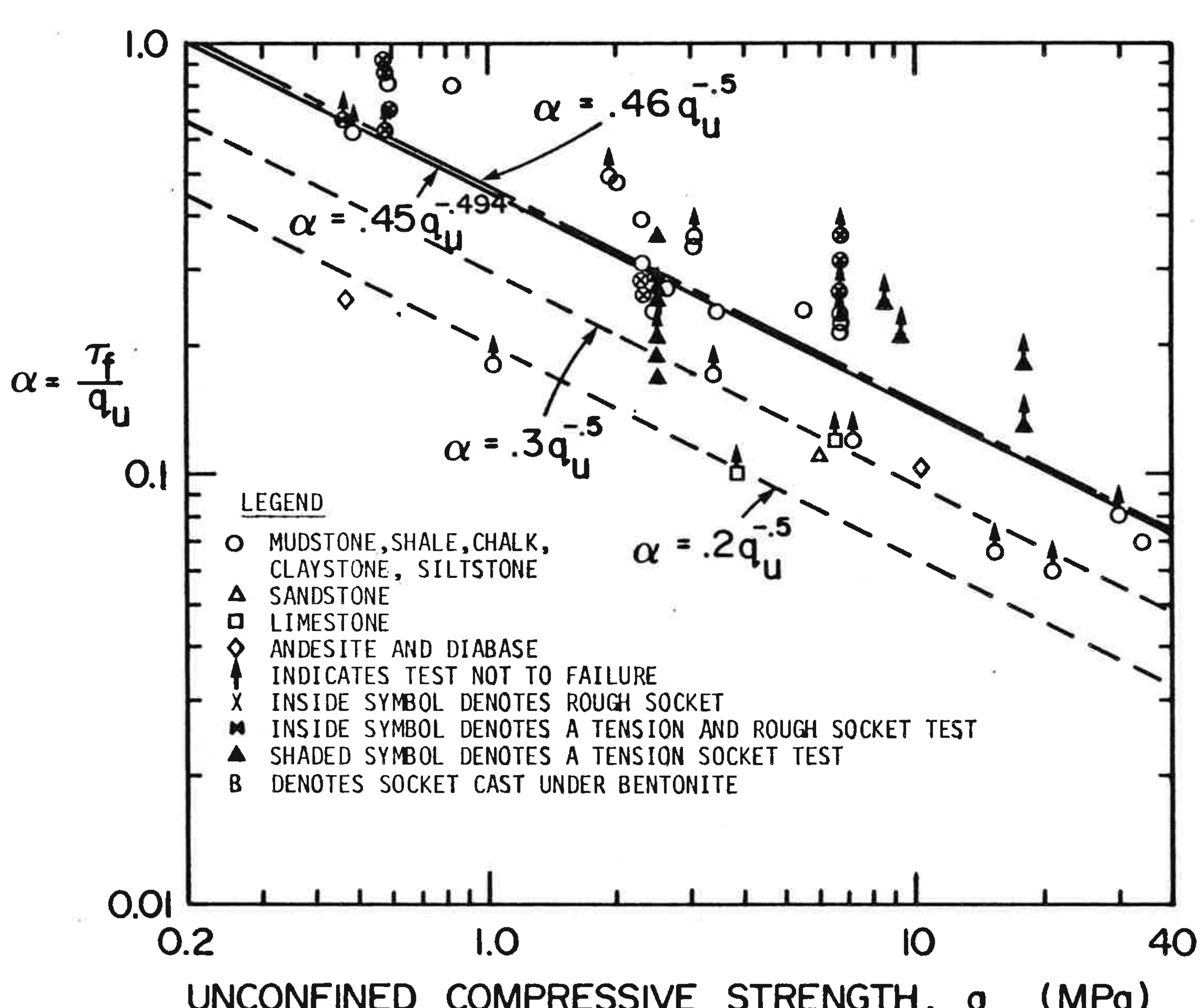


FIGURE 4.3 CORRELATIONS FOR COMPRESSION AND TENSION SOCKETS (DIA. > 350 mm, INCL. ROUGH SOCKETS)

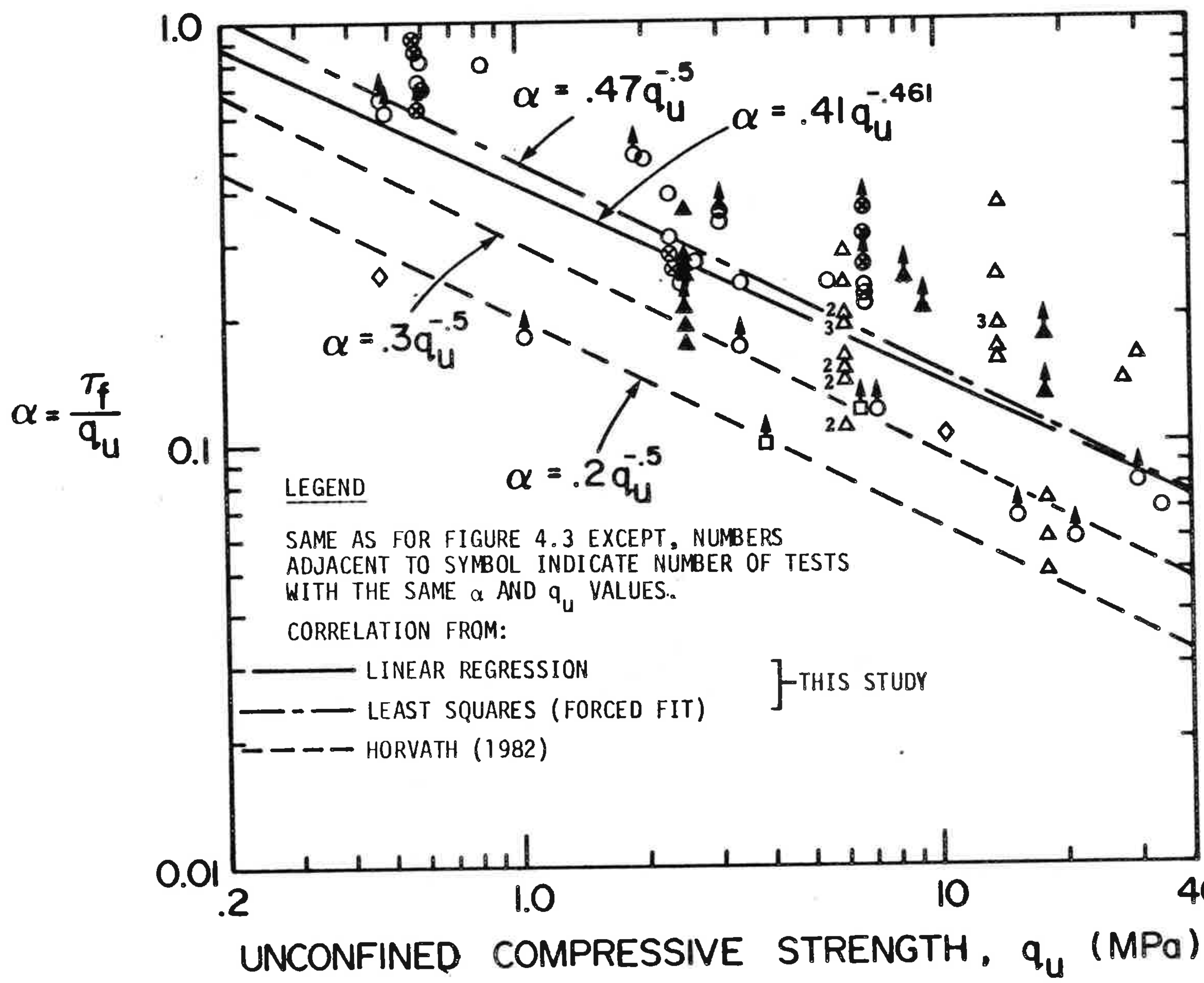


FIGURE 4.4 CORRELATIONS FOR ALL DIRECT DATA (DIA. > & < 350 mm, INCL. ROUGH SOCKETS)

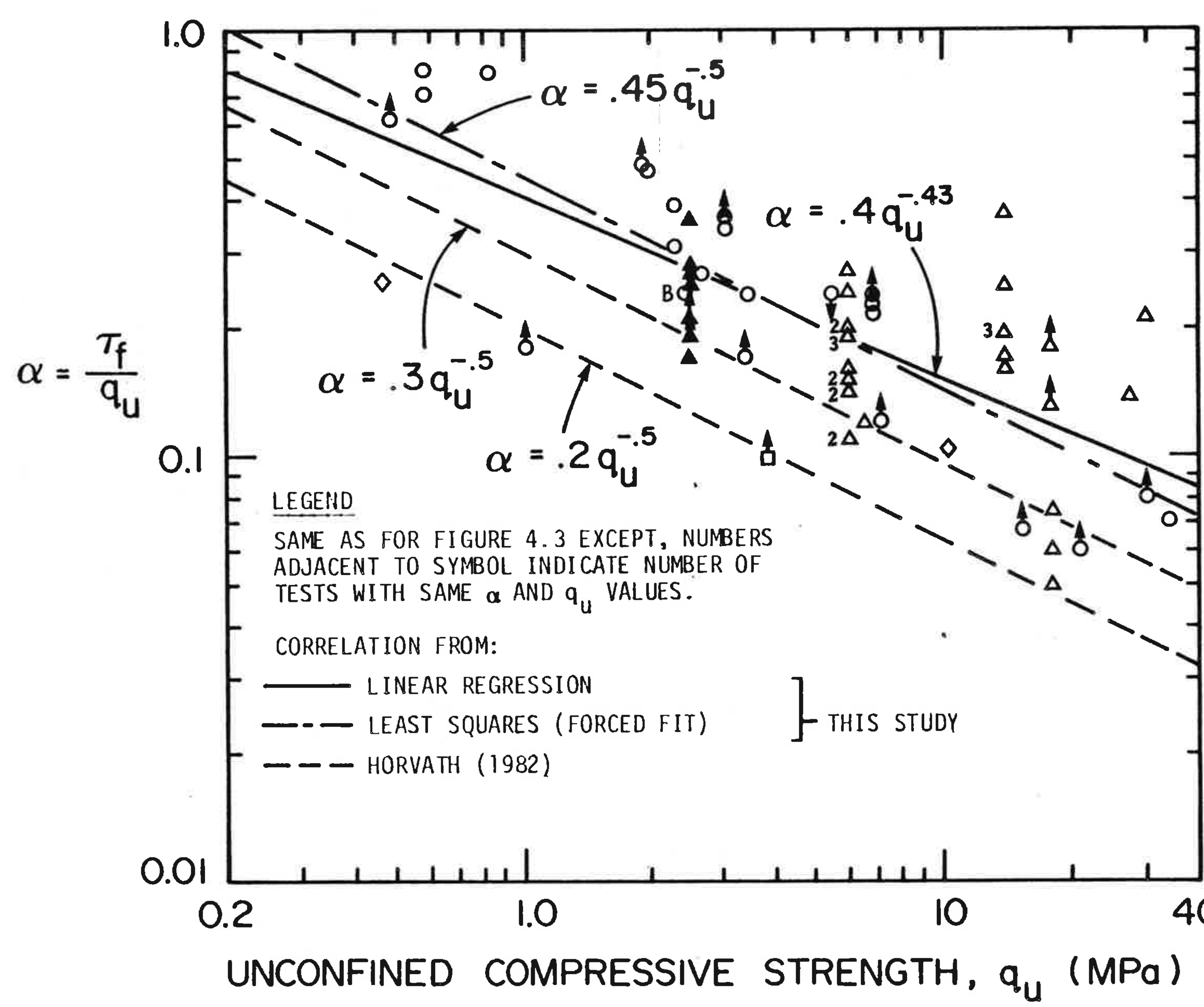


FIGURE 4.5 CORRELATIONS FOR ALL DIRECT DATA (DIA. < & > 350 mm, EXCL. ROUGH SOCKETS)

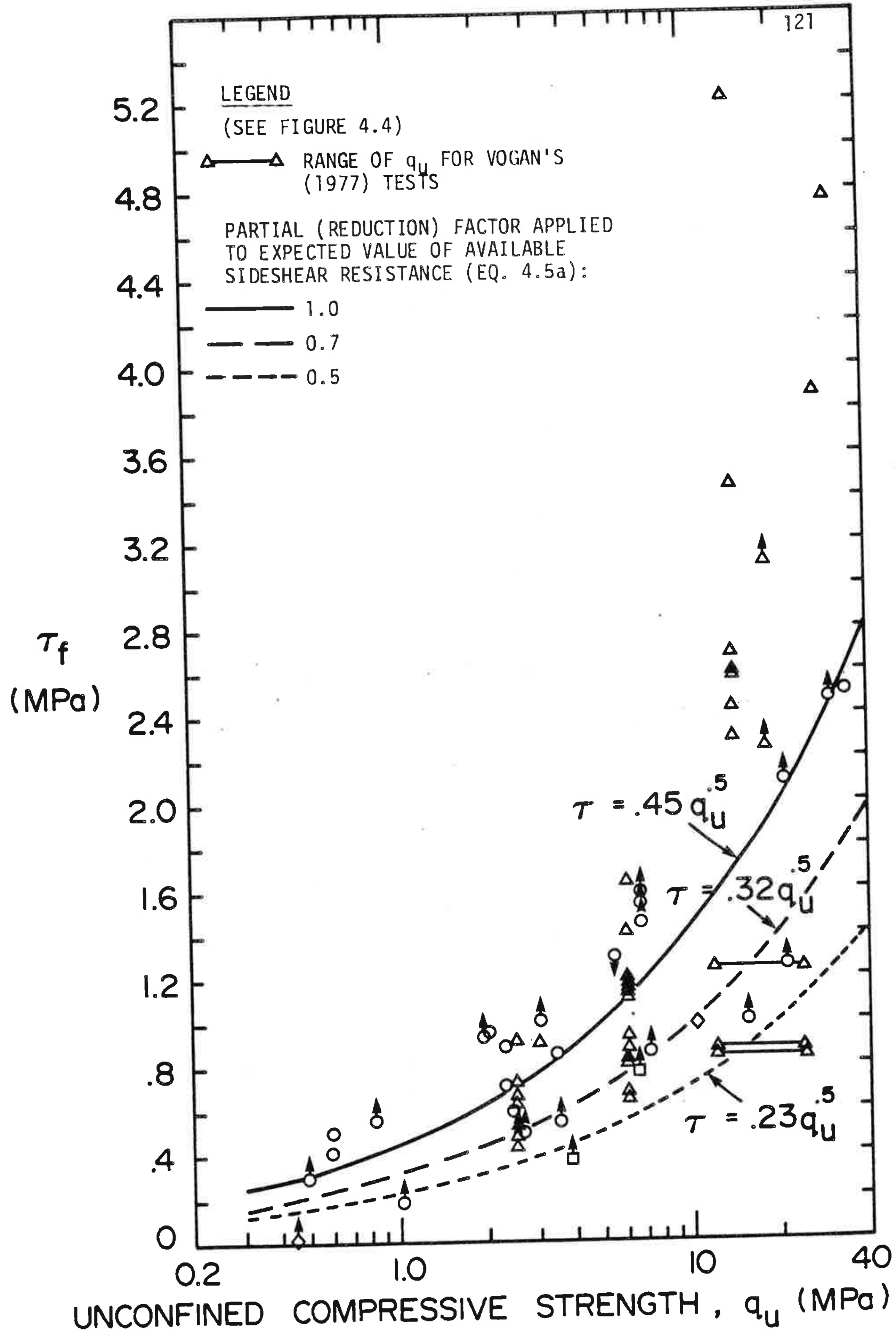


FIGURE 4.6 CORRELATIONS BETWEEN $\bar{\tau}$ AND q_u FOR ALL DIRECT DATA (DIA. < & > 350 mm, EXCL. ROUGH SOCKETS)

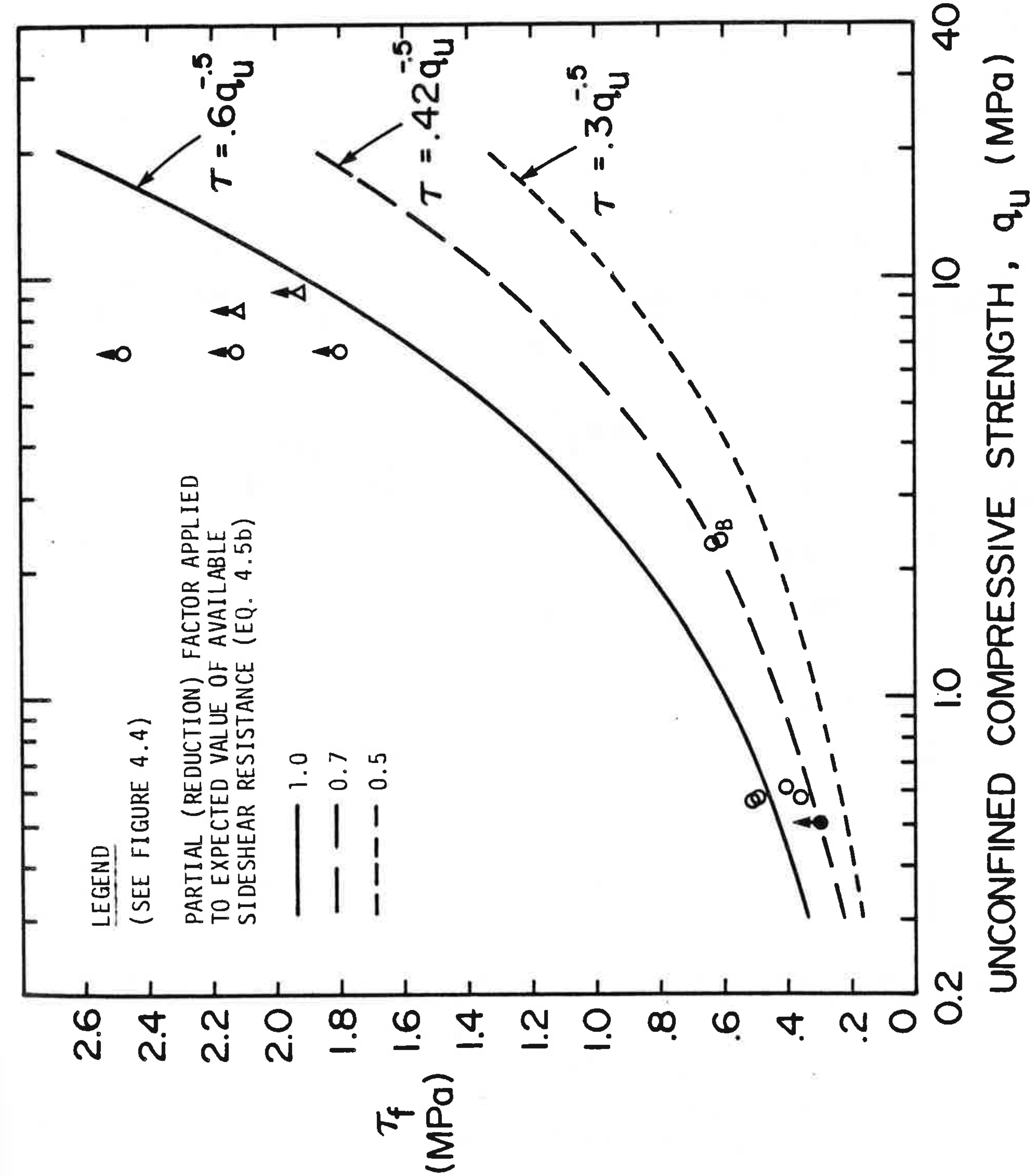


FIGURE 4.7 CORRELATIONS BETWEEN $\bar{\tau}$ AND q_u FOR ROUGH SOCKETS ONLY

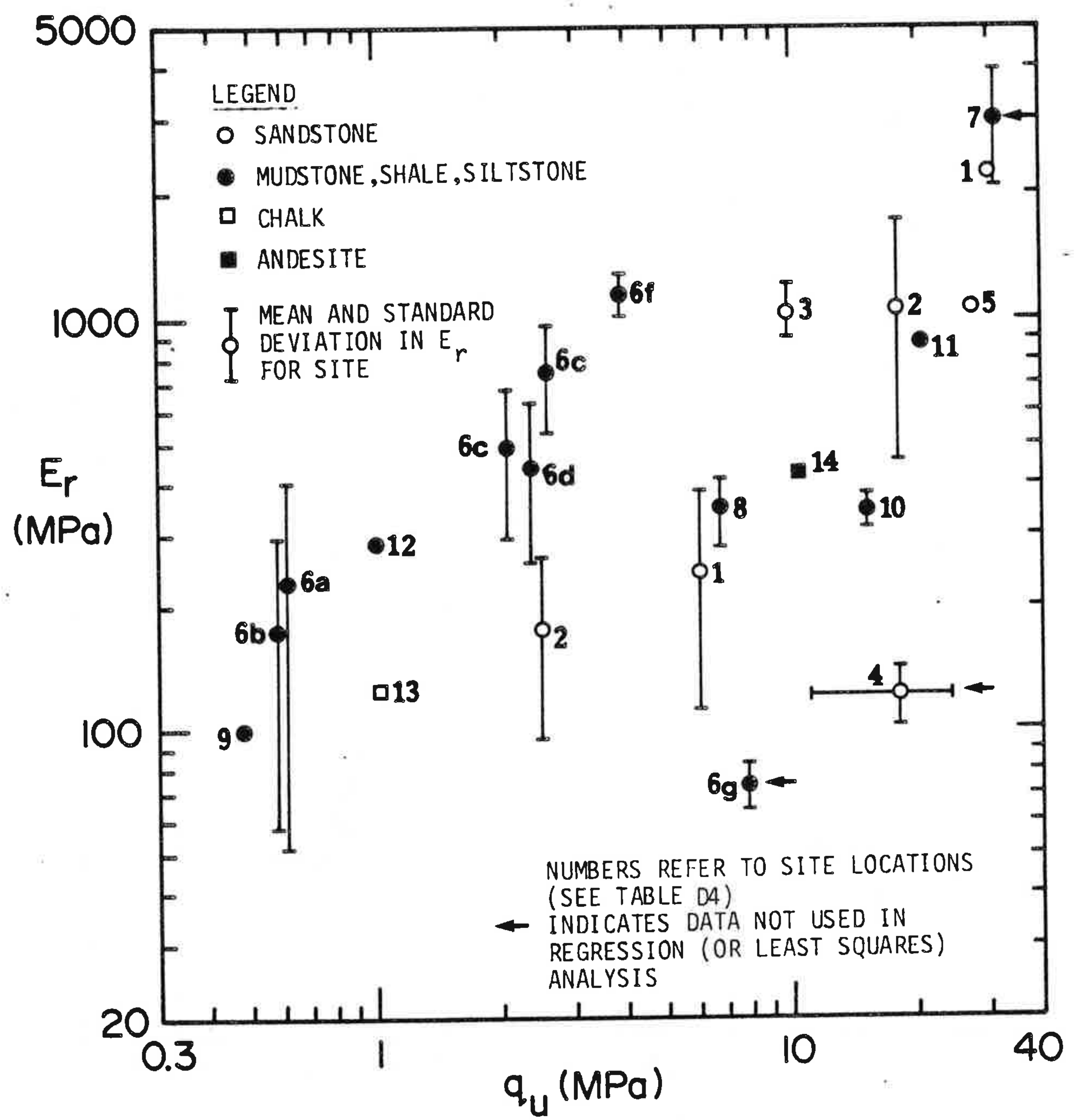


FIGURE 4.8 MEAN AND STANDARD DEVIATION OF BACKFIGURED ROCK MASS MODULUS (E_r) FOR AVERAGE VALUES OF q_u FROM VARIOUS TEST SITES

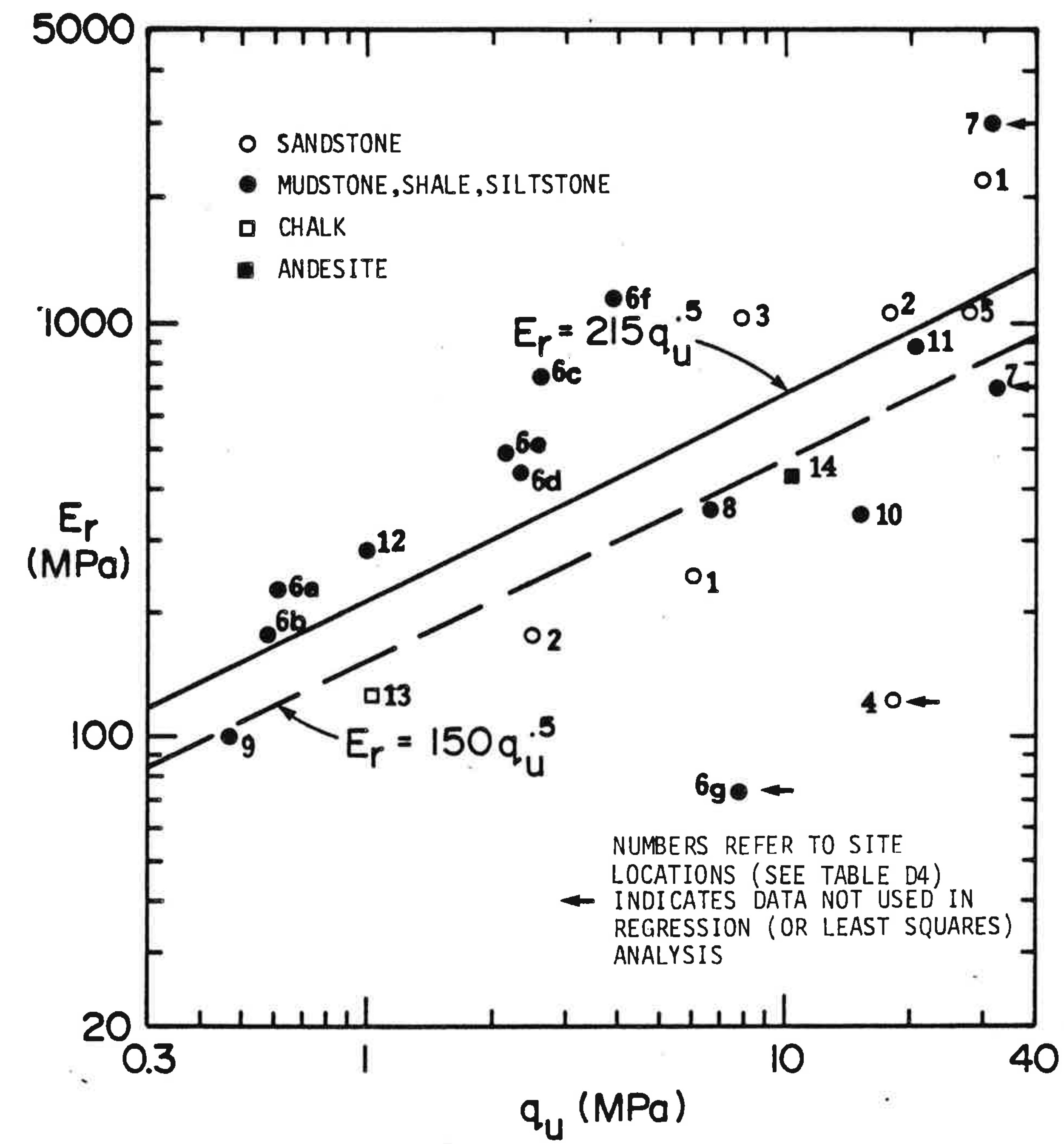


FIGURE 4.9 CORRELATION BETWEEN ROCK MASS MODULUS (E_r) AND UNCONFINED COMPRESSIVE STRENGTH (q_u) FROM LEAST SQUARES (FORCED FIT) ANALYSIS

CHAPTER 5

PROBABILISTIC ANALYSIS OF SOCKETED PILE BEHAVIOUR

5.1 GENERAL

Rock socketed piles are frequently designed using "allowable" or "design" values of parameters such as the average sideshear resistance and the rock modulus. These parameters generally incorporate an implicit "factor of safety" to take account of uncertainties regarding actual field values. However, the reduction in the sideshear resistance or modulus associated with this "factor of safety" is usually arbitrary and does not provide an indication of the likelihood that the design values may be underestimated.

In this chapter, consideration will be given to a simplified analysis which will provide an estimate regarding the reliability of the prediction of settlement response for a rock socketed pile. The probabilistic analysis will use design parameters obtained from the empirical equations suggested in Chapter 4, along with the results from a finite element analysis.

5.2 METHOD OF ANALYSIS

5.2.1 Description of the Probabilistic Analysis

A brief description of the model used to determine the predicted distribution of settlement for a complete rock socketed pile (i.e., involving both sideshear and endbearing) will be discussed in this section.

5.2.1.1 The Monte Carlo Simulation Technique

This technique was used to obtain an estimate of the probability distribution of pile settlement (the derived random variable) from randomly selected point estimates of the input parameters (eg. sideshear resistance and rock modulus) which conform to a known (or assumed) statistical distribution. (For a description of the Monte Carlo technique see Hammersley and Handscomb, 1964; or Sobol', 1974).

5.2.1.2 Finite Element Results

A series of finite element analyses similar to those described in Chapter 3, were performed to enable the probabilistic analysis to be carried out. The results were produced for a range of socketed pile geometries and pile to rock modulus ratios. Solutions were obtained for the case of a homogeneous rock mass (eg. $E_b/E_r = 1.0$).

The settlement of a socketed pile (for a particular length to diameter ratio) is influenced greatly by the pile to rock modulus ratio. For the purposes of the probabilistic settlement analysis, solutions for fifteen E_p/E_r ratios ranging from 1 to 200 were obtained for each L/D ratio considered. Pile to rock modulus ratios were selected such that the difference between any two E_p/E_r values would result in a relatively small (less than 5%) change in the observed settlement for the socketed pile at a particular applied load. Although additional intermediate values of E_p/E_r could be included, it is considered that the current number of analyses provide reasonable results without excessively increasing the costs of the analysis.

The rock and pile properties used in the finite element analysis are the same as those described in Chapter 3. The pile modulus E_p was assumed to be 35 GPa.

5.2.1.3 Probabilistic Analysis

The procedure adopted in this analysis may be summarized as follows:

(1) Using the empirical correlations suggested in Chapter 4, expected values of the sideshear resistance ($\overline{\tau}$) at the pile rock interface and rock mass modulus ($\overline{E}_r$) can be obtained (for a particular value of q_u) using

$$\overline{\tau} = 0.45\sqrt{q_u} \tag{5.1a}$$

or

$$\ell n\overline{\tau} = -.80 + .5\ell n q_u \tag{5.1b}$$

and

$$\overline{E}_r = 215\sqrt{q_u} \tag{5.2a}$$

or

$$\ell n\overline{E}_r = 5.37 + .5\ell n q_u \tag{5.2b}$$

where q_u, $\overline{\tau}$ and $\overline{E}_r$ are in MPa. The standard deviation of $\{\ell n\tau\}$ and $\{\ell nE_r\}$ ($\sigma_{\ell n\tau}$ and $\sigma_{\ell nE_r}$) were previously found to be 0.42 and 0.557 respectively (see Chapter 4).

(2) Using computer generated pseudo-random numbers, point estimates of $\{\ell n\tau\}_e$ and $\{\ell nE_r\}_e$ were determined assuming that both $\{\ell n\tau\}$ and $\{\ell nE_r\}$ are normally distributed with mean values $\{\ell n\overline{\tau}\}$ and $\{\ell n\overline{E}_r\}$ and standard deviations $\sigma_{\ell n\tau}$ and $\sigma_{\ell nE_r}$ given in step 1.

(3) Simulated values of τ and E_r (i.e., $\{\tau\}_e$ and $\{E_r\}_e$) were obtained by transforming the point estimates of $\{\ell n\tau\}$ and $\{\ell nE_r\}$, viz.

$$\{\tau\}_e = \exp\ \{\ell n\tau\}_e \qquad \text{(MPa)} \tag{5.3}$$

and $$\{E_r\}_e = \exp\ \{\ell nE_r\}_e \qquad \text{(MPa)} \tag{5.4}$$

(4) Each point estimate from the distribution for sideshear resistance and rock mass modulus (eg. $\{\tau\}_e$ and $\{E_r\}_e$ was used in the appropriate finite element analysis (i.e., for the closest value of E_p/E_r) to obtain a prediction of the settlement $\{\rho\}_e$ for the particular load level and socket geometry (L/D).

(5) Repeating steps 2 - 4 for M Monte Carlo experiments, a settlement histogram (which approximates the probability density function) can be drawn for each load level.

(6) "Design" values for sideshear resistance, τ_d, and rock mass modulus, E_d, were obtained by applying a reduction factor (RF) to the mean values of $\overline{\tau}$ and $\overline{E}_r$ given by Eqs. 5.1a and 5.2a,

eg. $$\tau_d = (RF)_\tau \cdot \overline{\tau} \tag{5.5}$$

$$E_d = (RF)_{E_r} \cdot \overline{E}_r \tag{5.6}$$

where $(RF)_\tau$ and $(RF)_{E_r} \leq 1.0$.

The design values were used in the appropriate finite element analysis (corresponding to E_p/E_d) to obtain a single value of the estimated "design settlement", ρ_d.

(7) The probability of exceeding the design settlement was estimated by comparing the number of generated settlements, $\{\rho\}_e$, which exceed the design settlement, ρ_d, i.e.,

$$PR(\rho > \rho_d) \simeq \frac{N(\{\rho\}_e > \rho_d)}{M} \times 100 \tag{5.7}$$

where

$PR(\rho > \rho_d)$ is the probability of exceeding the "design settlement in %

$N(\{\rho\}_e > \rho_d)$ is the number of generated settlements which exceed the design settlement

M is the total number of simulations (usually 500).

(8) Steps 4 - 7 were repeated for each load level.

(9) Steps 2 - 8 were repeated for each value of L/D.

(10) Steps 1 - 9 were repeated for each value of q_u.

5.3 PREDICTED SETTLEMENT DISTRIBUTIONS FOR SOCKETED PILES

In the present analysis, it has been assumed that the probability distribution of the settlement of rock socketed piles will depend primarily on the variability of the sideshear resistance and the mass modulus of the rock. Consideration was given to the effects of:

(i) Variability in sideshear resistance only,

(ii) Variability in rock mass modulus only,

(iii) Variability in both sideshear resistance and rock mass modulus.

The expected values, $\overline{\tau}$ and $\overline{E}_r$ used in the probabilistic analysis were obtained from Eqs. 5.1 and 5.2. Unless otherwise stated, the standard deviation in $\ell n\tau$ and ℓnE_r of 0.42 and 0.557 were used in the analysis. For the purpose of illustrating some of the general effects of the statistical variation in input parameters upon the settlement distribution, consideration will be given to the behaviour of a specific pile with L/D = 10 in a rock with q_u = 15 MPa. The reliability of design settlement predictions for a range of pile geometries and rock strength will be discussed in section 5.4.

5.3.1 Variability in Sideshear Resistance Only

Assuming for the moment that there is no variability in rock mass modulus (i.e. assuming the standard deviation of $\{\ell n E_r\}$ is zero), the variability in sideshear resistance will have different effects on the settlement distribution depending on the load level as shown in Figure 5.1. In this figure, the loads are expressed as a proportion of the load P_{ts} at which full slip is expected to occur (i.e., this is the load at which full slip occurs along the entire pile shaft for $\tau = \bar{\tau}$).

For any given pile load, there is a critical value of sideshear resistance τ_{cr} at which no slip will occur. Thus, the settlement will be the same for all values of shear strength greater than τ_{cr}. Now if the applied load is small compared to the full slip load P_{ts}, then the critical value of shear resistance τ_{cr} will also be small compared with the expected shear strength $\bar{\tau}$. Thus, most values of τ randomly selected from this distribution will exceed the critical value τ_{cr} and the settlement will correspond to the elastic solution. (The settlement can

never be less than that calculated for no slip). Under these circumstances, the probability density function can be approximated by a delta function as indicated in Figure 5.1a.

As the load level increases, the critical shear resistance also increases. At $P_t = P_{ts}$ approximately 25% of the randomly selected values of τ exceed the critical value τ_{cr} and give rise to the elastic solution. All values of τ less than τ_{cr} result in a settlement greater than the elastic value and the magnitude of the settlement depends on the specific value of τ. This gives rise to a settlement distribution which is skewed right as shown in Figure 5.1b.

At high load levels (eg. $P_t = 3P_{ts}$) less than 1% of the randomly selected values of τ exceed the critical value τ_{cr} giving rise to the elastic solution. For all values of $\tau < \tau_{cr}$, the settlement depends on the magnitude of τ and hence the settlement distribution (see Figure 5.1d) reflects the shear strength distribution which is skewed left.

For long sockets (eg. L/D = 10), the contribution of sideshear to the total load is greater than for short sockets. Consequently, the effect of variability in the available sideshear resistance will also be greater for long piles than for short piles.

5.3.2 Variability in Rock Mass Modulus Only

Since the settlement of a socketed pile can be written in the form

$$\rho = \frac{P_t I}{E_r D}$$

where I is an influence factor which depends on E_p/E_r, it is apparent that variability in modulus will have both a direct (since $\rho \propto 1/E_r$) and indirect (since I is a function of E_p/E_r) influence on the settlement distribution. Supposing that there was no variability in τ (i.e., the standard deviation in $\{\ell n \tau\}$ is zero), then the variability in E_r will produce a consistently skewed right distribution, which is similar to the distribution of $1/E_r$, for all load levels as shown by the short dashed lines in Figure 5.2.

5.3.3 Variability in Sideshear Resistance and Rock Mass Modulus

When consideration is given to the variability of both E_r and τ, the effect of modulus has the dominant effect on the resultant settlement distribution as may be appreciated by comparing the results given in Figures 5.1 and 5.2. The variability in sideshear resistance tends to increase the spread of the distribution of settlement and also modifies the shape. The effect is greatest at loads close to the full slip load P_{ts} and diminishes for loads either significantly less than or greater than P_{ts}.

5.4 PROBABILITY AND DESIGN SETTLEMENT

5.4.1 Probability of Exceeding Design Settlement

Probability distributions for pile settlements, such as those given in the previous section, may be useful for estimating the reliability of a design settlement (ρ_d) which is calculated using socket design parameters τ_d and E_d as defined by Eqs. 5.5 and 5.6.

As noted in the previous section, there will be an interaction between the effects of the variability in modulus and sideshear resistance. The effect of this interaction can best be appreciated by considering the probability that the settlement will exceed the settlement ρ_d calculated from a deterministic analysis using the mean (expected) values of the mass modulus $\overline{E}_r$ and average sideshear resistance $\overline{\tau}$ (i.e., $(RF)_\tau = (RF)_{E_r} = 1$ in Eqs. 5.5 and 5.6). If there was no variation in τ and if the influence factor I was independent of E_p/E_r, then the probability of exceeding the settlement ρ_d would be 50% at all load levels. However, the stress distribution, and hence the load required to cause partial slip along the pile does depend on E_p/E_r. This means that the influence factor I will vary with E_r even for a constant value of τ. The effect of the variability in E_r upon I is greatest at loads close to the full slip load P_{ts} and will diminish for loads greater than or less than P_{ts} as indicated by the results in Figure 5.2. As a consequence, the probability of exceeding the settlement ρ_d calculated using $\overline{E}_r$ and $\overline{\tau}$ will vary depending on the load as shown for a pile with L/D = 10 in rock with a q_u = 15 MPa by the short dashed curves in Figure 5.3. When variability in τ is neglected (i.e., assuming $\sigma_{\ell n \tau} = 0$), the probability of exceeding the settlement ρ_d decreases slightly at loads close to the full slip load P_{ts}. When the variation in both E_r and τ is considered, there is a substantial increase in the probability of exceeding the design settlement at loads close to the full slip load.

These results indicate that, in general, the settlement calculated using the expected values of the average sideshear resistance and rock modulus does not correspond to the expected settlement. Indeed, under some circumstances, the calculated settlement ρ_d may be considerably less than the expected (mean) settlement. This then raises the question as to which values of parameters τ_d, E_d should be used in deterministic design calculations to provide a reasonable degree of confidence that the actual settlement obtained in the field will not greatly exceed the design settlement ρ_d.

If a reduction factor $(RF)_{E_r} = 0.5$ is applied to the modulus used to calculate the design settlement (i.e., $E_d = 0.5\overline{E}_r$) then the probability of exceeding the design settlement is reduced to approximately 11% for all load levels if the variability in τ is neglected (see Figure 5.3). However, if variability in τ is considered then the probability of exceeding the design settlement can be as high as 33% even if the reduction factor of 0.5 is applied to the modulus.

Figure 5.4 shows the settlement distribution at loads of P_{ts} and $2P_{ts}$ together with the corresponding design settlements ρ_d calculated using $(RF)_{E_r} = 0.5$ and $(RF)_\tau = 1.0$.

In section 5.3.3 it was shown that the variability in τ leads to a considerable widening of the settlement distribution (as compared to that

obtained considering variability in E_r alone) at loads close to the full slip load but does not have a great effect at much higher or lower loads. In Figure 5.4, the effect of this is that 33% of the area of the settlement distribution curve is to the right of (i.e., above) the design settlement for $P_t = P_{ts}$ but only 11% of the area under the settlement distribution curve exceeds the design settlement for $P_t = 2P_{ts}$. If there was no variability in τ, we would expect the design settlement to be exceeded 11% of the time for all values of P_t given that $(RF)_{E_r}=0.5$.

Clearly, consideration must be given to the effect of applying a reduction factor to both the modulus and the available sideshear resistance. These results also suggest that settlement predictions which ignore the possibility of slip may be considerably in error in real situations where there will be variability in the available sideshear resistance and hence statistically some slip may be expected to occur.

5.4.2 Effect of the Choice of Reduction Factor

If it is assumed, as is often the current practice, that no slip will occur at the pile-rock interface, then the probability of exceeding a design settlement calculated using the rock modulus $E_d = (RF)_{E_r} * \overline{E}_r$ is independent of the length of the pile and can be simply expressed as a function of the rock modulus reduction factor $(RF)_{E_r}$ as shown in Figure 5.5. It can be seen that assuming $\sigma_{\ell nE_r}$ given in Chapter 4, the probability of exceeding the design settlement for reduction factors $(RF)_{E_r}$ of 0.7 and 0.5 are approximately 26% and 11% respectively. The corresponding probabilities of exceeding twice the design settlement are 3% and 0.5%.

When consideration is given to the statistical variability of available sideshear resistance τ then the probability of exceeding the design settlement ρ_d calculated using

$$E_d = (RF)_{E_r} * \overline{E}_r$$

$$\tau_d = (RF)_\tau * \overline{\tau}$$

varies with load level and the length of the pile as indicated in Figure 5.6.

The contribution of sideshear to the total load resistance is relatively smaller for short sockets(eg. L/D = 2) than it is for long sockets (L/D = 10). This, together with the differences in the shear stress distribution for short and long sockets combine to give the result that the probability of exceeding the design settlement is far less sensitive to the reduction factor $(RF)_\tau$ for short sockets than for long sockets. However, for both short and long sockets, the adoption of a sideshear reduction factor $(RF)_\tau$ of 0.7 or less can considerably dampen the effect of load level upon the probability of exceeding the design settlement. Thus, Figure 5.6 shows that for $(RF)_{E_r} = 0.5$, a value of $(RF)_\tau = 0.7$ will give a maximum probability of exceeding the design settlement of 11.5% and 14.5% for L/D = 2 and 10 respectively as compared with a probability of about 11% for a purely elastic analysis. A value

of $(RF)_\tau$ of 0.5 will ensure that the probability of exceeding the design settlement will not exceed 11% at all load levels for all socket lengths in the range 2D to 10D.

To this point, all the results have been for rock with an unconfined compressive strength q_u = 15 MPa. The expected values of both the rock modulus E_r, and available sideshear resistance both vary with q_u however the distribution of E_r and τ about these mean values has been assumed to be the same for all values of q_u. Thus if there is no slip, the probability of exceeding a design settlement is independent of q_u. However, the stress distribution along the socket, and hence likelihood of slip, do vary with the ratio of E_p/E_r and since E_r does vary with q_u, the probability of exceeding the design settlement will also vary somewhat with q_u. Nevertheless, this variation is small as indicated by the results in Figures 5.7 and 5.8 for q_u = 0.6 and 15 MPa.

Figure 5.7 shows that for a modulus reduction factor $(RF)_{E_r}$ of 0.5, the probability of exceeding the design settlement is unlikely to exceed 15% and 12% for $(RF)_\tau$ of 0.7 and 0.5 respectively for most pile geometries and values of rock modulus. Figure 5.8 shows the corresponding results for $(RF)_{E_r}$ = 0.7 and in this case the probability of exceeding the design settlement is unlikely to exceed 30% and 26% for $(RF)_\tau$ of 0.7 and 0.5 respectively.

As noted in Chapter 4, Meyerhof (1982, 1984) has recommended the use of partial factors (in this report these are referred to as reduction factors) applied to certain design parameters. Adopting a value of 0.7 as has been suggested for the serviceability limit state (i.e., settlements) would, as discussed above, give a probability of exceeding the design settlement of 30% or less for piles with $L/D \leq 10$. For these reduction factors, the probability of exceeding twice the design settlement is less than 3%.

5.5 SUMMARY

A probabilistic analysis using the Monte Carlo simulation technique has been used to provide a prediction of the distribution of settlement for a pile socketed into rock.

The analysis has been used to investigate the effects of statistical variability of two principal socket design parameters, namely sideshear resistance (τ) and rock mass modulus (E_r) on the predicted distributions of settlement. These distributions of settlement together with estimates of design settlement from deterministic analyses performed using design parameters τ_d, E_d were used to provide an indication of the reliability of a socketed pile design in terms of the probability of exceeding the specified design settlement.

The magnitude of the probability of exceeding design settlement was found to be influenced by the following factors:

(1) Variability in τ, particularly near load levels which cause slip at the pile-rock interface;

(2) Socket geometry (i.e., length to diameter ratio);

(3) The value of reduction factors, $(RF)_\tau$ and $(RF)_{E_r}$, which are applied to expected values of $\bar{\tau}$ and $\bar{E}_r$ to give $\tau_d = (RF)_\tau \bar{\tau}$ and $E_d = (RF)_{E_r} \bar{E}_r$.

The variability in the value of unconfined compressive strength (q_u) was found to produce relatively minor changes in the probability of exceeding the design settlement at all load levels.

Finally, probabilities of exceeding the design settlement that may be expected to occur for suggested design values of reduction factors $(RF)_\tau$ and $(RF)_{E_r}$, of 0.7 is less than 30% and the probability of exceeding the design settlement calculated over $(RF)_\tau = (RF)_{E_r} = 0.5$ is less than 11%.

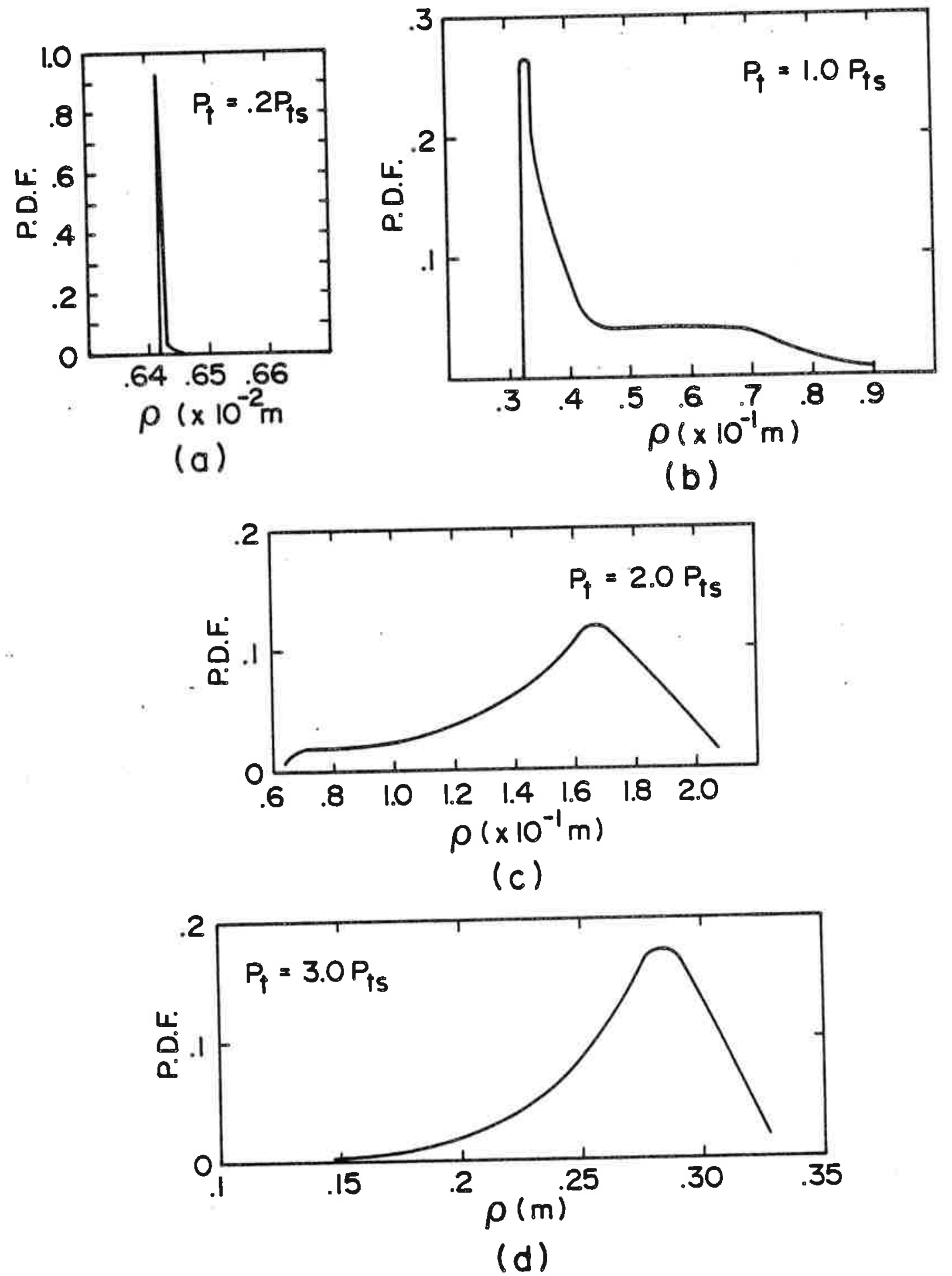


FIGURE 5.1 DISTRIBUTION OF SETTLEMENT OBTAINED CONSIDERING THE VARIABILITY IN τ ONLY
($L/D = 10$, $E_p = 35$ GPa, $q_u = 15$ MPa, $\overline{E}_r = 832$ MPa, $\overline{\tau} = 1.74$, $\sigma_{\ell nE_r} = 0$)

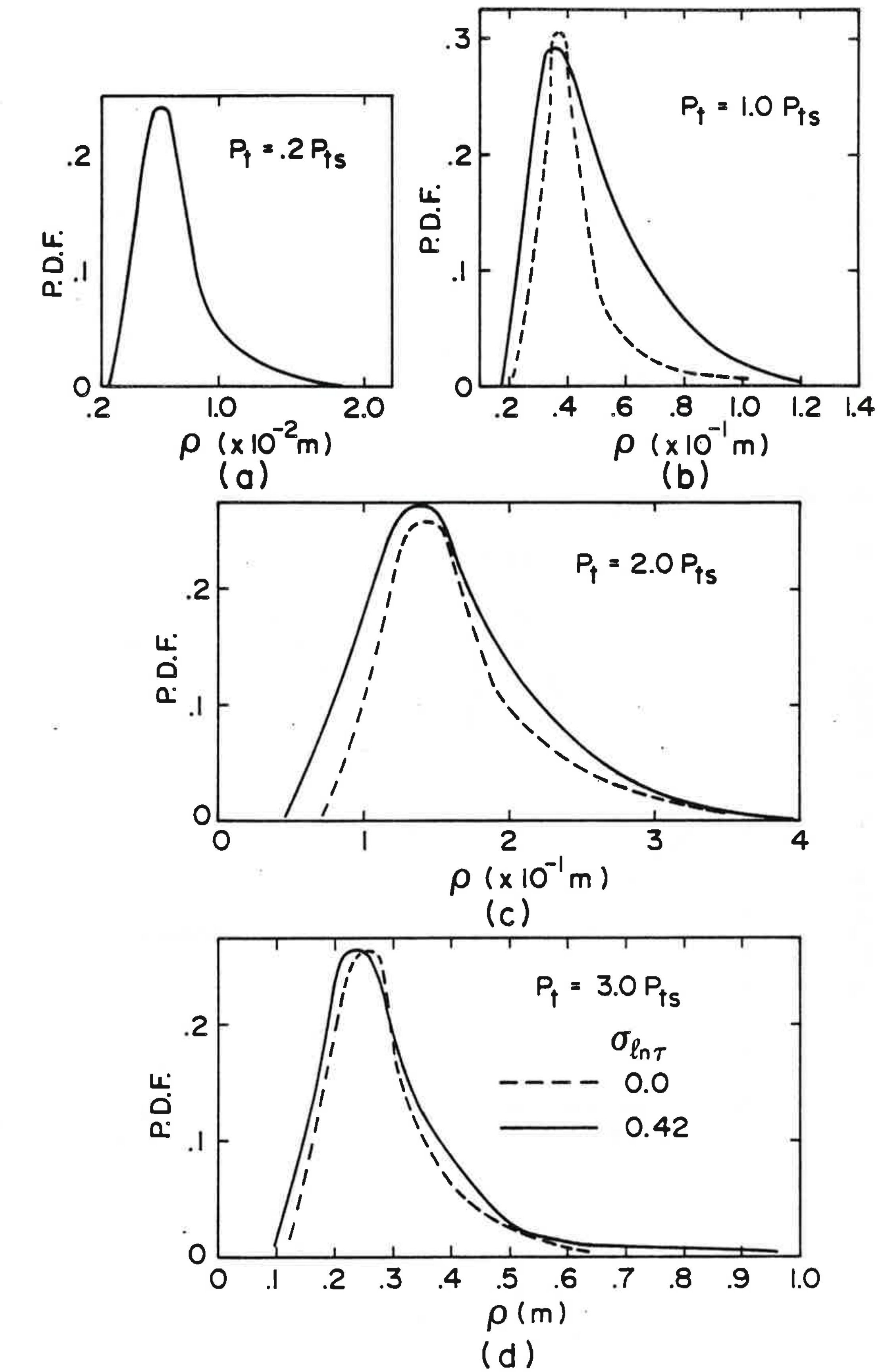


FIGURE 5.2 DISTRIBUTION OF SETTLEMENT FOR $\sigma_{\ell nE_r} = 0.557$
($L/D=10$, $E_p=35$ GPa, $q_u=15$ MPa, $\overline{E}_r=832$ MPa, $\overline{\tau}=1.74$ MPa)

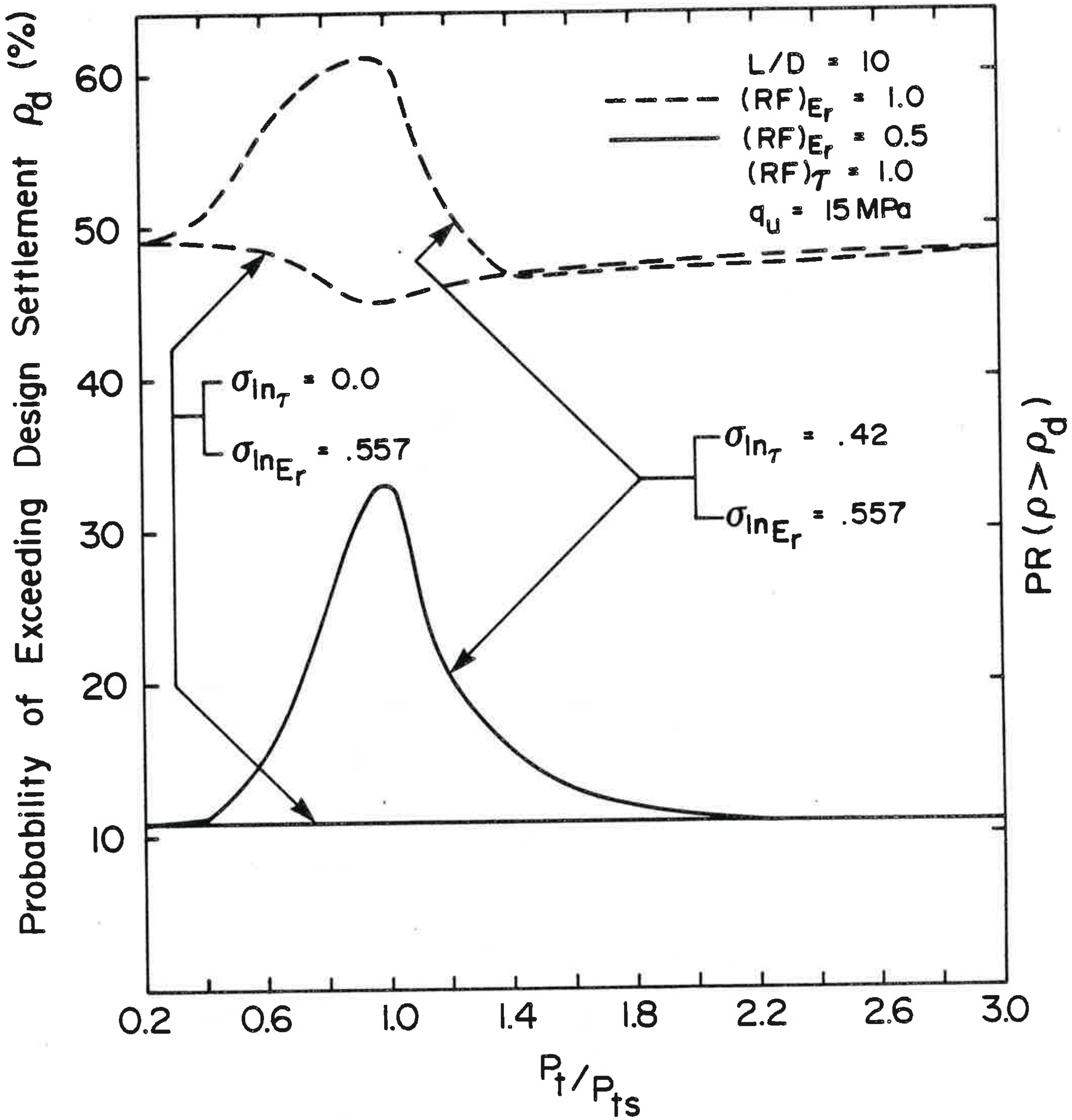


FIGURE 5.3 PROBABILITY OF EXCEEDING THE DESIGN SETTLEMENT AS A FUNCTION OF LOAD LEVEL FOR TWO VALUES OF REDUCTION FACTOR $(RF)_{E_r}$

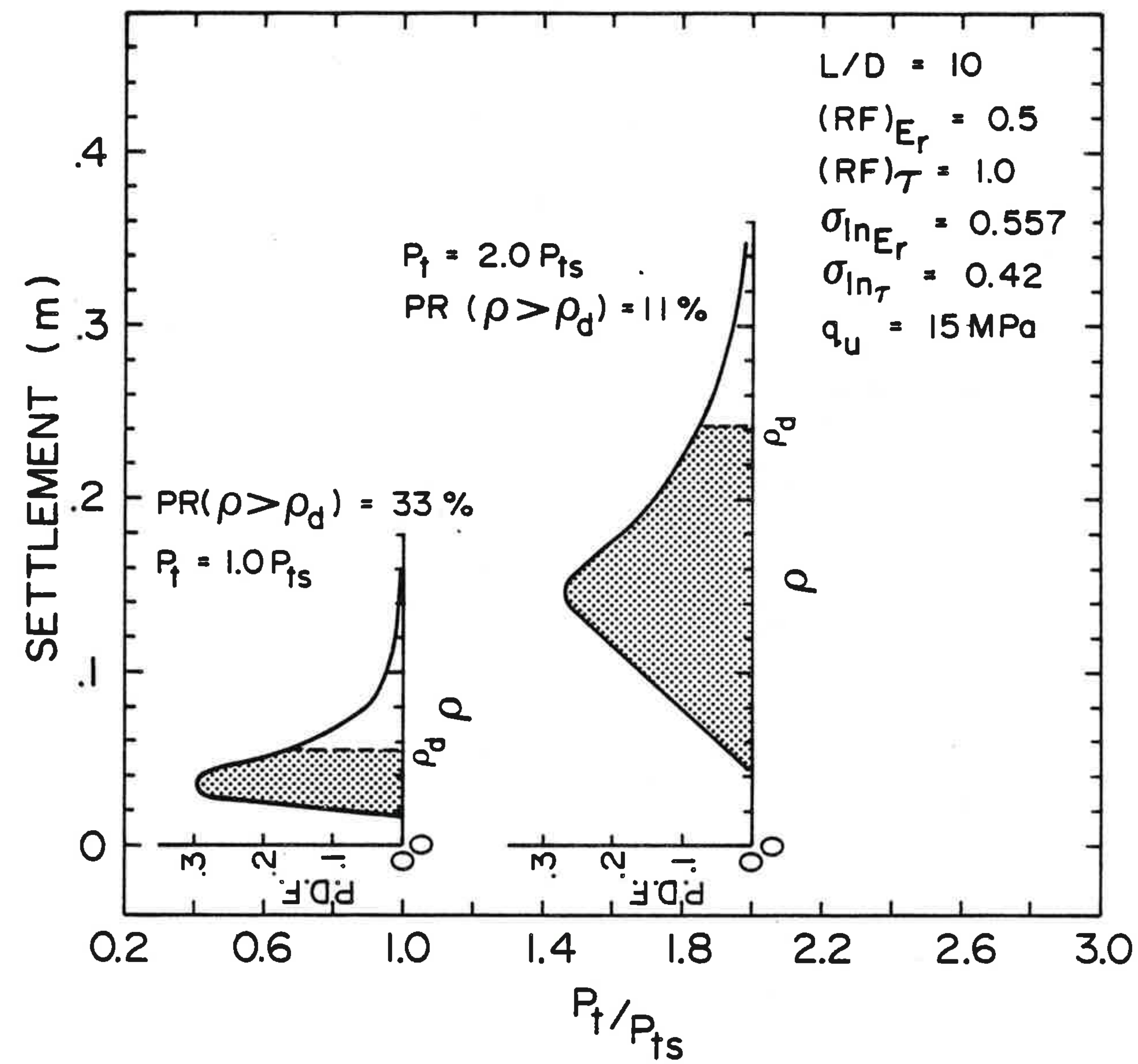


FIGURE 5.4 DISTRIBUTION OF SETTLEMENT AT TWO LOAD LEVELS

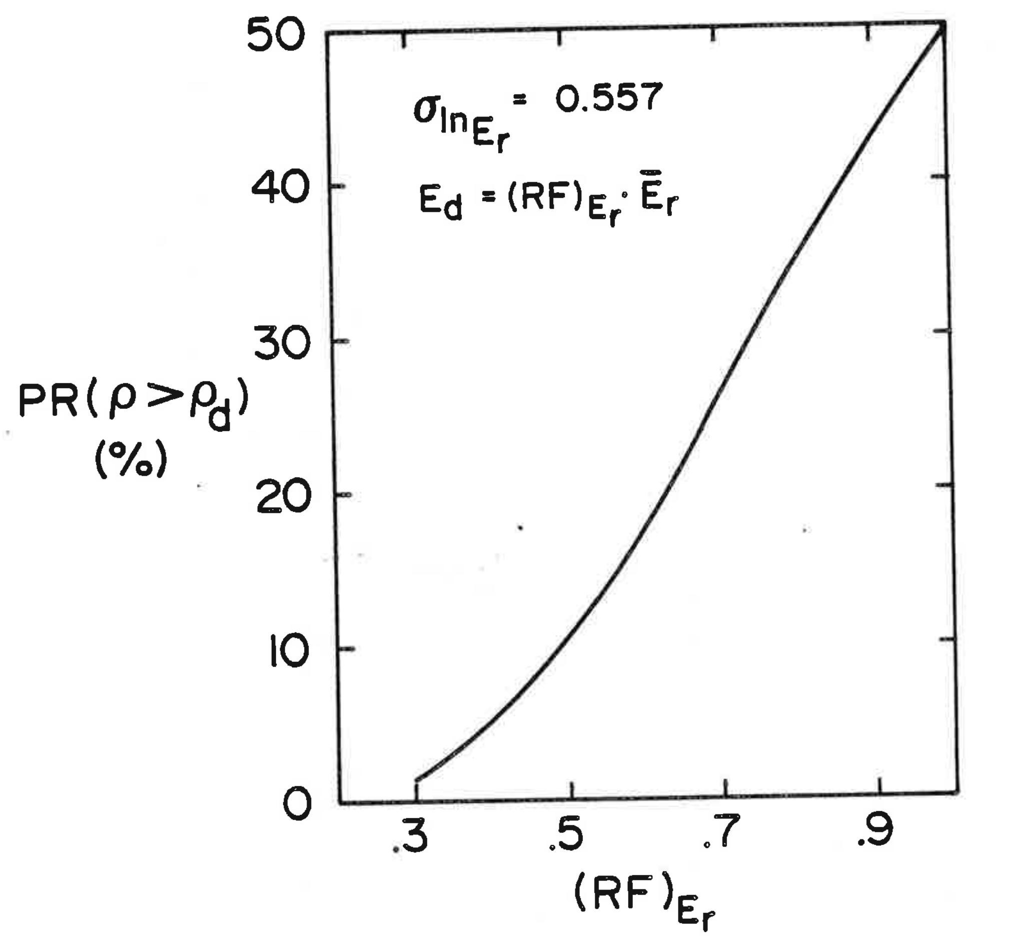


FIGURE 5.5 PROBABILITY OF EXCEEDING DESIGN SETTLEMENT IN AN ELASTIC ANALYSIS ($\sigma_{\ell nE_r}$ = 0.557 DETERMINED FROM THE CORRELATION GIVEN IN CHAPTER 4)

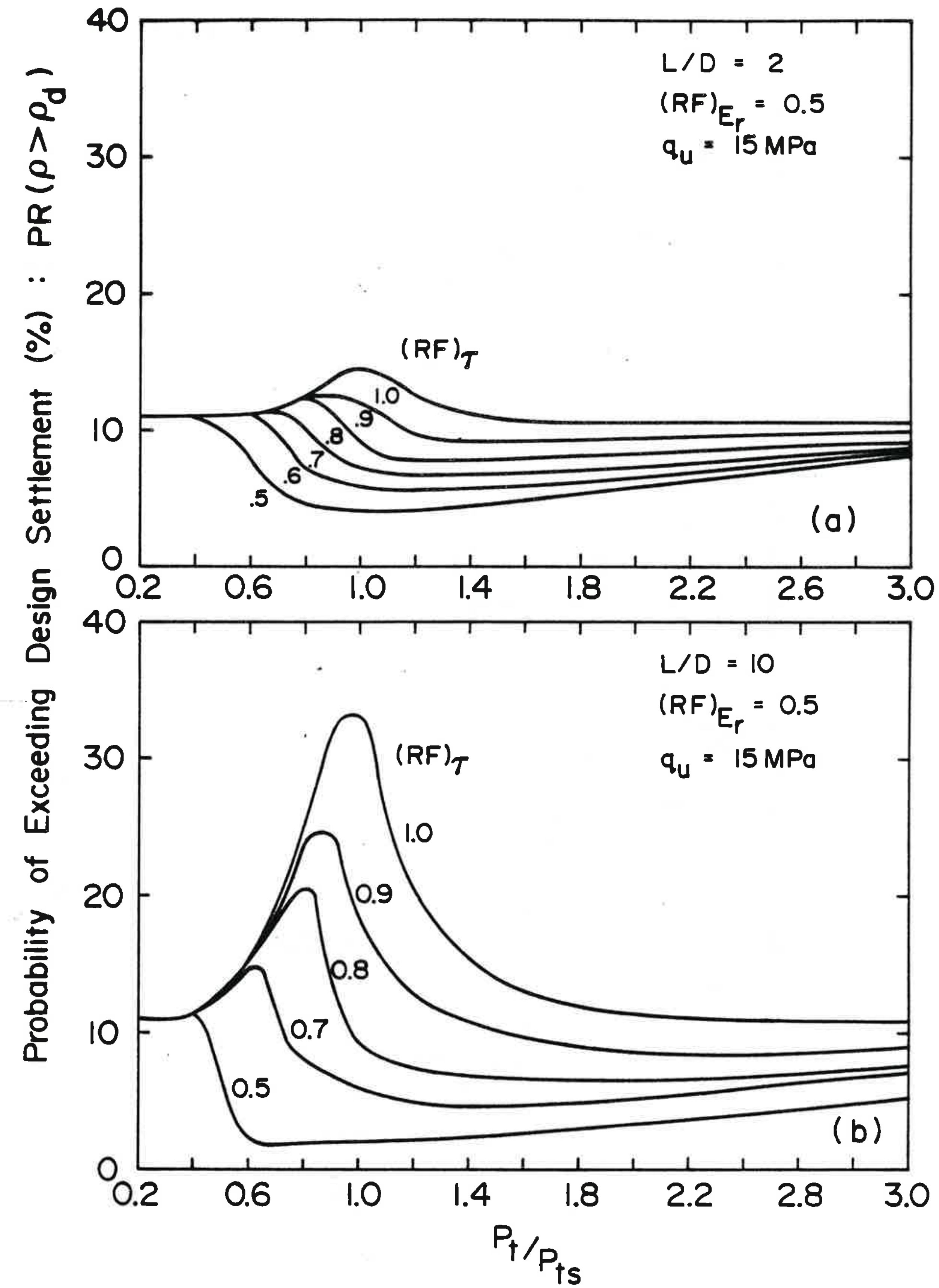


FIGURE 5.6 PROBABILITY OF EXCEEDING DESIGN SETTLEMENT AS A FUNCTION OF LOAD LEVEL AND REDUCTION FACTOR $(RF)_\tau$; $(RF)_{E_r}$ = 0.5

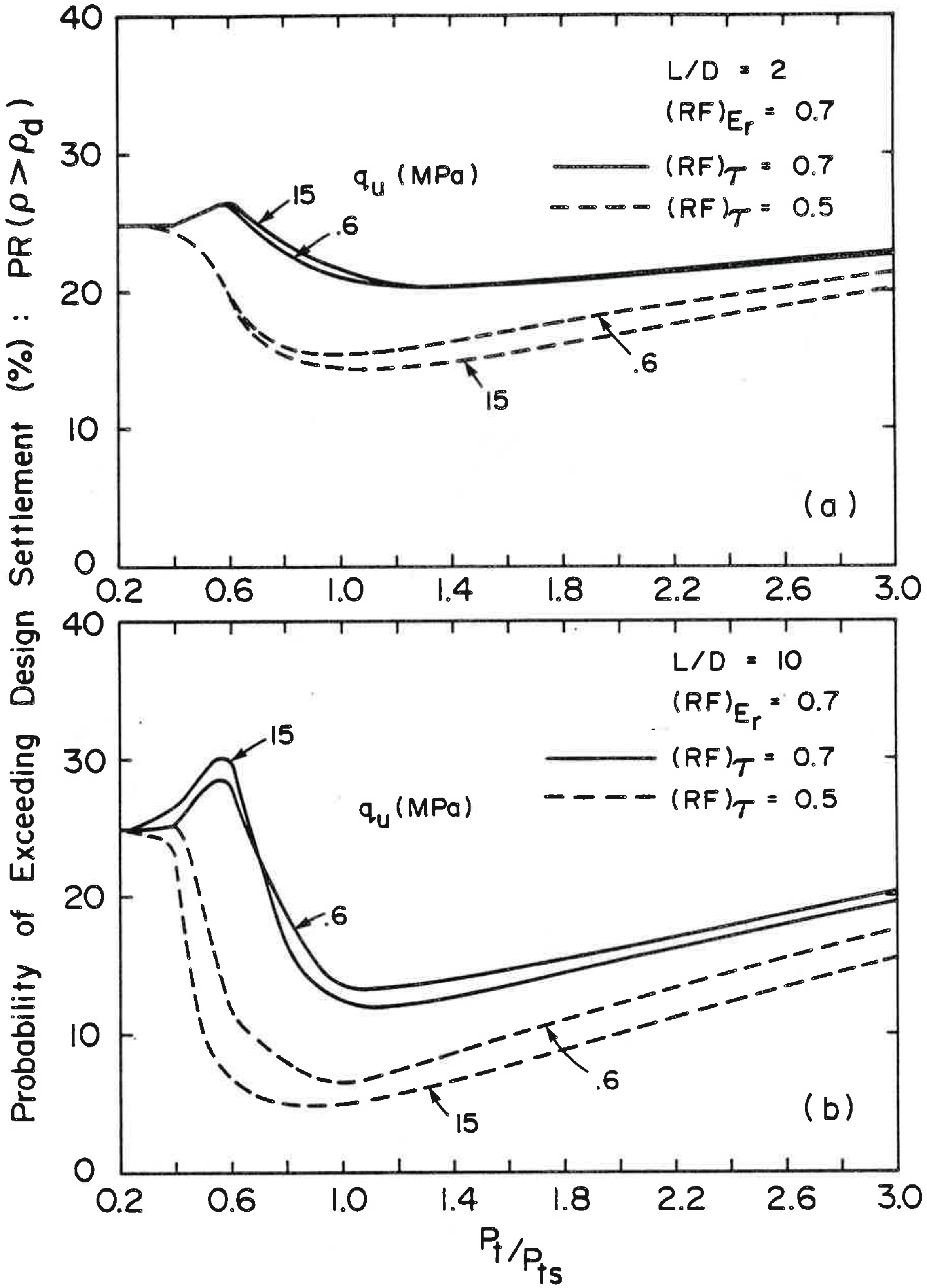


FIGURE 5.7 PROBABILITY OF EXCEEDING DESIGN SETTLEMENT AS A FUNCTION OF LOAD LEVEL AND REDUCTION FACTOR $(RF)_\tau$: $(RF)_{E_r} = 0.7$

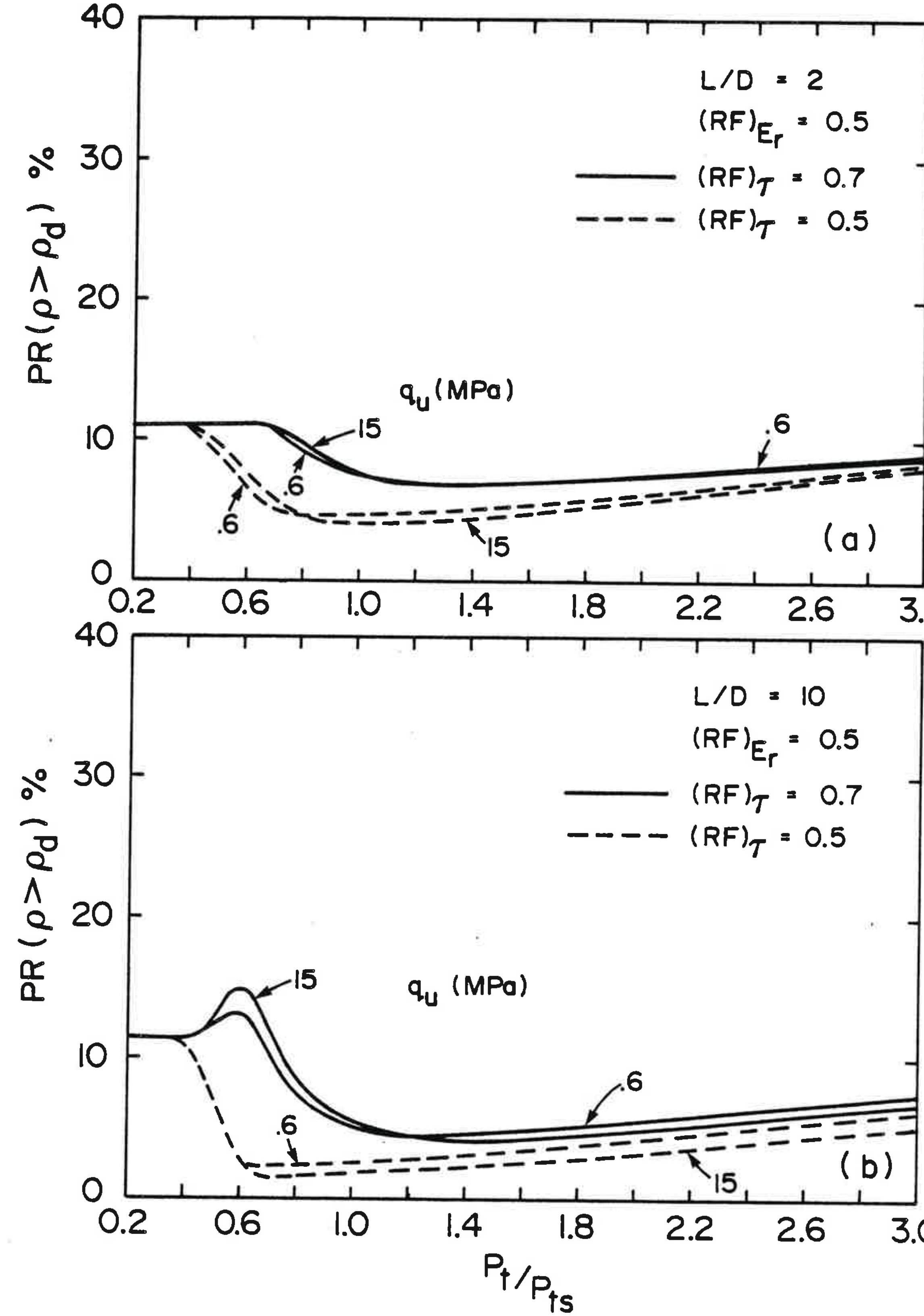


FIGURE 5.8 EFFECT OF UNCONFINED COMPRESSIVE STRENGTH q_u ON PROBABILITY OF EXCEEDING DESIGN SETTLEMENT: $(RF)_{E_r} = 0.5$

CHAPTER 6

DESIGN PROCEDURE FOR ROCK SOCKETED PILES

6.1 GENERAL

A number of design methods/guidelines are currently available from Canadian building codes, design manuals and from the published literature. In many cases, the primary design considerations used in these methods/guidelines involve the selection of a presumptive value of allowable endbearing capacity. Recognizing that in many instances this approach may lead to an overly conservative design, an alternative design method is proposed. This new approach uses an allowable design settlement as the governing criterion for the design of complete rock socketed piles (i.e., piles involving endbearing and sideshear).

The use of this design procedure will be illustrated by several worked examples. This will be complemented by consideration of the observed performance of instrumented piles which satisfy the design criteria.

6.2 EXISTING DESIGN METHODS

6.2.1 <u>Building Codes and Design Manuals</u>

General guidelines for the selection of design parameters and for the design of piles socketed into rock may be found in the:

(i) National Building Code of Canada (1980) - NBCC (1980)

(ii) Ontario Highway Bridge Design Code (1983) - OHBDC (1983)

(iii) Canadian Foundation Engineering Manual (1978) - CFEM (1978).

These design philosophies typically involve the design of the pile primarily on the basis of allowable endbearing pressures which are either (a) provided in tables (based on rock classification) and/or (b) are related to the results of unconfined compression tests on intact cores or in-situ pressuremeter tests (CFEM, 1978).

These codes give very little guidance for the selection of design values for the sideshear resistance mobilized at the pile-rock interface. Both the NBCC (1980) and CFEM (1978) recommend a small range of design values. Neither code indicates the rock conditions for which the recommendations are intended to apply. The suggestions given by the OHBDC (1983) for obtaining a design value of sideshear resistance appear more relevant to the design of piles in soils rather than rock since they rely on the parameters c and ϕ for the pile-rock interface. As noted in Appendix A, such parameters are rarely available (or determined) for socketed pile design.

As a consequence, the value of sideshear resistance used in the design of socketed piles based on these codes/manuals will generally be quite low and may result in overly conservative design for piles in weak rock.

6.2.2 Geotechnical Literature

Several alternative design procedures have been described in the geotechnical literature. Design methods which have been developed to reflect the elastic settlement response (i.e., assuming a fully bonded pile-rock interface; no slip) of an axially loaded socketed pile, have been described by Rosenberg and Journeaux (1976), Ladanyi (1977) and Pells and Turner (1979). By assuming a fully bonded interface between the pile and rock, these procedures implicitly assume that the available sideshear resistance is sufficient to avoid any slip.

As shown theoretically in Chapter 3 and as demonstrated by the field results of Williams (1980) and Horvath (1980), the majority of the load carried by a complete socketed pile will be carried in sideshear until slip occurs at the interface. Once slip occurs, the majority of additional load is then transferred to the base of the socket. Consideration of the fact that a large proportion of load is carried in sideshear, together with the fact that the shear stress distribution may be quite non-uniform while the available sideshear resistance may also vary from point to point, suggests that some slip can be expected to occur at the pile-rock interface unless extremely low (conservative) loads are applied. Recognizing that slip is likely to occur, irrespective of whether we consider it in design, it seems reasonable to adopt a design procedure which does explicitly consider the effect of slip.

One such procedure has been suggested by Williams et al. (1980). This procedure is intended to satisfy a settlement criterion which allows for nonlinear behaviour (i.e., yielding) at the pile-rock interface. There are however, two criticisms of their proposed method.

Firstly, the factor of safety used to arrive at a "design settlement" does not appear to give any indication of the reliability associated with final design. Secondly, since the design method was developed specifically for the low strength mudstones of Melbourne

(Australia), there has been some question as to whether their method is applicable to other rock types.

An alternative, more general approach for the design of piles socketed into rock will now be proposed.

6.3 SELECTION OF DESIGN PARAMETERS

6.3.1 Expected Sideshear Resistance

For preliminary design purposes, the expected sideshear resistance of clean sockets can be estimated by determining the average unconfined compressive strength q_u (in MPa) of the rock strata in which the socket is to be founded and then adopting an empirical correlation such as:

$$\bar{\tau}\ (\text{MPa}) = 0.45\ \sqrt{q_u} \quad \text{for regular sockets (roughness less than R4 as defined by Table 4.3)} \tag{6.1a}$$

$$\bar{\tau}\ (\text{MPa}) = 0.6\ \sqrt{q_u} \quad \text{for rough (R4) sockets} \tag{6.1b}$$

A clean socket is classified as a socket where there is no significant auger smear (i.e., zone of remoulded rock), bentonite, or other substance which will inhibit good bonding between the rock and concrete. In the absence of direct field test data, sideshear resistance should be neglected for any sockets which are not clean. This assumption may be appropriate for smooth sockets but can be quite conservative for sockets with roughness R3 or R4 (see Table 4.3 for a definition of these terms). The sideshear resistance of very rough sockets (this includes artificially roughened sockets; eg. see Horvath, 1980) may be close to that of

clean sockets but the actual available sideshear resistance should be verified for the expected field conditions from a field test.

The most reliable means of determining the expected sideshear resistance is by performing a field load test on a prototype pile. This test may serve to verify the adequacy of initial empirical estimates of available sideshear resistance or to justify the adoption of sideshear resistance values larger than those recommended above.

Parameters determined from laboratory scale tests (eg. small scale socket tests or direct shear tests on composite concrete-rock samples) should not be used in the design of full scale piles because of the difficulties of modelling the rock conditions and properties as they exist in the field. These difficulties have been discussed in more detail by Pells, Rowe and Turner (1980) and Williams (1980).

6.3.2 Expected Rock Mass Modulus

Frequently, the modulus of rock is determined from laboratory tests on intact rock cores. Values determined in this manner will generally overestimate the in-situ value since the influence of defects (i.e., joints, seams, etc.) contained in the rock mass are not directly considered. Consequently, a number of field tests have been developed to provide a better estimate of the rock mass modulus. Several field methods relevant to foundation design include,

(i) pressuremeter, Goodman jack tests,

(ii) in-situ plate load tests,

(iii) in-situ load test of a rock socketed pile.

In situations where the rock mass is relatively free from defects, and shows no evidence of anisotropy, methods (i) and (ii) may be expected to provide a reasonable estimate of the rock mass modulus. If the rock has defects (seams, open joints, etc.), the parameters determined from methods (i) and (ii) are generally not appropriate for socketed pile design. The most reliable method of estimating the rock mass modulus is to perform a field test on a prototype pile.

For the purposes of preliminary design, an estimate of the expected mass modulus can be obtained by determining the average unconfined compressive strength of the founding strata q_u (in MPa) and then using the empirical correlation

$$\overline{E}_r = 215\sqrt{q_u} \tag{6.2}$$

6.3.3 <u>Design Values for Sideshear Resistance and Mass Modulus</u>

Design values of sideshear resistance τ_d and rock mass modulus E_d should be obtained by applying a reduction factor (or partial factor) RF to the expected values $\overline{\tau}$, $\overline{E}_r$ viz.

$$\tau_d = (RF)_\tau \overline{\tau} \tag{6.3a}$$

$$E_d = (RF)_{E_r} \overline{E}_r \tag{6.3b}$$

As indicated in Chapter 5, the probabilities of exceeding the design settlement are less than 30% for $(RF)_\tau = (RF)_{E_r} = 0.7$ and less than 11% for $(RF)_\tau = (RF)_{E_r} = 0.5$. The probability of exceeding twice the design settlement is less than 3% in both cases.

To make allowance for potential variability in rock properties with both position and time, it is recommended that a reduction factor of at least 0.7 be applied even when the socketed pile design is based on parameters backfigured from a field test on a prototype pile.

Unless previous testing and experience indicate that the parameter empirically selected on the basis of Eqs. 6.1 - 6.3 are appropriate for a specific site, piles designed on the basis of these parameters should be proof tested to verify the design.

6.3.4 <u>Allowable Design Settlement</u>

Careful consideration should be given to the adoption of a reasonable allowable design settlement. It is well accepted that foundations on soil can experience some settlement and still perform adequately. The same philosophy should also apply to foundations on rock, recognizing that the majority of the deformations for foundations on most rock* will occur during construction.

The maximum groundline settlement should be selected to be the maximum settlement (at the surface of the soil/rock deposit) which is

---

* Exception must be made for rocks which may be particularly susceptible to creep.

consistent with good performance of the structure. The maximum pile head settlement, ρ_m, is then calculated by subtracting the compression of the pile or column above the actual socket (eg. due to the length L_e in Figure 3.17) from the maximum groundline settlement. (Note that the compression of the length L_e should be calculated using a concrete modulus and design load which have been factored in accordance with normal limit state design practice.) The design settlement for the socketed pile is then taken to be the maximum settlement divided by two, i.e., $\rho_d = 0.5\ \rho_m$.

In the design of socketed piles, differential settlement is likely to be of greater concern than the actual magnitude of the settlement. In general, piles should be designed to ensure that the settlement of each pile will be the same. If differential settlement is critical, the sockets of each pile should be carefully inspected in the field and the designs adjusted as necessitated by differences in the rock at different sockets (eg. variable weathering, seams, etc.).

6.3.5 Allowable Endbearing Pressures

There has been considerable debate concerning the bearing capacity of circular foundations on rock (see section 2.3.2). Numerous theories have been proposed (eg. see Pells and Turner, 1980; Couetdic and Barron, 1975) and there is a significant variation in the predicted bearing capacity. For example, the predicted bearing capacity of a circular footing on rock with $\phi = 35°$ ranges from 8 q_u to 32 q_u depending upon which theory is adopted. The scatter in experimental results is also quite high and the available data does not lend support to any particular

theory. Nevertheless, even adopting the lowest theoretical prediction and applying a factor of safety of three would give an "allowable bearing pressure" of 2.7 q_u. This, coincidentally, also approximately corresponds to the pressure expected to cause the onset of first failure based on incipient failure theory (eg. see Pells and Turner, 1980). Thus, based on purely theoretical considerations, an allowable bearing pressure at the base of the pile of 2.7 q_u would seem reasonable. (In considering the allowable bearing pressure, it should be remembered that we are only concerned here with preventing significant failure of the rock below the base of the pile. The magnitude of the settlement is considered separately in the following section and depends primarily on the mass modulus of the rock.)

Table 6.1 summarizes the results of a number of well documented field tests on recessed endbearing piles with diameters of 0.3 m or greater. In all cases, the load carried by the pile was still increasing when the test was terminated. In the tests reported by Horvath (1980), there was no sign of significant yield at the base of the pile when the tests were terminated at base bearing pressures ranging from 0.96 - 1.35 q_u.

In the tests reported by Glos and Briggs (1983), first yield occurred at base pressures of between 0.95 - 1.25 q_u although the load-displacement curve was still climbing steeply when the tests were terminated at base pressures of between 1.29 - 1.41 q_u.

The tests performed by Williams (1980) were carried to higher load levels (relative to q_u) and in these tests some yielding of the rock (as inferred by the change in shape of the load-displacement curve) was observed at base pressures of between 1.4 and 2.5 q_u. When these tests were terminated at loads ranging from 2.5 to 10.5 q_u, the load-displacement curve was still rising and collapse had not occurred.

For the purpose of designing piles socketed into weak rock (q_u < 35 MPa) where

(i) the base of the socket is at least one diameter below the rock surface

(ii) the rock to a depth of one diameter beneath the base of the pile is either intact or tightly jointed (i.e. no compressible/weak gouge filled seams) having an average unconfined compressive strength q_u

(iii) there are no solution cavities or voids beneath the pile

it appears adequate to adopt an allowable bearing pressure at the base of the pile q_{ba} given by

$$q_{ba} = q_u \tag{6.4a}$$

and a maximum bearing pressure at the base of the pile q_{ma} given by

$$q_{ma} = 2.5\ q_u \tag{6.4b}$$

provided that both q_{ba} or q_{ma} are also limited such that they do not exceed the allowable stress in the concrete.

In the design procedure to be described in detail in the following section, the base pressure q_b determined from the design charts will be

required to be less than the allowable bearing pressure q_{ba}; i.e.,

$$q_b \leq q_{ba} \tag{6.5}$$

This requirement is intended to ensure that the rock beneath the base of the pile behaves "elastically" (i.e., without significant yielding) under design conditions.

To provide some measure of safety in terms of the ultimate capacity of the rock socket, the maximum base pressure q_m, defined as

$$q_m = q_t - 4(RF)_s \frac{L}{D} \bar{\tau} \tag{6.6}$$

where q_t = the design pressure at the top of the socket

L = length of the socket

D = diameter of the socket

$\bar{\tau}$ = the expected sideshear resistance along the socket

$(RF)_s$ = a reduction factor for ultimate conditions

must be less than the maximum bearing pressure q_{ma}, i.e.,

$$q_m \leq q_{ma} \tag{6.7}$$

For the purposes of design, it is recommended that $(RF)_s$ be taken to be 0.3. Based on the statistical studies reported in Chapter 4, this implies that the probability of the mobilized shear resistance being less than that assumed in Eq. 6.6 is less than 0.5%. Thus, Eq. 6.6 is intended to ensure that even if only 30% of the expected sideshear resistance was mobilized at the pile-rock interface, the entire pile-socket system will still have an adequate factor of safety against collapse.

Note that for lightly loaded piles with long sockets, q_m may be very small (or possibly even negative). Under these circumstances Eq. 6.5 will govern.

An acceptable design should satisfy both Eqs. 6.5 and 6.7.

6.4 PROPOSED DESIGN PROCEDURE

6.4.1 Assumptions

(1) The proposed method is intended for the design of complete socketed piles (i.e., involving both endbearing and sideshear). This implies that the base of the pile is in direct contact with the underlying rock and no soft, compressible material impairs this contact. In addition, it has been assumed that the socket sidewalls are clean of any contamination (eg. rock smear, drilling mud etc.) which would result in a decrease in the peak average sideshear resistance at the pile-rock interface. The design procedure is also appropriate for design of piles where the socket sidewalls have been artificially roughened.

(2) It is assumed that there are no voids or open cavities within the zone of influence of the socketed piles.

(3) The procedure can be used in design where the applied axial load produces either an elastic pile response or results in slip at the pile-rock interface. Design charts given in Chapter 3 (and Appendix C) have been developed to specifically include such behaviour. These charts are also available for when the modulus of rock below the base of the pile is different from that surrounding the socketed pile.

6.4.2 Description of the Design Method

The design method described in this section provides a simple, yet rational approach for the design of rock socketed piles. An important feature of this design method is that a socketed pile can be designed quite rapidly once the initial design parameters are selected. This enables the designer to conduct a comparative study of various socket designs for a wide variety of preliminary design parameters. In addition, an estimate of the reliability associated with the resulting design(s) can be determined as discussed in Chapter 5.

A preliminary subsurface investigation should be conducted to assess the in-situ conditions of the rock mass and to determine a representative unconfined compressive strength. This information can be used together with the design procedure described below to develop a preliminary design. Field tests may then be performed to obtain improved estimates of rock properties and the design can be revised as necessary.

Design of a "Normal" Complete Socket

For a "normal" complete socket (i.e., where the pile is not recessed and where there are no clay filled seams along the walls of the socket), the design method proceeds as follows (see also Fig. 6.1):

STEP 1: (a) Select expected values of sideshear resistance and rock mass modulus as follows; either

(i) using a representative value of q_u determined from tests on intact core samples, determine $\bar{\tau}$ and $\bar{E}_r$.

eg. $\bar{\tau} = .45\ \sqrt{q_u}$ (for regular sockets - roughness less than R4)

or $\bar{\tau} = .6\ \sqrt{q_u}$ (for rough sockets - R4)

and $\bar{E}_r = 215\ \sqrt{q_u}$

Note that caution should always be adopted when using any empirical correlation. If a design is based on these empirical correlations, the design should be checked by proof loading in the field.

or

(ii) if more accurate field information is available then values of $\bar{\tau}_{field}$, $\bar{E}_{r_{field}}$ should be used.

(b) Apply reduction factors (i.e., $(RF)_\tau$ and $(RF)_{E_r}$) to the expected values found in step 1(a), to obtain the design parameters for sideshear resistance and rock mass modulus eg.

$$\tau_d = (RF)_\tau \cdot \bar{\tau}$$

$$E_d = (RF)_{E_r} \cdot \bar{E}_r$$

where recommended values for the reduction are given in Chapter 5 and section 6.3.3. The probability of exceeding the design settlement will depend on the reduction factors adopted.

STEP 2: Determine the following design parameters,

(i) allowable design settlement*, ρ_d (m)

(ii) socketed pile diameter, D (m)

(iii) applied axial load, P_t (MN)

(iv) modulus of pile, E_p (MPa)

* see section 6.3.4.

STEP 3: Calculate,

(i) the maximum socket lengths required, initially assuming that all load is to be carried by socket shear,

$$L_{dmax} = \frac{P_t}{\pi D \tau_d} \tag{6.8a}$$

(ii) the corresponding length to diameter ratios,

$$\frac{L_{dmax}}{D} = (L/D)_{dmax} \tag{6.8b}$$

(iii) the pile-rock modulus ratios,

$$E_p/E_d \tag{6.9}$$

and

$$E_b/\bar{E}_r \tag{6.10}$$

where E_b is the expected modulus of the rock below the base of the socket.

Note that the use of the ratio $E_b/\bar{E}_r$ ($E_t/\bar{E}_r$) rather than E_b/E_d (E_t/E_d) implies that the reduction factor $(RF)_{E_r}$ is applied to all the rock, i.e.

$$\frac{(RF)_{E_r} \cdot E_b}{E_d} = \frac{(RF)_{E_r} \cdot E_b}{(RF)_{E_r} \cdot \overline{E}_r} = \frac{E_b}{\overline{E}_r}$$

STEP 4: Calculate the design settlement influence factor, I_d,

$$I_d = \frac{\rho_d E_d D}{P_t} \tag{6.11}$$

STEP 5: Select the appropriate design charts (see Figures C2 - C5, Appendix C) for:

values of $\frac{E_p}{\overline{E}_r} = \frac{E_p}{E_d}$, and $\frac{E_b}{E_r} = \frac{E_b}{\overline{E}_r}$, and then,

(a) on the design chart, draw the "factored design line", which is a straight line between the coordinates ($L/D=0$, $P_b/P_t=100\%$) and ($L/D = L_{dmax}/D$, $P_b/P_t=0$) (see Figure 6.2)

(b) locate the intersection between the "factored design line" and the curve corresponding to the "design" settlement influence factor, I_d (see Figure 6.2). Then from the intersection point:

(i) Draw a vertical line to the L/D axis and read off the "design" length to diameter ratio $(L/D)_d$. Calculate the corresponding length $L_d = (L/D)_d \cdot D$.

(ii) Draw a horizontal line to the (P_b/P_t) axis and read off the "design" ratio of load carried to the base $(P_b/P_t)_d$.

Proceed to Step 6.

(c) If no intersection point can be established on this design chart, it is necessary to check whether the pile can be designed for the given conditions. Select the appropriate chart from Figure C1 (eg. see Figure 6.3) and draw a horizontal line for $I_n=I_d$. Find the intersection of this line with the curve for the appropriate value of E_p/E_r:

(i) If there is an intersection point on this curve then the pile can be designed elastically (i.e., negligible slip should occur under design conditions). The required $(L/D)_d$ can be obtained by dropping a vertical line from the point of intersection to the L/D axis. Calculate the design length $L_d = (L/D)_d \cdot D$. The corresponding proportion of load transferred to the base (P_b/P_t) may be determined from Figure C1b or d by drawing a vertical line from the selected value of $(L/D)_d$ to meet the curve for the appropriate value of E_p/E_r then move horizontally to read off the value of $(P_b/P_t)_d$ (eg. see Figure 6.4). Proceed to Step 6.

(ii) If there is no intersection point then no pile of diameter D will satisfy the design requirements for the specified conditions. Go back to Step 2 and either increase the design settlement (if the selected value was unrealistically low) or increase the diameter D.

NOTE: In situations where values of E_p/E_d lie between two design charts (or curves) given in Appendix C, interpolation will be necessary between the bounding design charts.

To illustrate the three possible situations which may occur in this step, consider a pile with $(L/D)_{dmax}$= 4.5 with E_p/E_r = 50 and E_b/E_r = 1 (where these values would have been determined in previous steps). In Step 5(a), $(L/D)_{dmax}$ = 4.5 is marked on the L/D axis and the "factored design line" is drawn between the points (L/D=0, P_b/P_t=100%) and ($(L/D)_{dmax}$,P_b/P_t=0). Now consider three possible values of I_d.

- If I_d=0.45 then the intersection point of the factored design line and the curve for I = 0.45 can be found (see Figure 6.2) and the values of $(L/D)_d$=2.2 and $(P_b/P_t)_d$=51% are read directly from the graph. We could now proceed to Step 6.

- If I_d=0.25 then an inspection of Figure 6.2 shows that there is no intersection point between the factored design line and the curve for I = 0.25. Thus we must follow the procedure described in Step 5(c). Going to Figure C1a (reproduced as Figure 6.3) and drawing a horizontal line for I_d=0.25, we find an intersection with the appropriate elastic curve for E_p/E_r=50 at $(L/D)_d$=4.7. Thus this pile can be designed elastically (i.e., no slip occurs at the interface). The corresponding value of (P_b/P_t) can be found from Figure C1b (reproduced as Figure 6.4) by moving vertically upwards from $(L/D)_d$=4.7 to the curve for E_p/E_r=50 and then horizontally to get $(P_b/P_t)_d$ = 13.5%. We could now proceed to Step 6.

- If I_d=0.15 then an inspection of Figure 6.2 shows that there is no curve for I = 0.15 on this figure. Thus we must follow the procedure described in Step 5(c). Going to Figure C1a (Figure 6.3 here) and drawing a horizontal line for I = 0.15, we find that there is no intersection of this line with the curve for E_p/E_r=50. Thus, a pile with this diameter D cannot be designed to satisfy the specified design conditions with I_d = 0.15. Thus we must go back to Step 2 and either reassess our choice of allowable settlement ρ_d or adopt a larger pile diameter (or both).

STEP 6: Calculate the maximum load at the base of the socketed pile

eg. $$q_b = (P_b/P_t)_d\, q_t \tag{6.12a}$$

$$q_m = q_t - 4(\tfrac{L}{D})_d (RF)_s \bar{\tau} \tag{6.12b}$$

where $$q_t = \frac{P_t}{(\pi D^2/4)} \tag{6.12c}$$

Using the recommendations of section 6.3.5, check that:

(i) $$q_b < q_{ba} \tag{6.13a}$$

(ii) $$q_m \leq q_{ma} \tag{6.13b}$$

(a) If both these conditions are satisfied then the design has been controlled by settlement considerations and a suitable pile with length L_d and diameter D has been selected.

(b) If either conditions (i) or (ii) are violated then endbearing controls the design and either the pile length or diameter must be increased to reduce the base pressure.

Design of a Recessed Complete Socketed Pile

If the pile is recessed, it should be initially designed ignoring recessment by following Steps 1-6 above. This design will be conservative for a recessed socketed pile (i.e., where $L_e \neq 0$). The effect of recessment can be ignored if the pile design involves slip (i.e., if it was designed using Figures C2-C5). However, the design obtained ignoring recessment may be a little too conservative if the pile design does not involve slip (i.e., if it was designed using Step 5(c) and Figures C1). In this case, the designer may adjust the design to take account of recessment as follows:

STEP 7: Determine the following additional design parameters:

(i) recessment length L_e (m)

(ii) modulus of the rock adjacent to the recessed part of the socket (see Figure 3.22) E_t (MPa)

and the corresponding ratios L_e/D, $E_t/E_r = E_t/\overline{E}_r$.

STEP 8: Having determined an initial estimate of $(L/D)_d$ from Step 5(c), determine a settlement reduction factor $(RF)_\rho$ from Figures C15-C18 for the appropriate values of L/D, L_e/D, E_t/E_r, E_p/E_r. Adjust the design value of I_d as follows

$$I_d^* = I_d/(RF)_\rho$$

and then repeat Steps 5 and 6 using this revised value of I_d^* instead of I_d. The adjusted value of $(L/D)_d^*$ should be used to compute a new value of $(RF)_\rho$. This value may be compared with that initially adopted and if necessary, Steps 5 and 6 may be repeated for a revised value of I_d^* calculated with the most recent estimate of $(RF)_\rho$.

Design of a Complete Socketed Pile in the Presence of Weathered Seams

In weak rocks, soft weathered seams may be encountered along the shaft of the socket. These seams will generally have a lower modulus and a lower shear strength than the adjacent rock. The effect of these seams will depend on these two properties together with the proportion of seams and the position of the seams along the socket. A number of possibilities will be considered in this section.

Suppose that an initial estimate of the socket length L_i has been obtained (eg. from a design neglecting seams by following Steps 1-6) and it is found that weathered seams will be present adjacent to the proposed socket.

1. If these seams are localized to a depth L_e, the designer may choose to neglect the rock with these seams and simply extend the pile to a depth L_i below the highly weathered zone (i.e., the length of the pile L_1 will be given by:

$$L_1 = L_i + L_e \tag{6.14}$$

However, if the length L_e is a significant proportion of the required length L_i and/or if a seam is located near the base of the socket, then this approach may be far too conservative and an alternative estimate of the required pile length should be determined as described below.

2. If the seams are evenly distributed down the socket of the pile or localized at some depth $L_e > D$,

(i) estimate - the proportion S of seams to total socket length

(i.e., $S = \Sigma$(seam thicknesses/L)

- the modulus of the weathered seam material E_s

- the pile-seam material interface strength τ_s

and calculate the corresponding ratios

$$\frac{E_s}{E_r} = \frac{E_s}{\overline{E}_r} ; \frac{\tau_s}{\tau_r} = \frac{\tau_s}{\overline{\tau}}$$

(ii) The same design procedure and design charts as for a homogeneous rock can be used for this case if the settlement influence factor $I_s = I_d$ (where I_d was determined in Step 4) is used to obtain a fictitious homogeneous settlement factor I_h which corresponds to the same load distribution P_b/P_t for a homogeneous rock. This can be achieved by selecting the design chart for the closest value of L/D and E_p/E_r from Figures C6 to C10 (interpolation may be necessary if charts are not given for the required values of L/D and E_p/E_r) and then from the known values of I_s, E_s/E_r, τ_s/τ_r and S, deduce the appropriate value of I_h.

(iii) Deduce a modified design sideshear resistance, viz.

$$\overline{\tau}^* = (S\tau_s + (1-S)\overline{\tau}) \qquad (6.15a)$$

$$\overline{\tau}_d^* = (RF)_\tau \cdot \overline{\tau}^* \qquad (6.15b)$$

and a modified maximum pile geometry

$$\left(\frac{L}{D}\right)^*_{dmax} = \frac{P_t}{\pi D \overline{\tau}_d^*} = \left(\frac{L}{D}\right)_{dmax} / \left(1-S + \frac{\tau_s}{\overline{\tau}} S\right) \qquad (6.16)$$

(iv) Using the value of $(L/D)^*_{dmax}$ from Eq. 6.16 and $I_d^* = I_h$ (as determined from (ii) above) proceed to Steps 5a and 5b and determine a length L_2 which will satisfy the design conditions. Check bearing consideration - Step 6.

(v) If no intersection point can be found in Step 5b, then

- deduce a modified modulus

$$E_d^* = \left(1-S + S\frac{E_s}{E_r}\right)(RF)_E \overline{E}_r = \left(1-S + S\frac{E_s}{E_r}\right)E_d \qquad (6.17)$$

and a new estimate of I_d^* viz.

$$I_d^* = \frac{\rho_d E_d^* D}{P_t} \qquad (6.18)$$

- proceed to Step 5c using the value of

$$I_d^* \text{ and } \frac{E_p}{E_r} = \frac{E_p}{E_d^*}$$

where E_d^* and I_d^* are given by Equations 6.17 and 6.18 and deduce a pile length L_2 which will satisfy the design conditions (if no pile exists, take $L_2 = \infty$). Check bearing considerations - Step 6).

(vi) Take the total pile length (as measured from the rock surface) to be the minimum of L_1 and L_2 as calculated above.

To illustrate this procedure, consider a pile to be designed for $E_p/E_r = 50$, $(L/D)_{dmax} = 4.5$ and $I = 0.45$. Neglecting seams, this pile could be designed as shown in Figure 6.2 for $(L/D)_d = 2.2$ and $P_t/P_b = 51\%$. Supposing that 20% of this socket length is composed of weathered seams (i.e., $S = 0.2$) with $E_s/E_r = 0.1$, $\tau_s/\tau_r = 0.1$.

1. If the seams are at the top of the socket, $L_e = 0.2$ L and

$$L_1 = (2.2 + 0.4)D = 2.6D$$

If the seams are distributed over the entire length L or near the base of the socket then $L_e = L$ and

$$L_1 = (2.2 + 2.2)D = 4.4D$$

2. Inspection of Figure 6.5 (i.e., Figures C7 and C10) indicates that for $I_s = I_d = 0.45$, $I_h/I_s = 1$ and 0.88 for $E_p/E_r = 10$ and 100 respectively. Since $E_p/E_r = 50$ in this case, interpolation is necessary giving $I_h/I_s = 0.91$ (assuming I_h/I_s varies with the logarithm of E_p/E_r).

Thus, $I_d^* = I_h = 0.91 \times 0.45 = 0.41$
and the modified maximum pile length for full slip design becomes

$$\begin{aligned}(L/D)^*_{dmax} &= (L/D)_{dmax}/(1-S + S\tau_s/\overline{\tau}) \\ &= 4.5/0.82 \\ &= 5.5\end{aligned}$$

Now repeating the procedure illustrated in Figure 6.2 but using $I_d^* = 0.41$ and $(L/D)_{dmax} = 5.5$ gives $(L/D)_d = 3.0$ and $(P_b/P_t)_d = 45\%$.
Thus $L_2 = 3D$.
If the seams are localized at the top of the pile
take $L = \min(L_1,L_2) = \min(2.6D,3D) = 2.6D$
If the seams are distributed along the pile,

$$L = \min(4.4D,3D) = 3D$$

Probability of Exceeding the Design Settlement

As indicated in section 6.3.3, the values of $(RF)_\tau$ and $(RF)_E$ used in the design will control the probability of the design settlement being exceeded. Thus for $(RF)_\tau = (RF)_{E_r} = 0.7$ and $(RF)_\tau = (RF)_{E_r} = 0.5$, the probability of exceeding the design settlement is less than 30% and 11% respectively at all load levels. In fact, at certain load levels, the probability of exceeding the design settlement ρ_d may be significantly smaller than these values. To check the probability of exceedance for a given design proceed as follows.

(i) Using the design chart for $E_p/\overline{E}_r$, extend a vertical from the L/D axis corresponding to $(L/D)_d$ found previously, to the full slip contour (upper dotted line), draw a horizontal line from this intersection point to the P_b/P_t axis and read the proportion of load transferred to the base of the pile at full slip $(P_b/P_t)_{fs}$ (eg. see Fig. 6.6).

(ii) Determine the ratio P_t/P_{ts};

- If there is full slip (i.e., $(P_b/P_t)_d > (P_b/P_t)_{fs}$) then

$$\frac{P_t}{P_{ts}} = (RF)_\tau \cdot \frac{[1-(P_b/P_t)_{fs}]}{[1-(P_b/P_t)_d]} \tag{6.19}$$

- If there is only partial or no slip then calculate

$$P_{ts} = \frac{P_s}{(1-(P_b/P_t)_{fs})} \tag{6.20}$$

where P_s = the load carried in sideshear at full slip
$= \pi DL\overline{\tau}$

and hence calculate P_t/P_{ts} from the known values of P_t and P_{ts}.

(iii) Using the appropriate charts (see Chapter 5) for the values of $(RF)_\tau$, $(RF)_{E_r}$, P_t/P_{ts}, q_u used in the design, estimate the probability of exceeding the design settlement (i.e., $PR(\rho > \rho_d)$).

To illustrate this consider the earlier example of a pile with $E_p/E_r = E_p/E_d = 50$, $(L/D)_{dmax} = 4.5$, $I_d = 0.45$, $(L/D)_d = 2.2$, $(P_b/P_t)_d = 51\%$. The probability of exceeding the design settlement will depend on the values of $(RF)_{E_r}$ and $(RF)_\tau$ used in the design.

If $(RF)_{E_r} = 0.5$, then

$$\frac{E_p}{E_r} = (RF)_{E_r} \cdot \frac{E_p}{E_d} = 0.5 \times 50 = 25$$

Thus from Figure C2c (reproduced as Figure 6.6) for $L/D = 2.2$, $E_p/E_r = 25$ the value of $(P_b/P_t)_{fs} = 24\%$ for this pile. Since $(P_b/P_t)_d > (P_b/P_t)_{fs}$ then from Eq. 6.19

$$\frac{P_t}{P_{ts}} = (RF)_\tau \frac{[1-0.24]}{[1-0.51]} = 1.55(RF)_\tau$$

$$= 0.78 \text{ for } (RF)_\tau = 0.5$$

Hence from Figure 5.8 (replotted as Figure 6.7) $PR(\rho > \rho_d) = 4.5\%$ (i.e., the probability that the design settlement will be exceeded is 4.5%).

If $(RF)_{E_r} = 0.7$ then $E_p/E_r = 0.7 \times 50 = 35$. Since no chart is given for $E_p/E_r = 35$, the value of $(P_b/P_t)_{fs}$ must be obtained by an interpolation between the values of 24% and 27% for $E_p/E_r = 25$ and 50 (refer to Figures C2c and C2d). Taking $(P_b/P_t)_{fs} = 26\%$

$$\frac{P_t}{P_{ts}} = (RF)_\tau \frac{[1-0.26]}{[1-0.51]} = 1.51(RF)_\tau$$

and if $(RF)_\tau = 0.7$ then $P_t/P_{ts} = 1.06$ and $PR(\rho>\rho_d) = 21\%$ from Figure 5.7 (i.e. the probability of exceeding the design settlement is 21%). If $(RF)_{E_r} = 0.7$ and $(RF)_\tau = 0.5$ then $P_t/P_{ts} = 0.8$ and $PR(\rho>\rho_d) = 16\%$ from Figure 5.7.

6.4.3 Comments Regarding the Design Procedure

The design procedure described in the previous section involves selecting a pile geometry such that:

(1) the probability of exceeding the design settlement ρ_d is acceptably small;

(2) the bearing pressure at the base of the pile under design conditions is less than a specified allowable value q_{ba}; and

(3) the maximum pressure that would develop at the base of the pile if only 30% of the expected sideshear resistance were mobilized will not exceed one third of the estimated base bearing capacity (i.e., $q_b < q_{ba}$).

Normally, one would want the probability of exceeding the design settlement to be less than 30% (this will give a probability of exceeding twice the design settlement of less than 3%). Consequently, the expected observed settlement of a pile designed to satisfy conditions (1) to (3) above will be less than the design settlement (i.e., the design settlement is not the settlement that would be predicted for piles with the selected geometry assuming the most probable rock pile properties).

Condition (1) ensures that the serviceability limit state criterion is satisfied. Condition (2) is intended to limit yielding of the rock beneath the base of the pile under design conditions.

Considerable research has been conducted into available sideshear resistance of pile-rock sockets under short term conditions as described in Chapter 4. However, very little research has been conducted into the long term load-transfer behaviour of socketed piles. Based on the very limited available data (eg. Ladanyi, 1977; Horvath, 1980), it would appear that some additional load transfer to the base of the socket will occur with time due to the time dependent characteristics of the rock and concrete. Although there is not enough data for any firm conclusions to be reached, it may be speculated that this time dependent behaviour will be more critical for smooth than for rough (eg. R4) sockets. Because of this uncertainty as to exactly how much load will eventually be carried in sideshear, condition (3) above is intended to ensure that even if only 30% of the expected sideshear resistance is mobilized, there will still be a "factor of safety" of at least 3 against bearing capacity failure of the pile.

When designing piles in rock which is considerably less stiff (i.e., $E_p/E_r > 10$), the design of piles to satisfy the settlement criteria is quite straightforward and invariably increasing the pile length will reduce settlement. Nevertheless, if the design settlement is too restrictive for the given rock, it may not be possible to design a pile of given diameter and reasonable length ($L/D < 10$) if the settlement

criteria are too restrictive. Weak rock should be regarded, in a sense, a very strong soil - i.e., if for a given structure constructed on soil you would be prepared to accept a settlement ρ, there is no reason why you should not be prepared to accept the same settlement ρ for the same structure constructed on weak rock. One cannot expect zero settlement for piles constructed on weak rock. Any attempt to achieve negligible settlement will necessarily involve large diameter piles and low applied pressures; this will be expensive.

When designing piles with a modulus approaching that of the concrete the design charts may give what at first sight appears to be strange results. For example, an inspection of Figure 6.8a shows that for $E_p/E_r = 1$, increasing pile length could actually increase settlement! This result arises from the fact that if the rock and concrete have the same modulus, and a socket is placed in the rock, then the interface between the rock and concrete represents a plane of weakness; the longer the pile the more the rock is weakened and the larger will be the settlement once slip is induced at the interface. Clearly, if this is the case, then it is better to have an endbearing pile at the surface of the rock (or perhaps socketed to a nominal $L/D = 1$), than to try and design a socketed pile.

For piles only slightly less stiff than the rock (eg. $E_p/E_r = 10$), some benefit (in terms of reduced settlement) may be derived from socketing the pile into the rock however the benefit of the higher concrete modulus can be partially offset by the weakening of the rock at the pile-rock interface. These competing effects can give rise to a situation where the pile length required to satisfy a given design condition (i.e., to give a specified I) may not be unique.

For example, the factored design line shown in Figure 6.8b for $(L/D)_{dmax}$ = 10 intersects the contour for I = 0.8 at L/D = 3.8 and 7.2. Closer inspection of these results shows that for this design line, increasing the pile length from L/D = 2 up to L/D = 5.2 will result in increasing I with increasing pile length. For pile lengths exceeding 5.2, there will be a decrease in I with increasing pile length. This still leaves the question: "If I am designing for I_d = 0.8 which value of L/D do I adopt". The answer is neither. If one is prepared to accept I_d = 0.8, then I_d < 0.8 is even better (for a given load this implies less settlement) and one could achieve I = 0.75 for an L/D = 1 for these conditions. This would give better settlement performance at lower cost than either a pile with L/D = 3.8 or 7.2. Of course, it still remains to check endbearing and it may be necessary to increase the pile length (or diameter) to satisfy this condition. If the designer chooses to increase the pile length to satisfy bearing considerations, care must be taken not to adopt a length which would give I > I_d (i.e., in this example we require L/D < 3.8 or L/D > 7.2).

The foregoing example illustrates a case for E_p/E_r = 10 where it may be better not to socket the pile into the rock to more than one diameter if $I_d \leq 0.8$ is acceptable. However, there are also situations where increasing the pile length will be beneficial for E_p/E_r = 10. For example, if we adopt the same design line as in the previous example, then when designing for I $\leq$ 0.7, there is a unique L/D ratio for the

specified value of I and increasing the socket length L will result in a corresponding decrease in I.

6.5 EXAMPLE DESIGN CALCULATIONS

To illustrate the use of the proposed design method, several sets of design calculations will be performed using data obtained from a socketed pile case history.

The case history considered for the present study has been described by Horvath (1982) and Horvath et al. (1983). Additional field data required for the design calculations (eg. borehole logs, results from laboratory and field tests on the in-situ rock) has been obtained from Horvath (1980).

These design calculations are intended to:

(1) Show how the proposed design method is used,

(2) Assess the merits of the proposed design method as illustrated by a comparison between the "designed" socketed piles and the observed behaviour in Horvath's tests.

6.5.1 Parameters Used in Design Calculations

The two field tested piles which have been used to provide a comparison with the "designed" socketed piles have been designated by Horvath (1980,1982) as P2 and P4. Both sockets were constructed in Queenston shale using conventional drilling methods. Socket P2 was a relatively smooth-sided socket while socket P4 was artificially roughened (i.e., a R4 roughness) using a special pneumatic roughening tool.

Figure 6.9 shows the general socket geometry of the two complete socketed piles. Table 6.2 provides the relevant design information. The backfigured modulus deduced from Horvath's load-displacement curves have been reinterpreted (see Chapter 4, section 4.3.2) and have been used in these design calculations.

Borehole logs for the two socket test locations are shown in Figures 6.10 and 6.11. Of particular interest is the fractured zone of rock at a depth of approximately 2.8 - 3.4 m (below ground surface) observed in the log for socket P2. This zone of rock may be expected to possess a lower rock mass modulus than the rock above or below it and is likely to have some influence on the pile behaviour. This may partly explain the difference between the backfigured rock mass modulus exhibited by sockets P2 and P4. The importance of considering this aspect in the design calculations will be discussed subsequently.

6.5.2 Detailed Design Calculations - Socket P2

The first set of design calculations are performed to illustrate how the design approach is used.

Socket P2 will be initially designed assuming,

(i) the rock mass is non-homogeneous (i.e. $0.5<E_b/E_r<1.0$).

(ii) a representative value of unconfined compressive strength for the rock mass determined by Horvath (1980), i.e., q_u = 6.75 MPa.

(iii) the reduction factors applied to $\overline{\tau}$ and $\overline{E}_r$ are chosen to meet a serviceability limit state (i.e., $(RF)_\tau = 0.7$ and $(RF)_{E_r} = 0.7$).

(iv) the design settlement ρ_d = 8.5 mm for a load P_t = 4.45 MN.

(v) the pile diameter corresponds to Horvath's piles P2, i.e., D = 0.71 m.

(vi) the modulus and sideshear resistance can be deduced from the proposed empirical correlations.

(vii) the pile is recessed to a depth of 0.6 m (to conform with Horvath's test).

STEP 1

q_u = 6.75 MPa (see Table 6.2)

for a regular socket (eg. roughness < R4)

(a) $\overline{\tau} = .45\sqrt{6.75} = 1.17$ MPa

and $\overline{E}_r = 215\sqrt{6.75} = 560$ MPa

(b) $\tau_d = 0.7(1.17) = .82$ MPa

$E_d = 0.7(560) = 390$ MPa

(for serviceability limit state)

STEP 2

ρ_d = 0.0085 m (8.5 mm)

D = .71 m

P_t = 4.45 MN

E_p = 37,000 MPa (Horvath, 1980)

STEP 3

An inspection of the bore log at socket P2 suggests:

(1) A weathered zone exists to a depth of approximately 0.5 m. The rock

in the weathered zone may be expected to have a modulus E_t less than the modulus of the rock adjacent to the socket E_r. The effect of recessing the socket to a depth of 0.6 m (as was done at socket P2) will therefore be neglected (i.e., this implies $E_t/E_r=0$).

(2) At a depth of approximately 2.8 - 3.4 m, a zone of highly fractured rock exists, where the RQD % (rock quality designation eg. see Deere, 1968) is substantially lower than values of RQD either above or below this zone. Consequently, this zone should be considered in designing the pile.

Based on these observations and the empirical correlations,Take

$E_p/E_d = 37000/390 = 95$

$E_b/E_r = 0.75$ (The weaker zone of rock at a depth of 2.8-3.4 m will likely possess a modulus ratio E_b/E_r between 1.0 and 0.5. Therefore an intermediate value will be used.)

$E_t/E_r = 0$ (Neglect the effect of recessment to a depth of 0.6 m.)

Therefore,

$$\left(\frac{L}{D}\right)_{dmax} = \frac{P_t}{\pi D^2 \tau_d} = \frac{4.45\text{ MN}}{\pi(0.71\text{m})^2(0.82\text{MPa})} = 3.4 .$$

STEP 4

$$I_d = \frac{\rho_d E_d D}{P_t} = \frac{(0.0085\text{m})(390\text{MPa})(0.71\text{m})}{(4.45\text{ MN})} = 0.53$$

STEP 5

For $E_p/E_d = 95$, $E_b/E_r = 1$ we can use design charts C2e and C3e for $E_p/E_r = 100$ (reproduced as Figure 6.12a and 6.12b) and construct a "factored design line" for $(L/D)_{dmax} = 3.4$. From the intersection of the design lines with the interpolated contour for $I_d = 0.53$, interpolation between the charts gives:

$(L/D)_d = 1.9$; $(P_b/P_t)_d = 45\%$.

Note 1: Linear interpolation between design charts for $E_p/E_r = 100$ and $E_b/E_r = 1.0, 0.5$ were used to estimate design values for $(L/D)_d$ and $(P_b/P_t)_d$ for $E_b/E_r = 0.75$.

Note 2: Since $50 < E_p/E_d = 95 < 100$ we could also deduce the values of $(L/D)_d$ and $(P_b/P_t)_d$ for $E_p/E_r = 50$ and interpolate between these results and the one obtained above for $E_p/E_r = 100$. In fact, the values of $(L/D)_d$ and $(P_b/P_t)_d$ obtained for $E_p/E_r = 50$ are 1.91 and 44% and it can be seen that in this case no interpolation was necessary to get results for $E_p/E_d = 95$).

STEP 6

For $(L/D)_d = 1.9$, the required socket length $L = 1.9\times0.71=1.35$ m and the socket will extend from depth 0.6 m to 1.95 m. The socket will terminate in the sound rock.

$$q_t = \frac{P_t}{\pi D^2/4} = \frac{4.45}{\pi \times 0.71^2 \times 0.25} = 11.2 \text{ MPa}$$

$$q_b = (P_b/P_t)_d\, q_t = 0.45 \times 11.2 = 5.1 \text{ MPa}$$

$$q_m = q_t - 4\left(\frac{L}{D}\right)_d (RF)_s \bar{\tau}$$

$$= 11.2 - 4\times1.9\times0.3\times1.17 = 8.5 \text{ MPa}$$

For this case

$$q_{ba} = q_u = 6.75 \text{ MPa}$$

$$\therefore \quad q_b < q_{ba} \qquad \text{OK}$$

$$q_{ma} = 2.5\ q_u = 2.5 \times 6.75 = 16.9 \text{ MPa}$$

$$\therefore \quad q_m < q_{ma} \qquad \text{OK}$$

$\therefore$ The pile is acceptable for a design settlement of 8.5 mm.

Probability of Exceeding Design Settlement

For $E_p/\overline{E}_r = 66$, the full slip load distribution for L/D = 1.9 is given by $(P_b/P_t)_{fs} = 24\%$

$$\therefore \quad \left(\frac{P_t}{P_{ts}}\right) = (RF)_\tau \frac{[1-(P_b/P_t)_{fs}]}{[1-(P_b/P_t)_d]} = 0.7 \times \frac{[1-0.24]}{[1-0.45]} = 0.97$$

Thus for $(RF)_{E_r} = 0.7$ and $(P_t/P_{ts}) = 0.97$, Figure 5.7 gives $PR(\rho > \rho_d)$ = 22% (i.e., the probability of exceeding the design settlement of 8.5 mm is 22%). The foregoing calculations are summarized in Table 6.3.

Observed Behaviour

If the conditions at socket P2 were considered representative of both sockets P2 and P4 (in fact the rock conditions at P4 are slightly better) then the displacement of both sockets which had an L/D = 1.93 may be compared with the design settlement ρ_d = 8.5 mm at the "design" load of 4.45 MN. In fact, the observed settlements at the head of the socket (i.e., at a depth of 0.6 m) were 8.4 mm and 5.9 mm at sockets P2 and P4 respectively. Clearly, the design criterion is satisfied in both cases.

6.5.3 Additional Design Calculations for Sockets P2 and P4

The previous section gave a detailed calculation for the design of a pile at location P2 using empirical correlations and a design settlement of 8.5 mm. If a field load test had been performed indicating that $\overline{E}_r$= 283 MPa and $\overline{\tau}$ = 1.45 MPa (which are in fact the results backfigured for socket P2) then the pile could be redesigned using these field values of $\overline{E}_r$ and $\overline{\tau}$ rather than empirical correlations.

Note that the backfigured modulus is lower than that deduced from empirical correlations. It is likely that this is the result of the zone of fractured rock beneath the base of socketed pile P2. However, since the presence of this zone is incorporated into the backfigured value, design calculations using this modulus should be performed by considering $E_b/E_r = 1.0$.

Since the backfigured modulus for socket P2 is lower than the value assumed in the previous calculation, the design of this socket for the same design settlement ρ_d = 8.5 mm would result in a socket larger than 1.9D. To permit a direct comparison with the field data, the socket will, therefore, be designed for ρ_d = 11.5 mm thereby giving a design with $(L/D)_d$ = 1.9 as shown in Table 6.3. Again, the observed settlement is less than the design settlement (8.4 mm as compared to 11.5 mm) and the design criterion is satisfied.

Similar calculations may be performed for socket P4 as shown in Table 6.4. An inspection of the bore log shows that this socket has negligible seams and the rock mass appears to be relatively free from significant fractures with depth, accordingly the backfigured modulus is higher than that at P2 (and closer to the value obtained from empirical correlations). Also, the mobilized sideshear resistance is higher because of artificial roughening of the socket. The design of a socket at P4 for a design settlement of ρ_d = 6.0 mm and 8.5 mm using empirical correlations and backfigured parameters respectively gives a length of 1.9D. The observed settlement of the socket was 5.9 mm and again the design criterion is satisfied in both cases.

Note that the design of a pile at P4 using empirical parameters involves slip (under the design conditions with reduction factor of 0.7 applied). The corresponding design using backfigured parameters involves no slip (i.e., an elastic response).

It can be seen from the foregoing calculations that the piles at locations P2 and P4 could have been safely designed to satisfy a design settlement criterion at a load P_t = 4.45 MN (using serviceability reduction factors of 0.7 for both modulus and sideshear resistance) either using empirical correlations and design adjustment for the effect of seams or using backfigured parameters from a field test (which implicitly includes the effect of seams and the zone of fractured rock). Both piles were eventually loaded to almost twice the "design load" without any sign of collapse when the test was terminated.

6.5.4 Comments

From the design calculations outlined above, it is concluded that:

1. A designer, familiar with the design approach, can perform a socket design for a given set of input parameters in 10-15 minutes.
2. The empirical parameters used to deduce $\overline{\tau}$ and $\overline{E}_r$ provided a reasonable design.

3. Parameters backfigured from a pile load test also provided a reasonable design. These parameters implicitly included the effects of the clay filled joints, seams and fractured rock zones and these need not be considered directly in the design provided that the prototype pile is representative of the final pile configuration.

6.6 SUMMARY

A brief review of the available design methods indicates that current approaches based on the use of allowable endbearing pressures may result in overly conservative designs.

A new design procedure has been proposed. This new procedure is relatively simple and a socket can be designed in less than 15 minutes once the designer is familiar with the technique. The design method is based on:

(1) satisfying a specified design settlement criterion
(2) checking to ensure there is an adequate factor of safety against collapse.

The design method has been illustrated by means of a number of hypothetical examples and by a series of detailed calculations relating to the complete sockets P2 and P4 tested by Horvath (1980). These calculations show that piles designed to have the same geometry as those tested by Horvath (1980) would have satisfied the design settlement criteria while having a proven "factor of safety" against collapse of at least 2. (Since the test piles were not brought to collapse the actual "factor of safety" is unknown.)

TABLE 6.1 OBSERVED BEARING PRESSURES FROM SEVERAL SOCKETED PILE CASE HISTORIES

Case	Diameter D(m)	Depth to Base $(\frac{L_e+L}{D})$	Unconfined Compressive Strength q_u(MPa)	First Yield Pressure q (MPa)	First Yield Pressure q/q_u	Displacement at First Yield (mm)	Maximum Sustained Pressure q (MPa)	Maximum Sustained Pressure q/q_u	Displacment at End of Test (mm)	Comment
Horvath (1980)										
P2	0.71	2.77	6.75	>9.1	>1.35	>23	9.1	1.35	23	Tests on complete socketed piles
P4	0.71	2.77	6.75	>6.5	>0.96	>12	6.5	0.96	12	
Glos & Briggs (1983)										
EAST	0.61	12.4	9.26	11.6	1.25	7	13.1	1.41	8.0	Tests on complete socketed piles
WEST	0.61	12.3	8.35	7.9	.95	7	10.6	1.29	9.7	
Williams (1980)										
S9	0.3	6.7	0.65	1.4	2.2	1	6.4	9.8	19.1	Tests on endbearing only socketed piles
S11	0.3	3.3	0.67	1.4	2.1	1.3	7.0	10.5	17.2	
M1	1.0	15.5	2.68	4.8	1.8	7.2	5.9	2.2	10.8	
M2	1.0	15.5	2.45	4.4	1.8	4.3	6.6	2.7	7.3	
M3	1.0	15.5	2.45	3.9	1.6	2.3	7.0	2.9	~ 6.0	
M4	1.0	15.5	2.68	3.8	1.4	3.2	6.7	2.5	7.3	
M6	0.6	3	1.93	3.3	1.7	2.9	9.2	4.8	84.3	
M7	1.0	3	1.4	3.5	2.5	15.8	7.1	5.0	109.0	

NOTE: (1) The load was still increasing when all of these tests were terminated.

(2) DISPLACEMENTS FOR HORVATH AND GLOS AND BRIGGS tests are referenced to the head of the socketed pile. WILLIAM'S tests are referenced to the base of the socketed pile.

TABLE 6.2 RELEVANT DESIGN INFORMATION FOR SOCKETED PILES P2 AND P4

	SOCKET P2	SOCKET P4
Average Unconfined Compressive Strength (at site) for Shale, q_u (MPa)	6.75	6.75
Backfigured Rock Mass Modulus, $E_{r_{field}}$ (MPa)	283.	377.
Measured Peak Average Sideshear Resistance, $_{field}$ (MPa)	1.45	> 1.8
Roughness Category	R2 (i.e. < R4)	R4
Modulus of Pile Concrete (GPa)	37	37
Length of Socketed Pile (m)	1.37	1.37
Diameter of Socketed Pile (m)	.71	.71
Recessed Pile Length (m)	.6	.6

TABLE 6.3: DESIGN CALCULATIONS FOR PILE P2

Quantity	Design Based on Empirical Correlations With Corrections for Seams		Design Based on Backfigured Parameters (which include the effect of seams)	
	Value	Notes	Value	Notes
1. Location	P2		P2	
2. ρ_d(m)	0.0085		0.0115	
3. P_t (MN)	4.45		4.45	
4. E_p (MPa)	37000		37000	
5. q_u (MPa)	6.75	Horvath (1980)	6.75	
6. $\bar{\tau}$ (MPa)	1.17	Eq. 6.1a	1.45	Backfigured
7. $\bar{E}_r$ (MPa)	560	Eq. 6.2	283	Backfigured
8. $(RF)_\tau$	0.7		0.7	
9. $(RF)_{E_r}$	0.7		0.7	
10. τ_d (MPa)	0.82	Eq. 6.3a	1.02	Eq. 6.3a
11. E_d (MPa)	390	Eq. 6.3b	198	Eq. 6.3b
12. E_p/E_d	95		187	
13. E_b/E_r	0.75	Assume weaker base rock	1	Assume effect of weaker base rock included in backfigured parameters
14. L_e (m)	0.6	Recessed socket	0.6	Recessed socket
15. E_t/E_r	0.0	Neglect recessing effect	0.0	Neglect recessing effect
16. D (m)	0.71	To conform to Horvath's pile	0.71	
17. $(L/D)_{dmax}$	3.4	Eq. 6.8	2.75	Eq. 6.8
18. I_d	0.56	Eq. 6.11	0.36	Eq. 6.11
19. S	~0	Minor occurrence of seams based on interpretation of the bore log. Neglect effect of seams.	-	Effect of any seams included in the backfigured parameters.
20. E_s/E_r	-		-	
21. τ_s/τ_r	-		-	
22. I_h/I_s	-		-	

Table 6.3 (cont'd)

	Value	Notes	Value	Notes
23. $(L/D)^*_{dmax}$	-		-	
24. I^*_d	-		-	
25. $(L/D)_d$	1.9	Interpolated from Fig. 6.12a,b	1.9	Interpolated from Fig. C2e and C2f
26. $(P_b/P_t)_d$(%)	45	"	32	"
27. L (m)	1.35		1.35	
28. q_t (MPa)	11.2	Eq. 6.12c	11.2	Eq. 6.12c
29. q_{ba} (MPa)	6.75	Eq. 6.4a	6.75	Eq. 6.4a
30. q_b (MPa)	5.1	Eq. 6.12a $<q_{ba}$,OK	3.6	Eq. 6.12a $<q_{ba}$,OK
31. $\bar{\tau}^*$ (MPa)	0.96	From [6]	1.45	From [6] - no seams
32. q_{ma} (MPa)	16.9	Eq. 6.4b	16.9	Eq. 6.4b
33. q_m (MPa)	8.5	Eq. 6.12b $<q_{ma}$,OK	7.9	Eq. 6.12b $<q_{ma}$,OK
34. $E_p/\bar{E}_r$	66		130	
35. $(P_b/P_t)_{fs}$(%)	24	Fig. C2d/C2e and C3d/C3e	31	Fig. C2e/C2f
36. P_t/P_{ts}	1.02	Eq. 6.19	0.71	Assumption that the standard deviation of material properties is the same as for empirical correlation.
37. $PR(\rho>\rho_d)$(%)	22	Fig. 5.7	24	Fig. 5.7
38. Observed Settlement (m)	0.0084	Less than ρ_d,OK	0.0084	Less than ρ_d,OK

TABLE 6.4: DESIGN CALCULATIONS FOR PILE P4

Quantity	Design Based on Empirical Correlations		Design Based on Backfigured Parameters	
	Value	Notes	Value	Notes
1. Location	P4		P4	
2. ρ_d (m)	0.006		0.0085	
3. P_t (MN)	4.45		4.45	
4. E_p (MPa)	37000		37000	
5. q_u (MPa)	6.75	Horvath (1980)	6.75	
6. $\bar{\tau}$ (MPa)	1.56	Eq. 6.1b for rough socket	1.8	Backfigured from field test data
7. $\bar{E}_r$ (MPa)	560	Eq. 6.2	377	"
8. $(RF)_\tau$	0.7		0.7	
9. $(RF)_{E_r}$	0.7		0.7	
10. τ_d (MPa)	1.09	Eq. 6.3a	1.26	
11. E_d (MPa)	390	Eq. 6.3b	264	Eq. 6.3b
12. E_p/E_d	95		140	
13. E_b/E_r	1	Sound base	1	Sound base
14. L_e (m)	0.6	Recessed socket	0.6	Recessed socket
15. E_t/E_r	0	Neglect recessing	0	Neglect recessing
16. D (m)	0.71	To conform with Horvath (1980)	0.71	
17. $(L/D)_{dmax}$	2.6	Eq. 6.8	2.2	Eq. 6.8
18. I_d	0.4	Eq. 6.11	0.36	Eq. 6.11
19. S	~0	Minor occurrence of seams based on interpretation of the bore log. Neglect effect of seams.	-	Effect of seams included in the backfigured parameters.
20. E_s/E_r	-	"	-	"
21. τ_s/τ_r	-	"	-	"
22. I_h/I_s	-	"	-	"

Table 6.4 (cont'd)

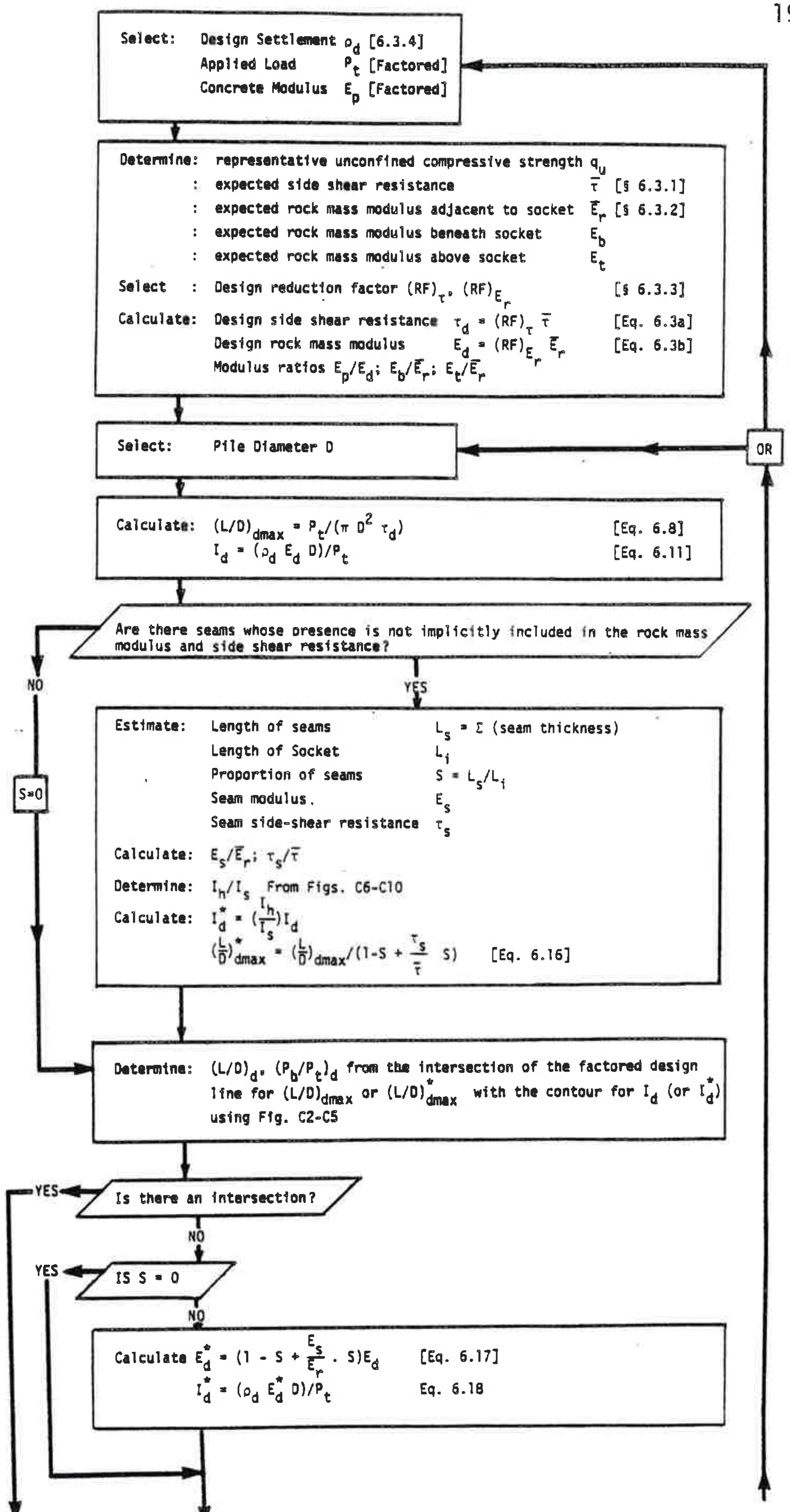
	Value	Notes	Value	Notes
23. $(L/D)^*_{dmax}$	-	"	-	"
24. I^*_d	-	"	-	"
25. $(L/D)_d$	1.9	Fig. 6.12a	1.9	Note: Fig. 6.12a will not give an intersection between the Factored Design Line and I_d = 0.36. Referring to Fig. 6.3 gives an elastic solution for $(L/D)_d$=1.9
26. $(P_b/P_t)_d$(%)	31	Fig. 6.12a	27	
27. L (m)	1.35		1.35	(P_b/P_t) from Fig. 6.4.
28. q_t (MPa)	11.2	Eq. 6.12c	11.2	Eq. 6.12c
29. q_{ba}(MPa)	6.75	Eq. 6.4a	6.75	Eq. 6.4a
30. q_b (MPa)	3.5	Eq. 6.12a $<q_{ba}$,OK	3.0	Eq. 6.12a $<q_{ba}$,OK
31. $\bar{\tau}^*$ (MPa)	1.56	From [6]	1.8	From [6] above
32. q_{ma}(MPa)	16.9	Eq. 6.4b	16.9	Eq. 6.4b
33. q_m (MPa)	7.6	Eq. 6.12b $<q_{ma}$,OK	7.1	Eq. 6.12b $<q_{ma}$,OK
34. $E_p/\bar{E}_r$	66		98	
35. $(P_b/P_t)_{fs}$(%)	29	Figs. C2d/C2e	31	Fig. C2e
36. P_t/P_{ts}	0.68	Eq. 6.19	0.57	From Eq. 6.20 (Note: Eq. 6.19 is not valid in this case since $(P_b/P_t)_d < (P_b/P_t)_{fs}$
37. $PR(\rho>\rho_d)$%	25	Fig. 5.7	26	Fig. 5.7
38. Observed Settlement (m)	0.0059	less than ρ_d,OK	0.0059	less than ρ_d,OK

TABLE 6.5: SUMMARY OF EQUATIONS

Equation No.	Equation
6.1a	$\bar{\tau}(\text{MPa}) = 0.45\ \sqrt{q_u}$
6.1b	$\bar{\tau}(\text{MPa}) = 0.6\ \sqrt{q_u}$ (Roughness R4)
6.2	$\bar{E}_r = 215\ \sqrt{q_u}$
6.3a	$\tau_d = (RF)_\tau \cdot \bar{\tau}$
6.3b	$E_d = (RF)_{E_r} \cdot \bar{E}_r$
6.4a	$q_{ba} = q_u$
6.4b	$q_{ma} = 2.5\ q_u$
6.8	$(L/D)_{dmax} = P_t/(\pi\ D^2\ \tau_d)$
6.11	$I_d = \rho_d\ E_d\ D/P_t$
6.12a	$q_b = (P_b/P_t)_d\ q_t$
6.12b	$q_m = q_t - 4(RF)_s\ (\frac{L}{D})_d\ \bar{\tau}$ $[(RF)_s = 0.3]$
6.12c	$q_t = P_t/(\pi\ D^2/4)$
6.13a	$q_b \leq q_{ba}$
6.13b	$q_m \leq q_{ma}$
6.14	$L_1 = L_i + L_e$
6.15a	$\bar{\tau}^* = (1 - S + S\ \tau_s/\bar{\tau}\ )\bar{\tau}$
6.15b	$\bar{\tau}_d^* = (RF)_\tau \cdot \bar{\tau}^*$
6.16	$(L/D)^*_{dmax} = (L/D)_{dmax}/(1 - S + S\ \tau_s/\bar{\tau})$
6.17	$E_d^* = (1 - S + S\ E_s/\bar{E}_r)E_d$

Table 6.5 (Continued)

Equation No.	Equation
6.18	$I_d^* = (\rho_d\ E_d^*\ D)/P_t$
6.19	$P_t/P_{ts} = (RF)_\tau\ (1 - (P_b/P_t)_{fs})/(1 - (P_b/P_t)_d)$
6.20	$P_{ts} = P_s/(1 - (P_b/P_t)_{fs})$
6.21	$P_s = \pi DL\bar{\tau}$



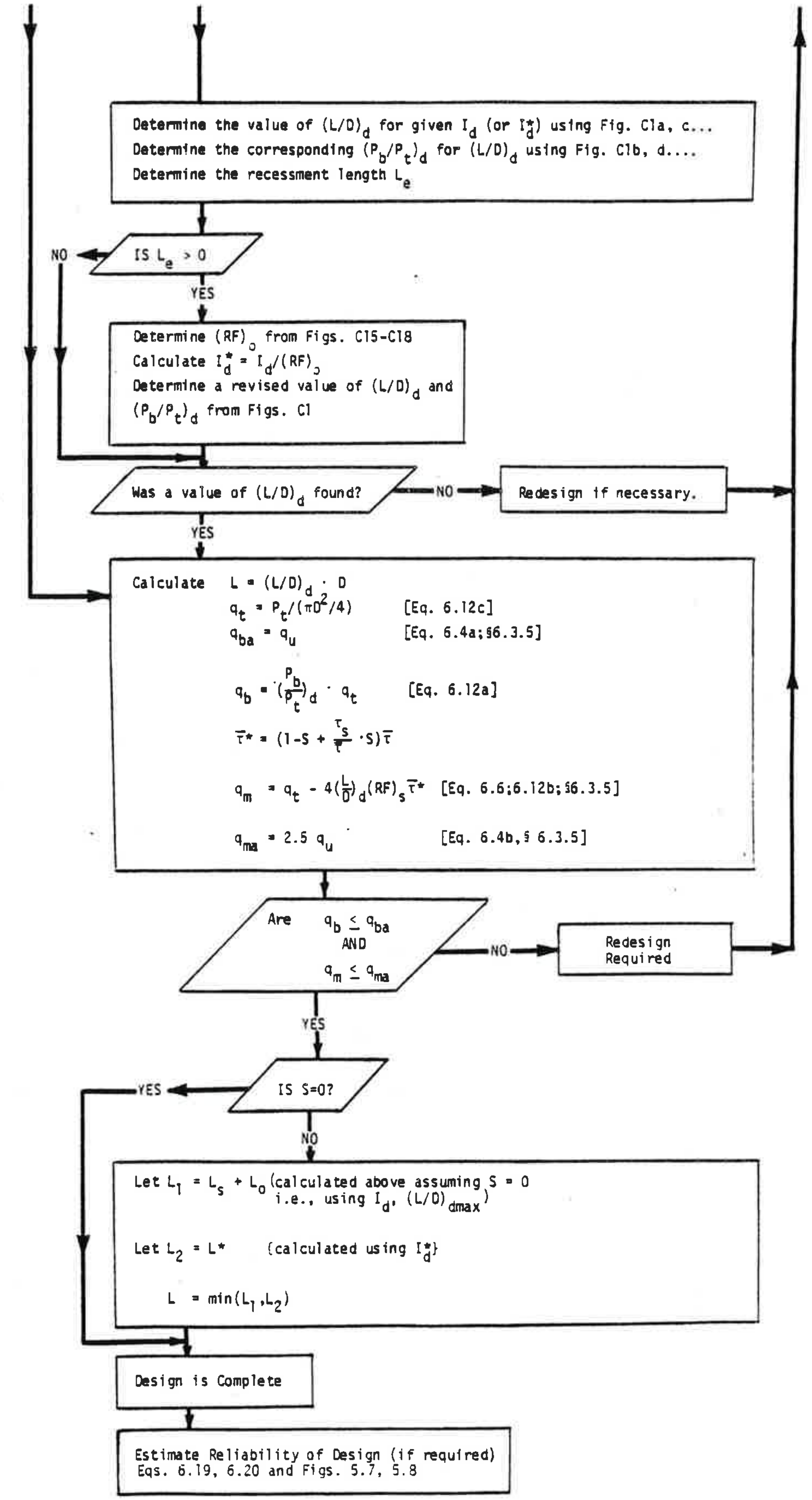


FIGURE 6.1 FLOW CHART FOR THE DESIGN PROCEDURE

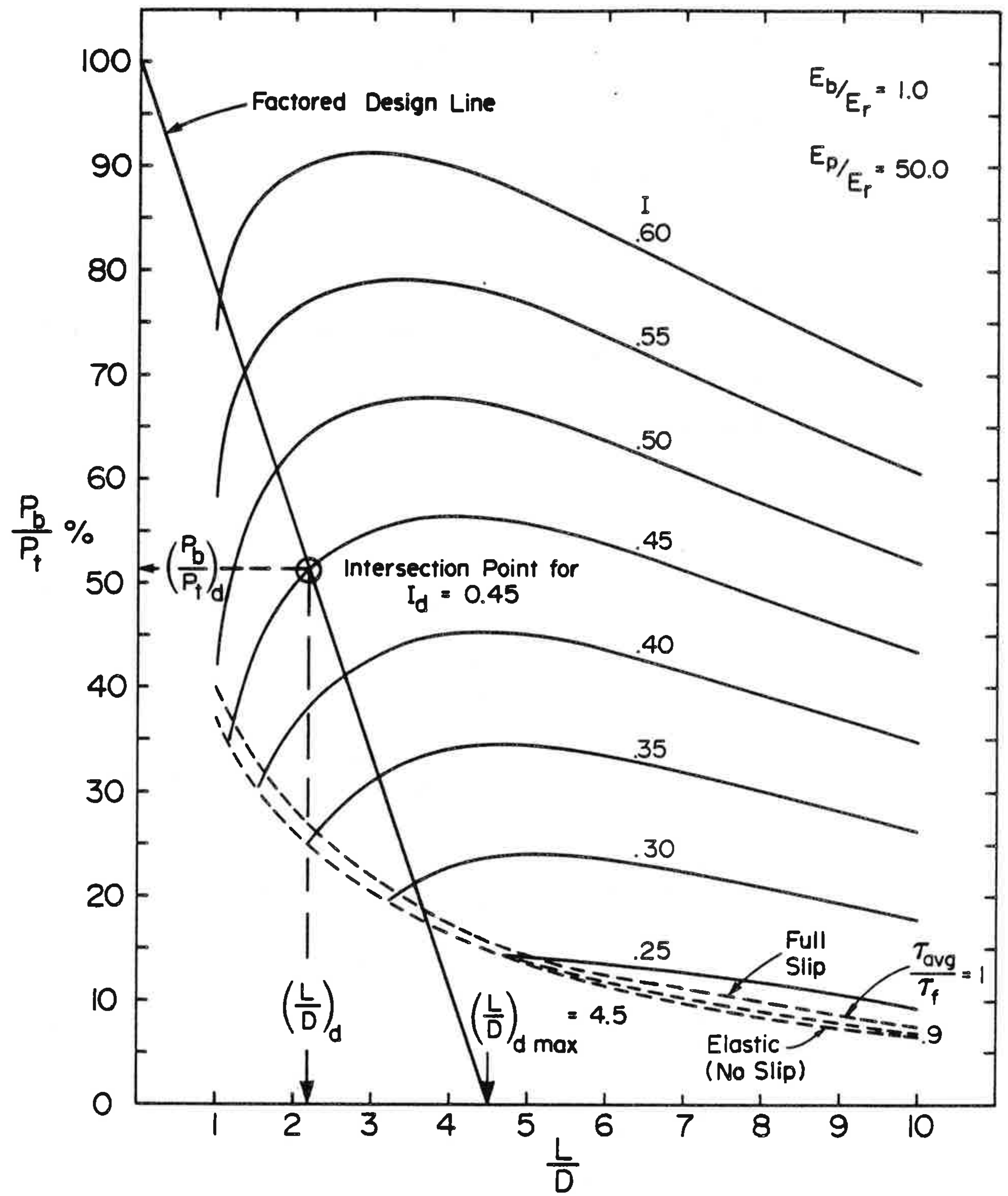


FIGURE 6.2 DESIGN OF A COMPLETE SOCKETED PILE ALLOWING FOR SLIP; $E_b/E_r = 1$ (Fig. C.2d)

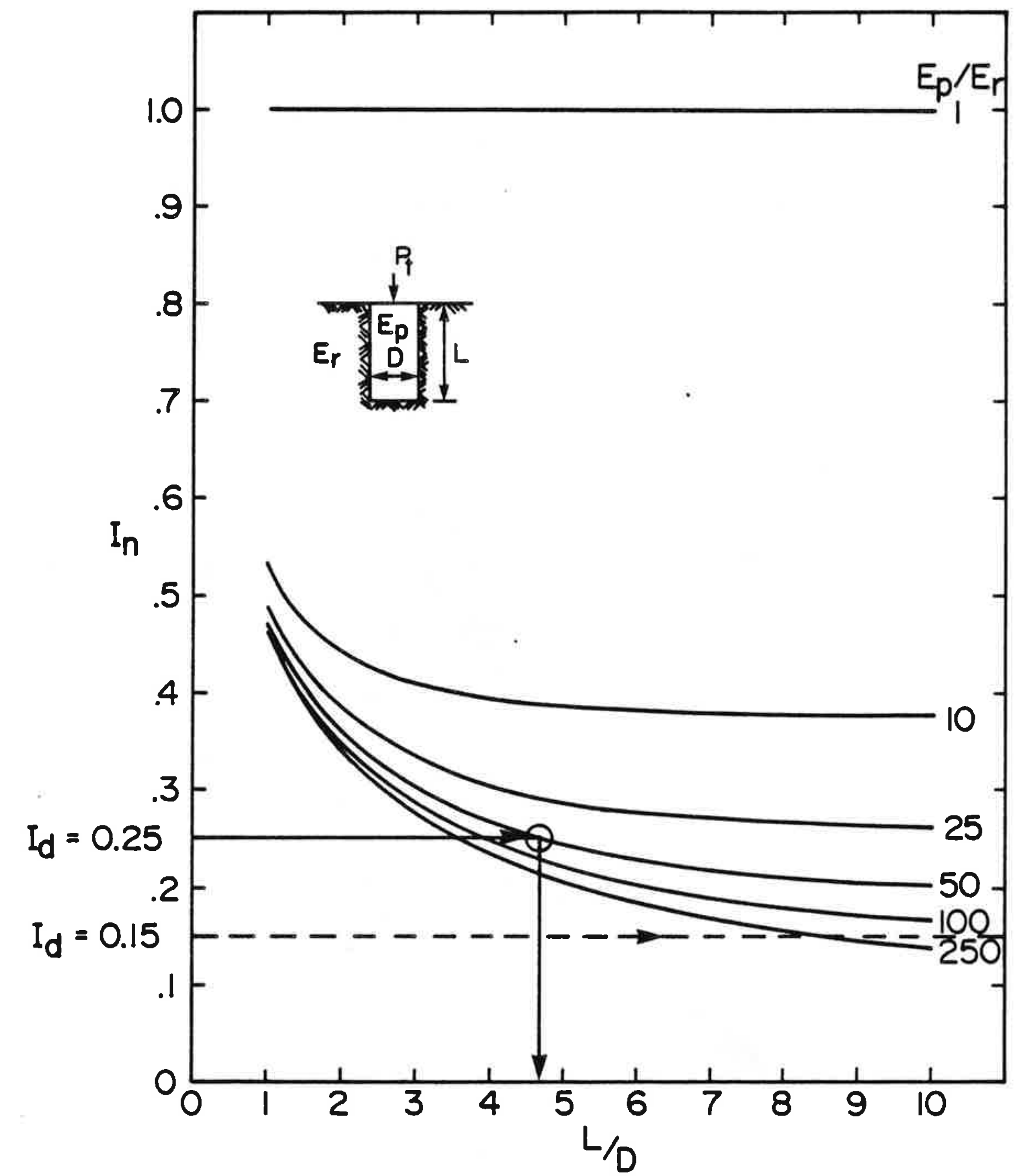


FIGURE 6.3 DESIGN OF A COMPLETE SOCKETED PILE FOR NO SLIP CONDITIONS; $E_b/E_r = 1$, (Fig. C.1a)

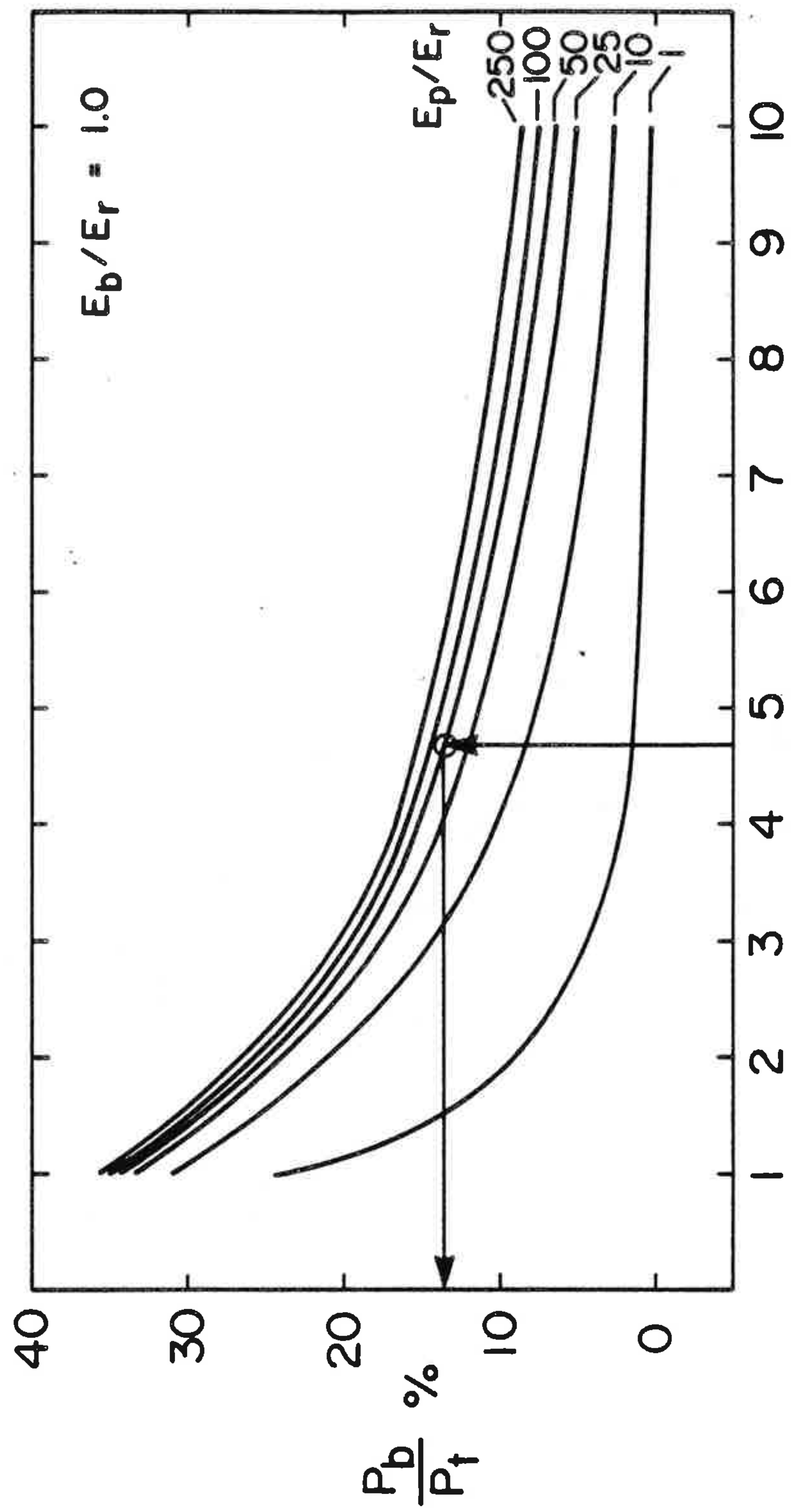


FIGURE 6.4 DESIGN OF A COMPLETE SOCKETED PILE FOR NO SLIP CONDITIONS; $E_b/E_r = 1$ (Fig. C.1b)

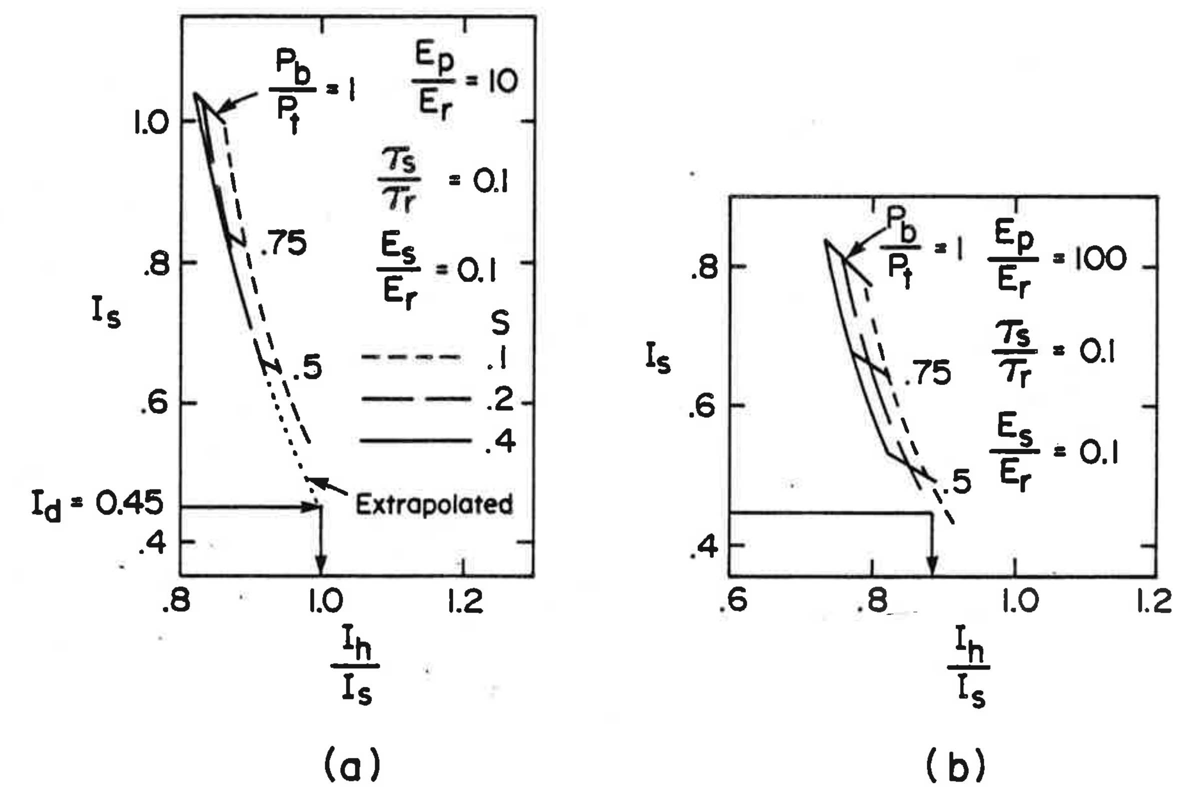


FIGURE 6.5 ADJUSTING THE DESIGN INFLUENCE FACTOR FOR THE EFFECT OF SEAMS - L/D = 2 (a) E_p/E_r = 10 (from Fig. C.7) (b) E_p/E_r = 100 (From Figure C.10)

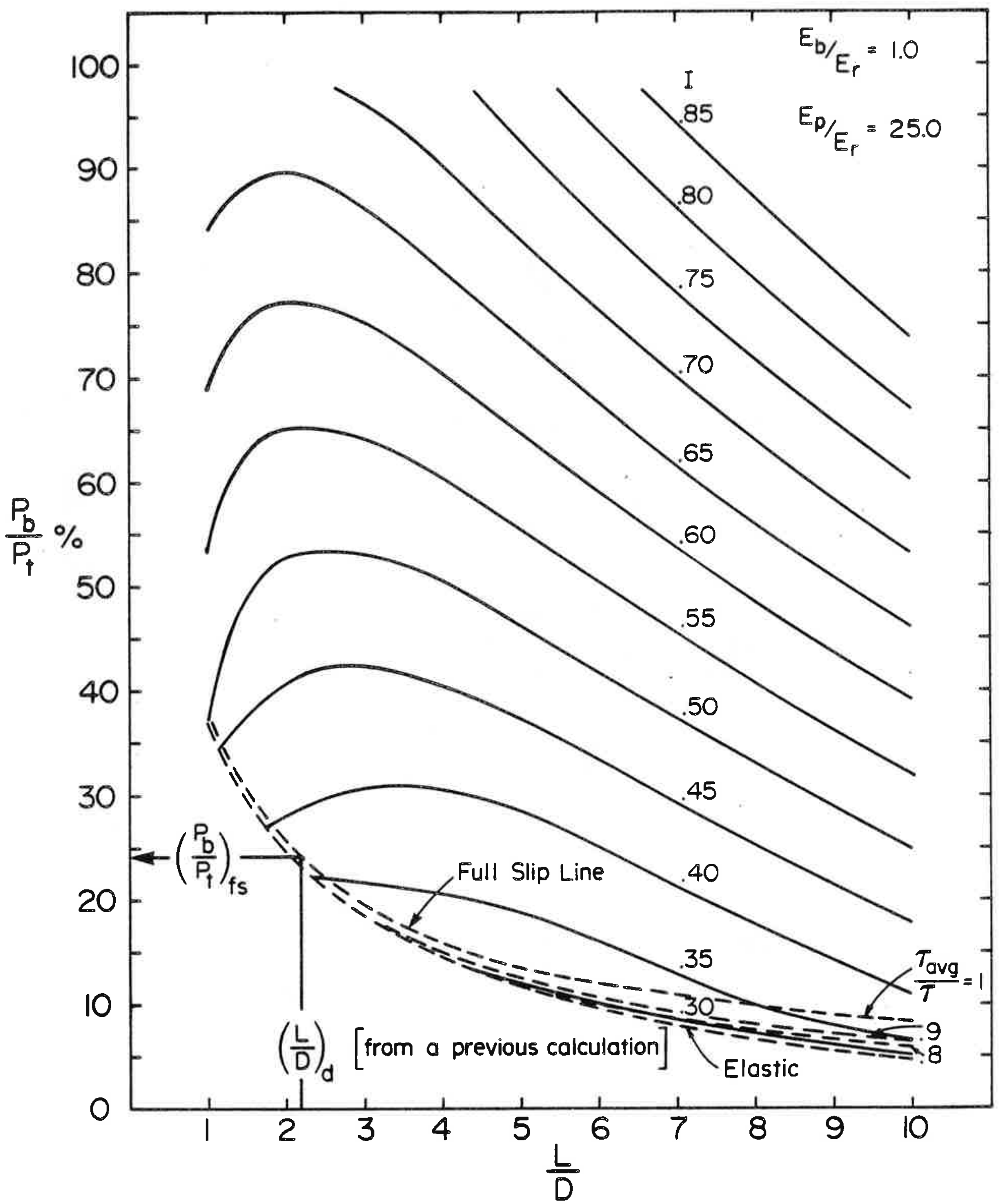


FIGURE 6.6 DETERMINING THE FULL SLIP RATIO $(P_b/P_t)_{fs}$ FOR A GIVEN SOCKET - $E_p/E_r = E_p/\bar{E}_r = 25$

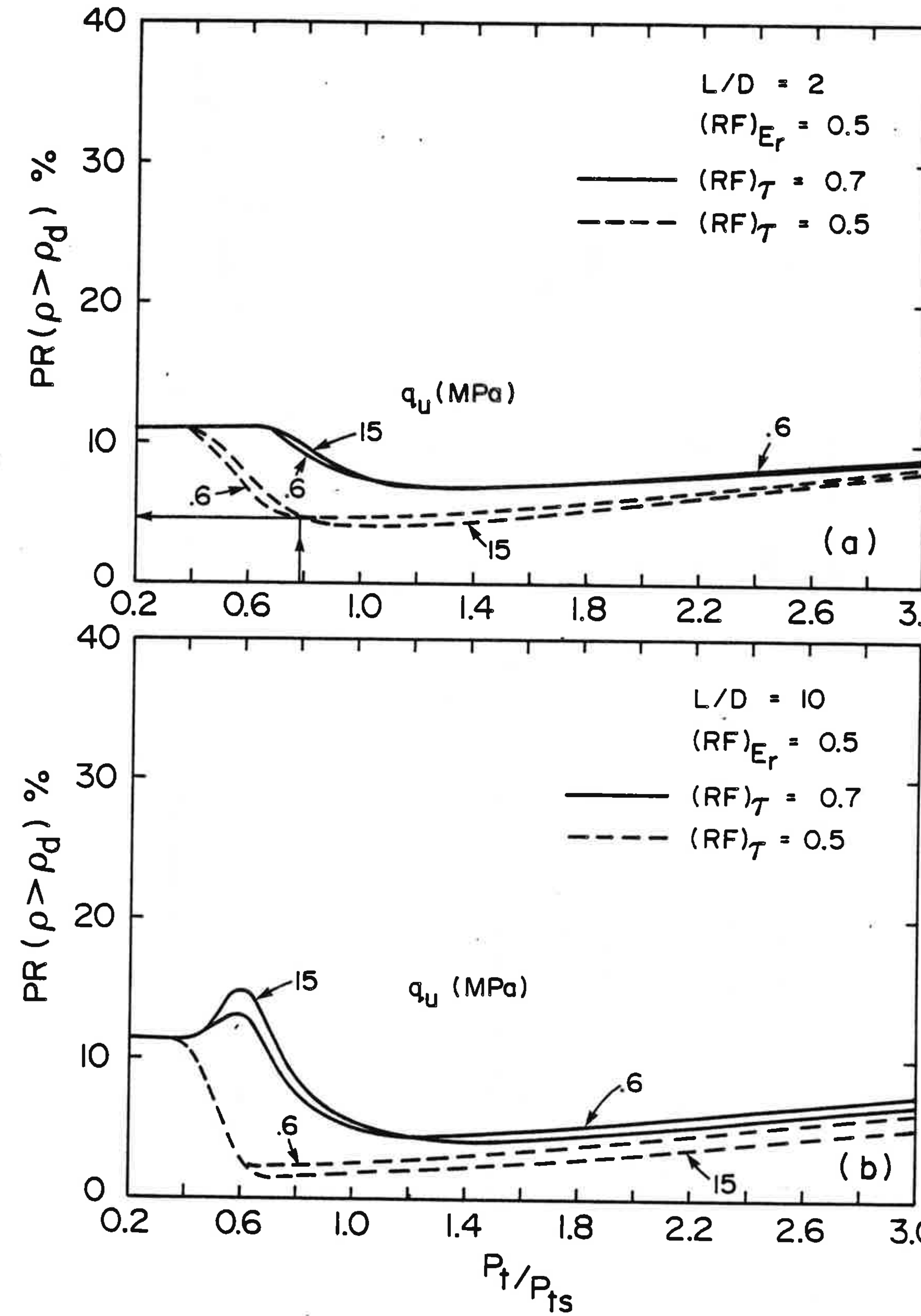


FIGURE 6.7 ESTIMATING THE PROBABILITY OF EXCEEDING DESIGN SETTLEMENT: $(RF)_{E_r} = 0.5$

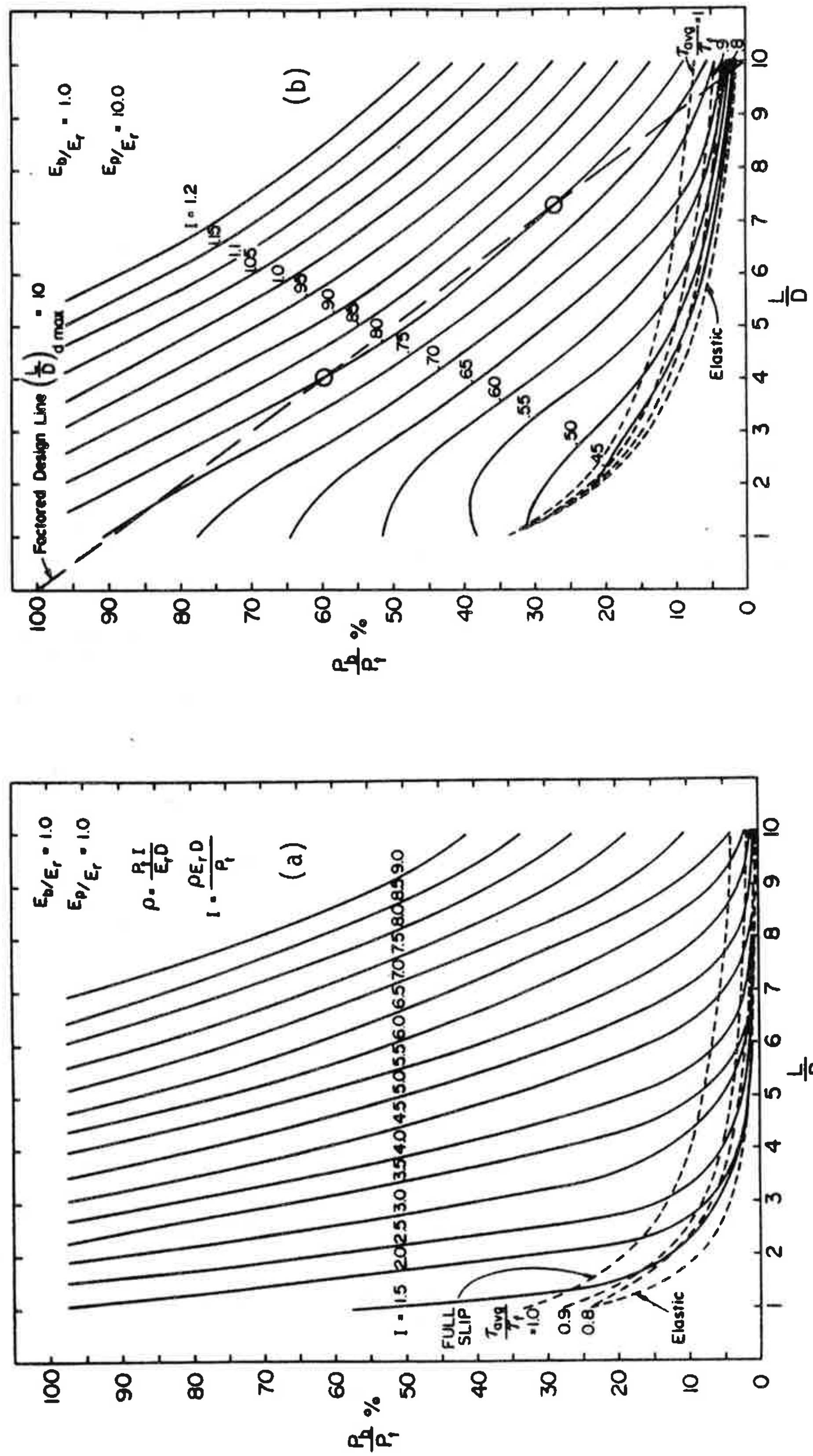


FIGURE 6.8 DESIGN CHARTS FOR E_p/E_r = 1 AND 10

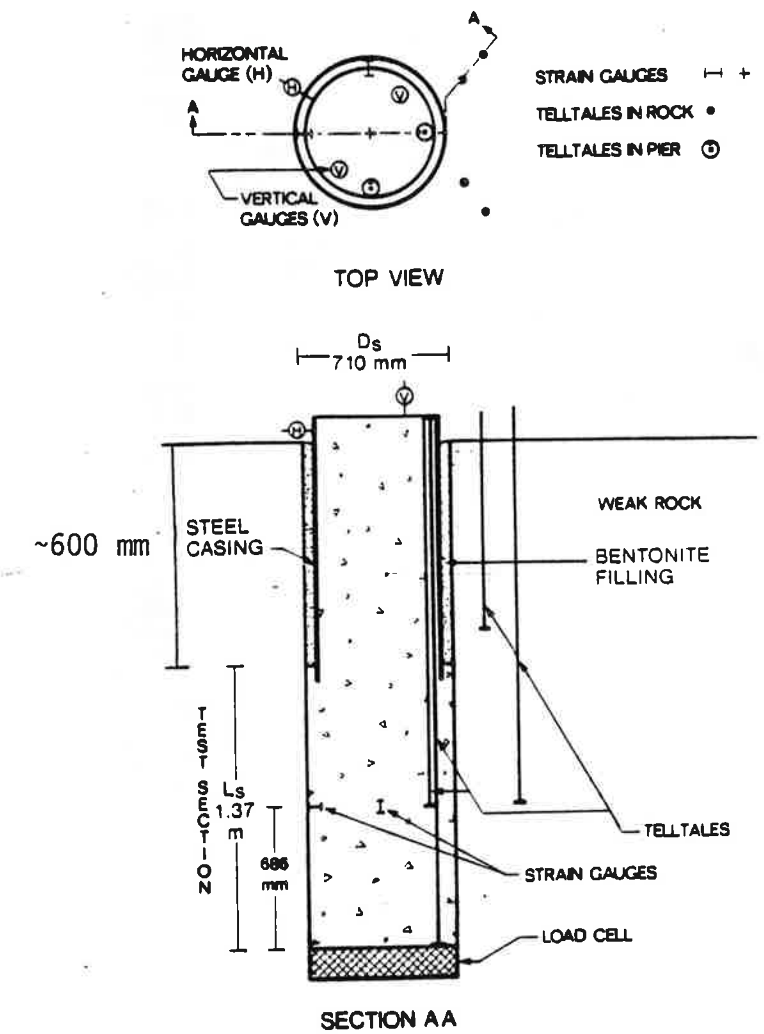


FIGURE 6.9 PLAN AND CROSS-SECTION OF GEOMETRY FOR SOCKETED PILES P2 AND P4 (from HORVATH, 1980)

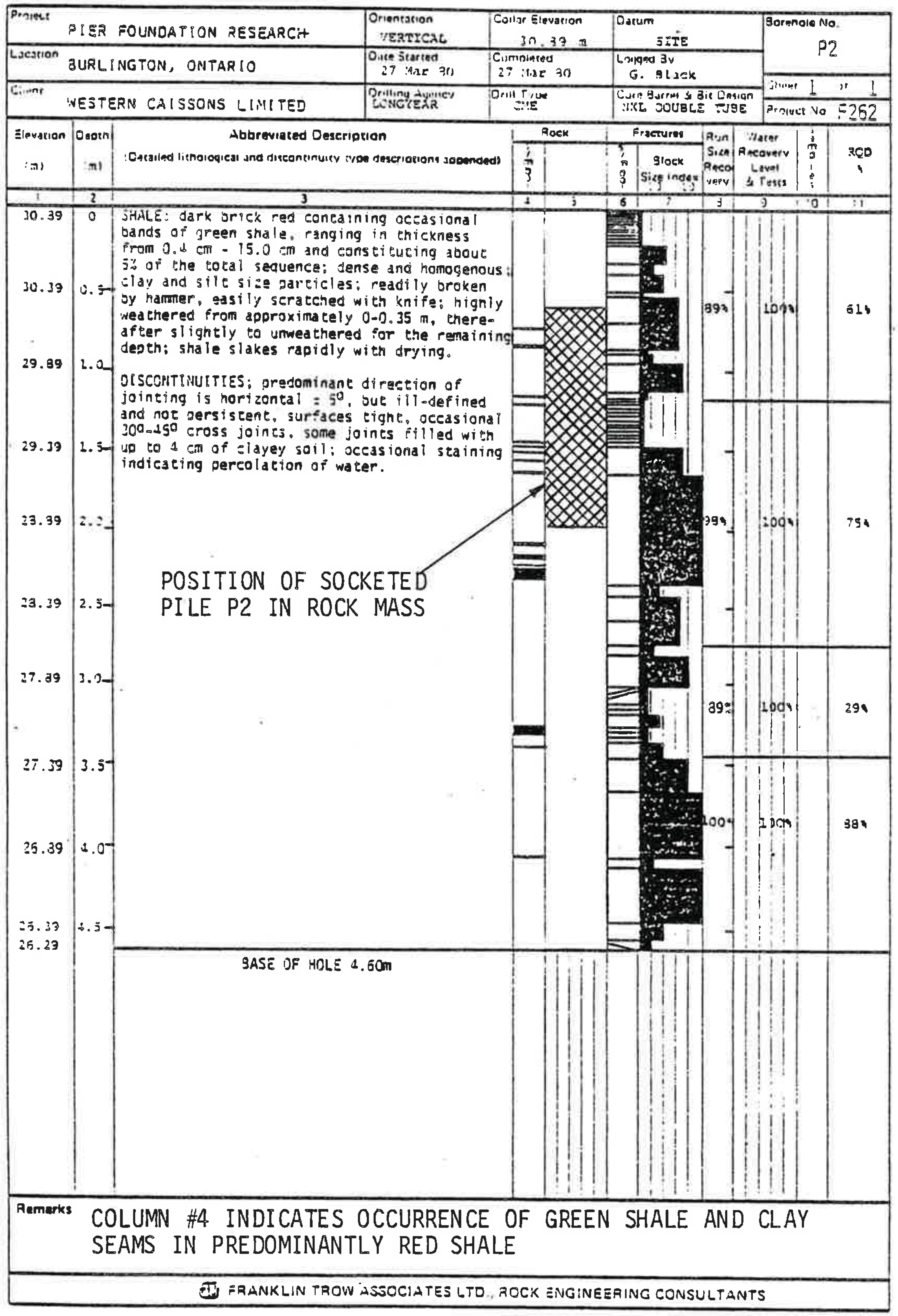


FIGURE 6.10 BOREHOLE LOG FOR SOCKET P2 (from HORVATH, 1980)

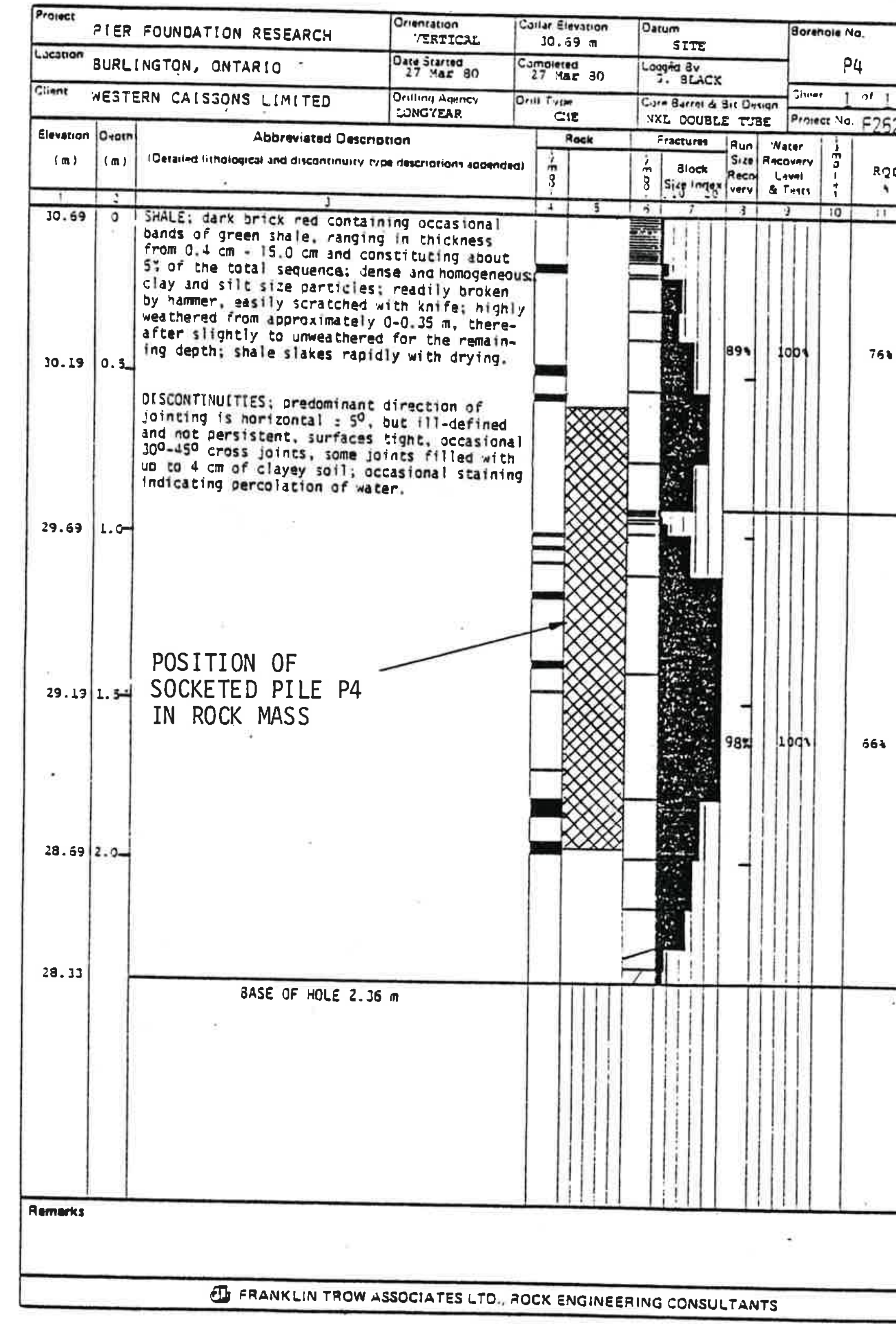


FIGURE 6.11 BOREHOLE LOG FOR SOCKET P4 (from HORVATH, 1980)

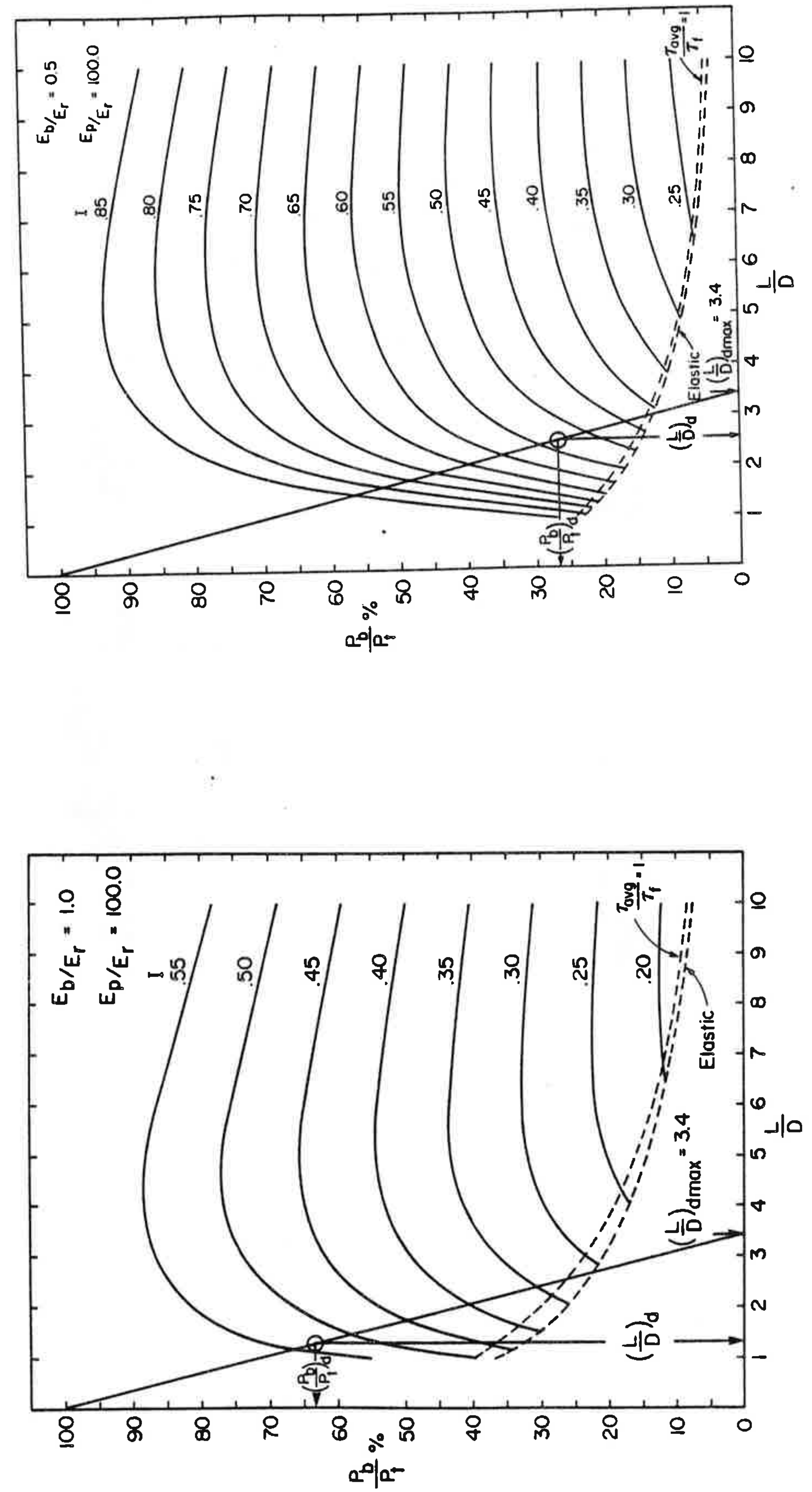


FIGURE 6.12 DETERMINATION OF DESIGN L/D AND P_b/P_t VALUES FOR $(L/D)_{dmax}$ = 3.4; I_d = 0.53

(a) E_b/E_r = 1.0 (b) E_b/E_r = 0.5

CHAPTER 7

DISCUSSION OF THREE SOCKETED PILE CASE HISTORIES

7.1 GENERAL

In this chapter, several important aspects concerning field load testing techniques and the interpretation of data from socketed pile load tests will be discussed. Information obtained from three well documented case studies by Glos and Briggs (1983), Williams (1980) and Horvath (1980) will be used in the analysis and subsequent discussion.

7.2 CASE HISTORY I (GLOS AND BRIGGS, 1983)

7.2.1 Introduction

Full scale field load tests on two complete rock socketed piles have been reported by Glos and Briggs (1983). These tests, which were performed for a project near Farmington, New Mexico USA, were used to modify the design of socketed piles which were originally designed on the basis of presumptive endbearing and sideshear values.

The sockets were constructed in a geological unit known as the Picture Cliffs Formation, which generally consists of interbedded sandstones and shales. From borehole logs (Glos, personal communication, 1983), the rock grades from a highly weathered state near the ground surface to a slightly weathered condition at 13.5 m below the ground surface. The test sockets (denoted East and West respectively) were located in this slightly weathered zone approximately 15.5 and 14.2 m below the ground surface. The discontinuities at the level of the socketed piles were described as being close to medium close (i.e., 50 mm - 1 m) and clean (i.e., no fracture filling material). Additional details concerning the measured properties of the rock mass can be found in the paper by Glos and Briggs (1983).

Figure 7.1 shows the geometry of the two piles and the location of instrumentation used in the tests. As indicated in this figure, the sidewalls of both the East and West sockets were artificially roughened to produce two sets of grooves 100 mm high and 76 mm deep.

While this case study provided valuable information regarding socketed pile behaviour, there are several points concerning the interpretation of their results which may require further discussion.

7.2.2 Preventing Endbearing on Rock

Although the piles tested by Glos and Briggs are complete socketed piles (i.e., involving endbearing and sideshear), the shaft of the recessed portion of the pile had a larger diameter than the actual socket (see Figure 7.1). Polystyrene discs were placed between the concrete and the rock in an attempt to prevent endbearing load transfer from the larger diameter pile length to the rock above the socket.

The technique of reducing pressure transmitted in endbearing by placing a polystyrene disc between a concrete pile and the rock has been adopted on numerous occasions. The possible effect of the compressibility of polystyrene upon the load transfer to rock has been investigated by Webb (1967) who, on the basis of unconfined compression tests on styrofoam discs, estimated that for his case up to 7% of the maximum applied load was carried to the rock beneath the styrofoam disc.

The findings by Webb raise the question as to how effective the polystyrene discs used by Glos and Briggs (1983) may have been in preventing load transfer to the rock above the socketed pile. To provide data concerning the compressibility of styrofoam, an unconfined compression test was performed on a 51 mm thick, 114 mm diameter disc of polystyrene (tradename "Styrofoam SM"). The results from this test are shown in Figure 7.2.

Considering Glos and Briggs's load tests, the initial stress applied to the styrofoam disc due to the weight of the concrete (assuming γ = 23.5 KN/m^3) in the recessed pile was calculated to be approximately 365 and 335 kPa for the East and West sockets respectively. The corresponding strains from Figure 7.2 are 43% and 28% for the two sockets. A tangent modulus for the styrofoam is calculated to be about 0.2 MPa. This value is relatively small in comparison with the average pile modulus ($E_p \approx 39 \times 10^3$ MPa) or rock modulus ($E_r \approx 890$ MPa). Based on the data provided by Glos and Briggs, it is estimated that the strain in the polystyrene increased by approximately 5-6% during the load test. An inspection of Figure 7.2 would suggest that relatively little additional load would be transferred to the rock beneath the polystyrene in this case. This conforms with Glos and Briggs's assumption.

7.2.3 Prediction of Base Loads for a Socketed Pile Assuming A Rigid Circular Footing

The stress measurements recorded at the base of the two piles tested by Glos and Briggs depends very significantly on the position of the instrumentation (eg. see Figures 11 and 12). This raises the question as to whether this variability is primarily due to the difference in modulus between the concrete and the rock or whether the readings and/or interpretation are suspect.

The large difference in modulus between the concrete and rock ($E_p/E_r \approx 40$) at this site would be expected to give a stress distribution at the base of the pile approaching that for a rigid circular die on an elastic halfspace, viz:

eg. $$\sigma_z(r) = \frac{q_{ave}\, a}{2\sqrt{a^2 - r^2}} \qquad (7.1)$$

where $\sigma_z(r)$ is the vertical stress beneath the circular footing at a radial distance r from the centreline of the footing
a = radius of the circular footing
q_{ave} = average applied pressure at surface of the footing

Based on information supplied by Glos and Briggs (1983) and Glos (personal communication, 1983), it would appear that the Carlson strain meters used in these tests were located approximately 230 mm ($\pm$ 15 mm) from the centre of the pile (i.e., r = 230 mm, a = 305 mm). The stresses deduced by Glos and Briggs at locations 4, 5, 6 (see Figure 7.1) are given in columns 3, 4 and 5 of Table 7.1. Assuming a rigid loading and knowing the stress σ_z at a particular point r, the average base pressure q_{ave} may be deduced using Eq. 7.1. The average base pressure q_{ave} deduced from Glos and Briggs's results corresponding to the strain measurement at locations 4, 5 and 6 are given in Table 7.1 columns 6, 7, 8 respectively. For pure vertical loading, all these pressures should be the same. In particular, since instruments 4 and 5 are located the same distance from the centreline (on opposite sides of the pile) it is reasonable to expect that the stresses at each position should be the same.

This is approximately so at the West socket but is not the case at the East socket.

Examination of the results for the East socket would suggest that there was some eccentricity associated with the loading of this pile and/or the location of the strain meters. This effect should be largely eliminated by considering the average base pressure deduced from the data at locations 4 and 5 (denoted by $\overline{q}_{4+5}$). Comparison of this value with the average base pressure deduced from the centreline readings (at location 6) gives reasonable agreement (i.e., to within 30%) as shown in Table 7.1.

At the West socket, the average base pressures deduced from the measurements at locations 4 and 5 are in reasonable agreement (i.e., within 15%) however these values are substantially different (by a factor of between 2 and 3) from the values calculated for the measurements at location 6. This finding casts some doubt on the reliability of either the gauge at location 6 for the West socket or the calculation of the stress σ_{z6}.

The load carried to the base of the socket (P_b) was calculated from the average of the values tabulated in columns 8 and 9 of Table 7.1 and is given in column 10 for both the East and West sockets. Because of the constantly low values of q_6 determined at the West socket, the base load P_b calculated on the basis of $\overline{q}_{4+5}$ alone (column 9) is given in column 11. The base loads P_b are plotted against the applied load P_t in Figure 7.3 together with the values reported by Glos and Briggs

(1983). The agreement at the East socket is generally quite good. At the West socket, the results given in columns 10 and 11 of Table 7.1 bracket Glos and Briggs's reported values. It is considered that the base loads calculated without considering the readings at gauge 6 are probably the most reliable.

7.2.4 Design Calculation for Glos and Briggs Site

To provide an additional example of the application of the proposed design procedure, the test piles at the location of the West socket were designed based on empirical correlations using reduction factors of 0.7 and a design settlement, ρ_d, of 5.3 mm for an applied load, P_t, of 4.5 MN. (This load was selected since it is close to the value used in the design of Horvath's piles in Chapter 6 and is typical of design load for the actual production piles at Glos and Briggs's site.) The design calculations for this case are given in Table 7.2. The required pile length of 1.46 m is close to the actual length of the West test pile (i.e., 1.47 m) which experienced a settlement of 2.5 mm at the load of 4.5 MN. Clearly, the design settlement of 5.3 mm is satisfied and there is also a substantial factor of safety against collapse of the pile which was eventually loaded to 8.9 MN without any sign of collapse.

It is also interesting to compare designs of the final production piles for Glos and Briggs's project as summarized in Table 7.3. For a design load of 4.5 MN and a diameter of 0.76 m, a socket 6.1 m in length would be required if the design was based on the original presumptive values of τ_d = 0.24 MPa, q_{ba} = 2.4 MPa proposed by Glos and Briggs's consultants. On the basis of their field test results, Glos and Briggs

eventually adopted design values of τ_d = 0.72 MPa and q_{ba} = 4.8 MPa which results in a socket length of 2.3 m.

If the sockets were designed using the proposed procedure for a design settlement of 5 mm and reduction factor of 0.5 (i.e., such that the probability of exceeding the design settlement is less than 11%), the required pile length would be 2.4 m (see Table 7.4). For the same design settlement and reduction factors of 0.7, the required pile length would be 1.35 m. This socket length is only slightly shorter (1.35 m versus 1.4 m at the East and 1.47 m at the West socket) and with a slightly larger diameter (0.76 m versus 0.61 m) than the test piles which both gave a settlement of 2.5 mm at a load of 4.5 MN. The authors would suggest, therefore, that the design based on a design settlement of 5 mm and RF = 0.7 would have proved adequate. A more conservative design adopting RF = 0.5 would have given a length corresponding to that which was obtained using Glos and Briggs's final design parameters. Both designs would be substantially cheaper than a design based on the initial presumptive values.

7.2.5 Conclusions

The case history reported by Glos and Briggs represents a valuable contribution to the engineering literature. Before conducting the pile load tests, the authors have performed a variety of simple field and laboratory investigations/tests in an attempt to establish the strength and deformation characteristics of the rock mass. It is believed that this type of investigation is a necessary and important part of a rational socketed pile design.

Based on the results of their load tests on two piles with artificially roughened rock sockets, the following conclusions regarding socketed pile behaviour can be drawn:

1) At low load levels (i.e. in the elastic range), the observed end-bearing loads (P_b/P_t) were consistent with theoretical solutions given by Pells and Turner (1979).
2) The maximum sideshear resistance mobilized by the test piles was approximately 8 times larger than empirical values initially used to design the production piles, thereby resulting in a savings of $2 million (US) in subsequent construction cost (Glos and Briggs, 1983,1984).
3) Design calculations performed using the empirical correlations developed in Chapter 4 produced socket lengths which are in good agreement to those actualy used for the production piles based on the same applied load, rock conditions and presumed settlement requirements.
4) Load distribution curves suggest that a large proportion of the applied axial load is resisted by socket shear in the upper portion of the roughened socketed pile. Such behaviour led the authors to conclude that socket lengths should not exceed 2-3 socket diameters. However, Horvath (1984) correctly points out that this statement may be misleading since the condition of the socket sidewalls (i.e., smooth or rough) as well as any specified settlement restrictions may require that the length to diameter ratio be greater than 3. Also, where difficulties arise in adequately removing debris from the base of the socket, pile lengths in excess of 3D

may be required if the entire applied load must be carried by socket shear.
5) Assumptions made by the authors regarding,
   (i) preventing endbearing on rock using a styrofoam disc
   (ii) base loads calculated assuming a rigidly loaded circular footing

   appear to be reasonable based on the previously described analyses (i.e., sections 7.2.2 and 7.2.3).
6) Glos and Briggs (1983) have concluded that values of rock mass modulus based on correlations with RQD (rock quality designation) or Schmidt Hammer results are not appropriate for this soft rock since they greatly overestimate those backfigured from the load-displacement curves from in-situ socket pile load tests. This conclusion appears to be well founded.

7.3 CASE HISTORY II (HORVATH, 1980, 1982)

7.3.1 Introduction

The results of six full scale socketed pile load tests in shale have been described by Horvath (1980, 1982). These load tests were conducted on both complete socketed piles and sideshear only piles. The purpose of these field load tests was to examine and compare the behaviour of socketed piles which were, (1) normally constructed with a standard single flight auger, (2) normally constructed, then artificially roughened using a special grooving device, (3) preloaded at the socket base in order to "stiffen" the load-displacement response. The tests were conducted at a quarry site near Burlington, Ontario, where the rock mass is exposed at the ground surface. The rock mass in which the sockets were constructed is known as the Queenston Formation, a predominantly red shale (with inclusions of harder green shale) of Upper Ordovician age. The heads of the socketed piles were located below the highly weathered surface rock at a depth of approximately 0.6 m. The rock below this level is described as being increasingly massive with depth, with predominantly horizontal joints typically 0.5 to 1 m apart and occasionally filled by up to 37 mm of clayey soil (see Horvath, 1980). Further details of rock properties and test results are available from Horvath (1980, 1982) or Horvath et al. (1983). The typical geometry of the test piles has been previously described in Chapter 6 (i.e., see Figure 6.9 and the two bore logs given in Figures 6.10 and 6.11).

Various aspects of the tests performed by Horvath have already been discussed. In this chapter a number of additional aspects pertaining to anchor interaction, the load transfer behaviour of the test sockets and the effects of preloading will be discussed.

7.3.2 The Effect of Anchor Interaction on Test Pile Results

As noted by Poulos and Davis (1980), errors in load test measurements may occur due to interaction effects between the pile being tested and the piles/anchors used to provide the load reaction. Due to the size and proximity of Horvath's anchor piles to his test piles (see Figure 7.4), it was felt that some consideration should be given to the likelihood of measurement errors due to interaction.

This interaction study was conducted using an elastic finite element analysis (and the principle of superposition) together with the

parameters E_r = 354 MPa, E_p = 35 GPa, ν_r = 0.3 and ν_p = 0.27. On the basis of the finite element analysis, the following corrections for the effect of the anchor piles are proposed:

(a) Settlement $\rho = \rho(\text{recorded}) + 0.206\ P_t$ (mm)

(b) Base Load $P_b = P_b(\text{recorded}) - 0.022\ P_t$ (MN)

where the recorded settlement and base load are in mm and MN respectively and the applied load P_t is in MN.

In the case of settlement, the correction is additive since interaction with the anchor pile will reduce the displacement of the test pile (i.e., the recorded settlements are too low). Interaction will increase the recorded base load P_b and hence the base load correction is negative.

The proposed corrections are relatively small as may be appreciated by comparing the recorded and corrected displacements and base loads for pile P2 is given in Tables 7.5 and 7.6. These results suggest that although there may have been some interaction between the anchor piles and the test piles, the effect is not great and considering the practical difficulties of increasing the spacing, the location of the anchor piles selected by Horvath (1980) seems reasonable.

7.3.3 Load Distribution in Socketed Piles

Horvath (1980, 1982) has presented load distribution curves for both relatively smooth and rough complete sockets.

The geometry of Horvath's test piles has previously been given in Chapter 6 (i.e., see Figure 6.9). Profiles of the socket sidewalls traced prior to placing the pile concrete for piles P2 and P4 are shown in Figure 7.5. Socket P2 was constructed using a single flight auger bit with hardened cutting teeth. As shown in Figure 7.5a, this produced a relatively smooth socket profile. In comparison, the profile for pile P4 is quite rough, as a result of the manual roughening technique described by Horvath (1980). The dimensions of these "grooves" are approximately 25 mm deep, 40 mm high with spacings of 150 mm.

The load distribution curves for these two piles have been reproduced in Figure 7.6. Data used to construct these figures were obtained from:

(1) Strain gauges located at the mid-depth of the socketed pile tests section (see Figure 6.9). As noted by Horvath (1980), these gauges were used to measure the radial and vertical strains from which the stresses were calculated.

(2) Load cells at the top and bottom of the pile (to measure the applied load and the load transferred to endbearing).

It is noted that these two different techniques may be expected to have different accuracies. In particular, the stresses (and hence load) determined by Horvath (1982) from the strain gauges at mid-depth should be viewed with particular caution since

(1) they are based on the assumption of a homogeneous elastic pile behaviour. Thus the calculation of stress from strains may be affected both (a) by changes in the modulus and Poisson's ratio of the rock as the load increases; and (b) by local stress concentration and microcracking of the concrete

(2) they are based on the assumption that the stress distribution across the pile is uniform

(3) they are based on the use of the stress-strain relationship

$$\sigma_z = \frac{E_p(1-\nu_p)}{(1-2\nu_p)(1+\nu_p)} [\varepsilon_z + \frac{\nu_p \varepsilon_r}{1-\nu_p} + \frac{\nu_p \varepsilon_\theta}{1-\nu_p}]$$

and it has been <u>assumed</u> that $\varepsilon_r = \varepsilon_\theta$ near the edge of the pile where the vertical and radial strains ($\varepsilon_z, \varepsilon_r$) were measured. Since the tangential strain ε_θ was not measured, one cannot be certain as to the validity of this assumption. However, based on theoretical considerations, one would not expect $\varepsilon_r = \varepsilon_\theta$ at the location of the instruments. At this location, the relationship between ε_r and ε_θ will depend on (a) the ratio of pile to rock modulus E_p/E_r and (b) the amount of slip which has occurred along the socket. [For example, if there is no slip, and $E_p/E_r \simeq 1$ then $\varepsilon_\theta \simeq 4\ \varepsilon_r$ for $\nu_p \simeq 0.27$.]

As shown in Figure 7.6, the load distribution curves have been normalized with respect to length of the socketed portion of the pile. The distribution of load for pile P2 suggests that the shear along the pile-rock interface is non-uniform at low load levels (i.e., in the elastic range of behaviour). This nonlinearity of shear stress along the pile shaft has been previously recognized by Williams et al. (1980) from instrumented piles socketed into mudstone near Melbourne, Australia.

Also from Figure 7.6a, it can be seen that as the applied load increases, the shape of the load distribution curve changes. The results imply that as the load increases, less and less load is carried in sideshear in the upper half of the socket while more and more of the load is supported by sideshear in the lower portion of the pile. At an applied load of 8 MN, the load distribution curve indicates that none of the applied load is carried between the head and mid-section of the pile. Therefore, the sideshear resistance along the periphery of the pile is zero. This implies a complete loss of sideshear resistance along the top half of the socket and is contrary to the test results given by Williams (1980) and Williams et al. (1980). For example, Figure 7.7 shows the results of a load test performed by Williams on a complete socketed pile. Even after significant yielding at the pile-rock interface (Figure 7.7a), the shear stress near the top of the pile is non-zero and is uniformly distributed along the pile shaft (Figure 7.7c). Thus the present writers would be rather hesitant to infer a complete loss of sideshear resistance along the top half of Horvath's pile P2 particularly in view of (a) the large sideshear which must otherwise have been mobilized along the lower half of the pile and (b) the potential of erroneous measurement of strains at the pile midpoint combined with the question of interpretation as previously discussed.

The distribution of vertical load for the roughened socket P4 deduced by Horvath (1982) is shown in Figure 7.6b. The interpretation of the midpoint load is even more difficult in this case than for the smoother socket P2 because of (a) erratic strain gauge readings reported by Horvath and (b) the assumptions used in deducing the stress from strain certainly do not seem appropriate for an artificially roughened socket.

Thus the present writers are hesitant to draw conclusions about the real distribution of load along the socket based on Horvath's data. However, considering the more reliable data at the top and bottom of the socket it does appear that

(a) only approximately 18% of the total load is carried to the base of the pile at low load levels;

(b) the proportion of load carried to the base increases with increasing load;

(c) at low load levels (i.e., in the elastic range), the proportion of load at the base of the pile is essentially the same for both the conventional (P2) and artificially roughened socket (P4). As the load increases, slip appears to occur more readily for the conventional socket and the proportion of load carried to the base increases relative to that for the artificially roughened sockets.

All of the foregoing findings are in agreement with theoretical expectations (see Chapter 3).

An additional point of note is the increase in load carried to the base of the pile which was observed when load was sustained for a period of 36 to 40 hours. At socket P2, the ratio P_b/P_t increased from 23 to 28% for a load P_t = 4.45 MN. At socket P4, the corresponding change appears to be from 27 to 30%. There is insufficient data to draw any firm conclusions however there does appear to be a time dependent load redistribution. Considerably more test data on full scale sockets is required before the effect of time dependent load transfer can be directly considered in design. Nevertheless, recognizing this possibility, the design procedure proposed in Chapter 6 allows for an adequate factor of safety against endbearing failure in the event that 70% of the load initially carried in sideshear will eventually be transferred to the base. Considering Horvath's piles at the load of 4.45 MN this would imply an adequate factor of safety against collapse if the P_b/P_t ratio increased with time from approximately 28% to 77%.

7.3.4 Preloaded Socketed Piles

Horvath (1982) described preloading of a socketed pile as:

> . . .the application of an expansion load between the bottom of the pier (pile) and the rock, which physically tends to force the pier base and rock apart.

Further to this he notes that,

> Improved load-displacement behaviour of a preload pier (pile) is anticipated because the load transfer between the pier (pile) and the socket should be more efficient. A larger portion of the applied load would be supported by the bottom of the socket at smaller displacements than for conventional piers (piles).

Previous attempts to investigate the concept of preloading laboratory scale piles have been described by Kenney et al. (1975). Evidence of a field application using this method to improve the load-displacement behaviour of piles has been given by Taylor (1975).

Horvath (1980,1982) has performed full scale load tests where an initial preload was applied to the base of conventionally constructed socketed piles (i.e. with relatively smooth socket sidewalls). Figure 7.8 shows the difference between the load-displacement behaviour of the preloaded piles and the conventional pile P2. (It should be noted that

the preload of .18 MN was used to ensure adequate "seating" of piles P2 and P4.)

From Figure 7.8, it can be seen that as the preload is increased, the pile exhibits smaller displacements at any particular value of applied load. Also, as shown by Horvath (1980), the percentage of load carried to the base of the pile is much larger than that exhibited by pile P2 at similar applied loads.

The results suggest that for regular constructed socketed piles (i.e., socket walls not intentionally roughened), there may be some advantages to preloading a pile, particularly when attempting to limit pile displacements. However, it is also useful to compare this behaviour with the results for an artificially roughened socket. As shown in Figure 7.9, the difference in load-displacement behaviour between the preloaded piles and the roughened socketed pile P4 are not nearly as evident as was the case for the conventional pile (P2). For example, at an applied axial load of 4 MN, the displacements (at the top of the pier) are:

SOCKETED PILE	δ (mm)
P2	7.2
P4	5.6
P5A	4.8
P5B	4.3

Also, as the load level increases, the difference between the displacements of pile P4 (neglecting the displacement due to the maintained load) and the preload piles becomes progressively smaller.

From a practical point of view, the advantages to be gained by using a preloaded socket rather than a roughened socket to achieve generally the same pile response, needs careful consideration. Several other comments regarding preloaded socketed piles are offered,

(1) It is likely that the practical difficulties and expense in preloading actual production piles will be greater than artificially roughening the walls of the socket.

(2) Preloading production piles may be useful in cases where the particular site exhibits vastly differing rock qualities and the possibility of differential settlement occurring becomes an important consideration. Such a situation has been described by Taylor (1975). However, when the conditions of the rock mass are generally similar over the site, the advantages of preloading rather than roughening the socket are not clearly evident.

(3) Both Kenney et al. (1975) and Taylor (1975) have expressed concerns over what effect time dependent behaviour has on the preloaded pile system.

Clearly, further study and clarification regarding the concept of a preloaded socketed pile is required.

7.3.5 Conclusions

The pile load tests conducted by Horvath (1980,1982) represent a significant contribution to the engineering literature. These load tests

were preceded by a geotechnical investigation. Again, the value of conducting a preliminary site investigation to aid in the design of socketed piles should not be understated. As was shown in Chapter 6, reasonable socket designs could be deduced using only a representative value of the unconfined compressive strength of the rock mass and information from a detailed core log.

From the preceding discussion, a number of conclusions can be drawn:

(1) From an elastic finite element analysis, it was found that the interaction between the test pile and the reaction anchor piles can influence the magnitude of measured settlement and base load. Although the effect is not large in the case of Horvath's testing arrangement, due consideration should be given to this aspect when interpreting data from pile load tests.

(2) The magnitude of average available sideshear resistance is greater for piles constructed in roughened sockets compared to that mobilized in conventionally constructed (i.e., not intentionally roughened) sockets.

(3) No firm conclusion could be reached regarding the distribution of load along the shaft of the socketed pile however it is evident that the general performance of artificially roughened sockets was substantially better than for the conventional sockets.

(4) Comparison of load distribution curves suggests that there may be an optimum socket roughness which would result in superior load-displacement behaviour.

(5) Tests involving preloaded piles (constructed in non-roughened sockets) gave marginally lower settlements (at the same applied

load) than tests on piles in artificially roughened sockets. In many applications, artificial roughening may be the most appropriate way of improving socketed pile performance.

(6) More research into the time dependent load transfer behaviour of conventional, artificially roughened and preloaded piles is required.

7.4 CASE HISTORY III (WILLIAMS, 1980)

7.4.1 Introduction

The case history reported by Williams (1980) represents one of the most extensive socketed pile testing programmes undertaken to date. These tests have covered a wide range of socketed pile geometries, socket roughness and pile types (eg. endbearing only, complete piles etc.).

The load tests were conducted at various sites in a rock formation known locally as Melbourne mudstone (Melbourne, Australia). Melbourne mudstone consists of interbedded claystone (rare), siltstone and sandstone, with siltstone clearly predominating (Williams, 1980). Both the unconfined compressive strength of the rock and the frequency of discontinuities vary from each test site considered. The two sites of particular interest are referred to as the 1) Stanley Ave. site and 2) Middleborough Rd. site. The two locations are where the majority of Williams's tests were performed. At the Stanley Ave. site, the piles were constructed in highly weathered mudstone, where the unconfined compressive strength of the rock ranges from approximately 0.44-0.83 MPa, and joint frequency is generally less than 5 joints/meter (see Williams, 1980). The piles at Middleborough Road were constructed in either medium or highly weathered mudstone. Compressive strengths ranged from 2.0-3.4 MPa with joint frequency less than 1 joint/m.

In the following sections, several aspects concerning the observed behaviour and interpretation of Williams's results will be discussed. Also, an illustrative pile design using the method proposed in Chapter 6 will be used in a comparison with one of Williams's complete socketed piles.

7.4.2 Backfigured Rock Mass Modulus at Stanley Ave. Site

As shown in Table 7.7, the rock mass modulus backfigured from load-displacement curves for each of the piles tested is significantly different at the Stanley Ave. site. The reason for this behaviour is not clear since the piles are not located at great distances from one another (i.e., boundaries of the test site enclosed by an approximately 20 m by 6 m rectangular area) and the two available core logs do not indicate any significant changes in the stratigraphy of the rock mass.

The backfigured modulus was calculated by Williams by:

i) using load and displacement values from the initial linear slope of the load-displacement curves.

ii) selecting an appropriate settlement influence factor (I_ρ) and reduction factor RF from available elastic solutions (i.e., Pells and Turner, 1979).

iii) the equation given in Chapter 4 (i.e., Eq. 4.7a) is then used to deduce the rock modulus.

While there is no disagreement with the method used to backfigure the rock mass modulus, several comments concerning the results are warranted.

Site Description

The location of the test piles at Stanley Ave. are shown in Figure 7.10. Figures 7.11 to 7.13 show the site geology and the two core logs at each end of the test site.

Categories for Backfigured Rock Mass Modulus

As mentioned, the backfigured rock mass modulus varies widely over the test site. The modulus values given in Table 7.7 are categorized into three groups, depending on their magnitude as follows,

ZONE	RANGE OF BACKFIGURED MODULUS (E_r)
I	$E_r \leq 100$ MPa
II	$100 < E_r < 250$
III	$E_r \geq 250$

These zones are graphically displayed on Figure 7.10. It is interesting to note that these zones correspond to the central, north and south sections of the test site respectively. Several factors which may contribute to this behaviour are discussed below.

(i) Since only two core logs are available, it is difficult to accurately assess what type of rock surrounds and/or lies beneath the test piles. The north-core log (Figure 7.12) shows beds of highly-extremely weathered zones of mudstone between highly weathered (pink and yellow) mudstone. Also, a bed of sandstone (approximately 200-300 mm thick) is located about 2.5 m below the rock surface. Consequently, it is thought that some variation in the backfigured modulus could be expected depending on the position of the test pile in the rock mass.

(ii) Possibly of greater significance is the testing arrangement and methods used to perform the pile load tests.

(a) As shown in Table 7.7, piles in Zones I and II were all tested using a hydraulic jack reacting against a fixed reaction beam. Also, the diameters of the sockets are all less than .4 m. Endbearing was prevented in the sideshear sockets (for Zones I and II) by dissolving a styrofoam disc placed between the rock and pile base.

(b) In Zone III (see Figure 7.10 and Table 7.7), the piles all had diameters of 0.6 m or larger. The method of developing reaction for the applied load on piles S1-S5 are considerably different from other pile tests at the Stanley Ave. site. The method adopted to prevent endbearing for the sideshear sockets consisted of a collapsable steel base. However, calculated vertical stresses (from strain gauge measurements) near the base of pile S1, indicated significant endbearing loads (eg. $P_b/P_t = 48\%$ at $P_t = 1.76$ MN).

These observations suggest that the difference in backfigured rock mass modulus at the Stanley Ave site may be influenced by,

(1) Interaction effects due to the selected testing arrangement;

(2) Variations in the rock mass at each socket location;

(3) Diameter effects;

(4) The method used to prevent endbearing on rock for sideshear resistance piles (particularly piles S1, S3, and S5).

7.4.3 Design Calculations for Pile M8

The results of design calculations for a complete socketed pile are described in this section. Complete piles (i.e., involving endbearing and sideshear) were tested only at the Middleborough Road site, therefore, the redesign of pile M8 (see Williams, 1980) will be considered.

The design calculations for pile M8 were based upon empirical correlations using the following data as given by Williams:

q_u = 2.0 MPa

E_p = 35,000 MPa

L_e = .6 m

D = .66 m

From core logs, the socket is to be located in medium weathered mudstone with a joint frequency less than 1 joint/m with no evidence of clay seams. The test pile was recessed .6 m to avoid a zone of highly weathered mudstone. Tables 7.8 and 7.9 summarize the design calculations.

In Table 7.8a, a design settlement of 10 mm at an applied load of 3.5 MN resulted in a socket geometry similar to the test pile M8. The design settlement is found to be much greater than the observed settlement of 2.6 mm at an applied load of 3.5 MN. However, a check on the endbearing requirements given in Chapter 6 (see section 6.3.5) indicates

that the values of q_{ba} and q_{ma} are exceeded for this combination of L, D and P_t. Thus, following the design procedure proposed in Chapter 6, it is necessary to increase the pile length (or diameter) to reduce the endbearing pressures. However, it should also be noted that the actual test pile with the same geometry was loaded to P_t = 5 MN before the design settlement of 10 mm was reached and was eventually loaded to 8 MN without collapse (the load displacement curve was still rising). Thus, even though the pile is judged unacceptable in the current design procedure (due to excessive endbearing load), the "factor of safety" against exceeding the design settlement was 1.4 (i.e., a load 40% greater than the design load would be required to give a settlement of 10 mm) and the factor of safety against collapse was greater than 2.25.

Because the pile geometry selected for a design settlement of 10 mm gave excessive endbearing pressure (compared with the values recommended in section 6.3.5) the pile length was increased to ensure that both endbearing criteria were satisfied. Table 7.8b shows that adopting a more stringent design settlement of 4.5 mm results in a design socket geometry (i.e. $(L/D)_d$) which is nearly 2.5 times larger than the length of the test pile. While the observed settlement is not known for an $(L/D)_d$ = 7, it is likely to be less than the 2.6 mm exhibited by the actual pile M8 with (L/D) = 2.7. The redesigned pile now satisfies both settlement and allowable endbearing requirements.

The second example illustrates how the design method proceeds for a recessed socketed pile. For an applied load of 2.0 MN (i.e., within the elastic range of behaviour) and a design settlement of 4 mm results in an initial designed pile geometry (i.e., $(L/D)_d$ of 3).

Since the pile is recessed approximately .6 below the ground (rock) surface with a modulus ratio E_t/E_r of .5 (deduced from core hole log) a settlement reduction factor (i.e., $(RF)_\rho$) of approximately 0.93 is obtained from the appropriate charts in Appendix C. Consequently, a modified design socket geometry of L/D = 2.7 is calculated. The design is shown to satisfy settlement and endbearing requirements.

7.4.4 Conclusions

Williams has reported an extensive laboratory and field investigation into socketed pile behaviour and the properties of Melbourne mudstone. The large number of pile tests conducted provide a great deal of information concerning values of average available sideshear resistance and backfigured rock mass modulus. However, Williams has not explained why there is such a large variation in the backfigured rock mass modulus for piles at the Stanley Ave. site. Following a review of Williams's borehole logs, testing procedure and pile geometries, it was suggested that these variations may have resulted from 1) variable rock properties over the test site, 2) the diameter of the socket, 3) the type of loading arrangement adopted for the test, 4) uncertainty regarding the use of a collapsable steel cylinder to prevent endbearing on rock.

Design calculations for pile M8, using the empirical correlations for sideshear resistance and rock mass modulus, have been performed. The results indicate that a reasonable and safe pile design is obtained and satisfies both settlement and endbearing requirements. It was also shown that reasonable pile design is obtained when consideration is given to recessment of the socketed pile below the rock surface.

TABLE 7.1: TEST DATA AND CALCULATED VALUES FOR ENDBEARING STRESS FOR GLOS AND BRIGGS' (1983, 1984) EAST AND WEST SOCKETED PILES

SOCKET	APPLIED LOAD	DATA FROM GLOS AND BRIGGS' (1983) (at Bottom Instrument Level)			CALCULATED [a]AVERAGE BASE PRESSURE (q_{avg}) DEDUCED FROM EQ. 7.1 BASED ON DATA AT LOCATIONS 4, 5, 6					
		σ_{z4}^b	σ_{z5}	σ_{z6}	q_4	q_5	q_6	$\bar{q}_{4+5}$	P_b^c	P_b^d
	(MN)		(MPa)			(MPa)			(MN)	(MN)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
EAST	2.22	2.4	1.7	.96	3.2	2.3	2.0	2.8	.70	-
	4.45	6.3	4.4	2.8	8.4	5.8	5.6	7.1	1.9	-
	6.67	9.7	7.5	4.6	12.9	10.0	9.2	11.5	3.0	-
	8.9	14.4	12.2	6.8	19.0	16.1	13.6	17.6	4.6	-
WEST	2.22	1.1	1.2	.24	1.5	1.6	.5	1.6	.31	.5
	4.45	3.8	3.3	1.1	5.1	4.4	2.2	4.8	1.0	1.4
	6.67	6.8	6.0	2.1	9.0	8.0	4.2	8.5	1.9	2.5
	8.9	10.8	9.6	2.9	14.3	12.7	5.8	13.5	2.8	3.9

[a] assuming r = 230 mm (with a = 305 mm)

[b] vertical stress measured at location of instrumentation (see Figure 7.1)

[c] P_b determined using $(q_6 + \bar{q}_{4+5})/2$

[d] P_b determined using $(\bar{q}_{4+5})$

TABLE 7.2: DESIGN CALCULATIONS WEST PILE, GLOS & BRIGGS' SITE

Quantity	Value	Notes	Quantity	Value	Notes
1. Location	East or West		20. E_s/E_r	-	From borehold log; no seams present
2. ρ_d (m)	.0053		21. τ_s/τ_r	-	" " " " "
3. P_t (MN)	4.5		22. I_h/I_s	-	" " " " "
4. E_p (MPa)	39,000		23. $(L/D)^*_{dmax}$	-	" " " " "
5. q_u (MPa)	8.9	Glos & Briggs (1983)	24. I^*_d	-	" " " " "
6. $\bar{\tau}$ (MPa)	1.79	Eq. 6.1b	25. $(L/D)_d$	2.4	Fig. C1a
7. $\bar{E}_r$ (MPa)	640	Eq. 6.2	26. $(P_b/P_t)_d$(%)	23	Fig. C1b
8. $(RF)_\tau$	.7		27. L (m)	1.46	
9. $(RF)_{E_r}$	.7		28. q_t (MPa)	15.4	Eq. 6.12c
10. τ_d (MPa)	1.25	Eq. 6.3a	29. q_{ba} (MPa)	8.9	Eq. 6.4a
11. E_d (MPa)	450	Eq. 6.3b	30. q_b (MPa)	3.55	Eq. 6.12a < q_{ba}
12. E_p/E_d	87		31. $\bar{\tau}^*$ (MPa)	1.79	From [6] - no seams
13. E_b/E_r	1.0	Sound base	32. q_{ma} (MPa)	22.3	Eq. 6.4b
14. L_e (m)	~13	Recessed socket	33. q_m (MPa)	10.2	Eq. 6.12b < q_{ma}, OK
15. E_t/E_r	0	Neglect recessing effect due to highly weathered rock above pile	34. E_p/E_r	61	
16. D (m)	.61	To conform with Glos & Briggs test piles	35. $(P_b/P_t)_{fs}$(%)	25.5	Fig. C2d and C2e
17. $(L/D)_{max}$	3.1	Eq. 6.8	36. P_t/P_{ts}	.67	Eq. 6.20
18. I_d	.323	Eq. 6.11	37. $PR(\rho>\rho_d)$ (%)	25	Fig. 5.7
19. S	0	From borehole log; no seams present	38. Observed Settlement (m) East socket (L = 1.4 m) West socket (L = 1.47 m)	.0025 .0025	Less than ρ_d, OK Less than ρ_d, OK

TABLE 7.3: COMPARISON OF DIFFERENT PILE DESIGNS FOR P_t = 4.5 MN, D = 0.76 m, Glos & Briggs' Site

Method	Design Length (m)
1. Presumptive Pressures (Glos & Briggs') τ_d = 0.24 MPa, q_{ba} = 2.4 MPa	6.1
2. Allowable pressures based on load test results τ_d = 0.72 MPa, q_{ba} = 4.8 MPa	2.3
3. Proposed Design Method ρ_d = 5 mm, $(RF)_\tau = (RF)_{E_r}$ = 0.5	2.4
4. Proposed Design Method ρ_d = 5 mm, $(RF)_\tau = (RF)_{E_r}$ = 0.7	1.35

TABLE 7.4: DESIGN CALCULATIONS FOR P_t = 4.5 MN, D = 0.76 m, ρ_d = 5mm GLOS AND BRIGGS' SITE

	Design Based on Empirical Correlations $(RF)_\tau = (RF)_{E_r} = 0.5$		Design Based on Empirical Correlations $(RF)_\tau = (RF)_{E_r} = 0.7$	
Quantity	Value	Notes	Value	Notes
1. Location				
2. ρ_d (m)	.0050		.0050	
3. P_t (MN)	4.5		4.5	
4. E_p (MPa)	39,000		39,000	
5. q_u (MPa)	8.9		8.9	
6. $\bar{\tau}$ (MPa)	1.79	Eq. 6.1b	1.79	Eq. 6.1b
7. $\bar{E}_r$ (MPa)	640	Eq. 6.2	640	Eq. 6.2
8. $(RF)_\tau$	.5		.7	
9. $(RF)_{E_r}$	.5		.7	
10. τ_d (MPa)	.90	Eq. 6.3a	1.25	Eq. 6.3a
11. E_d (MPa)	320	Eq. 6.3b	450	Eq. 6.3b
12. E_p/E_d	122		87	
13. E_b/E_r	1.0		1.0	
14. L_e (m)	~13		~13	
15. E_t/E_r	0		0	
16. D (m)	.76	Production Socket Diameter	.76	Production Socket
17. $(L/D)_{dmax}$	2.76	Eq. 6.8	2.0	Eq. 6.8
18. I_d	.27	Eq. 6.11	.38	Eq. 6.11
19. S	0	No seams	0	No seams
20. E_s/E_r	-		-	
21. τ_s/τ_r	-		-	
22. I_h/I_s	-		-	

Table 7.4 (Continued)

Quantity	Value	Notes	Value	Notes
23. $(L/D)^*_{dmax}$	-		-	
24. I^*_d	-		-	
25. $(L/D)_d$	3.2	From Fig. C1a	1.8	From Fig. C1a
26. $(P_b/P_t)_d$(%)	19	Fig. C1b	27.5	Fig. C1b
27. L (m)	2.4		1.35	
28. q_t (MPa)	9.9	Eq. 6.12c	9.9	Eq. 6.12c
29. q_{ba} (MPa)	8.9	Eq. 6.4a	8.9	Eq. 6.4a
30. q_b (MPa)	1.9	Eq. 6.12a < q_{ba}, OK	2.7	Eq. 6.12a < q_{ba}, OK
31. $\bar{\tau}$* (MPa)	1.79	From [6]	1.79	From [6]
32. q_{ma} (MPa)	22.3	Eq. 6.4b	22.3	Eq. 6.4b
33. q_m (MPa)	3.0	Eq. 6.12b < q_{ma}, OK	6.0	Eq. 6.12 < q_{ma}, OK
34. $E_p/\bar{E}_r$	61		61	
35. $(P_b/P_t)_{fs}$(%)	21	Fig. C2d and C2e	30	Fig. C2d and C2e
36. P_t/P_{ts}	0.35	Eq. 6.20	0.54	Eq. 6.20
37. $PR(\rho>\rho_d)$(%)	11	Fig. 5.8	26	Fig. 5.7

TABLE 7.5 INFLUENCE OF DISPLACEMENT CORRECTION ON MEASURED VALUES FOR PILE P2

APPLIED LOAD (MN)	MEASURED DISPLACEMENT (mm)	CORRECTED DISPLACEMENT (mm)
2.0	3.8	4.2
4.0	8.1	8.9
6.0	13.9	15.1
8.0	23.9	25.5

Displacement correction, ρ_c = +.206 mm/MN

TABLE 7.6 INFLUENCE OF LOAD TRANSFER CORRECTION ON MEASURED VALUES FOR PILE P2

APPLIED LOAD (MN)	MEASURED RATIO P_b/P_t	CORRECTED RATIO P_b/P_t
2.0	.15	.13
4.0	.19	.17
6.0	.34	.32
8.0	.45	.43

TABLE 7.7 SUMMARY OF WILLIAMS'S SOCKET PILE LOAD TEST RESULTS AT STANLEY AVENUE

ZONE	SOCKET DESIGNATION	q_u (MPa)	D (m)	BACKFIGURED ROCK MASS MODULUS (MPa) (initial Tangent Modulus) Williams	This Study	TYPE OF PILE	TYPE OF TESTING ARRANGEMENT USED (see Williams (1980) for details)
	S6	.60	.10	210	-	Endbearing Only	Hydraulic Jack Against
	S7	.44	.10	143	-	"	a Fixed Reaction Beam
II	S9	.65	.30	184	-	"	"
	S11	.67	.30	165	-	"	"
	S12	.58	.335	161	141	Sideshear Only	"
	S10	.75	.10	74	-	Endbearing Only	"
	S13	.57	.10	98	-	"	"
	S14	.58	.395	90	88	Sideshear Only	"
I	S15	.58	.395	100	70	"	"
	S16	.58	.395	100	83	"	"
	S19	.45	.10	97	-	Endbearing Only	"
	S22	.52	.10	77	-	"	"
	S1	.83	.66	259	306	Sideshear Only	Central Cable Anchor
	S2	.54	.60	256	-	Endbearing Only*	2 Anchor Cables,Reaction Beam
III	S3	.55	1.17	586	406	Sideshear Only	Central Reaction Anchor
	S4	.57	1.0	473	-	Endbearing	2 Anchor Cables,Reaction Beam
	S5	.59	1.12	562	501	Sideshear Only	Central Cable Anchor

* at surface

TABLE 7.8: DESIGN CALCULATIONS FOR PILE M8
(Based on Empirical Correlations - neglecting recessment)

	QUANTITY	(a) VALUE	(a) NOTES	(b) VALUE	(b) NOTES
1.	Location	M8		M8	
2.	ρ_d(m)	.010		0.0045	
3.	P_t (MN)	3.5		3.5	
4.	E_p (MPa)	35,000		35,000	
5.	q_u (MPa)	2.0	Williams (1980)	2.0	Williams (1980)
6.	$\bar{\tau}$ (MPa)	.64	Eq. 6.1a	.64	Eq. 6.1a
7.	$\bar{E}_r$ (MPa)	304	Eq. 6.2	304	Eq. 6.2
8.	$(RF)_\tau$	.7		.7	
9.	$(RF)_{E_r}$	.7		.7	
10.	τ_d (MPa)	.45	Eq. 6.3a	.45	Eq. 6.3a
11.	E_d (MPa)	213	Eq. 6.3b	213	Eq. 6.3b
12.	E_p/E_d	164		164	
13.	E_b/E_r	1.0	Assume sound base	1.0	Assume sound base
14.	L_e (m)	0.6	Recessed socket	0.6	Recessed socket
15.	E_t/E_r	0.5	Neglect recessing effect since slip is likely	0.5	Neglect recessing effect
16.	D (m)	.66		.66	
17.	$(L/D)_{dmax}$	5.68	Eq. 6.8	5.68	Eq. 6.8
18.	I_d	.40	Eq. 6.11	.18	Eq. 6.11
19.	S	0	No seams	0	No seams
20.	E_s/E_r	—		—	

TABLE 7.8 Continued

	QUANTITY	(a) VALUE	(a) NOTES	(b) VALUE	(b) NOTES
21.	τ_s/τ_r	—		—	
22.	I_h/I_s	—		—	
23.	$(L/D)^*_{dmax}$	—		—	
24.	I^*_d	—		—	
25.	$(L/D)_d$	2.8	Fig.C2e & C2f	7.0	Fig. C1a
26.	$(P_b/P_t)_d$(%)	48	Fig. C2e & C2f	10	Fig.C1b
27.	L (m)	1.85		4.6	
28.	q_t (MPa)	10.2	Eq. 6.12c	10.2	Eq. 6.12c
29.	q_{ba} (MPa)	2.0	Eq. 6.4a	2.0	Eq. 6.4a
30.	q_b (MPa)	4.9	Eq. 6.12a $>q_{ba}$, N.G.	1.	Eq. 6.12a $<q_{ba}$, O.K.
31.	$\bar{\tau}$ * (MPa)	.64	From (6)	.64	From (6)
32.	q_{ma} (MPa)	5.0	Eq. 6.4b	5.0	Eq. 6.4b
33.	q_m (MPa)	8.0	Eq.6.12b$>q_{ma}$, N.G. redesign	4.8	Eq.6.12b$<q_{ma}$, O.K.
34.	$E_p/\bar{E}_r$			115	
35.	$(P_b/P_t)_{fs}$(%)			11	Fig. C2e, C2f
36.	P_t/P_{ts}			.51	
37.	PR($\rho > \rho_d$)(%)			~ 27	Fig. 5.7
38.	Observed Settlement (m)	2.6	Less than ρ_d, O.K.	<2.6	Less than ρ_d, O.K.

TABLE 7.9 DESIGN CALCULATIONS FOR PILE M8

Design Based on Empirical Correlations and including socket recessment

	QUANTITY	VALUE	NOTES
1.	Location	M8	
2.	ρ_d (m)	.004	
3.	P_t (MN)	2.0	
4.	E_p (MPa)	35,000	
5.	q_u (MPa)	2.0	Williams (1980)
6.	$\bar{\tau}$ (MPa)	.64	Eq. 6.1a
7.	$\bar{E}_r$ (MPa)	304	Eq. 6.2
8.	$(RF)_\tau$	.7	
9.	$(RF)_{E_r}$	.7	
10.	τ_d (MPa)	.45	Eq. 6.3a
11.	E_d (MPa)	213	Eq. 6.3b
12.	E_p/E_d	164	
13.	E_b/E_r	1.0	Assume sound base
14.	L_e (m)	0.6	Recessed Socket
15.	E_t/E_r	0.5	Based on core log information
16.	D (m)	.66	
17.	$(L/D)_{dmax}$	3.25	Eq. 6.8
18.	I_d	.28	Eq. 6.11
19.	S	0	No seams
20.	E_s/E_r	-	From Fig. C2e and C2f we see that the design is elastic. From C1a $(L/D)_d = 3$
21.	τ_s/τ_r	-	From C16b, $(RF)_\rho = 0.88$ for $E_t/E_b = 1$
22.	I_h/I_s	-	From C18 it is clear that linear interpolation for E_t/E_b is conservative
23.	$(L/D)^*_{dmax}$	-	$\therefore (RF)_\rho \doteq 0.94$
24.	I^*_d	.30	$\therefore I^*_d = I_d/(RF)_\rho = 0.3$
25.	$(L/D)_d$	2.7	From C1a $(L/D)_d \doteq 2.7$

TABLE 7.9 (cont'd)

	QUANTITY	VALUE	NOTES
26.	$(P_b/P_t)_d$(%)	21	Fig C1b
27.	L (m)	1.8	
28.	q_t (MPa)	5.8	Eq. 6.12c
29.	q_{ba} (MPa)	2.0	Eq. 6.4a
30.	q_b (MPa)	1.23	Eq. 6.12a $< q_{ba}$, O.K.
31.	$\bar{\tau}^*$ (MPa)	.64	From (6)
32.	q_{ma} (MPa)	5.0	Eq. 6.4b
33.	q_m (MPa)	3.7	Eq. 6.12b $< q_{ma}$ O.K.
34.	$E_p/\bar{E}_r$	115	
35.	$(P_b/P_t)_{fs}$(%)	25.5	Fig. C2e, C2f
36.	P_t/P_{ts}	.63	Eq. 6.20
37.	PR($\rho > \rho_d$) (%)	∿ 26	Fig. 5.7
38.	Observed Settlement (m)	.0017	Less than ρ_d, O.K.

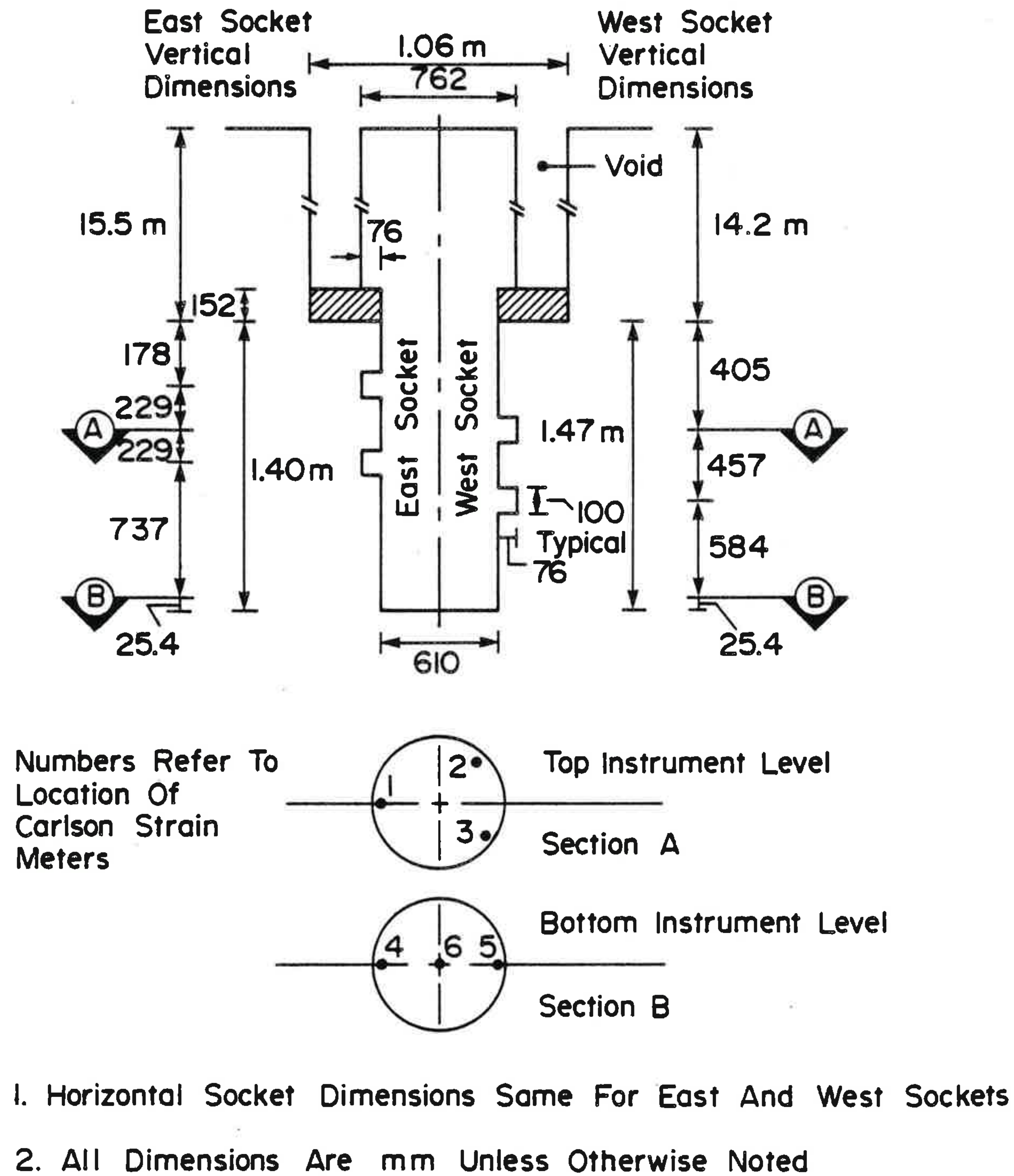


FIGURE 7.1 GEOMETRY AND INSTRUMENT LOCATIONS FOR EAST AND WEST SOCKETS (from Glos and Briggs, 1983) (not to scale)

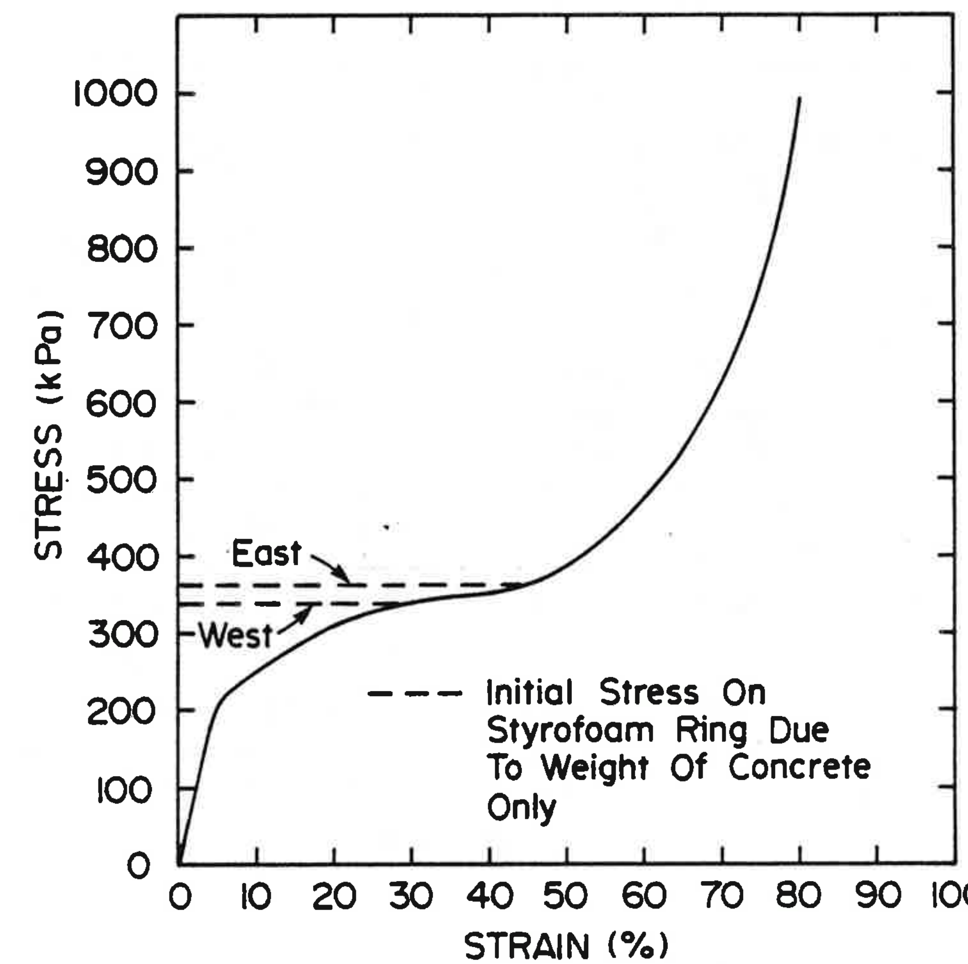


FIGURE 7.2 STRESS-STRAIN RESULTS FOR A COMPRESSION TEST ON A 51 mm THICK, 114 mm DIAMETER POLYSTYRENE DISC

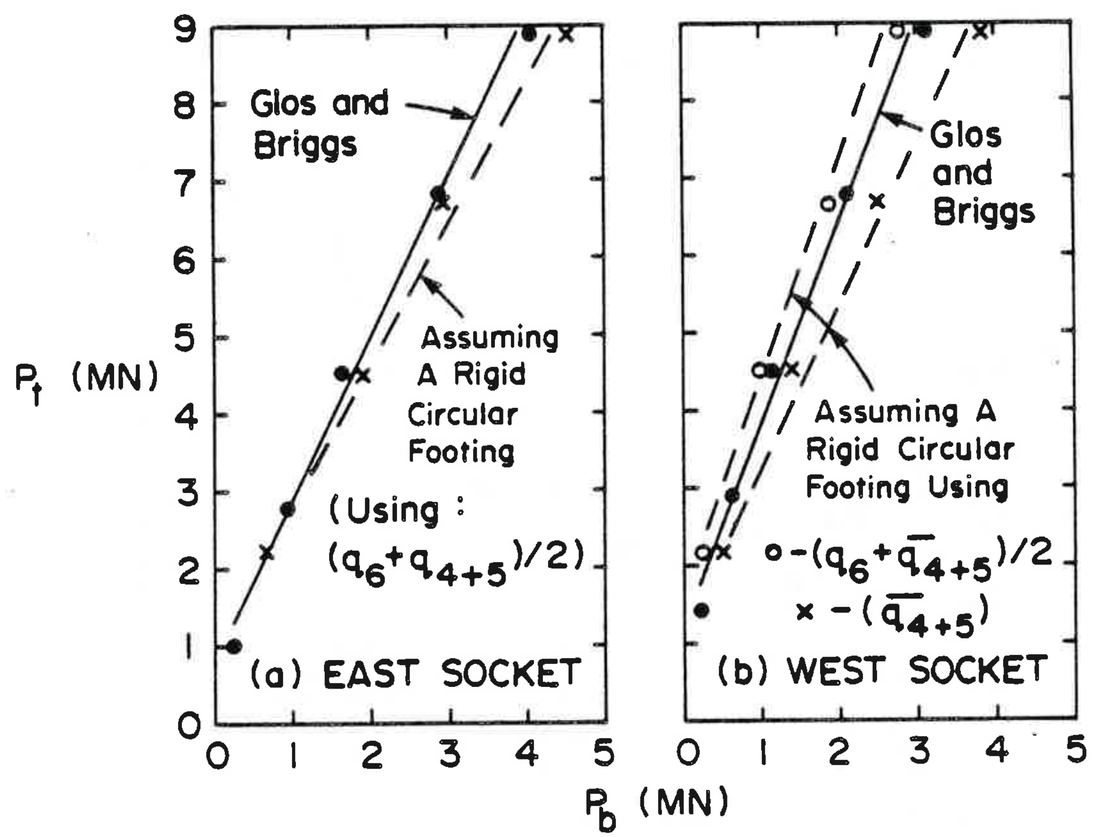


FIGURE 7.3 VARIATION BETWEEN APPLIED LOAD AND LOAD TRANSFERRED TO PILE BASE USING,

i) DATA SUPPLIED BY GLOS AND BRIGGS (1983, 1984)

ii) ASSUMPTION OF A RIGIDLY LOADED CIRCULAR FOOTING

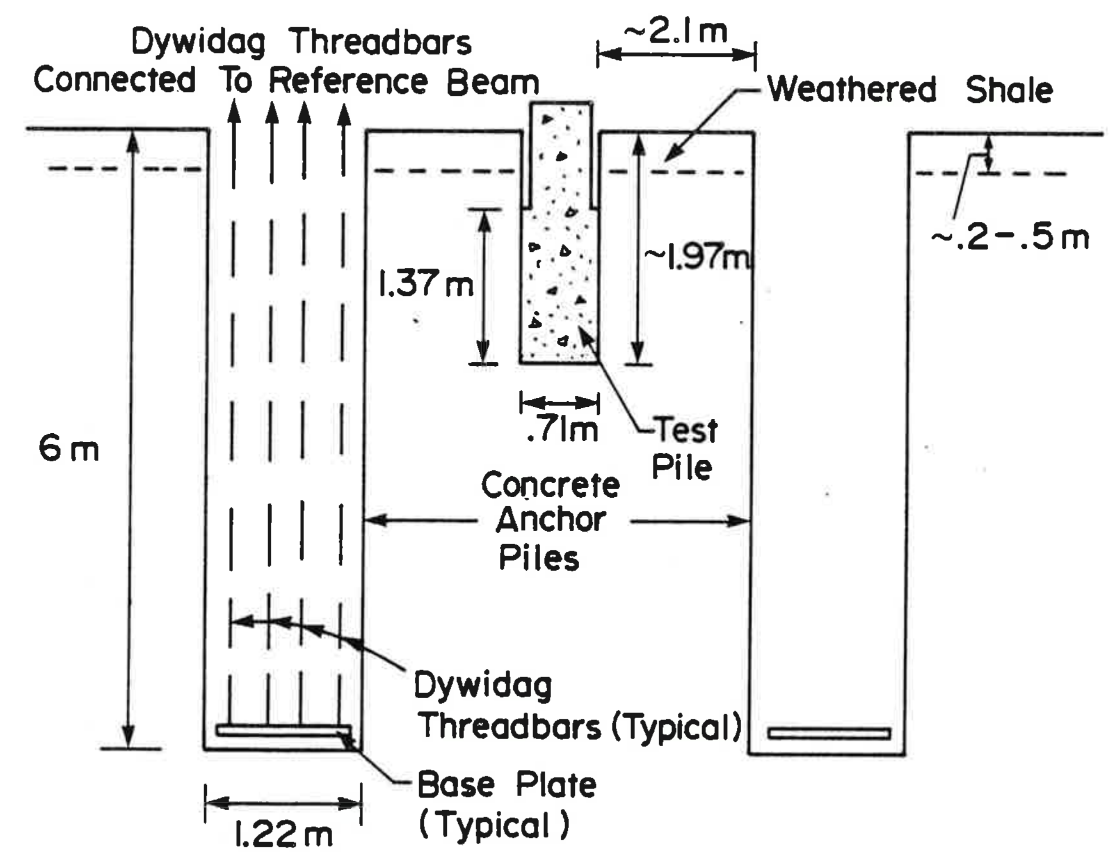


FIGURE 7.4 TYPICAL ARRANGEMENT FOR HORVATH'S TEST PILES

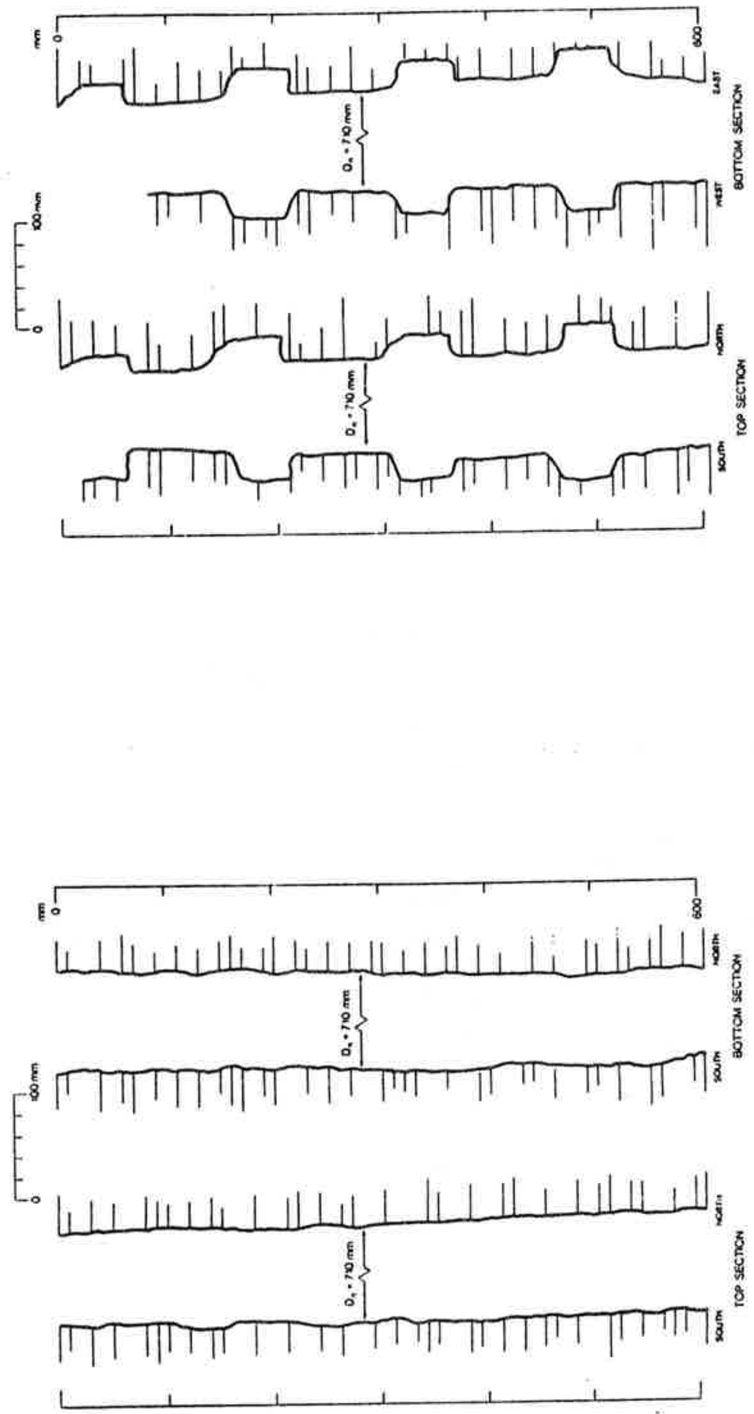
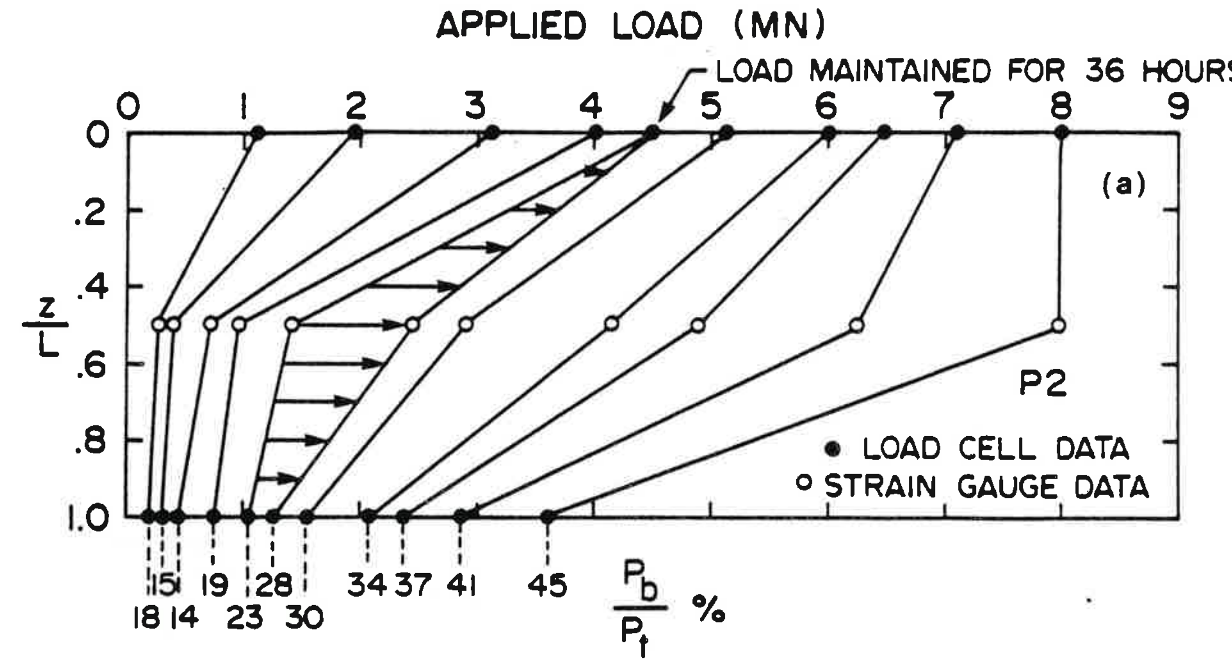


FIGURE 7.5 PROFILES TRACED FROM THE SOCKET SIDEWALLS OF PILES P2 AND P4 (From Horvath, 1982)



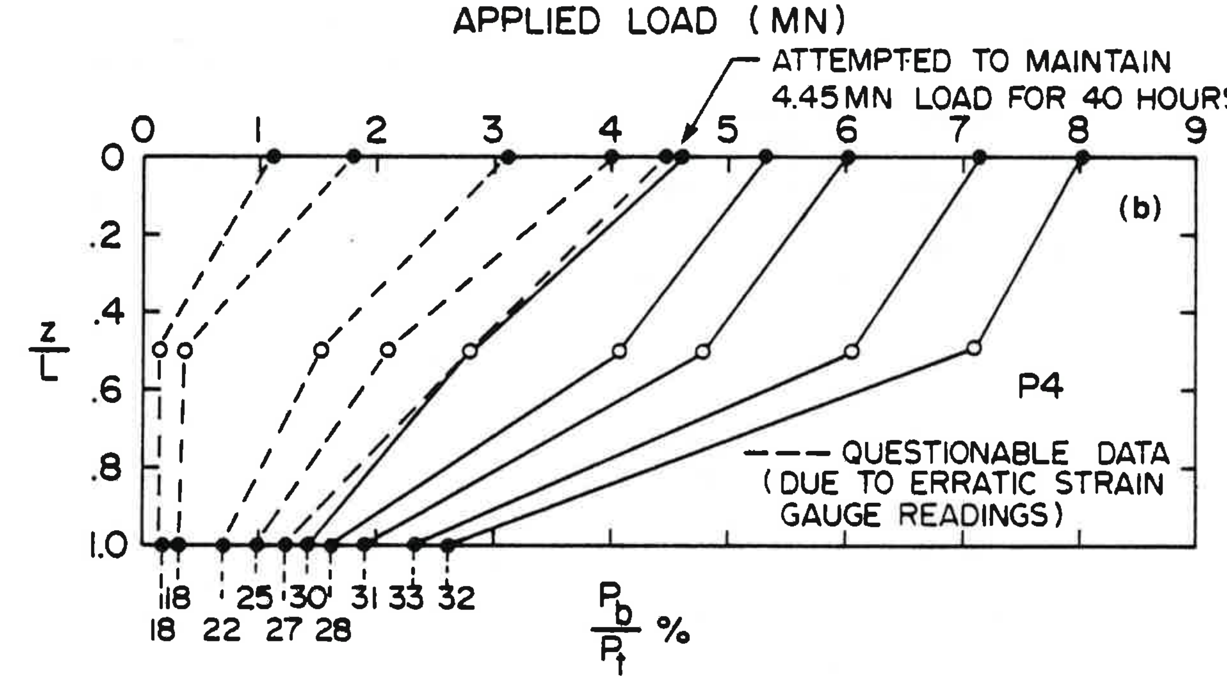
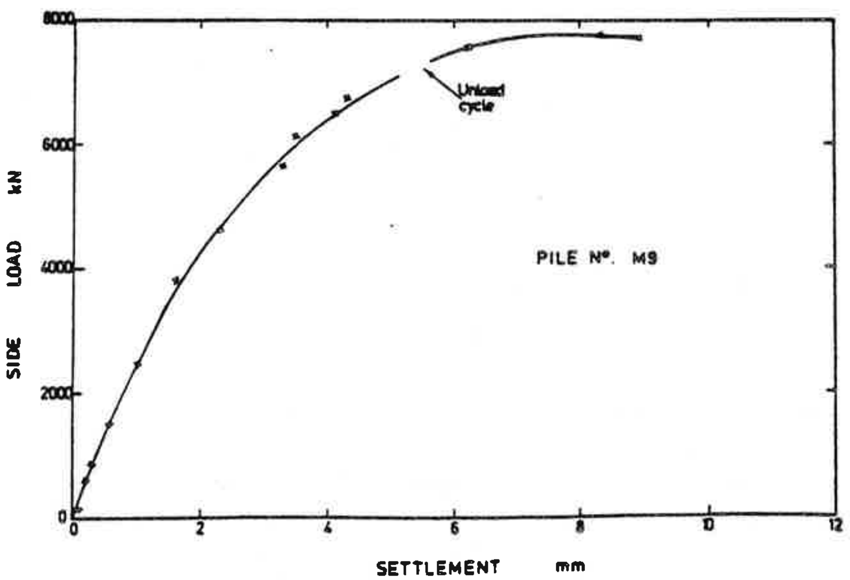
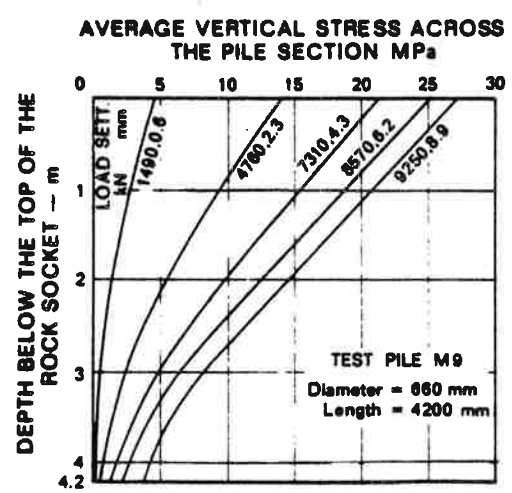


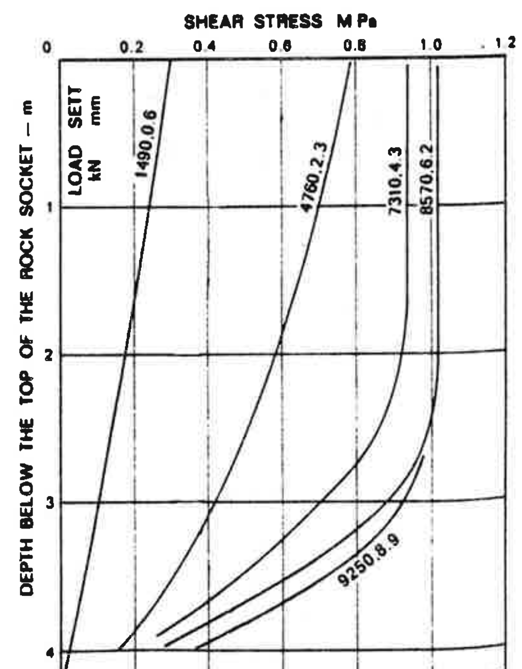
FIGURE 7.6 VERTICAL LOAD DISTRIBUTION CURVES FOR FIELD TESTED COMPLETE SOCKETED PILES PERFORMED BY HORVATH (1980,1982) (a) CONVENTIONAL SOCKET (b) ROUGHENED SOCKET



(a) LOAD-SETTLEMENT CURVES FOR PILE M9 - SIDE RESISTANCE COMPONENT
(from Williams, 1980)



(b) Variations in load distribution during yielding of pile M9.
(from Williams et al., 1980)



(c) An example of non-uniform development of side resistance. The results are from pile M9.
(from Williams et al., 1980)

FIGURE 7.7 LOAD TEST RESULTS FOR WILLIAMS' COMPLETE SOCKETED PILE M9

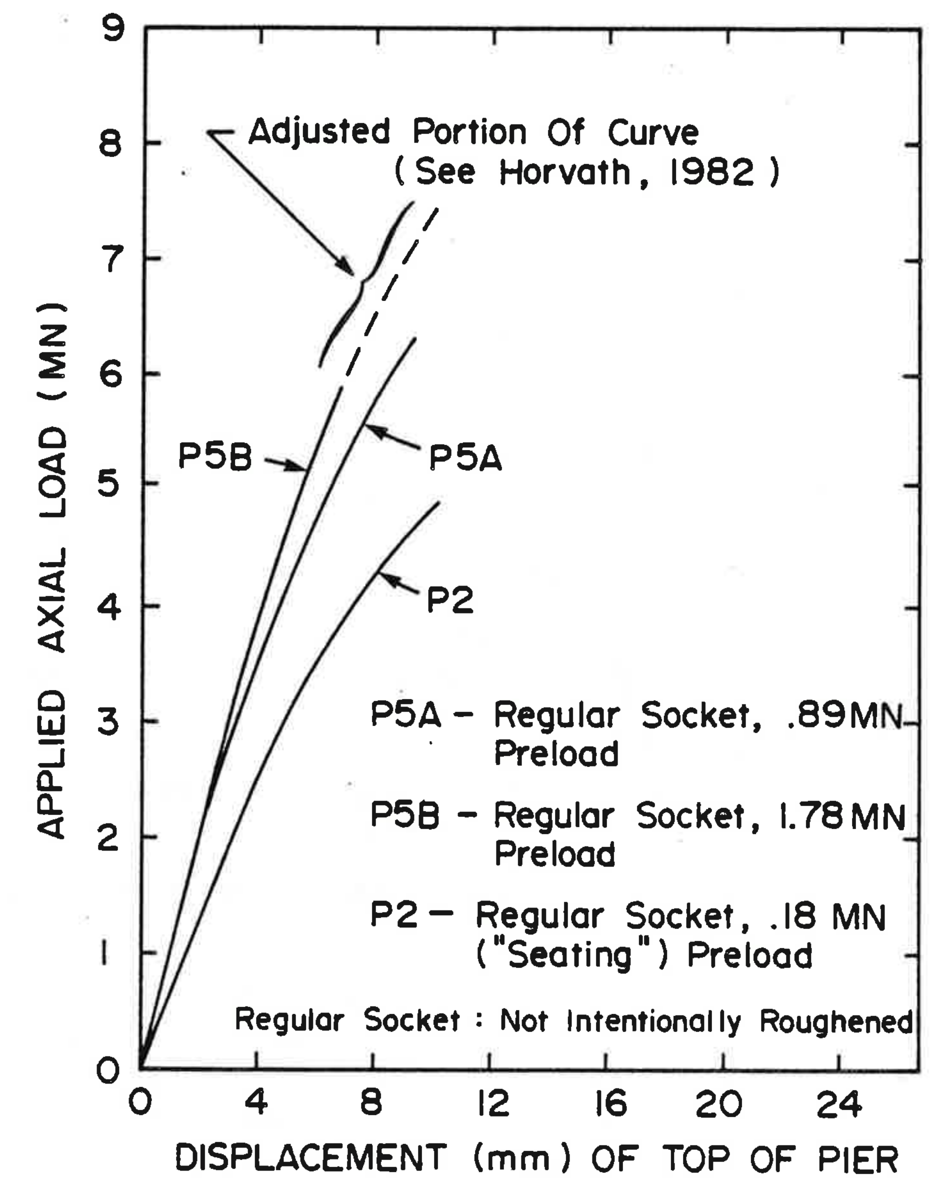


FIGURE 7.8 COMPARISON OF LOAD DISPLACEMENT BEHAVIOUR FOR PRELOADED PILES, P5A AND P5B WITH A CONVENTIONAL SOCKETED PILE, P2 (data from Horvath, 1982)

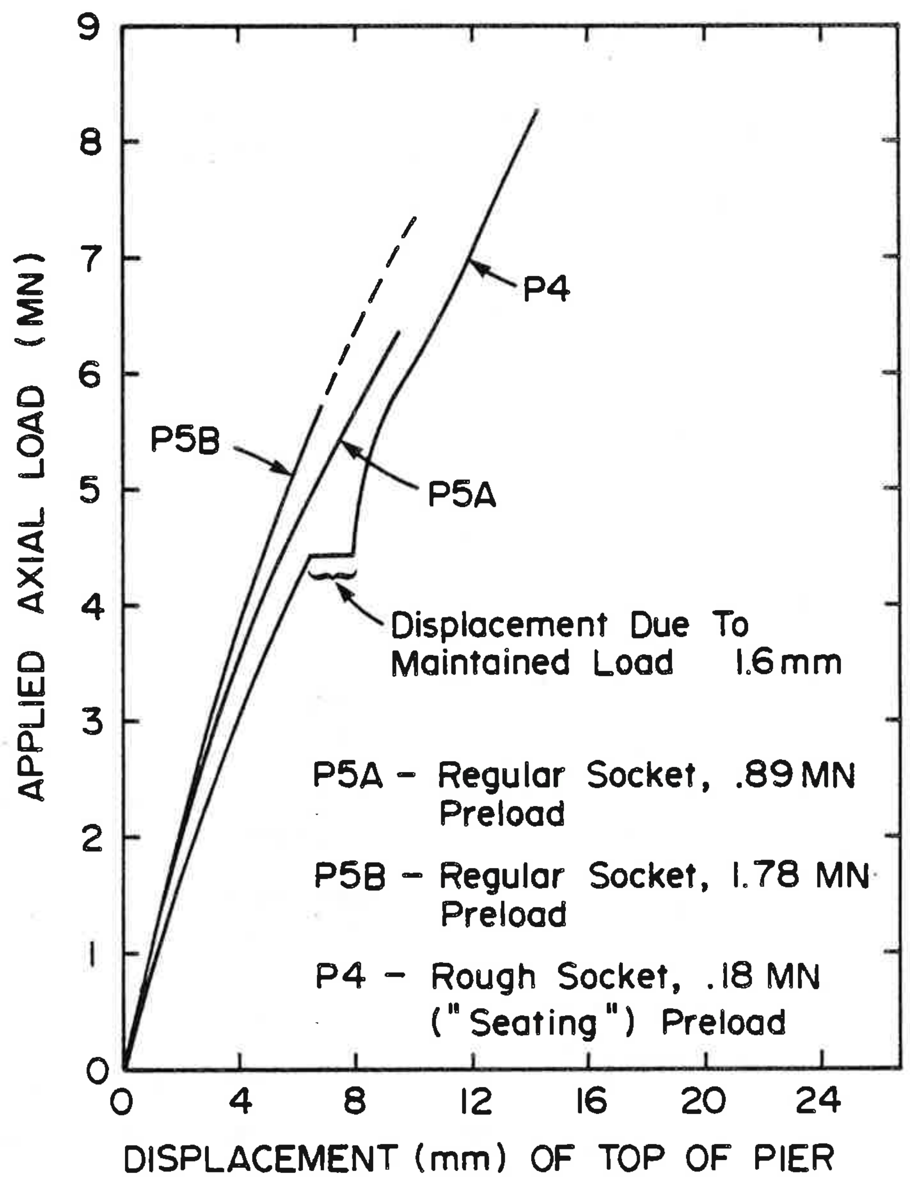


FIGURE 7.9 COMPARISON OF LOAD-DISPLACEMENT BEHAVIOUR FOR PRELOADED PILES, P5A AND P5B WITH A ROUGHENED SOCKETED PILE, P4 (data from Horvath, 1982)

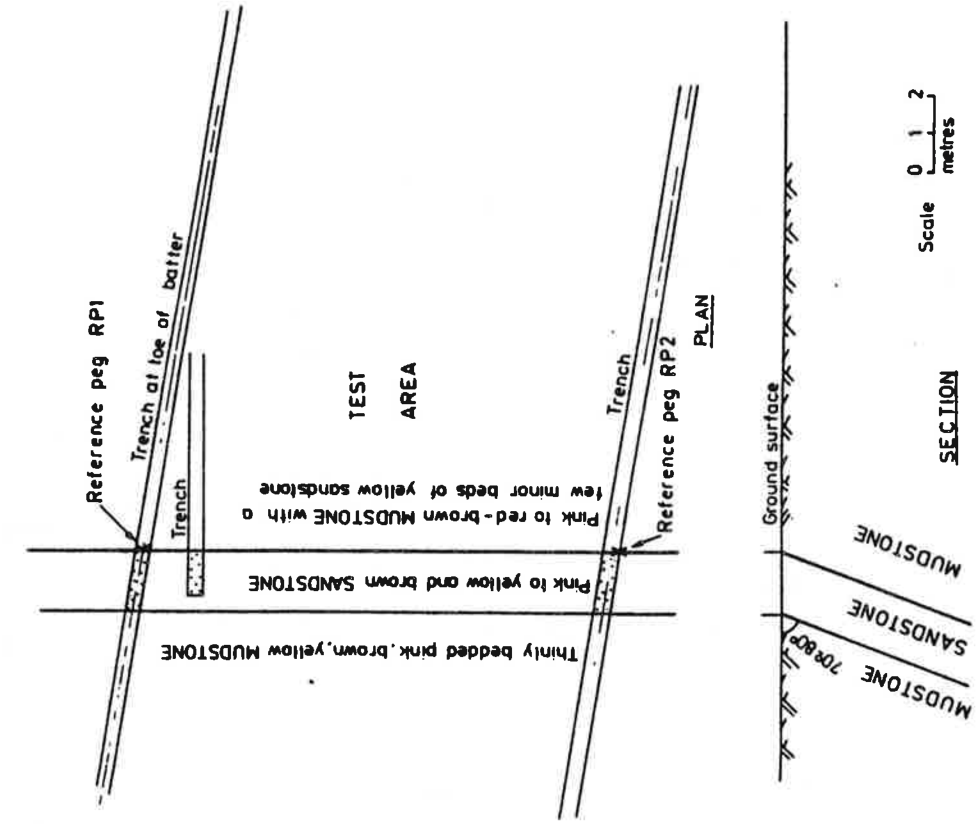


FIGURE 7.11 SITE GEOLOGY PLAN FOR STANLEY AVENUE (from Williams, 1980)

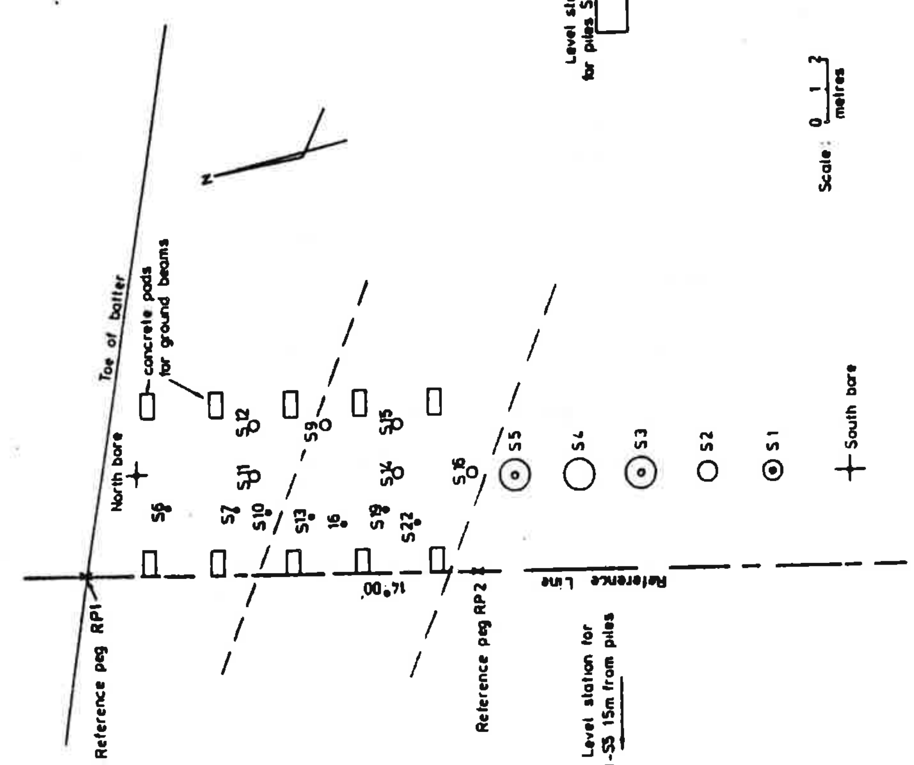


FIGURE 7.10 PILE LOCATION PLAN FOR STANLEY AVENUE (from Williams, 1980)

MATERIALS RESEARCH DIVISION

BORE LOG-ROCK

Highway/Municipality	Bridge/Stream	Bore No
MULGRAVE FREEWAY	STANLEY AVE.	NORTH
Section/Road: TEST PILE SITE	Bore Location: SEE SITE PLAN	Job No: 89234
RL: — / Drill Method: NMLC with water	Angle of Bore (from vertical): 0	Trend of Bore: —

Depth (m)	Log	Lithology	Structure, Hardness, Etc.	Dip°	J.T.F.V. No/m	JT Description	R%
0		SILTSTONE (Silurian)	Pink Highly weathered	65° to 70°			88
							.95
1			1.1 Mottled pink white & yellow HW-XW				
			1.4 Pink HW				90
2			2.14 Cream XW				2.34
			2.34				100
		SANDSTONE	2.75 Brown-yellow HW-MW				
3		SILTSTONE	2.95 Pink HW				3.16
							100
			3.5 Pink & yellow HW				3.5
4			4.0 Yellow HW				93
			4.5 END of BORE				4.5
5			Note: core badly disturbed ∴ no assessment of joint frequency or joint description				

Logged by [illegible]
Date Dec 77

MRD 44 Feb 1978

FIGURE 7.12 **BORE LOG FOR THE NORTH BORE AT STANLEY AVENUE.** (from Williams, 1980)

MATERIALS RESEARCH DIVISION

BORE LOG-ROCK

Highway/Municipality	Bridge/Stream	Bore No
MULGRAVE FREEWAY	STANLEY AVE.	SOUTH
Section/Road: TEST PILE SITE	Bore Location: SEE SITE PLAN	Job No: 89234
RL: — / Drill Method: NMLC with water	Angle of Bore (from vertical): 0	Trend of Bore: —

Depth (m)	Log	Lithology	Structure, Hardness, Etc.	Dip°	J.T.F.V. No/m	JT Description	R%
0		SILTSTONE (Silurian)	Mottled red & white HW	78 approx			
			0.15 Pink HW				100
1							1.05
							100
							1.58
							100
2							1.83
		SANDSTONE	2.3 Red-brown HW				90
		SILTSTONE	2.6 Pink HW				
3							3.0
							67
			3.41 Pink and yellow				3.41
4							93
			4.5 END of BORE				4.5
5			Note: The core was badly disturbed ∴ no assessment of joint frequency or joint description.				

Logged by [illegible]
Date Dec 77

MRD 44 Feb 1978

FIGURE 7.13 **BORE LOG FOR SOUTH BORE AT STANLEY AVENUE** (from Williams, 1980)

CHAPTER 8

SUMMARY AND CONCLUSIONS

This research programme has been concerned with the development of improved, rational procedures for the design of piles socketed into rock where load is carried both in sideshear and endbearing. The following paragraphs summarize the various aspects considered and the key findings.

(1) Current design procedures were reviewed and it was found that in the majority of cases (with some notable exceptions) socketed piles are presently designed on the basis of presumptive allowable endbearing and sideshear values which may be very conservative.

(2) A theoretical approach for the prediction of the behaviour of piles socketed into weak rock has been described. This approach allows consideration of plastic failure within the rock, as well as slip, strain softening and dilatancy at the pile-rock interface.

(3) The fundamental difference between tests commonly used for determining average sideshear resistance has been discussed. It was suggested that there will generally be a discrepancy between the average sideshear obtained from displacement controlled tests on sideshear sockets and that obtained from load defined tests on endbearing piles. This discrepancy is small for moderately long and rough sockets however the results obtained from sideshear sockets may underestimate the available residual sideshear resistance of endbearing piles. The magnitude of this discrepancy will depend upon the relative stiffness of the pile and rock as well as the length and roughness of the socket. In theory, the discrepancy may be more

than an order of magnitude for typical parameters and a perfectly smooth socket although in practice, this variation would not be expected because of the finite roughness of even "smooth" sockets.

(4) The effect of a number of fundamental parameters upon the average peak and residual sideshear values was examined for endbearing piles subjected to a load defined test. It was shown that the magnitude of the average sideshear depends not only upon the strength parameters, but also upon dilatancy at the interface, the length to diameter ratio and the relative modulus of the pile and rock. The average sideshear deduced using typical values of the fundamental parameters for two types of rock were of similar magnitude to those observed in field tests. These results suggest that the apparent scatter of results obtained from different field tests on rock with apparently the same strength characteristics, may be partly due to the effect of parameters such as pile length and modulus ratio.

(5) A series of solutions were presented for a pile socketed into a homogeneous rock and for a pile bearing on either stiffer or weaker rock. In addition to the elastic response, these solutions provide a means of estimating the load at which full slip occurs and the corresponding pile head settlement; as well as the pile head displacement for any given load following full slip. Nonlinearity of the bearing strata may be approximated by the selection of an appropriate secant modulus.

(6) The effect of weak horizontal seams along the pile shaft was examined and theoretical solutions for assessing the significance of these seams upon the pile response were presented.

(7) A theoretical analysis which illustrates the settlement behaviour of complete sockets recessed below the rock surface has been presented. The results of this analysis are given as a series of design charts which will provide an estimate of the reduction in settlement experienced by a socketed pile recessed in either a homogeneous or non-homogeneous rock mass.

(8) The results of published socketed pile case histories have been carefully reviewed and subsequently placed into one of two distinct categories. The data judged to be most reliable was then used to develop empirical correlations between the unconfined strength of intact rock and (1) the measured (average) sideshear resistance and (2) the backfigured rock mass modulus. These correlations have been presented in the form of several simple equations which can be used to select preliminary values of sideshear resistance and rock mass modulus for socketed pile design. Caution should be adopted in using any empirical correlations. If the correlations presented in this report are used, the design should be checked by proof loading.

(9) A probabilistic analysis using the Monte Carlo simulation technique was used to investigate the effects of statistical variability of two principal socket design parameters, namely sideshear resistance (τ) and rock mass modulus (E_r) on the predicted distributions of settlement. These distributions of settlement together with estimates of design settlement from deterministic analyses performed using design parameters τ_d, E_d were used to provide an indication of the reliability of a socketed pile design in terms of the probability of exceeding the specified design settlement.

(10) The probability of a exceeding design settlement was found to be influenced by the following factors:

(a) Variability in τ, particularly near load levels which cause slip at the pile-rock interface;

(b) Socket geometry (i.e., length to diameter ratio);

(c) The value of reduction factors, $(RF)_\tau$ and $(RF)_{E_r}$ which are applied to expected values of sideshear and modulus $\bar{\tau}$ and $\bar{E}_r$ to give the design values $\tau_d = (RF)_\tau \bar{\tau}$ and $E_d = (RF)_{E_r} \bar{E}_r$.

(11) The variability in the value of unconfined compressive strength (q_u) was found to produce relatively minor changes in the probability of exceeding the design settlement at all load levels.

(12) The probability of exceeding the design settlement for the suggested design values of reduction factors $(RF)_\tau$ and $(RF)_{E_r}$, of 0.7 is less than 30% and the probability of exceeding the design settlement calculated for $(RF)_\tau = (RF)_{E_r} = 0.5$ is less than 11%. The probability of exceeding twice the design settlement is less than 3% in both cases.

(13) A new design procedure has been proposed. This new procedure is relatively simple and a socket can be designed in less than 15 minutes once the designer is familiar with the technique. The design method is based on:

(a) satisfying a specified design settlement criterion

(b) checking to ensure there is an adequate factor of safety against collapse.

(14) The design method has been illustrated by means of a number of hypothetical examples and by a series of detailed calculations relating to the complete sockets P2 and P4 tested by Horvath (1980).

These calculations show that piles designed to have the same geometry as those tested by Horvath (1980) would have satisfied the design settlement criteria while having a proven "factor of safety" against collapse of at least 2. (Since the test piles were not brought to collapse the actual "factor of safety" is unknown.)

(15) Design calculations using the procedure described in Chapter 6 produced socket lengths which are in good agreement with those actually used by Glos and Briggs (1983). These calculations show that the piles designed for $P_t = 4.5$ MN to have the same geometry as those tested by Glos and Briggs would have satisfied the design settlement criteria while having a proven "factor of safety" against collapse of at least 2 (again the piles were not brought to collapse).

(16) Design calculations (Chapter 6) show that a pile designed to have the same geometry as Williams's (1980) pile M8 would have satisfied the design settlement criteria. In this case, the design procedure would have required a longer socket to satisfy endbearing considerations. Nevertheless, as designed for $P_t = 3.5$ MN, the actual pile had a factor of safety against collapse of at least 2.25 (again the pile was not loaded to collapse).

(17) Based on the available evidence, it would appear that piles designed using the proposed procedure would have performed adequately at the sites described by Williams (1980), Horvath (1980), and Glos and Briggs (1983).

(18) Additional aspects of three key case histories (Glos and Briggs, 1983; Williams, 1980; Horvath, 1980) such as the effect of styrofoam

to eliminate endbearing, the stress distribution at the base and along the socket, and the effect of anchor piles have been discussed and conclusions regarding these factors are given in Chapter 7.

CHAPTER 9

RECOMMENDATIONS

Based on the results of this study, the following recommendations are made:

1. It is recommended that the proposed design procedure be adopted in the design of a number of full scale piles in soft/weak rock ($q_u <$ 35 MPa). This design should be performed by an experienced geotechnical engineer. The design should be preceded by an adequate preliminary site investigation and should be supplemented by an inspection of the drilled sockets prior to casting of the socket. If the actual socket conditions in the field do not conform with the design assumptions based on the preliminary site investigation (eg. if unexpected seams are encountered), the socket should be redesigned (i.e., the socket length increased) as necessary. The proposed design procedure permits rapid redesign of piles under these conditions. Until adequate experience has been gained with the new design procedure, representative piles should be proof loaded to ensure the allowable design settlement is not exceeded under the expected loading conditions.
2. One of the major uncertainties with regard to the design of complete socketed piles is the degree of load transfer from sideshear to endbearing with time. It is recommended that a number of long term full scale tests be performed to monitor the redistribution of load with time. The results of these tests may permit the adoption of

even less conservative procedures than those proposed in this report.

3. This study has excluded consideration of socketed piles in rock which may exhibit significant time dependent deformations (eg. rock salt, potash). It is recommended that the proposed procedure should not be used for sockets in these rocks until more reliable field data has been obtained.
4. The proposed design procedure does not consider preloading of the base of socketed piles. It is considered that more long term full scale test data is required before design procedures can be confidently recommended for socketed piles which have been subjected to significant preloading at the base. It is recommended that additional research be conducted in the area.

ACKNOWLEDGEMENTS

The research described in this report was supported by the Department of Supply and Services Canada and the National Research Council of Canada under contract No. ISU83-00082.

Many thanks are due to Dr. M. Bozozuk, of the National Research Council, who acted as Scientific Advisor on this project. Thanks are also due to Mr. P.J.N. Pells for many useful discussions over many years.

Finally, the authors wish to express their appreciation to Ms. J. Lemon for her patience and accurate, speedy typing of the report and to Mr. G. Lusk for tracing the figures. Both provided excellent work under very tight time constraints.

REFERENCES

Aurora, R.P. and Reese, L.C. 1977. Field tests of drilled shafts in clay-shales. Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol. 2, pp. 371-376.

Baecher, G.B. and Rackwitz, R. 1982. Factors of safety and pile load tests. International Journal for Numerical and Analytical Methods in Geomechanics, Vol. 6, No. 4, pp. 409-424.

Ballivy, G. and Dupuis, M. 1980. Laboratory and field evaluation of the bonding strength of grouted rock anchors. Proceedings of the 13th Canadian Rock Mechanics Symposium, Toronto, pp. 97-102.

Ballivy, G., Poulin, B., Guertin, J. and Corriveau, R. 1981. Techniques d'essais en chantier et en laboratoire pour la conception des fondations ancrées à la roche. Canadian Geotechnical Journal, Vol. 18, No. 4, pp. 577-583.

Barton, N. 1976. The shear strength of rocks and rock joints. International Journal of Rock Mechanics and Mining Science, Vol. 13, No. 9, pp. 255-279.

Barton, N., Lien, R. and Lunde, J. 1974. Engineering classification for the design of tunnel support. Rock Mechanics, Vol. 6, No. 4, pp. 189-236.

Bauer, G.E. 1980. The measurement and analysis of load transfer from a caisson to its rock socket. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 1, pp. 235-240.

Bertok, J. and Berezowski, M. 1983. Pile load tests for the Calgary Air Terminal Building. Canadian Geotechnical Journal, Vol. 20, No. 2, pp. 353-361.

Bieniawski, Z.T. 1976. Rock mass classification in rock engineering. Proceedings of the Symposium on Exploration for Rock Engineering, Johannesburg, pp. 97-106.

Bieniawski, Z.T. 1978. Determining rock mass deformability: Experience from case histories. International Journal of Rock Mechanics and Mining Science, Vol. 15, No. 5, pp. 237-247.

Bowles, D.S. and Ko, H-Y. (Eds.) 1984. Probabilistic characterization of soil properties: Bridge between theory and practice. Proceedings of a symposium sponsored by ASCE, ASCE Convention, Atlanta, Georgia.

Brown, E.T. and Trollope, D.H. 1970. Strength of a model of jointed rock. Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 96, No. SM2, pp. 685-704.

Burland, J.B. 1970. Discussion, Session A. Proceedings, Conference on In Situ Investigations in Soils and Rocks, British Geotechnical Society, London, England, pp. 61-62.

Burman, B.C. and Hammett, R.D. 1975. Design of foundations in jointed rock masses. Proceedings of the 2nd Australia-New Zealand Conference on Geomechanics, Brisbane, pp. 83-88.

Bushell, T.D. and Baker, C.N. 1982. Caissons socketed in sound mica schist: Discussion. Journal of the Geotechnical Engineering Division, ASCE, Vol. 108, No. GT7, pp. 988-991.

Buttling, S. 1976. Estimates of shaft and end loads in piles in chalk using strain gauge instrumentation. Geotechnique, Vol. 26, No. 1, pp. 133-147.

Canadian Foundation Engineering Manual. 1978. Canadian Geotechnical Society, J.D. Scott (Ed.), Montreal.

Chappell, B.A. and Maurice, R. 1980. Classification of rock mass related to foundations. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 1, pp. 29-42.

Chiu, H.K. and Dight, P.M. 1983. Prediction of the performance of rock-socketed side-resistance-only piles using profiles. International Journal of Rock Mechanics and Mining Science, Vol. 20, No. 1, pp. 21-32.

Christian, J.T. and Carrier III, W.O. 1978. Janbu, Bjerrum and Kjaernsli's charts reinterpreted. Canadian Geotechnical Journal, Vol. 15, No. 1, pp. 123-127.

Coates, D.F. and Gyenge, M. 1966. Plate-load testing on rock for deformation and strength properties. Testing Techniques for Rock Mechanics, ASTM STP 402, pp. 19-35.

Coates, D.F. 1981. Rock Mechanics Principles, Monograph 874, Energy, Mines and Resources Canada, Ottawa.

Coates, D.F. and Yu, Y.S. 1970. Rock anchor design mechanics. Mining Research Centre, Department of Energy, Mines and Resources, Mines Branch, Ottawa, Canada, Internal Report MR 70/63-LD.

Costa Nunnes, J.A. and Fernandes, C.E. 1981. Computed friction in a pile's rock-socket. Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Vol. 2, pp. 677-680.

Coon, R.F. and Merritt, A.H. 1970. Predicting in situ modulus of deformation using rock quality indexes. Determination of the In Situ Modulus of Deformation of Rock, ASTM STP 477, pp. 154-173.

Couetdic, J.M. and Barron, K. 1975. Plate-load testing as a method of assessing the in-situ strength properties of Western Canadian coal. International Journal of Rock Mechanics and Mining Science, Vol. 12, No. 10, pp. 303-310.

D'Appolonia, E., D'Appolonia, D.J. and Ellison, R.D. 1975. Chapter 20, Drilled Piers, in Foundation Engineering Handbook, Ed. by H.F. Winterkorn and H.Y. Fang, Van Nostrand Reinhold Co., New York.

Davis, E.H. 1968. Theories of Plasticity and Failure of Soil Masses. Chapter 6, Soil Mechanics - Selected Topics. Ed. by I.K. Lee, Butterworths, London.

Davis, E.H. 1980. A note on some plasticity solutions relevant to the bearing capacity of brittle and fissured materials. Proceeding of the International Conference on Structural Foundations on Rock, Sydney, Vol. 2, pp. 83-90.

Deere, D.V. 1968. Geological Considerations, Chapter 1, Rock Mechanics in Engineering Practice. Ed. by K.G. Stagg and O.C. Zienkiewicz, John Wiley and Sons, New York.

Dight, P.M. and Chiu, H.K. 1981. Prediction of shear behaviour of joints using profiles. International Journal of Rock Mechanics and Mining Science, Vol. 18, No. 3, pp. 369-386.

Donald, I.B., Sloan, S.W. and Chiu, H.K. 1980. Theoretical analyses of rock socketed piles. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 1, pp. 303-316.

Douglas, D.J. 1980. Modulus values for weathered rock from plate load tests. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 1, pp. 17-22.

Freeman, C.F., Klajnerman, D. and Prasad, G.D. 1972. Design of deep socketed caissons into shale bedrock. Canadian Geotechnical Journal, Vol. 9, No. 1, pp. 105-114.

Fox, E.N. 1948. The mean elastic settlement of a uniformly loaded area at a depth below the ground surface. Proceedings of the 2nd International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, Vol. 1, pp. 129-132.

Geological Society Working Group. 1970. Working Party report on the logging of rock cores for engineering purposes. Quarterly Journal of Engineering Geology, Vol. 3, No. 1, pp. 1-24.

Gibson, G.L. 1973. Field and laboratory investigation for design of rock socketed caissons on a multi-storey building founded in sound limestone. M.Eng. Thesis, Carleton University, Ottawa, Ontario, Canada.

Gibson, G.L. and Devenny, D.W. 1973. Concrete to bedrock bond testing by jacking from bottom of a bore hole. Canadian Geotechnical Journal, Vol. 10, No. 2, pp. 304-306.

Glos III, G.H. and Briggs Jr., O.H. 1983. Rock sockets in soft rock. Journal of Geotechnical Engineering, ASCE, Vol. 109, No. 4, pp. 525-535.

Glos III, G.H. and Briggs Jr., O.H. 1984. Rock sockets in soft rock: Discussion. Journal of Geotechnical Engineering, ASCE, Vol. 110, No. 10, pp. 1528-1529.

Gupton, C.P., O'Brien, J.F. and Logan, T.J. 1982. Design of drilled shafts in South Florida limestone. Proceedings of the Conference on Engineering and Construction in Tropical Residual Soils (ASCE), Honolulu, pp. 403-422.

Hammersley, J.M. and Handscomb, D.C. 1964. Monte Carlo Methods. John Wiley and Sons, New York.

Hartley, W.F. 1981. The capacity of piers drilled into rock: Discussion. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 2, p. 55.

Hobbs, N.B. 1975. Factors affecting the prediction of settlement of structures on rock: With particular reference to the Chalk and Trias. Settlement of Structures, Pentech Press, London, pp. 579-610.

Horvath, R.G. 1978. Field load test data on concrete-to-rock bond strength for drilled pier foundations. University of Toronto, Department of Civil Engineering Publication 78-07 (ISSN 0316-7968).

Horvath, R.G. and Kenney, T.C. 1979. Shaft resistance of rock-socketed drilled piers. ASCE Annual Convention, Atlanta, Georgia, Preprint No. 3698.

Horvath, R.G., Trow, W.A. and Kenney, T.C. 1980. Results of tests to determine shaft resistance of rock-socketed drilled piers. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 1, pp. 349-361.

Horvath, R.G. 1980. Research project report on load transfer system for rock-socketed drilled pier foundations. For the National Research Council of Canada, DSS File No. 10X5.31155-9-4420, Contract Serial No. 1SX79.000531.

Horvath, R.G. 1982. Behaviour of rock-socketed drilled pier foundations. Ph.D. Thesis, University of Toronto, Toronto, Ontario, Canada.

Horvath, R.G. and Kenney, T.C. 1982. Caissons socketed in sound mica schist: Discussion. Journal of the Geotechnical Engineering Division, ASCE, Vol. 108, No. GT7, pp. 991-993.

Horvath, R.G., Kenney, T.C. and Kozicki, P. 1983. Methods of improving the performance of drilled piers in weak rock. Canadian Geotechnical Journal, Vol. 20, No. 4, pp. 758-772.

Horvath, R.G. 1984. Rock sockets in soft rock: Discussion. Journal of Geotechnical Engineering, ASCE, Vol. 110, No. 10, pp. 1525-1528.

Jackson, W.T., Perez, J.Y. and Lacroix, Y. 1974. Foundation, construction and performance for a 34-storey building in St. Louis. Geotechnique, Vol. 24, No. 1, pp. 63-90.

Jaeger, L.G. and Bakht, B. 1983. Number of tests versus design pile capacity. Journal of Geotechnical Engineering, ASCE, Vol. 109, No. 6, pp. 821-831.

Janbu, N., Bjerrum, L. and Kjaernsli, B. 1956. Veiledning ved losning av fundamentering soppgaver. Norwegian Geotechnical Institute Publication 16, Oslo, pp. 30-32.

Johnston, I.W., Donald, I.B., Bennet, A.G. and Edwards, J.W. 1980. The testing of large diameter pile rock sockets with a retrievable test rig. Proceedings of the 3rd Australia-New Zealand Conference on Geomechanics, Wellington, Vol. 1, pp. 105-108.

Jorden, E.E. and Dobie, M. 1976. Tests on piles in Keuper Marl for foundations of a blast furnace at Redcar. Geotechnique, Vol. 26, No. 1, pp. 105-113.

Kaderabek, T.J. 1982. Tall silos on soft limestone. Project Report, K.B.C. Consultants, Coconut Grove, Florida.

Kay, J.N. 1976. Safety factor evaluation for single piles in sand. Journal of the Geotechnical Engineering Division, ASCE, Vol. 102, No. GT10, pp. 1093-1108.

Kenney, T.C., Lee, L.C. and Gagg, C. 1975. Laboratory investigation of preloading as a method of improving the load-displacement performance of socketed piles. University of Toronto, Department of Civil Engineering, Publication 75-09 (ISSN 0316-7968).

Kenney, T.C. 1977. Factors to be considered in the design of piers socketed into rock. University of Toronto, Department of Civil Engineering, Publication 77-06 (ISSN 0316-7968).

Koutsoftas, D.C. 1981. Caissons socketed in sound mica schist. Journal of the Geotechnical Engineering Division, ASCE, Vol. 107, No. GT6, pp. 743-757.

Kulhawy, F.H. 1978. Geomechanical model for rock foundation settlement. Journal of the Geotechnical Engineering Division, ASCE, Vol. 104, No. GT2, pp. 211-227.

Ladanyi, B. and Roy, A. 1971. Some aspects of bearing capacity of rock mass. Proceedings of the 7th Canadian Rock Mechanics Symposium, Edmonton, pp. 161-190.

Ladanyi, B. 1977. Friction and endbearing tests on bedrock for high capacity socket design: Discussion. Canadian Geotechnical Journal, Vol. 14, No. 1, pp. 153-155.

Ladanyi, B. and Domingue, D. 1980. An analysis of bond strength for rock socketed piers. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 1, pp. 363-373.

Lee, I.K., White, W. and Ingles, O.G. 1983. Soil Variability, Chapter 2, Geotechnical Engineering, Pitman Publishing Co., Boston.

Lumb, P. 1966. The variability of natural soils. Canadian Geotechnical Journal, Vol. 3, No. 2, pp. 74-97.

Lumb, P. 1970. Safety factors and the probability distribution of soil strength. Canadian Geotechnical Journal, Vol. 7, No. 3, pp. 225-242.

Lumb, P. 1971. Precision and accuracy of soil tests. Proceedings of the 1st International Conference on Application of Statistics and Probability to Soil and Structural Engineering, Hong Kong, pp. 330-345.

Mackenzie, I.M. 1969. Foundation load tests on Sydney sandstone. Rock Mechanics Symposium, University of Sydney, pp. 132-134.

Madhav, M.R. and Krishna, K.S.R. 1980. A probabilistic analysis of foundations on rock. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 1, pp. 173-180.

Mallard, D.J. and Ballantyne, J.L. 1976. The behaviour of piles in Upper Chalk at Littlebrook D Power Station. Geotechnique, Vol. 26, No. 1, pp. 115-132.

Matich, M.A.J. and Kozicki, P. 1967. Some load tests on drilled cast-in-place concrete caissons. Canadian Geotechnical Journal, Vol. 4, No. 4, pp. 367-375.

Mattes, N.S. and Poulos, H.G. 1969. Settlement of a single compressible pile. Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 95, No. SM1, pp. 189-207.

McMahon, M.D. and McMahon, B.K. 1980. Foundation investigations and monitoring of the MLC Centre Tower, Sydney. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 1, pp. 153-160.

Meyerhof, G.G. 1953. The bearing capacity of concrete and rock. Magazine of Concrete Research, April, pp. 107-116.

Meyerhof, G.G. 1970. Safety factors in soil mechanics. Canadian Geotechnical Journal, Vol. 7, No. 4, pp. 349-355.

Meyerhof, G.G. 1982. Limit states design in geotechnical engineering. Structural Safety, Vol. 1, No. 1, pp. 67-71.

Meyerhof, G.G. 1984. Safety factors and limit states analysis in geotechnical engineering. Canadian Geotechnical Journal, Vol. 21, No. 1, pp. 1-7.

Moore, W.W. 1964. Foundation design. Civil Engineering, ASCE, Vol. 34, No. 1, pp. 33-35.

National Building Code of Canada. 1980. National Research Council of Canada, Ottawa.

Nemec, J.M. 1979. Skin resistance tests of model piles in hard rock. Proceedings of the 4th International Congress of the International Society of Rock Mechanics, Montreux, Vol. 2, pp. 475-478.

Olson, R.E. and Dennis, N.D. 1983. Reliability of pile foundations. ASCE Annual Convention, Houston, Texas, Preprint No. SC-3.

Ontario Highway Bridge Design Code. 1983. Ministry of Transportation and Communication, Toronto.

Osterberg, J.O. and Gill, S.A. 1973. Load transfer mechanism for piers socketed in hard soils or rock. Proceedings of the 9th Canadian Rock Mechanics Symposium, Montreal, pp. 235-262.

Palmer, D.J. 1977. Discussion: Session 1 - Chalk. Piles in Weak Rock, Institution of Civil Engineers, London, pp. 189-192.

Palmer, J.H.L. and Lo, K.Y. 1976. In-situ stress measurements in some near-surface rock formations - Thorold, Ontario. Canadian Geotechnical Journal, Vol. 13, No. 1, pp. 1-7.

Pariseau, W.G. 1973. Influence of rock properties variability on mine opening stability analysis. Proceedings of the 9th Canadian Rock Mechanics Symposium, Montreal, pp. 141-165.

Peck, R.B., Hanson, W.E. and Thornburn, T.H. 1974. Foundation Engineering, 2nd edition, John Wiley and Sons, New York.

Peck, R.B. 1976. Rock foundations for structures. Rock Engineering for Foundations and Slopes, Proceedings of a Specialty Conference sponsored by ASCE, Vol. II, Boulder, Colorado, pp. 1-21.

Pells, P.J.N., Douglas, D.J., Rodway, B., Thorne, C. and McMahon, B.K. 1978. Design loadings for foundations on shale and sandstone in the Sydney Region. University of Sydney, Research Report No. R315.

Pells, P.J.N. and Turner, R.M. 1979. Elastic solutions for the design and analysis of rock-socketed piles. Canadian Geotechnical Journal, Vol. 16, No. 3, pp. 481-487.

Pells, P.J.N. and Turner, R.M. 1980. Endbearing on rock with particular reference to sandstone. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 1, pp. 181-190.

Pells, P.J.N., Rowe, R.K. and Turner, R.M. 1980. An experimental investigation into sideshear for socketed piles in sandstone. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 1, pp. 291-302.

Pells, P.J.N. and Rowe, R.K. 1981. A design method for rock socketed piles. Symposium on Socketed Foundations on Weak Rock, Institution of Engineers, Australia (Australian Geomechanics Society), pp. 1-12.

Pells, P.J.N. 1983. Plate loading tests on soil and rock. Proceedings of an Extension Course on In-Situ Testing for Geotechnical Investigations, Sydney, pp. 73-86.

Poulos, H.G. and Davis, E.H. 1980. Pile Foundation Analysis and Design. John Wiley and Sons, New York.

Rehnman, S.E. and Broms, B.B. 1971. Bearing capacity of piles driven into rock. Canadian Geotechnical Journal, Vol. 8, No. 2, pp. 151-162.

Rosenberg, P. and Journeaux, N.L. 1976. Friction and endbearing tests on bedrock for high capacity socket design. Canadian Geotechnical Journal, Vol. 13, No. 3, pp. 324-333.

Rowe, R.K., Booker, J.R. and Balaam, N.P. 1978. Application of the initial stress method to soil-structure interaction. International Journal of Numerical Methods in Engineering, Vol. 12, No. 5, pp. 873-880.

Rowe, R.K. and Pells, P.J.N. 1980. A theoretical study of pile-rock socket behaviour. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 1, pp. 253-264.

Sales, A. 1983. Channel Island access bridge - test pile. Technical Report, Roads Division, Materials and Research Section, Department of Transport and Works, Northern Territory, Australia.

Seychuk, J.L. 1970. Load tests on bedrock. Canadian Geotechnical Journal, Vol. 7, No. 4, pp. 464-470.

Singh, A. 1971. How reliable is the factor of safety in foundation engineering. Proceedings of the 1st International Conference on Applications of Statistics and Probability to Soil and Structural Engineering, Hong Kong, pp. 390-424.

Sobol', I.M. 1974. The Monte Carlo Method. The University of Chicago Press, Chicago.

Spanovich, M. and Garvin, R.G. 1979. Field evaluation of caisson-shale interaction. Behaviour of Deep Foundations, ASTM STP 670, pp. 537-557.

Spencer, C.B. 1944. Deep caissons support river-side power plant. Civil Engineering, ASCE, Vol. 14, No. 2, pp. 67-68.

Stocker, P.T. 1981. Application of modulus based design for piles in jointed rock. Symposium on Socketed Foundations on Weak Rock, Institution of Engineers, Australia (Australian Geomechanics Society), pp. 13-33.

Stroud, M.A. 1977. Discussion: Session 2 - rocks other than chalk in Piles in Weak Rock, Institution of Civil Engineers, London, pp. 211-215.

Taylor, P.W. 1975. Preloaded pier foundations for a city building. New Zealand Engineering, Vol. 30, No. 11, pp. 320-325.

Terzaghi, K. 1943. Theoretical Soil Mechanics. John Wiley and Sons, New York.

Thorburn, S. 1966. Large diameter piles founded on bedrock. Proceedings of the Symposium on Large Bored Piles, Institution of Civil Engineers, London, pp. 121-129.

Thorne, C.P. 1980. The capacity of piers drilled into rock. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 1, pp. 223-233.

Tomlinson, M.J. 1971. Some effects of pile driving on skin friction. Proceedings of the Conference on the Behaviour of Piles, The Institute of Civil Engineers, London, pp. 107-114.

Tomlinson, M.J. 1977. Discussion: Session 3 - testing, instrumentation and construction in Piles in Weak Rock, Institution of Civil Engineers, London, pp. 232-233.

Vesic, A.S. 1969. Foundations of buildings in clay: Discussion. Main Session 2. Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico, Vol. 3, pp. 242-244.

Vogan, R.W. 1977. Friction and endbearing tests on bedrock for high capacity socket design: Discussion. Canadian Geotechnical Journal, Vol. 14, No. 1, pp. 156-158.

Wagner, H. and Schumann, E.H.R. 1971. The stamp-load bearing strength of rock, an experimental and theoretical investigation. Rock Mechanics, Vol. 3, No. 4, pp. 185-207.

Webb, D.L. 1976. The behaviour of bored piles in weathered diabase. Geotechnique, Vol. 26, No. 1, pp. 63-72.

Webb, D.L. and Davies, P. 1980. Ultimate tensile loads of bored piles socketed into sandstone rock. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 1, pp. 265-270.

Weir-Jones, I. and Lauga, H. 1973. The use of 'in-situ' moduli determination as an aid to underground opening design and their relationship to laboratory determined values. Proceedings of the 9th Canadian Rock Mechanics Symposium, Montreal, pp. 1-20.

Williams, A.F. 1980. The design and performance of piles socketed into weak rock. Ph.D. Thesis, Monash University, Melbourne, Australia.

Williams, A.F., Donald, I.B. and Chiu, H.K. 1980. Stress distributions in rock socketed piles. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 1, pp. 317-325.

Williams, A.F., Johnston, I.W. and Donald, I.B. 1980. The design of sockets in weak rock. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, Vol. 1, pp. 327-347.

Williams, A.F. and Pells, P.J.N. 1981. Side resistance rock sockets in sandstone, mudstone and shale. Canadian Geotechnical Journal, Vol. 18, No. 4, pp. 502-513.

Wilson, L.C. 1976. Tests of bored and driven piles in Cretaceous mudstone at Port Elizabeth, South Africa. Geotechnique, Vol. 26, No. 1, pp. 5-12.

Zienkiewicz, O.C. 1977. The Finite Element Method, 3rd Edition. McGraw Hill, New York.

APPENDIX A

CURRENT PRACTICE FOR THE DESIGN OF SOCKETED PILES:

RESPONSE TO A SURVEY

INTRODUCTION

In January 1983, a survey was sent to 92 consultants and Government agencies who were considered to be likely to have had some involvement with the design of piles socketed into rock. This survey forms part of an ongoing programme of research into the behaviour of socketed piles being conducted at The University of Western Ontario. It was intended that this survey complement an earlier study by Horvath (1978). In particular, it was hoped that this survey would provide data relating to the elements which directly influence current design strategies together with an indication of the impact of recent research (eg. Rosenberg & Journeaux, 1976; Ladanyi, 1977; Horvath and Kenney, 1979; Pells and Turner, 1979; Rowe and Pells, 1980; Williams et al., 1980) upon design practice in Canada.

As of May 30, 1984, a total of 34 (37%) surveys had been returned (see Table A1) of which 17 provided a detailed response suitable for inclusion in this report. Some of the responses represent the experiences from several regional offices of the same consulting firm. While the number of completed replies accounts for only 18 percent of the total number distributed, it is felt that those received represent a reasonably broad cross-section of current Canadian experience with socketed piles. From the response, it appears that the majority of socketed pile applications are in Central and Western Canada. Both replies from the Maritime Provinces indicated that the use of socketed piles is quite limited in that region.

TABLE A1 DISTRIBUTION AND RESPONSE TO A SURVEY RELATING TO THE DESIGN OF ROCK SOCKETED PILES

Province or Territory	Surveys Distributed	Surveys Returned	
		Detailed Responses	Other Replies
British Columbia	12	2	3
Alberta	25	5	8
Saskatchewan	10	1	-
Manitoba	8	2	2
Ontario	14	2	2
Quebec	12	3	-
Newfoundland	1	1	-
Nova Scotia	8	1	1
New Brunswick	1	-	-
Yukon Territories	1	-	1
Totals	92	17	17

TABLE A2 FOUNDING ROCK TYPES FOR SOCKETED PILE CONSTRUCTION

Rock Type	Number of Respondents Indicating Experience in This Type of Rock
Shale (mudstone)	11
Limestone	8
Sandstone	7
Schist	3
Gneiss	3
Siltstone	3
Slate	2
Diorite	1
Granite	1

QUESTIONNAIRE

The questionnaire consisted of 19 questions relating to the founding rock type, criteria used in design, socket construction considerations and the extent of the geotechnical investigation prior to design. Generally, each question required only a "yes" or "no" reply; however, in some cases, the respondents were asked to check the appropriate answer using the letters "T" or "P" depending on whether the response applied to a typical site (T) or to a site where particular emphasis (P) was placed on the pile/rock performance. Specific reference will be made to cases where the majority of consultants responded to questions in that manner. Several questions in the survey required a slightly more detailed response.

It should be emphasized that this questionnaire is only concerned with the situation in which the majority of the pile capacity is developed by side friction and/or endbearing in the rock socket.

ROCK TYPES FOR SOCKETED PILE FOUNDATIONS

Table A2 shows that the predominant founding rocks for socketed piles are shales, limestones and sandstones, although some experience has been gained with a wide range of rock types.

DESIGN OF PILES SOCKETED INTO ROCK

Design Criteria

General design procedures for rock socketed piles do not appear to be well established. The Canadian Foundation Manual (1978) provides a simplified design approach as does the National Building Code of Canada (1980). However, from

TABLE A3 BASIS OF DESIGN CRITERIA FOR SOCKETED PILES

Criterion for Design	Number of Respondents: Total Responses	Typical Site (T)	Important Site (P)	Both (T&P)	Not Specified
(a) Canadian Foundation Manual (1978) only	0	-	-	-	-
(b) National Building Code of Canada (1980) only	0	-	-	-	-
(c) Previous Experience only	12	2	5	2	3
(d) (a) and (c)	5	2	-	1	2

TABLE A4 USE OF PREVIOUS PERFORMANCE RECORDS AS A GUIDE FOR DESIGN

	Responses: Yes	No	Not Specified
Are performance records or case histories used as a guide for design?	13	4	-
Is this information from:			
(a) instrumented piles, for short-term load-settlement behaviour?	11	2	4
(b) instrumented piles, for long-term load-settlement/transfer behaviour?	3	3	11

Table A3, it would appear that these procedures are not sufficient and that most respondents chose to rely on previous personal or company experience to guide them in subsequent socket designs.

Pile load tests and reference to geotechnical reports and the published literature were listed as major factors influencing design. Table A4 shows that the majority of respondents used previous performance records as a guide to subsequent socket designs. However, it was also apparent that in many cases these tests were proof tests and the parameters backfigured from the test may represent a lower bound.

While it is recognized that short-term load tests conducted on rock sockets yield a great deal of information for subsequent pile designs, it is worth noting the use by some consultants of long-term test results as well. Some literature dealing with the long-term load-transfer behaviour of rock sockets is available (eg. Ladanyi, 1977; Bauer, 1980). However, the practical implementation into the design process and the benefits to be derived from considering this behaviour, do not appear to be firmly established.

The specific criteria governing the design of socketed piles are given in Table A5. As might be expected, construction constraints were a major factor to be considered in design. Among the construction constraints given, the most frequently mentioned were problems associated with inflow of water into the socket during construction, availability of equipment and materials and the condition of the rock upon excavation. Others listed the ability to clean and inspect the socket and constraints on preferable length/diameter ratios, as problems encountered.

Although it appears that design criteria based on endbearing capacity is

TABLE A5 FACTORS GOVERNING SOCKETED PILE DESIGN

Criterion	Number of Responses
(a) Bearing Capacity	3
(b) Settlement	3
(c) Construction Constraints	1
(d) (a) & (c)	6
(e) (b) & (c)	2
(f) no response	2

TABLE A6 METHOD UTILIZED TO CARRY THE APPLIED SOCKET LOAD

Applied Load Carried By:	Total Responses	Typical Site (T)	Important Site (P)	Both (T&P)	Not Specified
(a) Endbearing Only	1	1	-	-	-
(b) Sideshear (Bond) Only	3	1	-	1	1
(c) Combination of End-bearing and Sideshear	7	-	1	2	4
(d) (a) or (b)	1	-	-	-	1
(e) (b) or (c)	3	1	-	2	-
(f) (a),(b) and (c)	1	-	-	1	-
(g) no response	1	-	-	-	-

Relative contribution provided by endbearing and sideshear as assigned by designers who selected response (c), (e) or (f)

(percentage) endbearing/sideshear	
75/25	1
50/50	2
25/75	1
Variable	7

most common among consultants, there may be a trend developing toward using settlement as the controlling factor in the design of socketed piles. One respondent with considerable experience in socketed pile design, stated that he had never used bearing capacity as a governing factor in design and further commented that he had never heard of a case where a bearing failure of a caisson in rock had occurred.

Three of the four respondents who indicated that they had used a settlement criterion as the basis for their design, had also used published settlement charts of either Poulos and Davis (1974) or Pells and Turner (1979) or both, as an aid in their designs. In addition, two respondents stated that their socketed piles had been designed to meet a specific settlement criterion (ie. <6.4 mm and 15 mm respectively).

Of particular interest in socketed pile design is the mechanism by which the applied load is carried. Table A6 summarizes the replies concerning which type of load carrying mechanism they have assumed in their designs. The majority of respondents use a combination of endbearing and sideshear to carry the applied load. The contribution of each component varied considerably depending on factors such as socket length, probable water conditions and the soundness of the rock. The factors influencing the choice of a particular supporting mechanism are given in Table A7. The category "others" included factors such as the economy of large endbearing caissons versus slender sideshear piles and the potential for inspection/cleaning of the socket.

DESIGN PARAMETERS

(a) Allowable Endbearing Values

Selection of allowable endbearing pressures used in design are generally the

TABLE A7 FACTORS AFFECTING THE CHOICE OF SUPPORT MECHANISMS

Factor Influencing Selection		Total Responses
(a) Past Experience		17
(b) Degree of Weathering or Shattering of Rock		11
(c) Visual Inspection of:	(i) Excavated Sockets	10
	(ii) Boreholes	9
(d) Rock Quality Designation (RQD-%)		7
(e) Condition of Socket Sidewalls		6
(f) Unconfined Compression Tests		6
(g) Others		5

TABLE A8 RANGE OF ALLOWABLE ENDBEARING PRESSURES USED IN DESIGN

Rock Type	Range of Values		Average of Respondents Values		Number of Responses
	(MPa)	(ksf)	(MPa)	(ksf)	
Shale (mudstone)	.48-9.57	10-200	2.84	59	11
Limestone	2.87-9.57	60-200	4.99	104	7
Sandstone	.96-9.57	20-200	3.14	66	7
Schists	2.39-9.57	50-200	4.19	88	2
Gneiss	7.18-9.57	150-200	8.38	175	2
Slate	2.87-3.83	60-80	3.35	70	2
Siltstone	1.0-1.53	21-32	1.27	27	2
Granite	9.57	200	9.57	200	1
Factor of Safety	2-6				

result of the individual's previous experience. Individual experience in selecting design values often required consideration of information obtained from load tests on sockets, locally governing (city) bylaws, allowable settlements obtained at working loads and the compatibility of endbearing pressure with the structural capacity of the socket.

The values of allowable endbearing pressures used in design, are undoubtedly influenced by the condition of the rock at the base of the socket. A range and average of these values for the specific rock types previously mentioned are presented in Table A8. The factor of safety perceived to be incorporated into these values was highly variable, ranging from 2 to 6.

It is noted that there is considerable scope for debate regarding the "true" bearing capacity of piles socketed into rock and the variation in the "Factor of Safety" can be partially attributed to this uncertainty. The authors would suggest that the use of these allowable endbearing pressures really represents an indirect limited settlement criterion rather than a stability criterion.

(b) Allowable Sideshear (Bond) Values

Allowable sideshear values were generally selected on the basis of previous experience based on results backfigured from pile load tests. Nevertheless, 40% of respondents did indicate that they had made use of published empirical correlations (eg. Rosenberg and Journeaux, 1976; Horvath and Kenney, 1979; Williams and Pells, 1981) to estimate the available sideshear from a knowledge of the unconfined compression strength of the rock.

The reported range and average values of allowable sideshear are given in Table A9. The perceived "factor of safety" associated with these values ranged

TABLE A9 RANGE OF ALLOWABLE SIDESHEAR (BOND) VALUES USED IN DESIGN

Rock Type	Range of Values MPa	Range of Values psi	Average MPa	Average psi	Number of Responses
Shale (mudstone)	.10 -1.03	14-150	.42	60	12
Limestone	.35 -1.03	50-150	.92	133	8
Sandstone	.14 -1.03	20-150	.60	87	7
Schist	.14 - .69	20-100	.47	67	2
Gneiss	.52 -1.38	75-200	.87	125	2
Slate	.35 -1.38	50-200	.95	138	2
Siltstone	.35 - .48	50-70	.41	60	1
Granite	1.03 -1.38	150-200	1.21	175	1
Diorite	1/30 of concrete's 28 day compressive strength but <1.38 MPa (200 psi)				1
Factor of Safety	2 - 4				

TABLE A10 ROCK TYPES AND RANGE OF STRENGTH PROPERTIES REPORTED (after Horvath and Kenney, 1979)

Rock Type	No. of Tests	Unconfined Compressive Strength MPa	Unconfined Compressive Strength psi	Mobilized Shaft Resistance MPa	Mobilized Shaft Resistance psi
Shale or Mudstone	50	.35-110	50-16,000	.12-3+	17-440+
Limestone or Chalk	17	1-7+	150-1,000+	.12-2.8+	17-418+
Sandstone	8	7-24+	1000-3,500+	.17-6.5	25-940
Igneous	4	.35-10.5+	50-1,500+	.12-6.3	18-920
Metamorphic	8			.47-1.9	68-273

from 2 to 4. For comparison purposes, Table A10 gives the values of mobilized shaft resistance obtained from load test data which were accumulated by Horvath (1978) and Horvath and Kenney (1979). These data were collected primarily from sources in North America, England and South Africa.

Since a number of respondents had designed socketed piles in the same rock formation and/or at similar geographic locations, a comparison of their selected design values is provided in Table A11. (It should be noted that only shales, limestones, sandstones and slate have been included in this table, since insufficient data were obtained to allow a meaningful comparison for other rock types.)

(c) Factors Affecting Choice of Design Parameters

Respondents were asked what situations were implicitly accounted for in their choice of allowable endbearing and sideshear values. Where a specific situation was not incorporated in these values, they were asked to provide information regarding the adjustments that would have to be made to the original values. Table A12 summarizes these responses.

SOCKETED PILE DIMENSIONS AND APPLIED LOADS

Although the dimensions used for socketed pile design vary depending on the nature of both the founding rock and the project requirements, there appears to be a consistent range of values as shown in Table A13. Results from Horvath's (1978) survey are included for comparison.

The applied load used in socketed pile design depends upon the nature of the structure and the column loads the socket is required to support. Typical ranges

TABLE A11 DESIGN PARAMETERS AS SPECIFIED BY RESPONDING CONSULTANTS

Rock Type	Geological Formation	Geographic Location	Responses (allow. endbearing (MPa)/allow. sideshear (MPa)) 1	2	3	Formation or Location Average
SHALE/(mudstone)						
	UTICA	Montreal	2.39-4.79/.35	3.83/.35-.69		4.47/.43
	not specified	Montreal	2.39-9.57/.14-.69			
	BILLINGS & EASTVIEW	Ottawa	N/A /.52-1.03			5.98/.60
	not specified	Ottawa	2.39-9.57/.14-.69			
	DUNDAS	Toronto	7.18/1.03			7.18/1.03
	QUEENSTON	Burlington	7.18/1.03			
	PASKAPOO	Calgary	<1.91/<.24	1-2/.1-.2	1.44/.172-.345	1.67/.18
			2-4/.1-.22			
	not specified	Calgary	.48/.10			
	KITSILANO	Vancouver	~1.53/.35-.48			1.53/.41
LIMESTONE						
	TRENTON	Montreal	7.18/.35-.52	4.79/1.03		6.38/.78
	not specified	Montreal	4.79-9.57/.69-1.03			
	OTTAWA	Ottawa	N/A /.69-1.03			7.18/.86
	not specified	Ottawa	4.78-9.57/.69-1.03			
	RED RIVER	Winnipeg	2.87/1.03			3.95/1.03
	not specified	Winnipeg	2.87/1.03	7.18/1.03	2.87/1.03	

TABLE A11 (cont'd)

Rock Type	Geological Formation	Geographic Location	Responses (allow. endbearing (MPa)/allow. sideshear (MPa)) 1	2	3	Formation or Location Average
SANDSTONE						
	not specified	P.E.I.	2.39-9.57/.14-1.03			5.98/.59
	POTSDAM	Nova Scotia	3.83/.52-.86			3.83/.69
	POTSDAM	Beauharnois Quebec	3.83/.52-.86			4.91/.64
	not specified	Gatineau PQ	2.39-9.57/.14-1.03			
	PASKAPOO	Calgary	<3.83/<1.03	2-4/.2-.3	2.87/1.03	2.66/.63
	not specified	Calgary	.96/.19			
	BURRARD	Vancouver	~1.53/.35-.48			~1.53/.41
SLATE						
	HALIFAX	Halifax	3.83/.35-.69			3.35/.95
	not specified	Halifax	2.87/1.38			

TABLE A12 SITUATIONS IMPLICITLY ACCOUNTED FOR IN THE SELECTED ALLOWABLE VALUES

Situation	Is This Situation Accounted for in Allowable Design Value (Tables 8 & 9) Yes	No	Adjustment and/or Comment
a) Depth of embedment	11	-	.a socket length of 1.5 times the diameter is generally required as a minimum
b) Differences in the modulus of rock (E_m) and pile concrete (E_p)	10	-	.judgement on E_m
c) Difference between the rocks intact (E_i) and mass (E_m) modulus	6	-	.difficult to predict unless a downhole geophysical study is carried out
d) Roughness of socket sidewalls	10	-	
e) Occurrence of seams, joints or bedding planes in zone of socket influence	11	1	.inspected - if significant extend the socket
f) Weathered or fractured rock in zone of socket influence	7	5	.reduce allowable stress .provide a deeper socket .top 2-3 ft often neglected .discount length of socket in the fractured rock if localized, or if necessary reduce allowable sideshear value
g) Possibility of incomplete bond at interface of pile and rock	7	3	.ensure proper cleaning by inspection
h) Possibility of debris at the base of the socket	7	5	.base inspected, cleaned prior to concreting
i) Others			.if unable to inspect/clean discount contribution of endbearing and use allowable bond strength of 90 psi (for Limestone) .for small jobs, a conservative bearing capacity is given which takes into account the overall condition of the rock .ability to dewater in order to inspect, if not possible, reduce the allowable sideshear (and thereby lengthen the socket) .for a),d),f) - judgement on % endbearing versus sideshear

TABLE A13 RANGE OF TYPICAL DIMENSIONS USED IN DESIGN OF COMPRESSION SOCKETS (both endbearing and sideshear only)

Dimension	Sideshear Only Sockets	Full (Endbearing & Sideshear) Sockets This Survey	Full (Endbearing & Sideshear) Sockets Horvath (1978)
Length	.91-6.1 m	1-12 m	0.56-12.2 m
Diameter	305-460 mm	610-2440 mm	229-1050 mm
L/D	.2-13.3	1-8	.8-20

TABLE A14 RANGE OF TYPICAL APPLIED SOCKET LOADS

Socket Type	Applied Socket Load (MN)
Sideshear Only	.45 - 6.67
Full (endbearing and sideshear)	1.15 - 22

TABLE A15 METHODS OF CLASSIFICATION USED FOR RECOVERED ROCK CORES

Classification	Responses to Each Method				
	Total Responses	Typical Site (T)	Important Site (P)	Both (T&P)	Not Specified
a) Degree of Weathering or Shattering of Rock	15	4	4	1	6
b) Rock Quality Designation (RQD - %)	13	3	5	1	5
c) A Published Geomechanics Classification (eg. Barton et al. (1974) or Bieniawski (1976)	2	-	2	-	-
d) Geological Society Eng. Group Working Party Report (1970)	2	1	-	-	1
e) Other Methods	8	. scale of hardness . unconfined compressive strength . geological core analysis and interpretation . % core recovery . Deere's classification . company classification			

of values for sideshear only and endbearing plus sideshear piles are given in Table A14. (This table does not include loads of 35 MN and 44.5 MN reported in two particular cases because these values were much higher than the range of values reported in all other designs.)

GEOTECHNICAL INVESTIGATION FOR SOCKETED PILE DESIGN

Drilling and analyses of rock cores recovered from field boreholes represent the most common form of preliminary geotechnical investigation. Fifteen of the responses to this series of questions indicated they had obtained and classified rock cores, however, several stated that this classification was carried out only in cases of more important projects. Table A15 summarizes the responses to the methods usually adopted for classifying these rock cores.

Other forms of geotechnical investigation received limited application. Table A16 illustrates the variability of test methods used in both the field and laboratory, as well as their perceived significance in determining the engineering properties of a particular founding rock formation. From Table A16 it may be inferred that there exists no definite trend towards a systematic series of field or laboratory tests which has been adopted by the Canadian consulting community.

Of the laboratory tests mentioned, the unconfined compression test seems to be the most popular for determining the strength properties of the intact rock. These values may also be used as a guide to drilling rate and to estimate interface shear properties using empirical correlations, as previously discussed.

Most respondents indicated that an inspection of a borehole or the rock socket prior to concreting constitutes a basic part of their geotechnical investigation. This visual inspection was carried out for a variety of reasons

TABLE A16 SUMMARY OF FIELD AND LABORATORY TESTS CONDUCTED ON THE ROCK

	Affirmative Responses	Remarks
FIELD		
Tests conducted to obtain the elastic properties of the rock mass:		
a) Borehole pressuremeter/dilatometer test	5	.in very soft rock
b) Goodman jack test	2	
c) Plate loading test	2	.+18" dia. plate at depth . 2 ft^2 plate at depth
d) Others: (i) pile load tests	2	.conducted prior to contract pile const
(ii) downhole geophysical	1	.on important jobs
(iii) load test on a concrete plug poured on a styrofoam pad	1	.to determine rock-concrete bond
(iv) Lugean type pressure packer test	1	.to determine secondary permeability
(v) isolated pullout tests	1	
LABORATORY		
Tests conducted to determine properties of the rock:		
a) Unconfined compression test	12	
b) Brazilian tensile test	2	
c) Point load test	2	.parallel and perpendicular to bedding planes
d) Triaxial compression test	2	.limited
e) Direct shear tests	1	.on composite rock/concrete samples
f) Absorption tests	1	.to indicate softening potential
g) Atterberg index tests	2	.for shales and mudstones
h) Hardness index	1	
i) Moisture content	1	
j) Slaking	1	

given in Table A17.

SOCKETED PILE CONSTRUCTION

During the construction phase of a socketed pile project, field dimensions of the socket may often require modification. These modifications are usually required as a result of conditions revealed by the in-situ inspection or pre-construction pile (socket) load test results. Thirteen of the sixteen respondents indicated that preliminary design had, at some time, required modification. Table A18 lists the most common reasons for these changes.

In the case of full sockets (endbearing and sideshear), it is generally accepted that adequate cleaning at the base of the excavation prior to concreting is necessary in order to fully develop the allowable endbearing pressure. The value of cleaning and/or roughening the socket sidewalls met with varying viewpoints among the respondents. Only 6 of the 16 responses indicate that manual cleaning of socket sidewalls is conducted, and of these none had carried out this operation in an attempt to achieve a desired socket roughness. While some consultants presented reasons as to why this procedure was not adopted, recent investigations (Pells, Rowe and Turner, 1980; Horvath, 1982) indicate that consideration of this aspect of socket construction may be justified under certain circumstances.

Equipment used to construct socketed piles in rock is highly variable and may be dependent on a number of factors. One such consideration in terms of excavation is the relative strength of the founding rock formations. Lower strength rocks such as some shales may be conveniently excavated by augering, while stronger rocks such as sandstone and limestone may require the use of churn

TABLE A17 INSPECTION TO ESTABLISH IN-SITU CONDITIONS OF THE ROCK

	Responses Yes	No	Not Specified	Remarks
Type of visual inspection carried out:				
i) inspection of rock socket	6	5	6	
ii) inspection of borehole	15	1	1	.also diamond core drilling at base .proves casing has reached rock
Inspection carried out in order to determine:				
a) general rock condition	12	2	3	
b) frequency of jointing	12	3	2	
c) condition of joints (eg. weathered, clay filled etc.)	9	4	4	
d) base of socket is clear of debris	9	-	8	
e) roughness of rock socket sidewall	6	5	6	
f) groundwater conditions	1	-	-	
g) grooving feasibility	1	-	-	

TABLE A18 REASONS FOR FIELD MODIFICATIONS TO AS-DESIGNED SOCKET DIMENSIONS

Reason for Modification	Total Response	Remarks
a) Poor Quality Rock	8	.coring at base of socket indicated presence of seams .significant mud-filled joints present .increase length due to poor rock .bedrock softer than anticipated
b) Inability to Hand Clean Socket	2	.due to water problems, socket increased in diameter by hand methods (ie. jackhammer) .unable to inspect/clean due to dewatering problems
c) Unexpected Loading Condition Changes	3	.structural engineer changed loading conditions .doubts about achieving the design bond based on inspection .poor bond on small diameter piles
d) Result of Pile Load Test	1	.socket shortened in light of test results

drills. For significantly stronger rock (eg. Diorite, Gneiss) the use of large air rotary drills have been specified by some consultants. Frequently mentioned commercial names of equipment include Williams, Hughes LDH, crane mounted Calweld, Reed-Taurus and Benoto. Some other factors which may influence the equipment used to construct socketed piles include local availability and contractor experience.

CONCLUSIONS

The responses to a questionnaire sent to consultants across Canada have been summarized. This survey revealed that:

(1) In Canada, the founding rock types commonly encountered in socketed pile applications are quite variable however, sedimentary formations such as shale, limestone and sandstones are the most common.

(2) Bearing capacity considerations are perceived to be the governing design criterion by many designers although the authors would suggest that for endbearing piles this criterion is really an implicit settlement controlled design. Several consultants did prefer to use a direct settlement controlled design approach. In addition, many respondents indicated the necessity of considering various construction constraints which may be imposed on their designs.

(3) In the majority of cases, socket loads are carried by a combination of endbearing and sideshear. No typical trend appeared to exist as to the relative contribution each is assigned in design.

(4) Allowable endbearing and sideshear parameters chosen for design were selected primarily on the basis of the respondents' previous experience. Of note were the differences that existed in design parameters selected for rock

formations located in the same geographical region. The rock type that exhibited the least variation in design values was limestone, irrespective of its geographic location.

(5) Socket dimensions generally varied among designers although the calculated length to diameter ratios appeared to fall into a typical range of values from 1 to 8, with ratios of 1 to 3 being most common for full sockets.

(6) The extent of the geotechnical investigation carried out in conjunction with a socketed pile project consisted mainly of field investigations rather than any extensive laboratory testing program. Appraisal of in-situ rock cores, field inspection of borehole and socket and load testing figured prominently in the design. In many cases the field inspection resulted in modifications to socket dimensions and re-evaluation of design parameters.

(7) In light of recently published research, reconsideration of some aspects of socketed pile design may be justified.

APPENDIX B

THEORETICAL DEVELOPMENT FOR THE FINITE ELEMENT ANALYSIS

The following discussion provides a brief and simplified description of the theory associated with the rock-pile interaction analysis conducted in Chapter 3.

The method of analysis allows for slip at a cohesive-frictional dilatant rock-pile interface as well as providing for a strain-softening response at the interface when required.

Within the finite element mesh considered, a series of dual nodes are located along the pile-rock interface with one of each pair attached to the concrete (node 1) and rock (node 2) respectively. If there is no slip, then normal and tangential compatibility conditions exist at the interface. For a pile whose base remains in contact with the rock these compatibility conditions may be written in the form

$$\sum_{j=1}^{m} I_{Nj} \dot{F}_{uj} = (\dot{\rho}_{2Nk} - \dot{\rho}_{1Nk}) \tag{B1a}$$

$$\sum_{j=1}^{m} I_{Tj} \dot{F}_{uj} = (\dot{\rho}_{2TK} - \dot{\rho}_{1Tk}) \tag{B1b}$$

where $\dot{F}_{uj}$ are the (unknown) incremental forces at the pile-rock interface

I_{Nj} and I_{Tj} are normal and tangential influence coefficients which can be determined initially and will remain constant for each iteration and load step

$\dot{\rho}_{1Nk}$, $\dot{\rho}_{2Nk}$, $\dot{\rho}_{1Tk}$, $\dot{\rho}_{2Tk}$ are the known displacements at the interface due to the known forces (including residual forces arising from the initial stress formulation in an elasto-plastic analysis); and

m is the number of degrees of freedom at the interface.

Details regarding the analysis of the more general case where the pile may separate from the rock at the base, as might be expected for uplift conditions, are given by Rowe, Booker and Balaam (1978). If there is no slip, the compatibility equation (Eq. B1) can be solved to determine $\dot{\underset{\sim}{F}}_u$ and once these quantities are known, the displacements within both the rock and the pile may be directly determined. A displacement defined analysis (allowing unloading) may be performed by specifying the pile head displacement with the total load being an unknown which is determined from equilibrium considerations.

If it is assumed that the pile-rock interface behaviour is governed by a Mohr-Coulomb failure criterion, then slip will occur when the mobilized shear stress τ reaches the strength of the material i.e.

$$\tau = c_p + \sigma_n \tan \phi_p \qquad \text{(B2)}$$

where c_p is the peak interface "adhesion"

σ_n is the normal stress at the pile-rock interface

ϕ_p is the peak angle of friction of the pile-rock interface

A simple model of the interface behaviour would involve the degradation of the pile-rock interface parameters after the peak stress has been reached,

thus

$$\tau = c + \sigma_n \tan \phi \qquad \text{(B3)}$$

where c and ϕ are a post peak interface adhesion and friction angle respectively. In general, the degradation from peak to residual strength at a point will occur with increasing relative displacement between the two sides of the rupture. For convenience, it is assumed here that c and tan ϕ degrade linearly with relative displacement and that the residual parameters (c_r, ϕ_r) are attained after a relative displacement δ_r. Once slip occurs, the tangential compatibility condition (Eq. B1b) is replaced by the failure condition (Eq. B3).

If an interface has a dilatancy angle ψ (Davis, 1968) at a point when slip occurs, then increments in the normal and tangential displacement may be related by this dilatancy angle giving:

$$\sum_{j=1}^{m} (I_{Nj} - \tan \psi \, I_{Tj}) \, \dot{F}_{uj} = (\dot{\rho}_{2Nk} - \dot{\rho}_{1Nk}) - \tan \psi \, (\dot{\rho}_{2Tk} - \dot{\rho}_{1Tk}) \qquad \text{(B4)}$$

Once slip occurs, the normal compatibility condition (Eq. B1a) is replaced by the dilatancy equation (B4) and as previously, equation (B1b) is replaced by the appropriate form of equation (B3). Experimental evidence (Williams, 1980; Pells and Rowe, 1981) indicates that the pile-rock interface will dilate until a limited normal displacement (which depends on rock stiffness) is reached, and will then deform at constant volume. To sufficient accuracy, the experimentally observed behaviour may be idealised as a dilation with a constant dilatancy angle ψ until the limiting normal displacement (dilation) δ_d is attained after which the dilatancy angle at that point is reduced to zero (ψ = 0 implies constant volume deformation).

APPENDIX C

DESIGN CHARTS FOR ROCK SOCKETED PILES

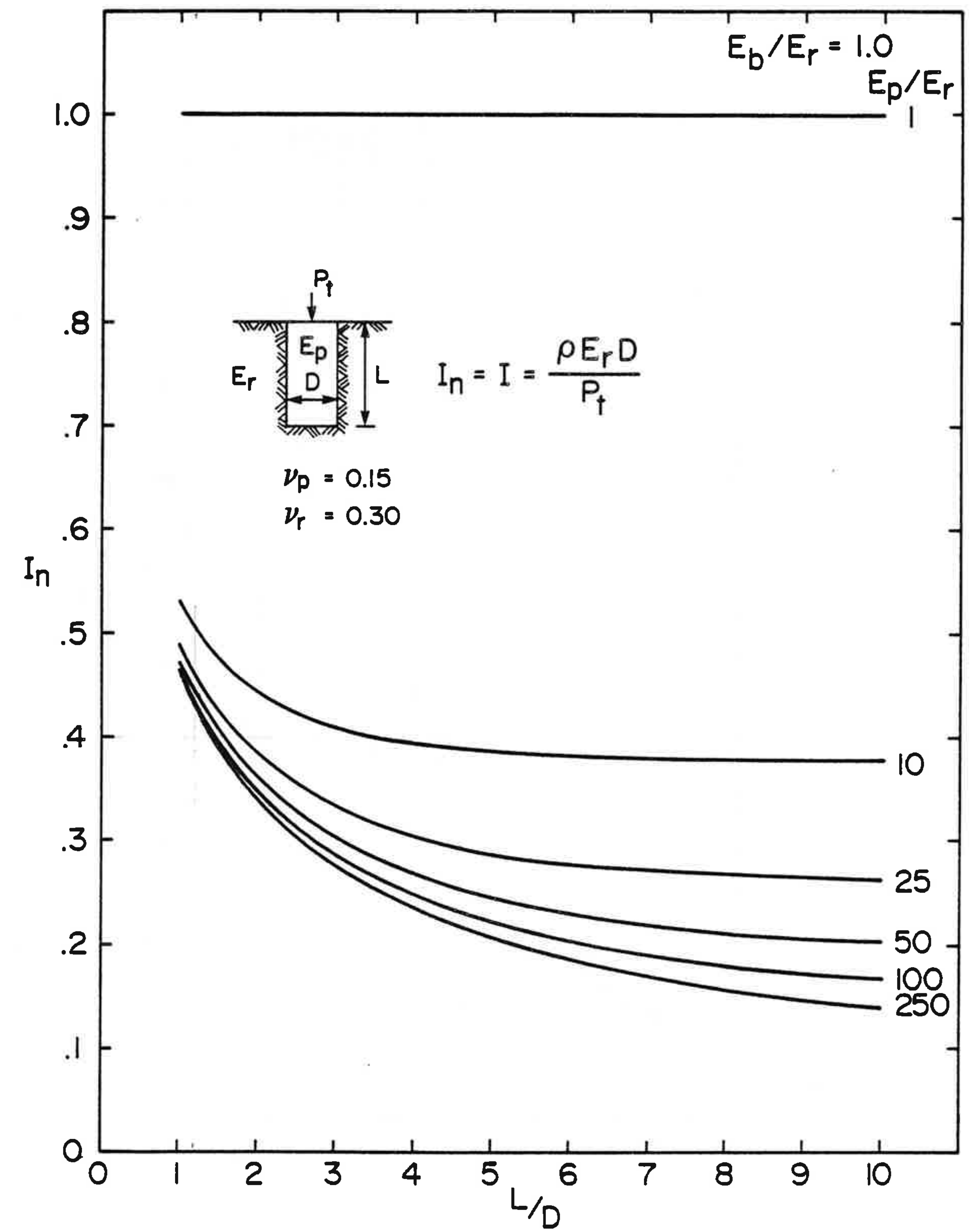


FIGURE C.1a ELASTIC SETTLEMENT INFLUENCE FACTORS FOR A COMPLETE SOCKETED PILE: $E_b/E_r = 1$

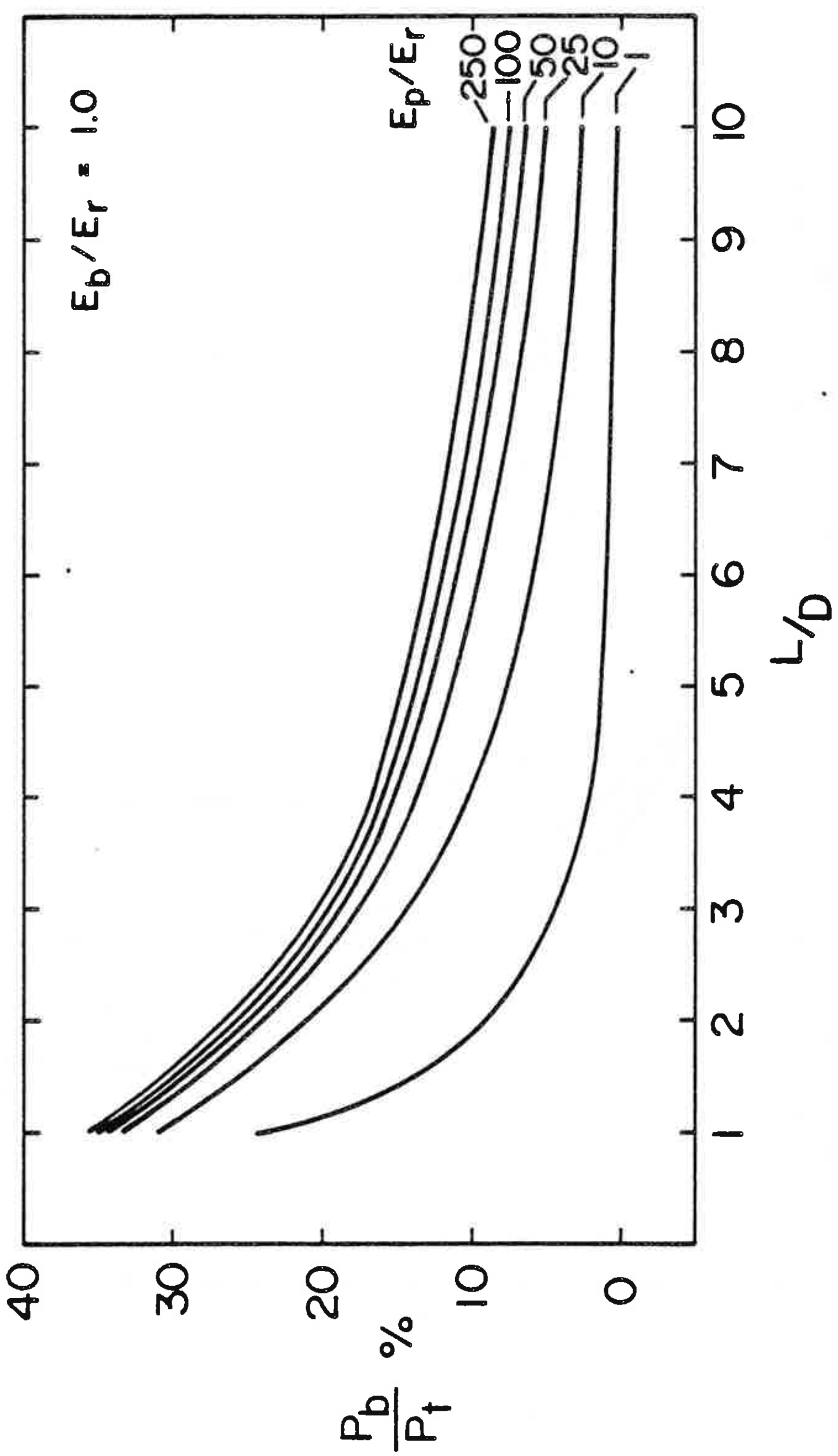


FIGURE C.1b ELASTIC LOAD DISTRIBUTION CURVES FOR A COMPLETE SOCKETED PILE (HOMOGENEOUS: E_b/E_r = 1.0)

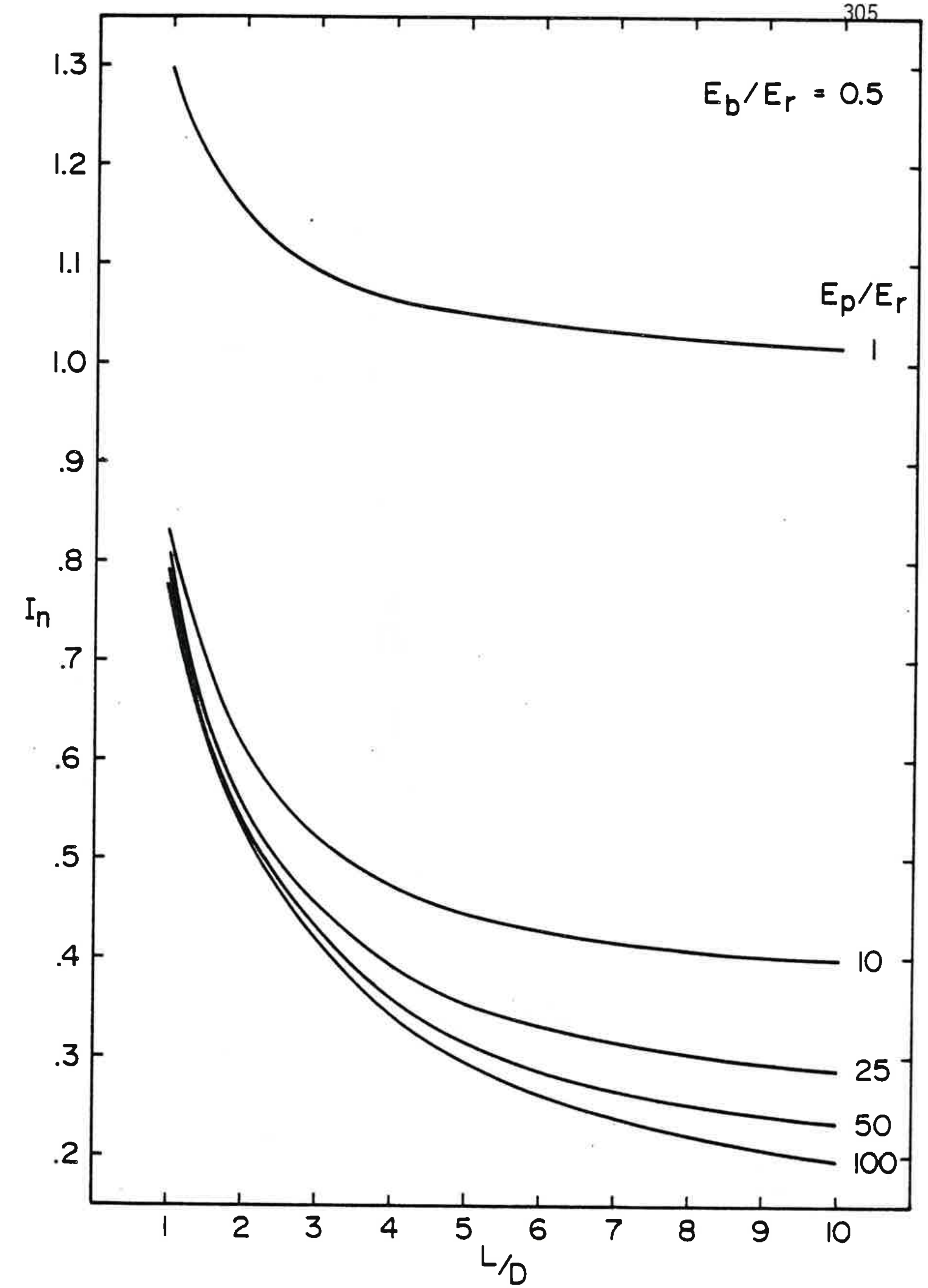


FIGURE C.1c ELASTIC SETTLEMENT INFLUENCE FACTORS FOR A COMPLETE SOCKETED PILE: E_b/E_r = 0.5

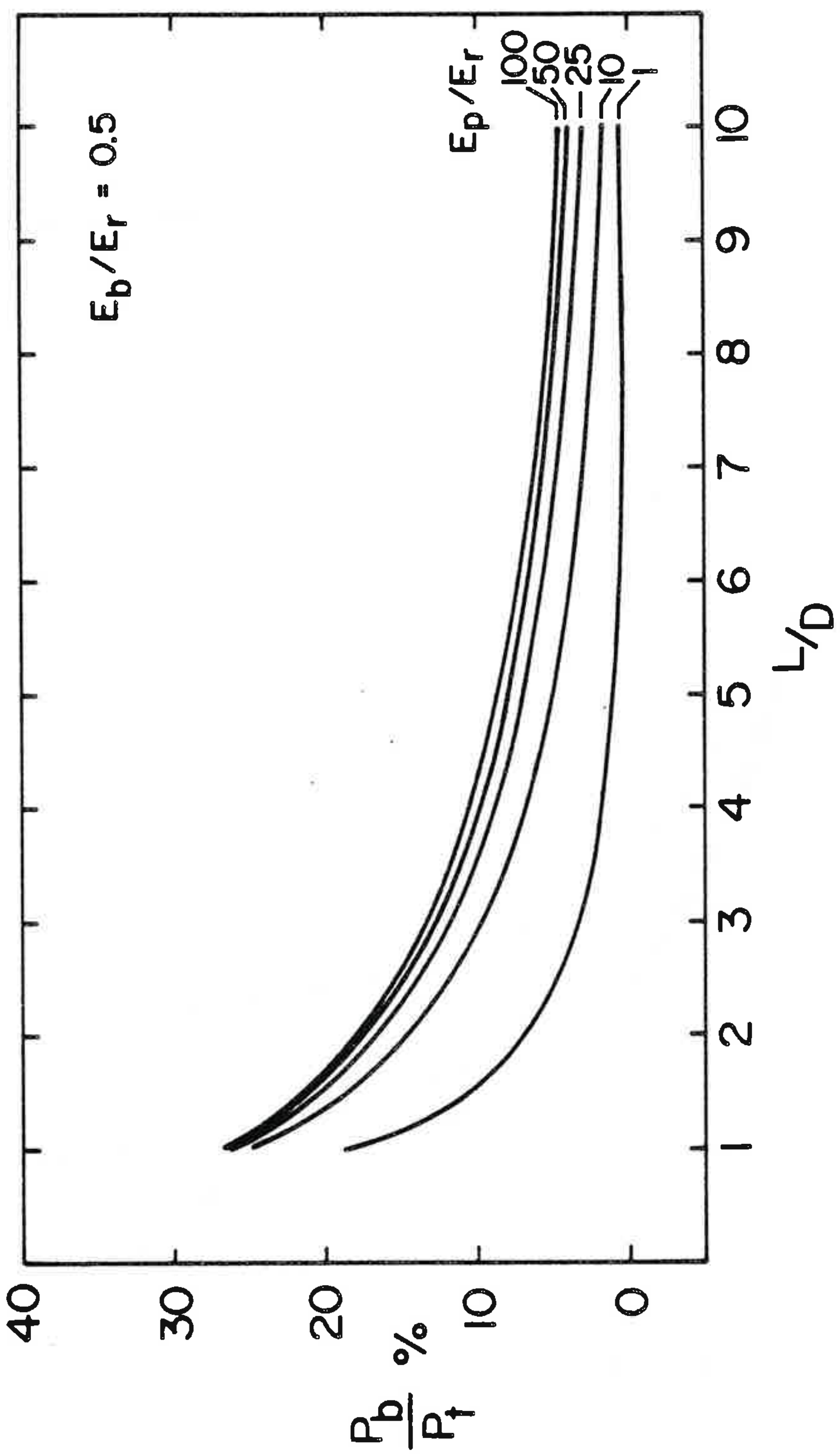


FIGURE C.1d ELASTIC LOAD DISTRIBUTION CURVES FOR A COMPLETE SOCKETED PILE: E_b/E_r = 0.5

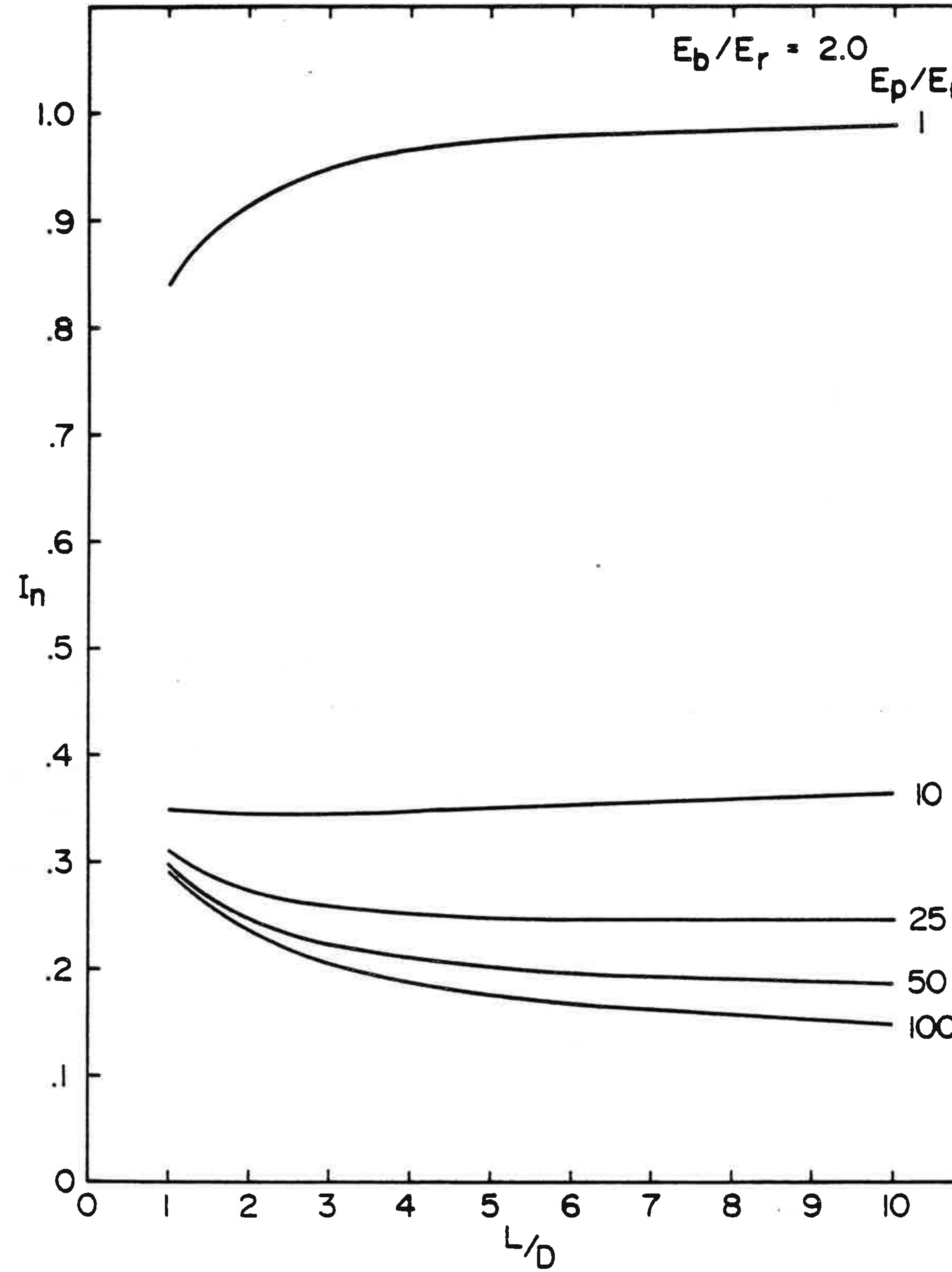


FIGURE C.1e ELASTIC SETTLEMENT INFLUENCE FACTORS FOR A COMPLETE SOCKETED PILE: E_b/E_r = 2.0

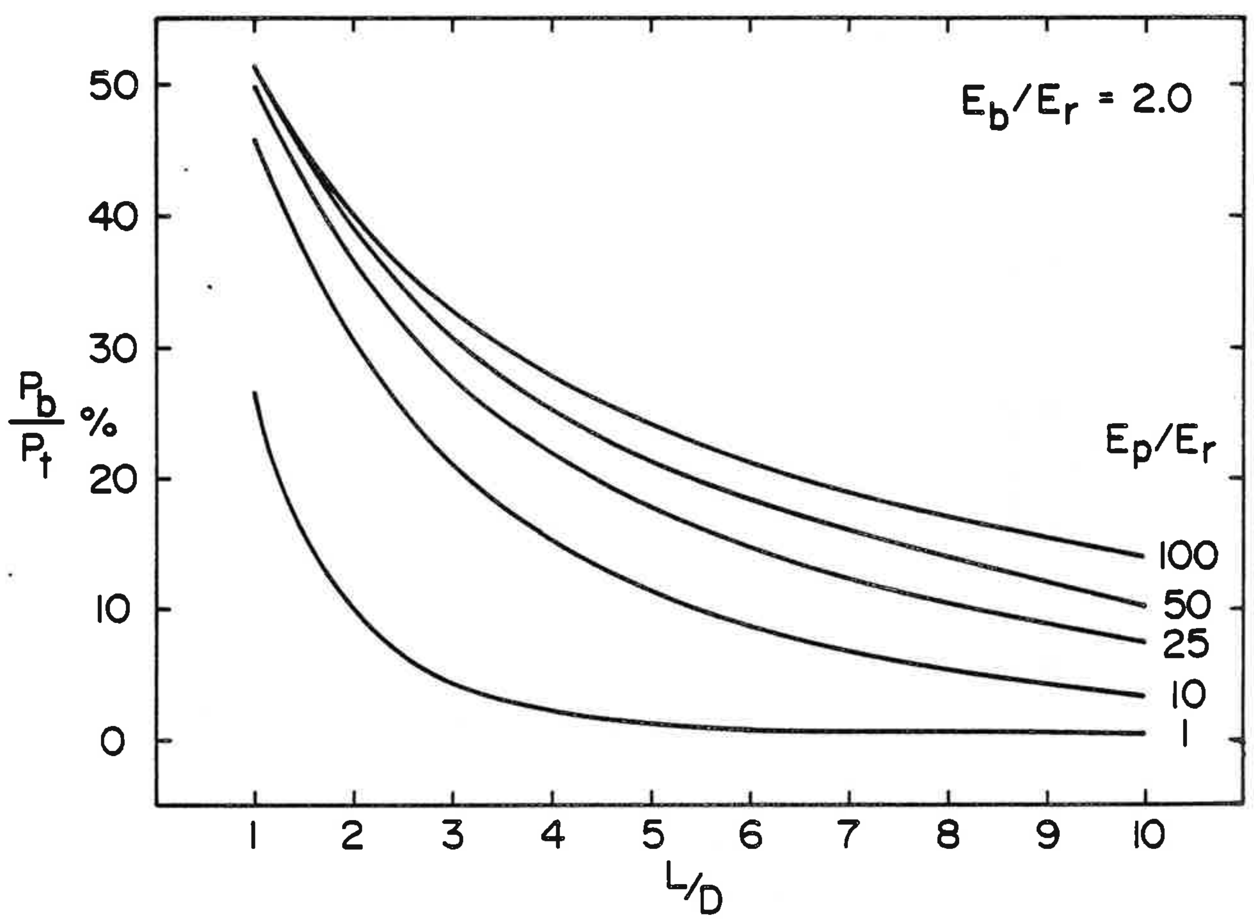


FIGURE C.1f ELASTIC LOAD DISTRIBUTION CURVES FOR A COMPLETE SOCKETED PILE: E_b/E_r = 2.0

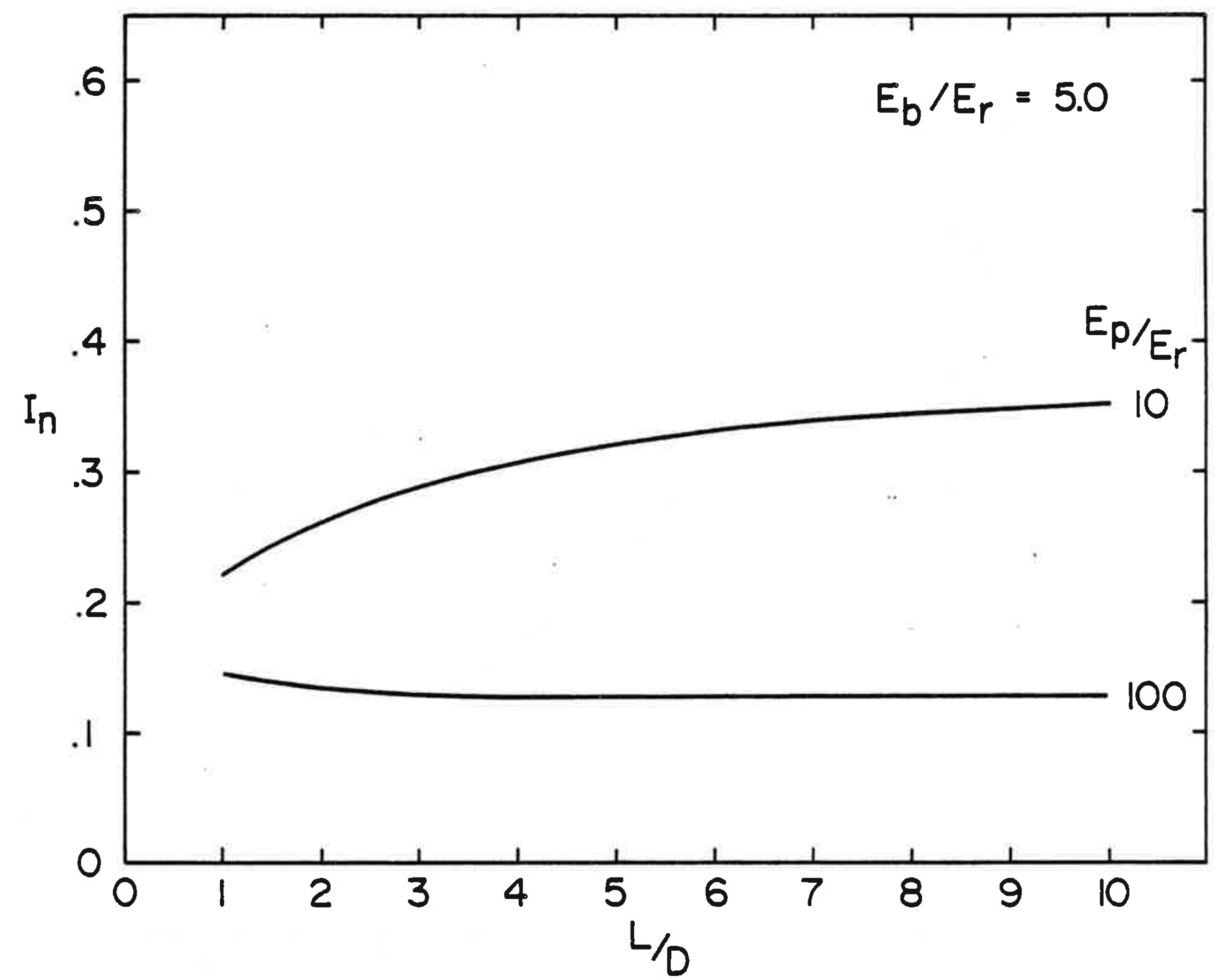


FIGURE C.1g ELASTIC SETTLEMENT INFLUENCE FACTORS FOR A COMPLETE SOCKETED PILE: E_b/E_r = 5.0

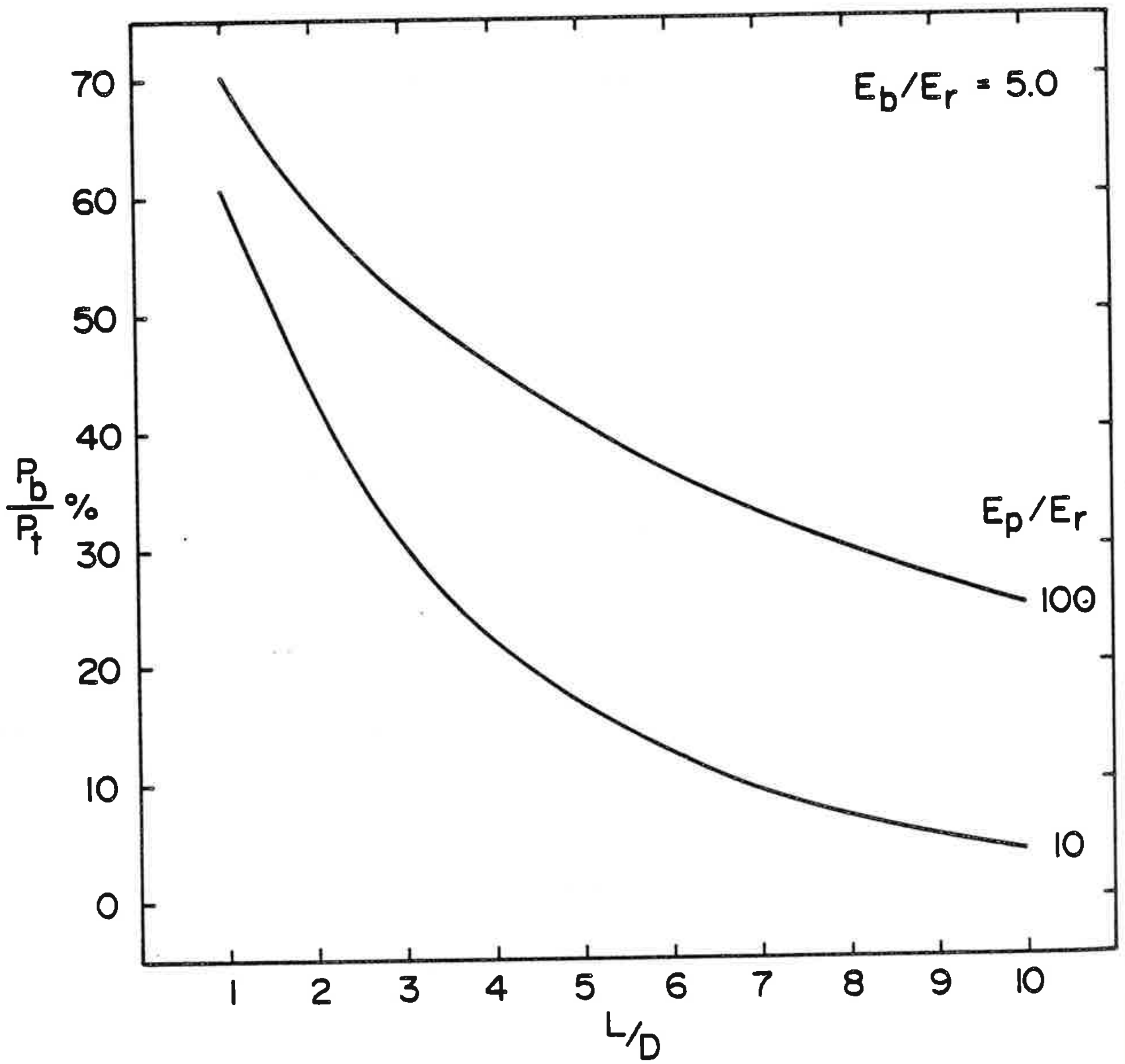


FIGURE C.1h ELASTIC LOAD DISTRIBUTION CURVES FOR A COMPLETE SOCKETED PILE: E_b/E_r = 5.0

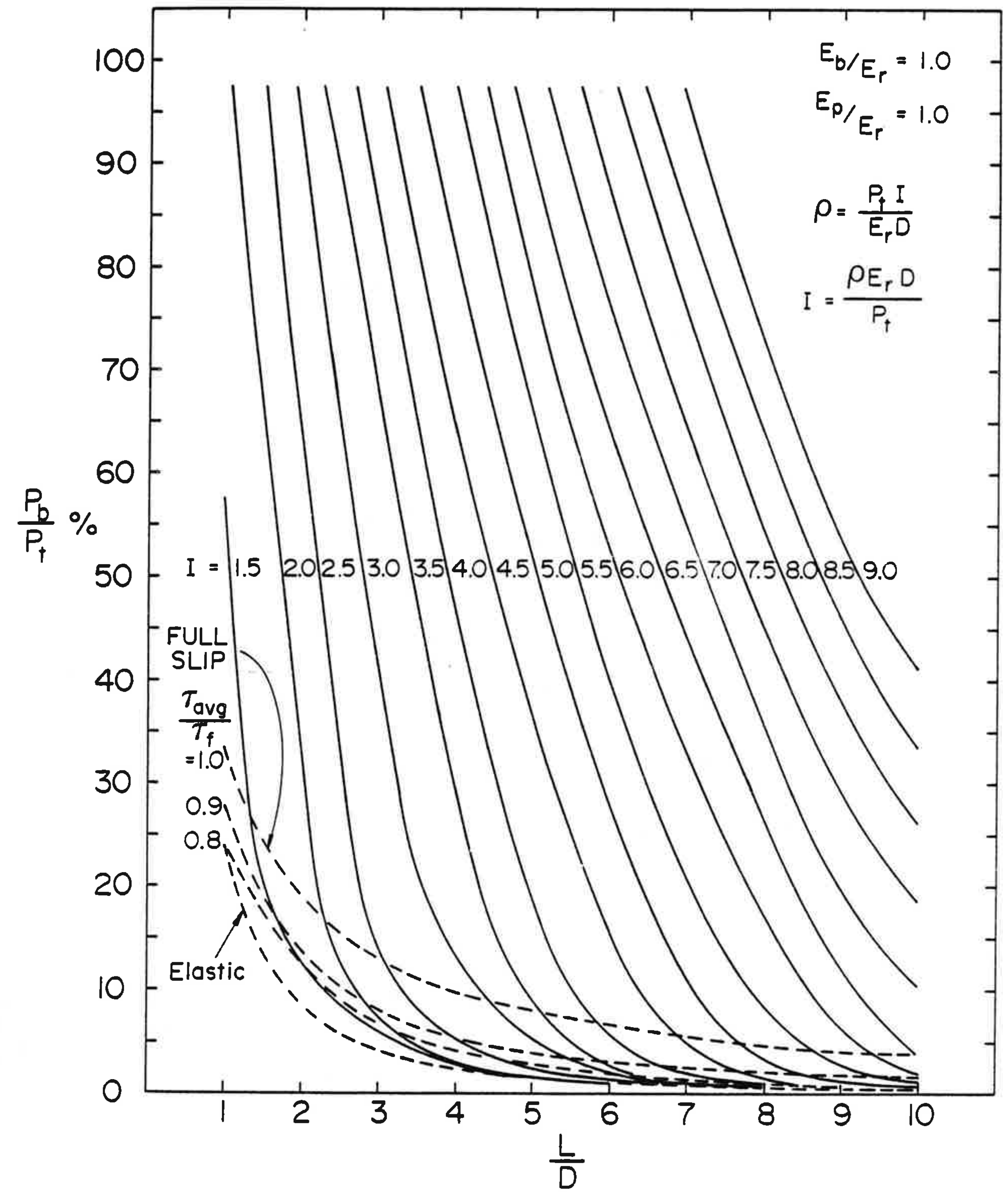


FIGURE C.2a DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

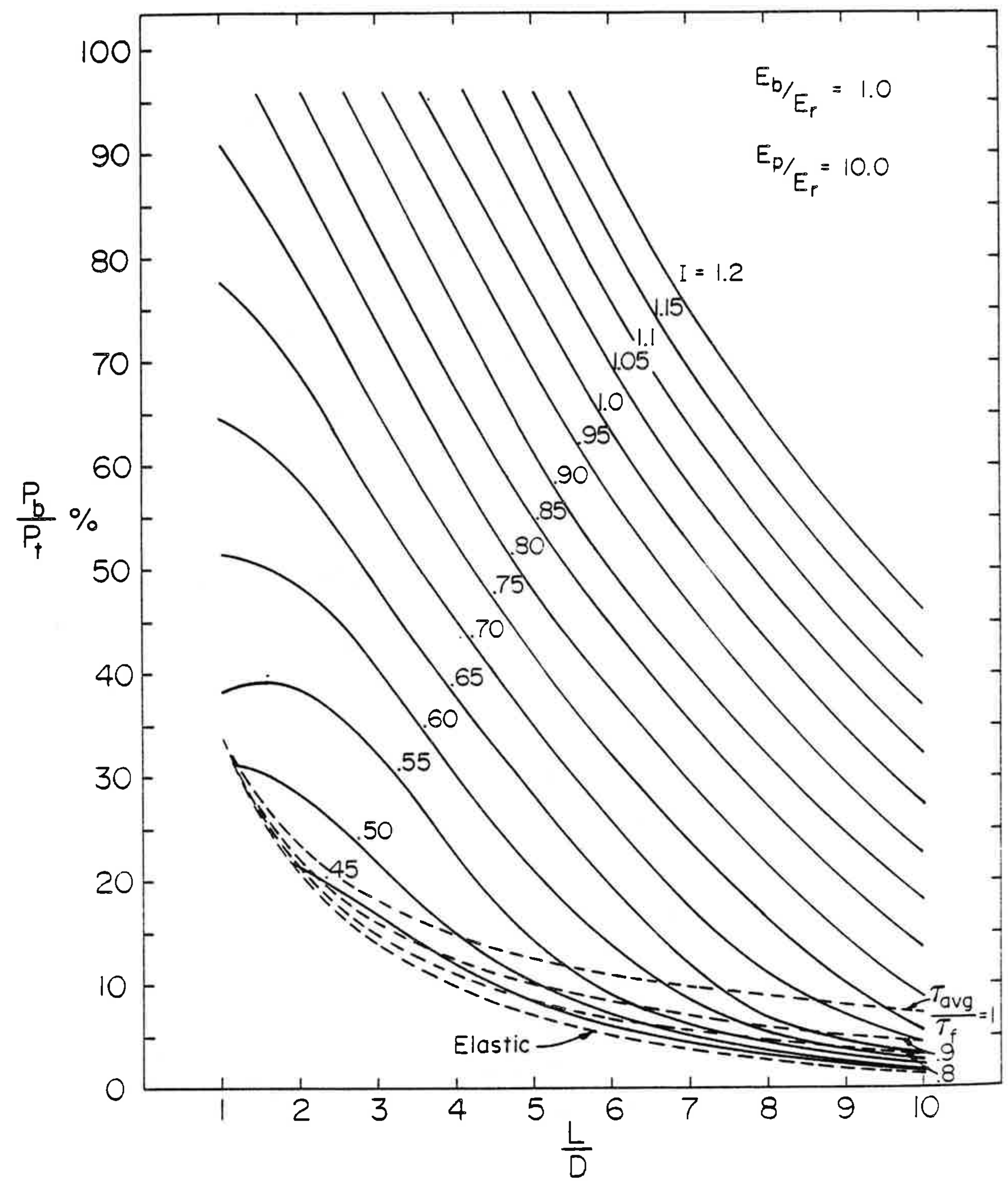


FIGURE C.2b DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

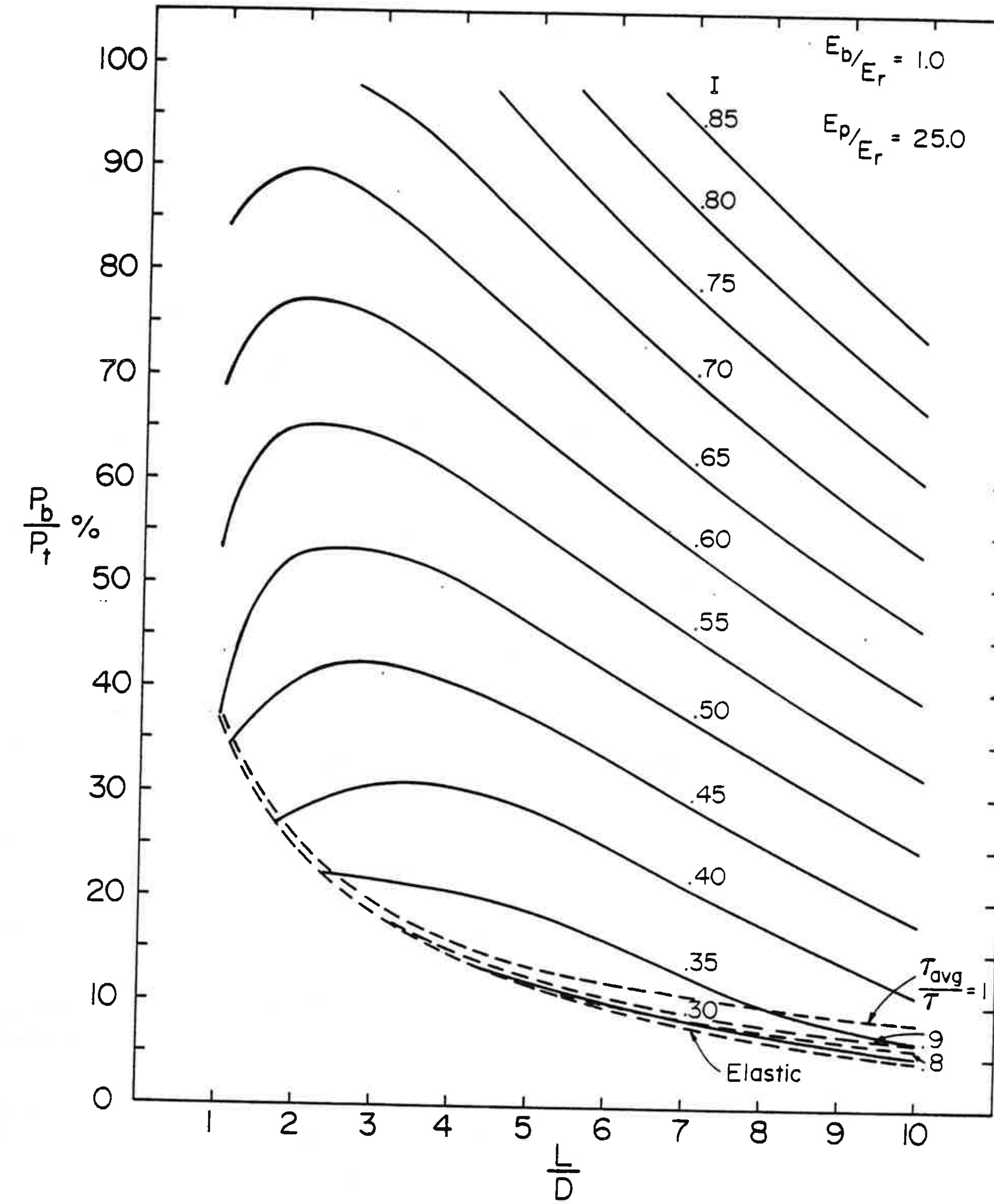


FIGURE C.2c DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

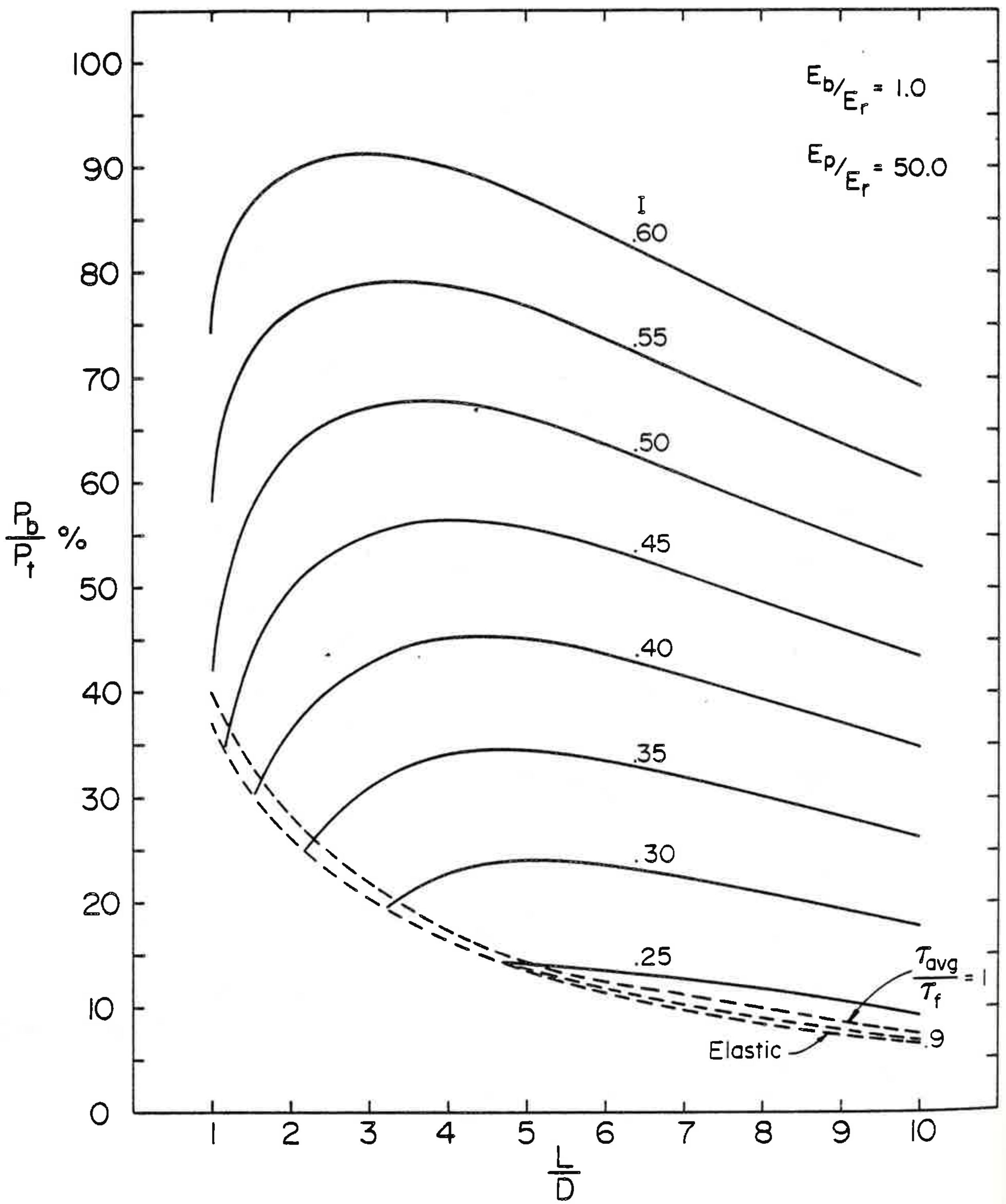


FIGURE C.2d DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

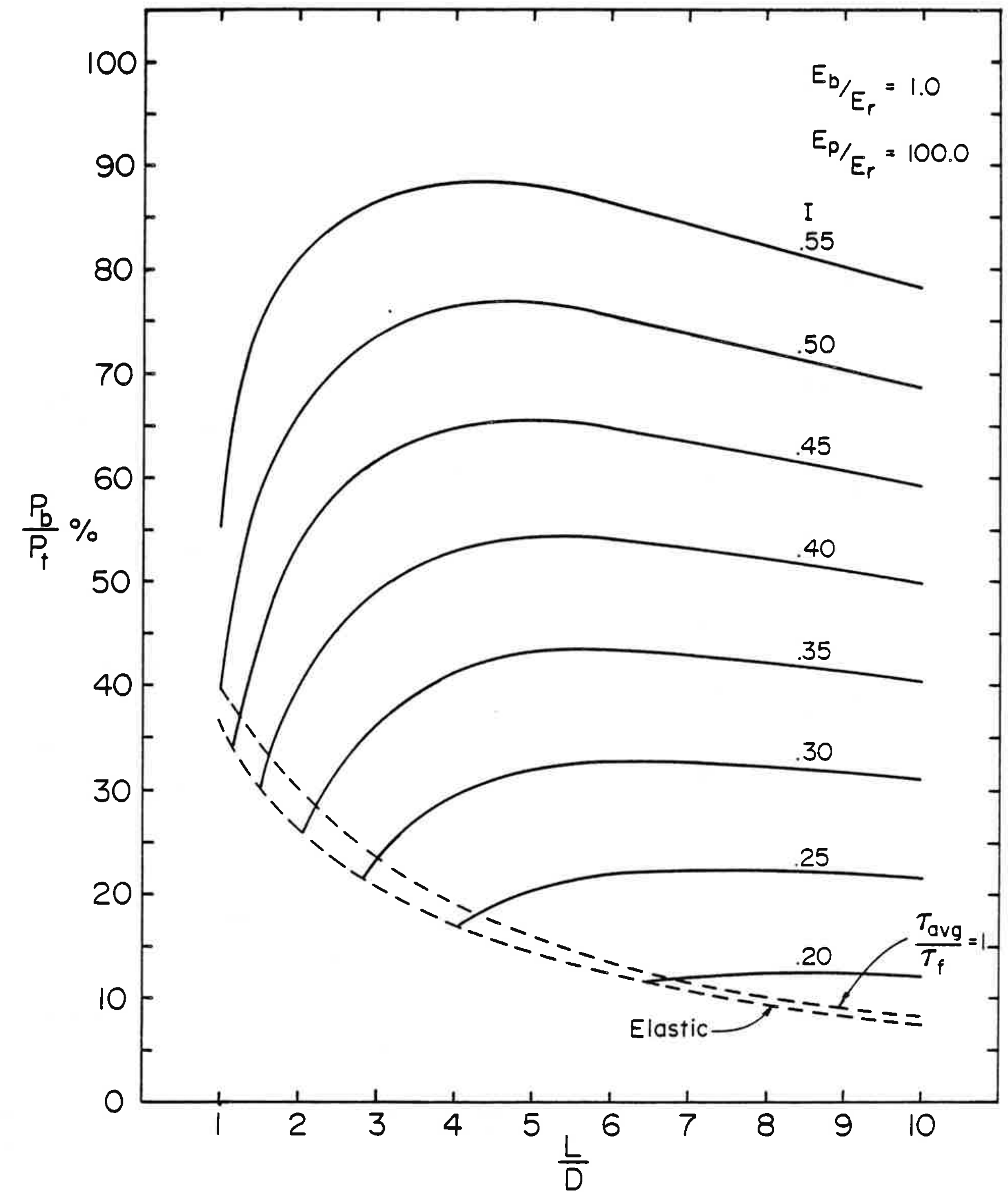


FIGURE C.2e DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

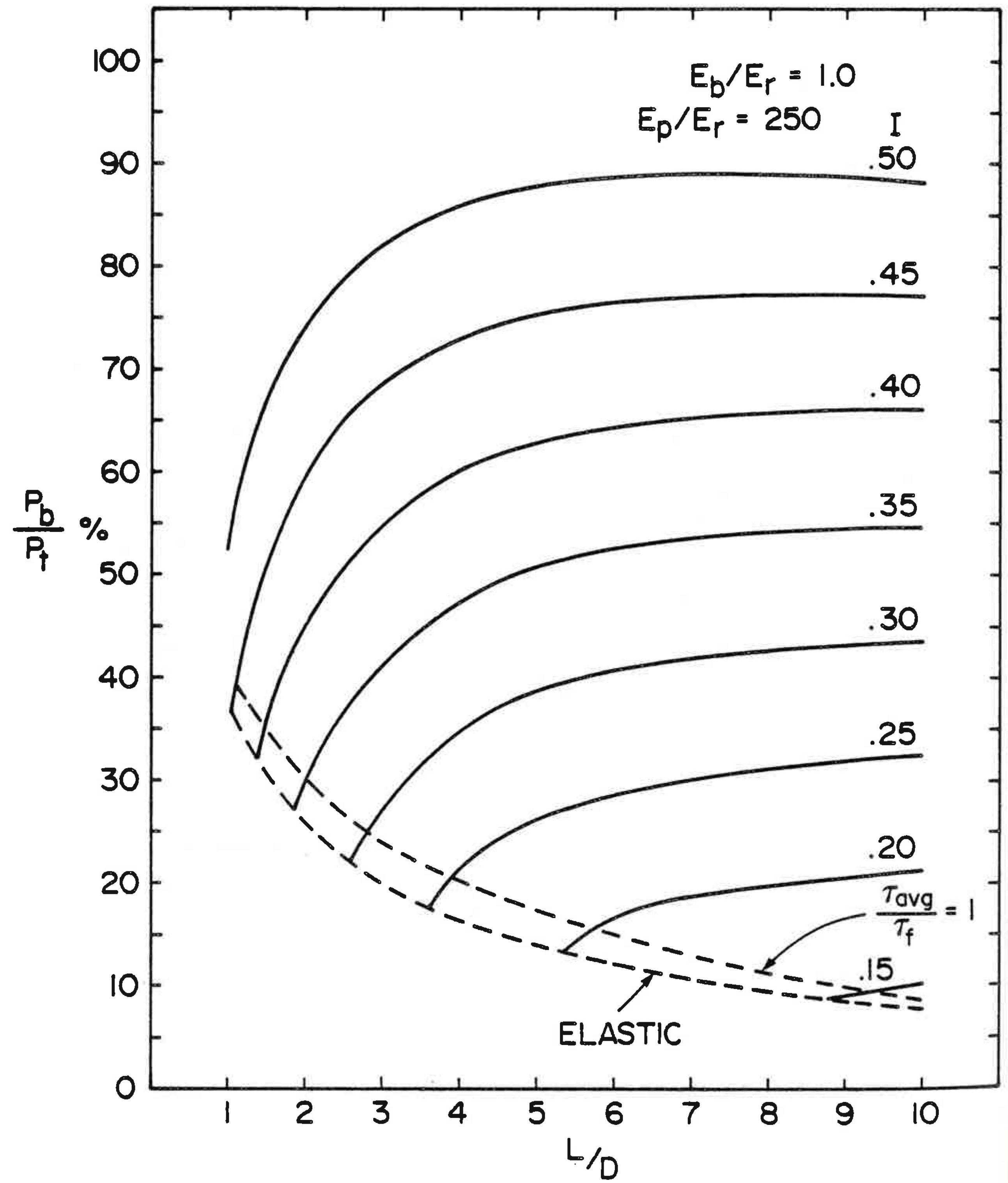


FIGURE C.2f DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

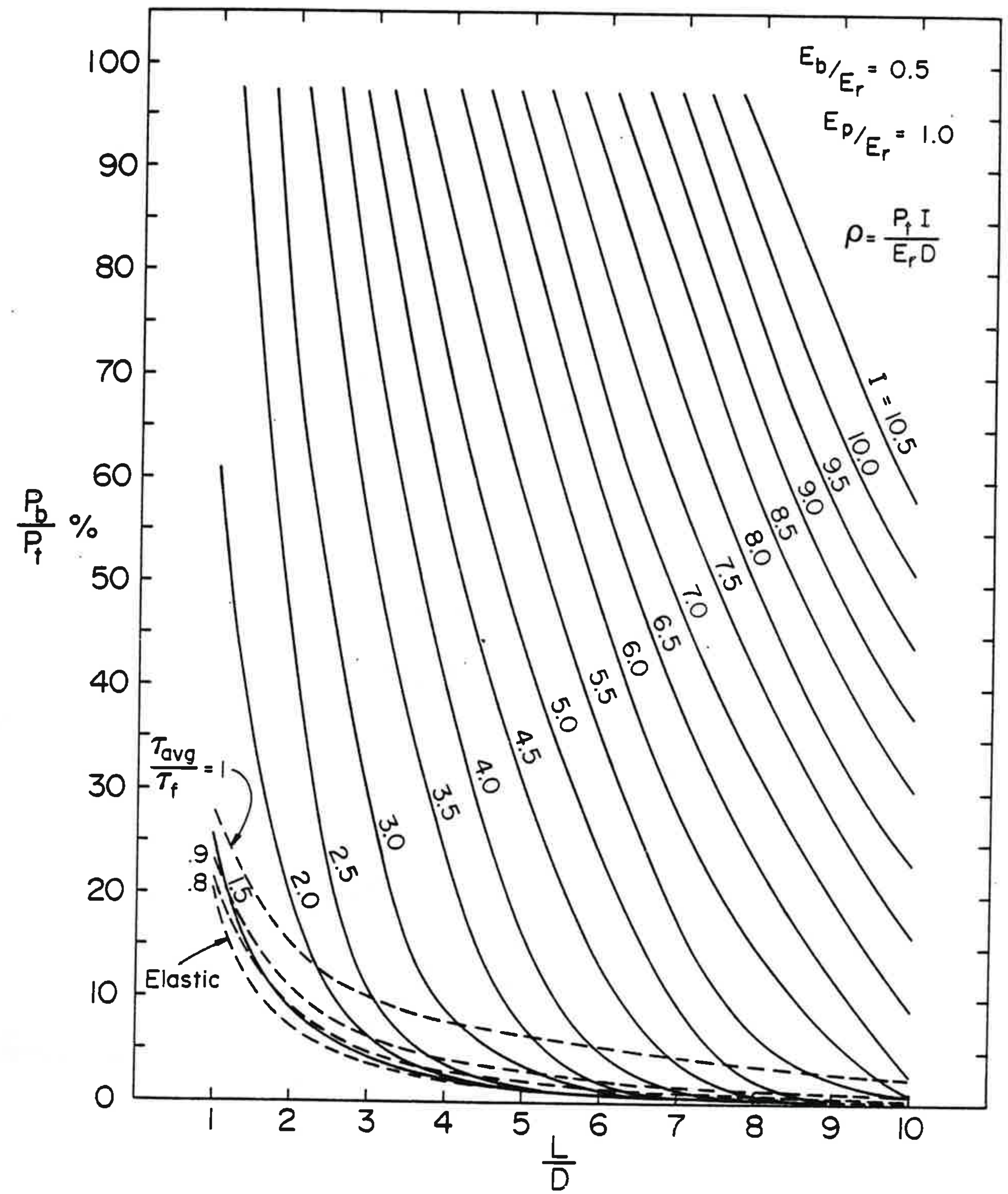


FIGURE C.3a DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

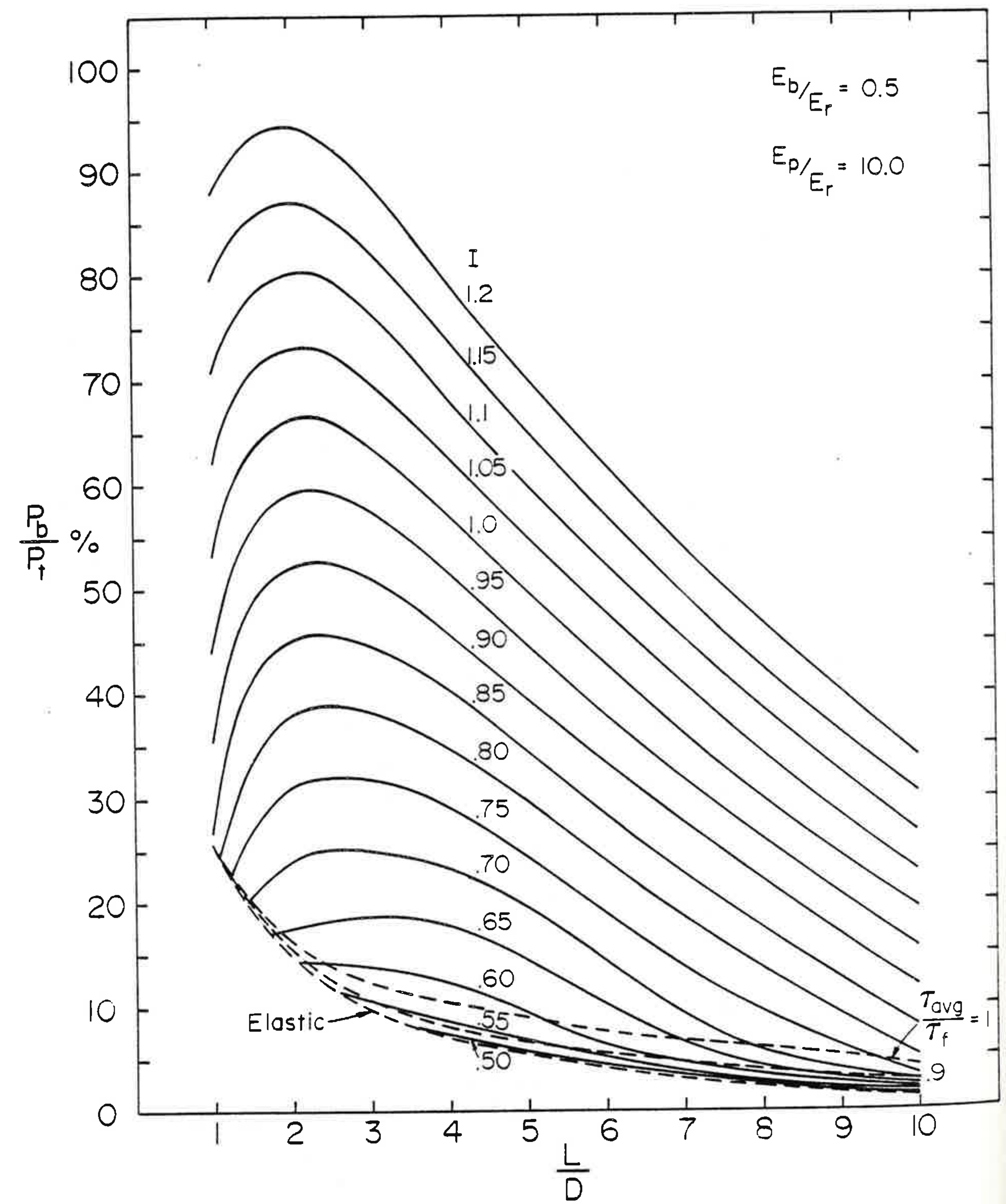


FIGURE C.3b DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

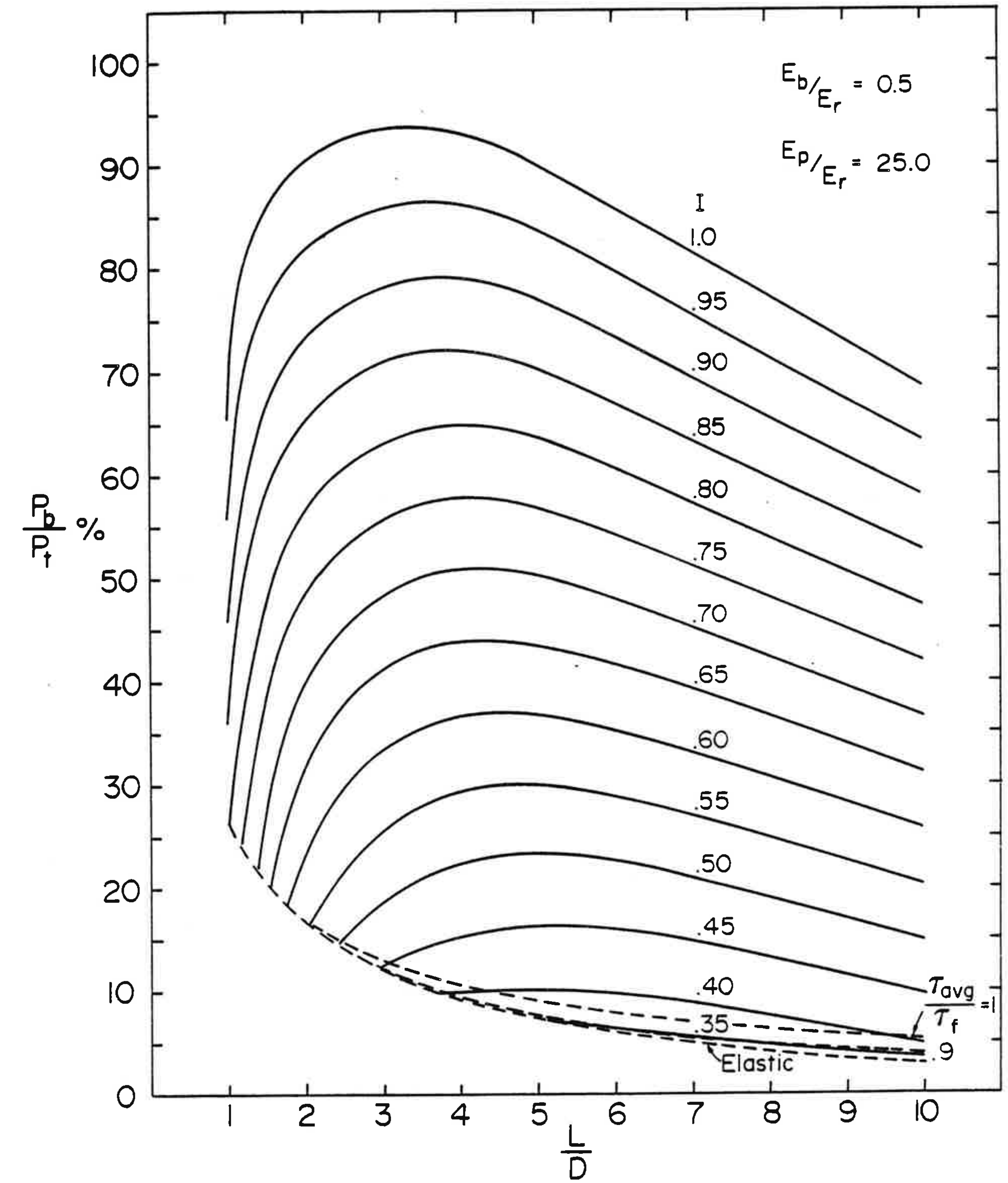


FIGURE C.3c DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

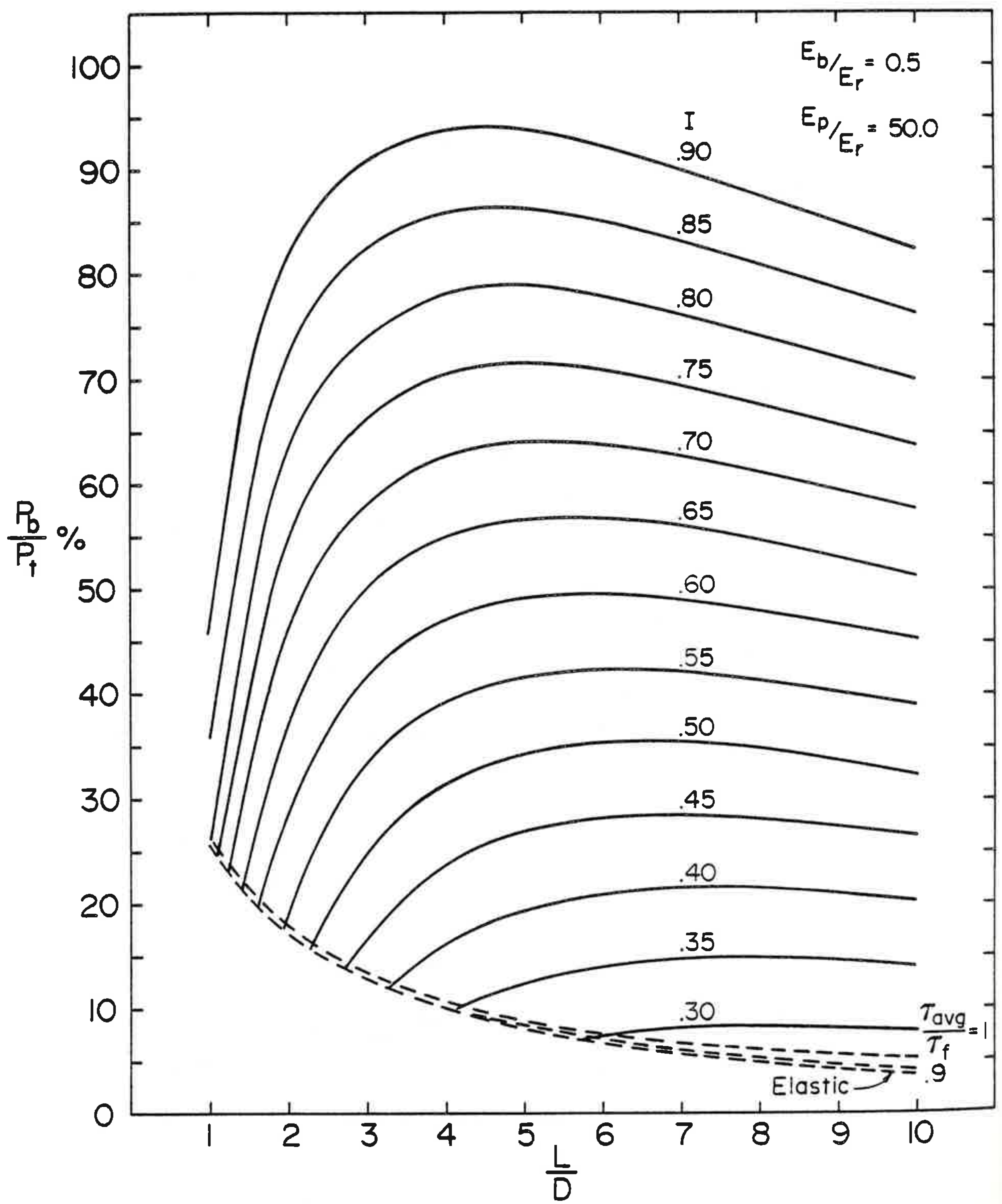


FIGURE C.3d DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

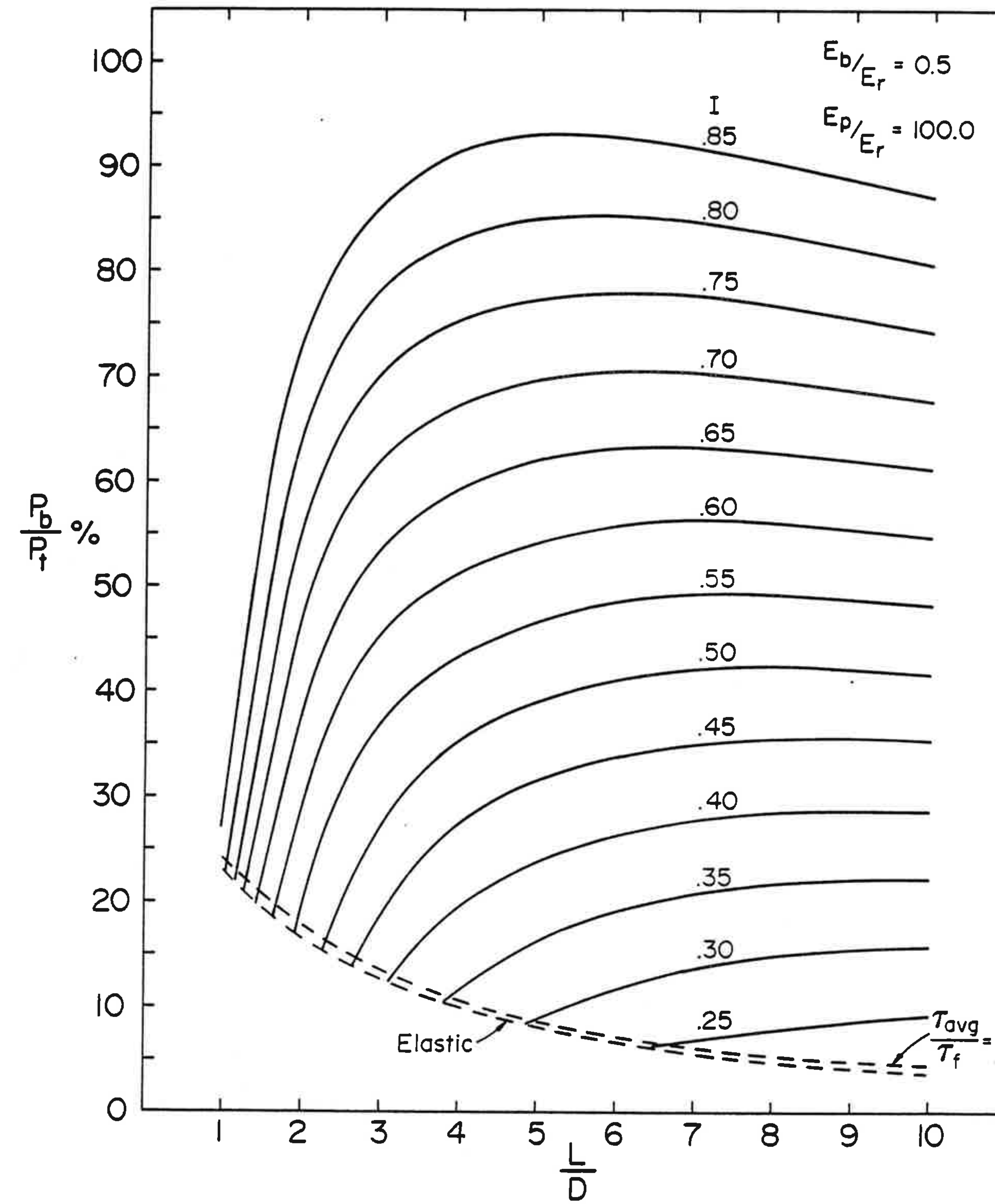


FIGURE C.3e DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

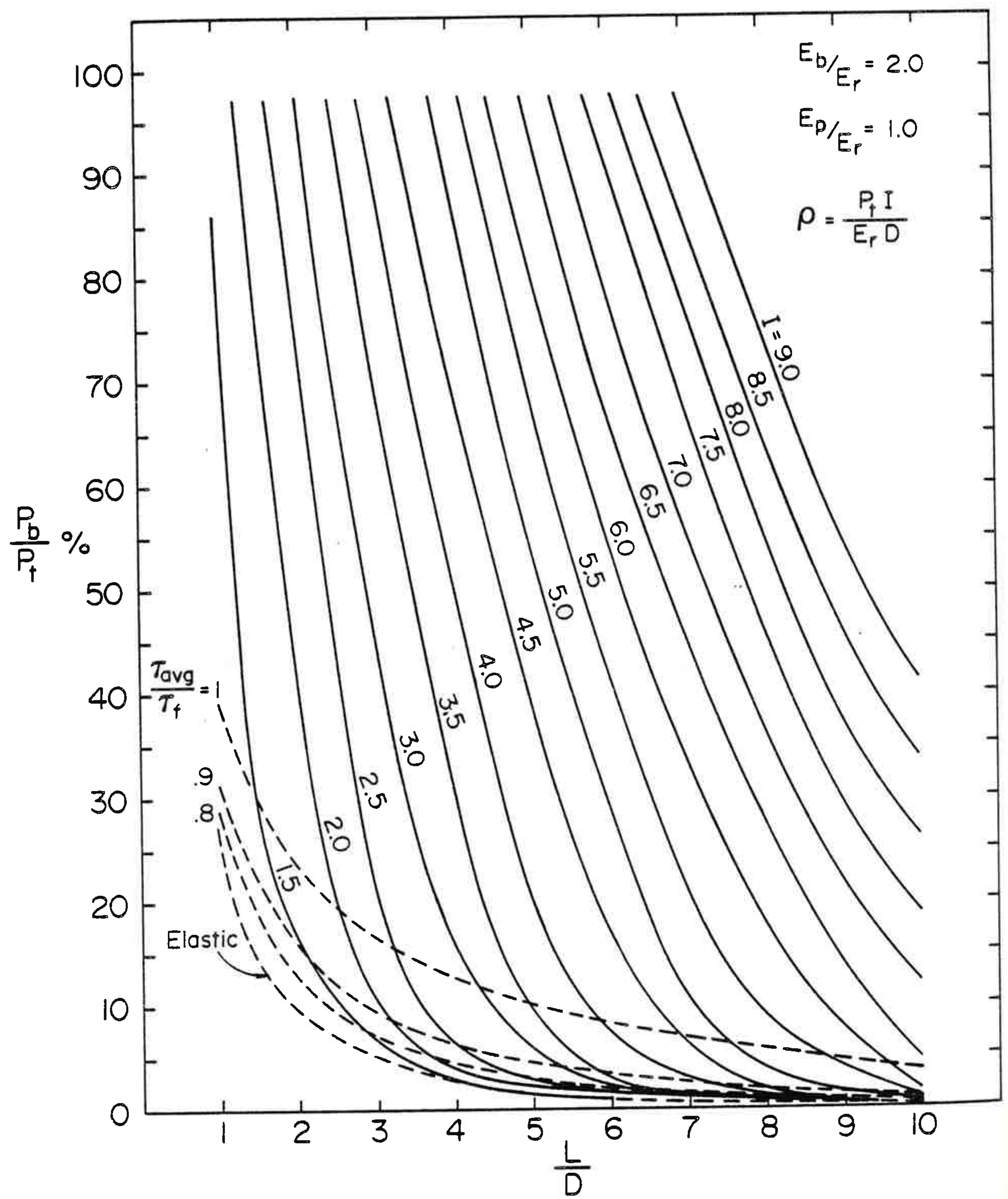


FIGURE C.4a DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

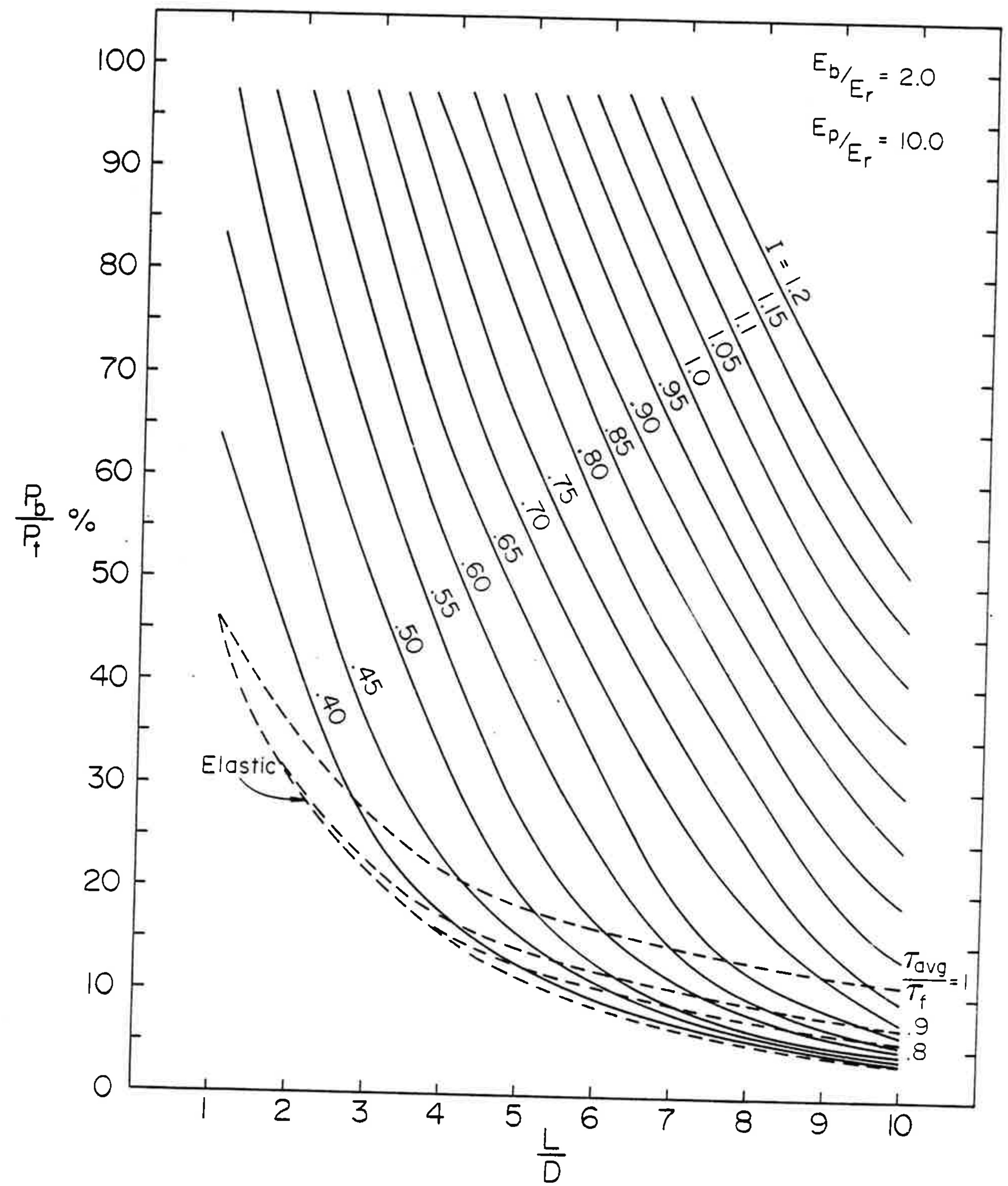


FIGURE C.4b DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

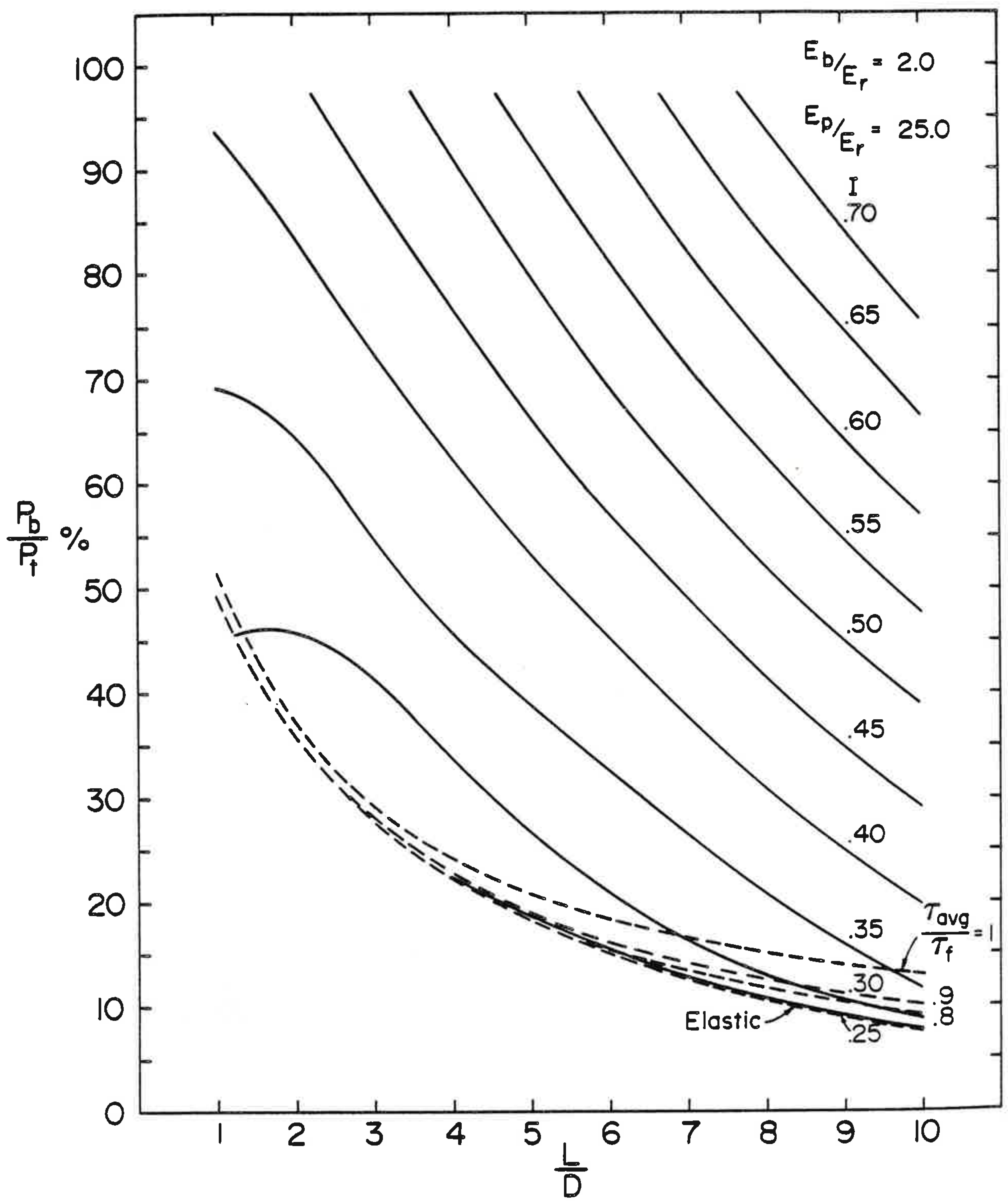


FIGURE C.4c DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

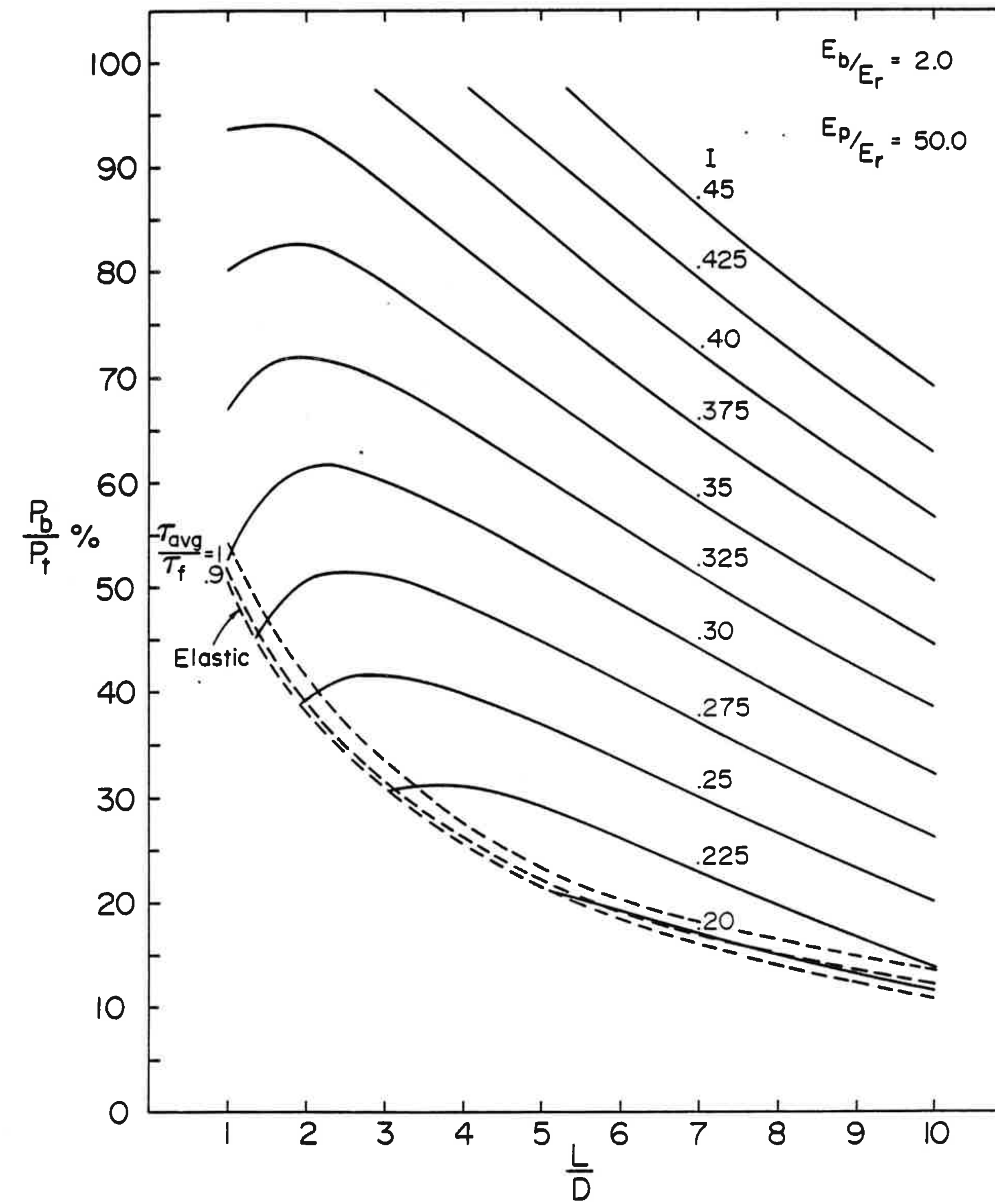


FIGURE C.4d DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

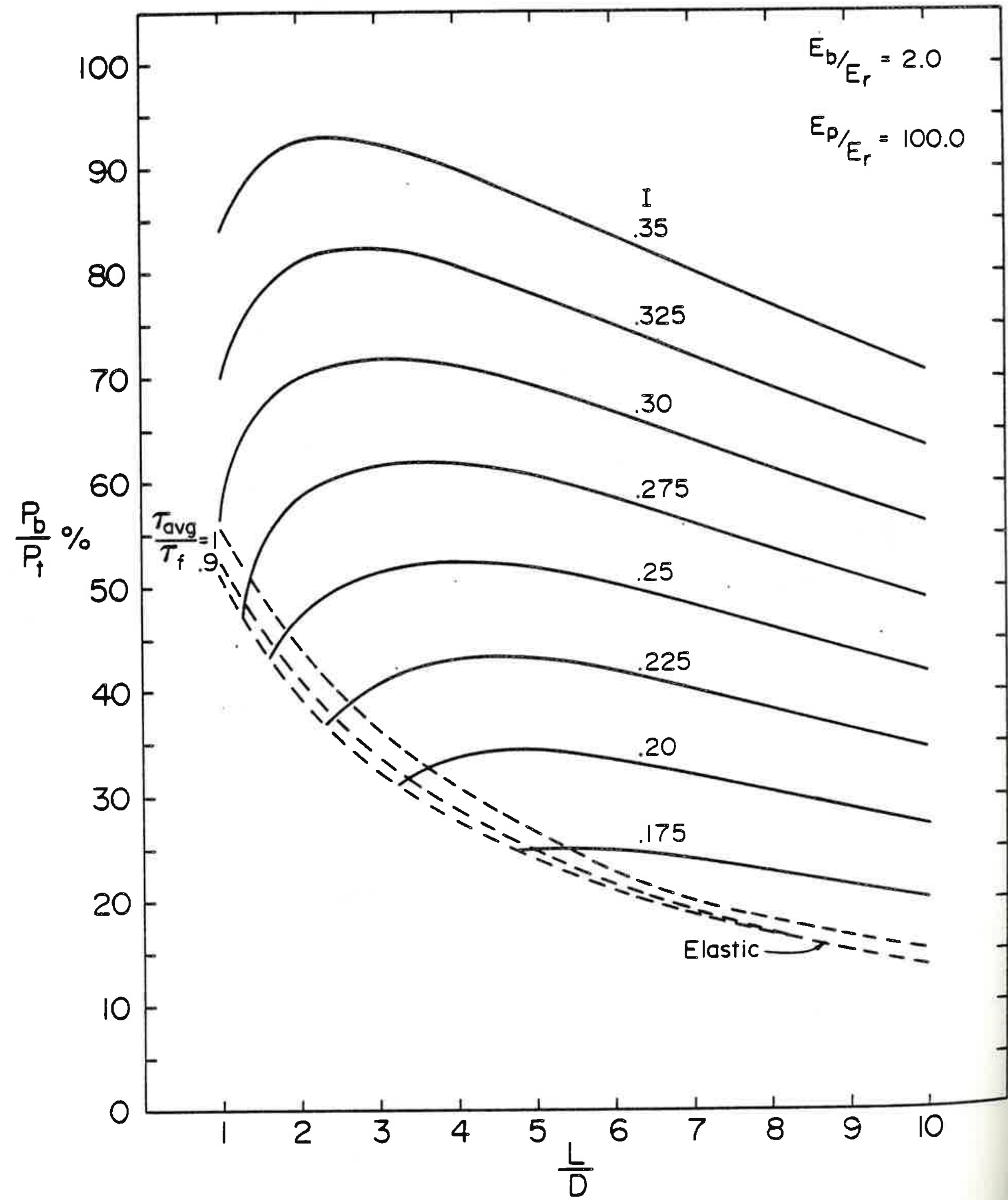


FIGURE C.4e DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

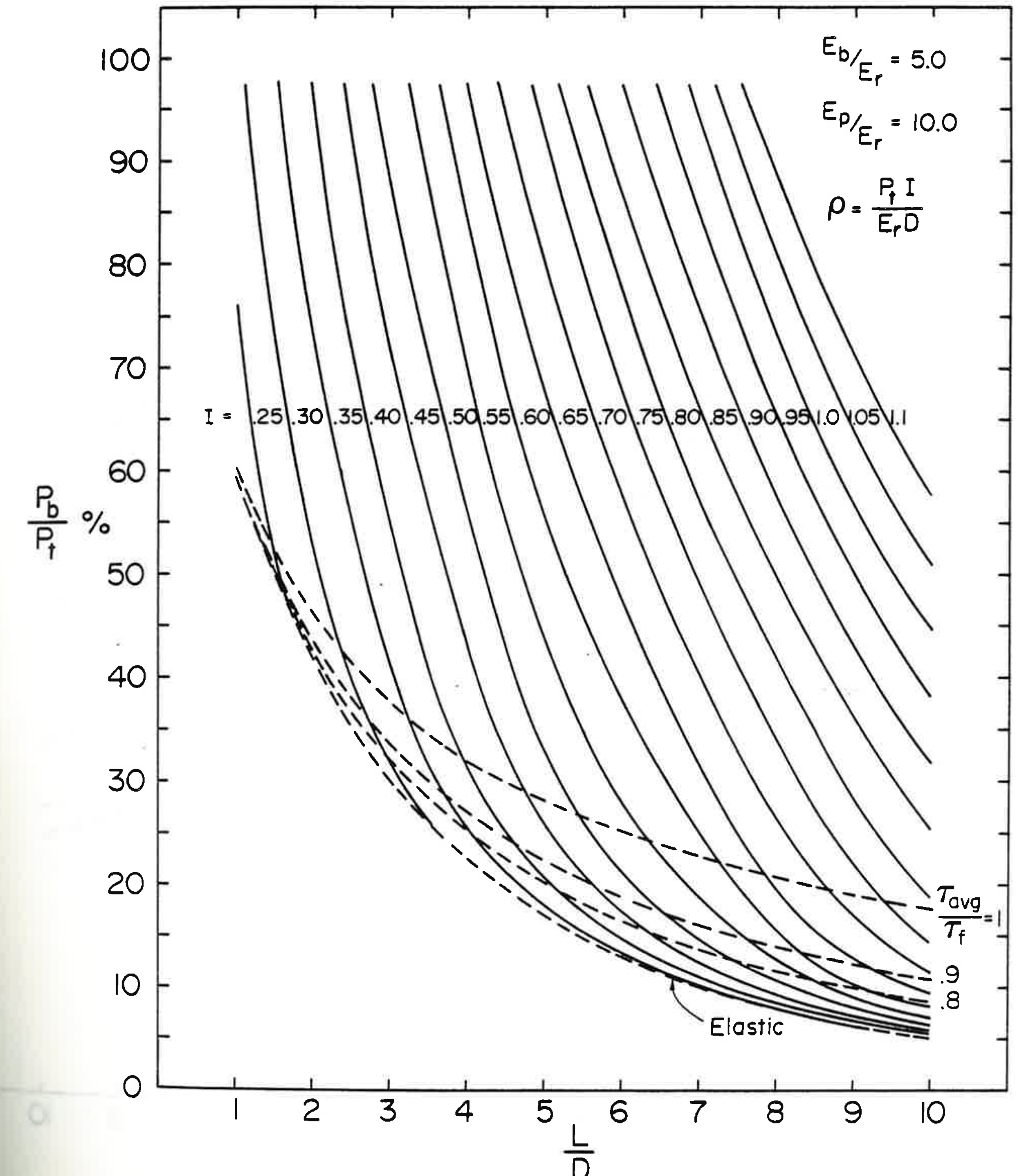


FIGURE C.5a DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

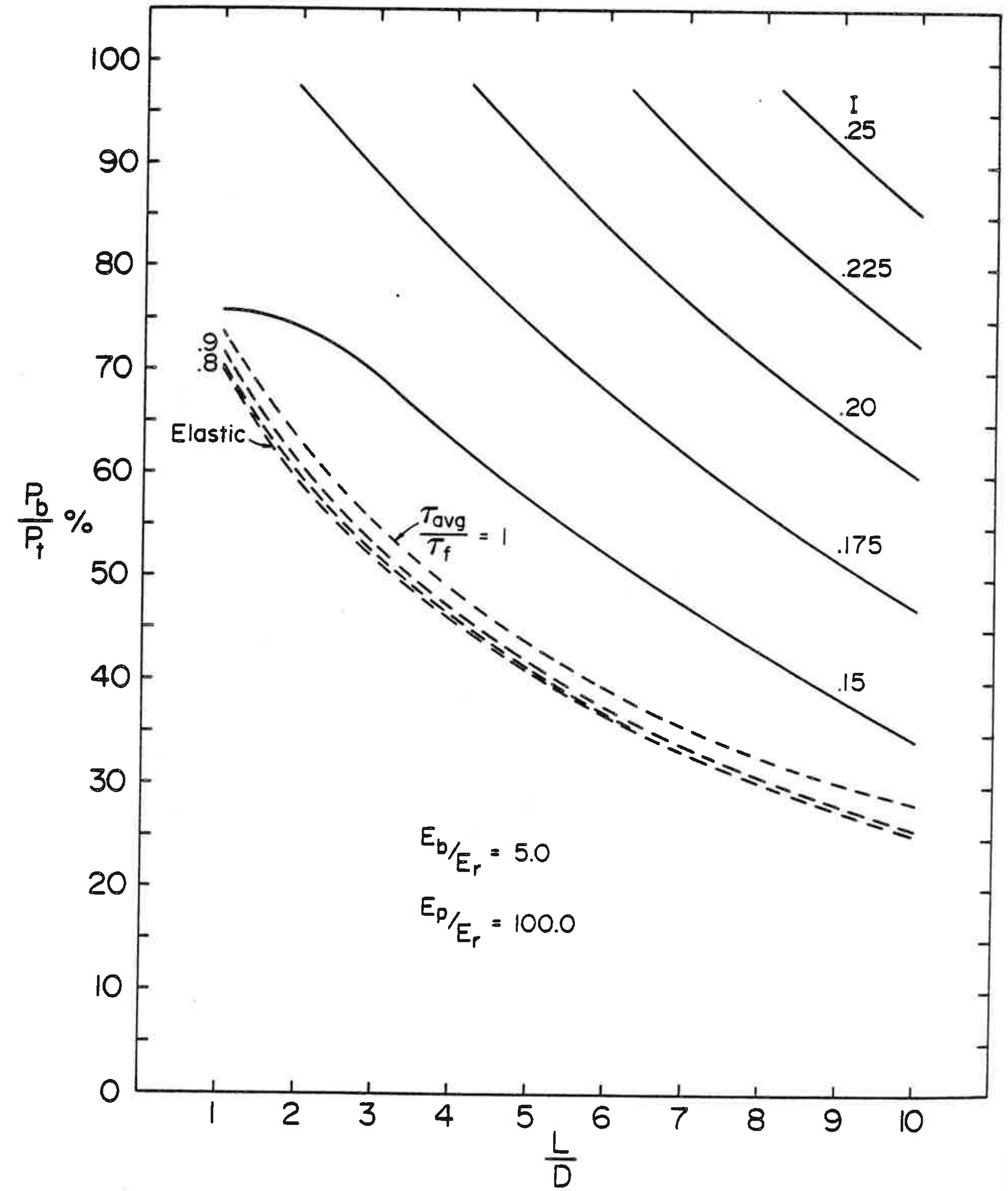


FIGURE C.5b DESIGN CHARTS FOR A COMPLETE SOCKETED PILE

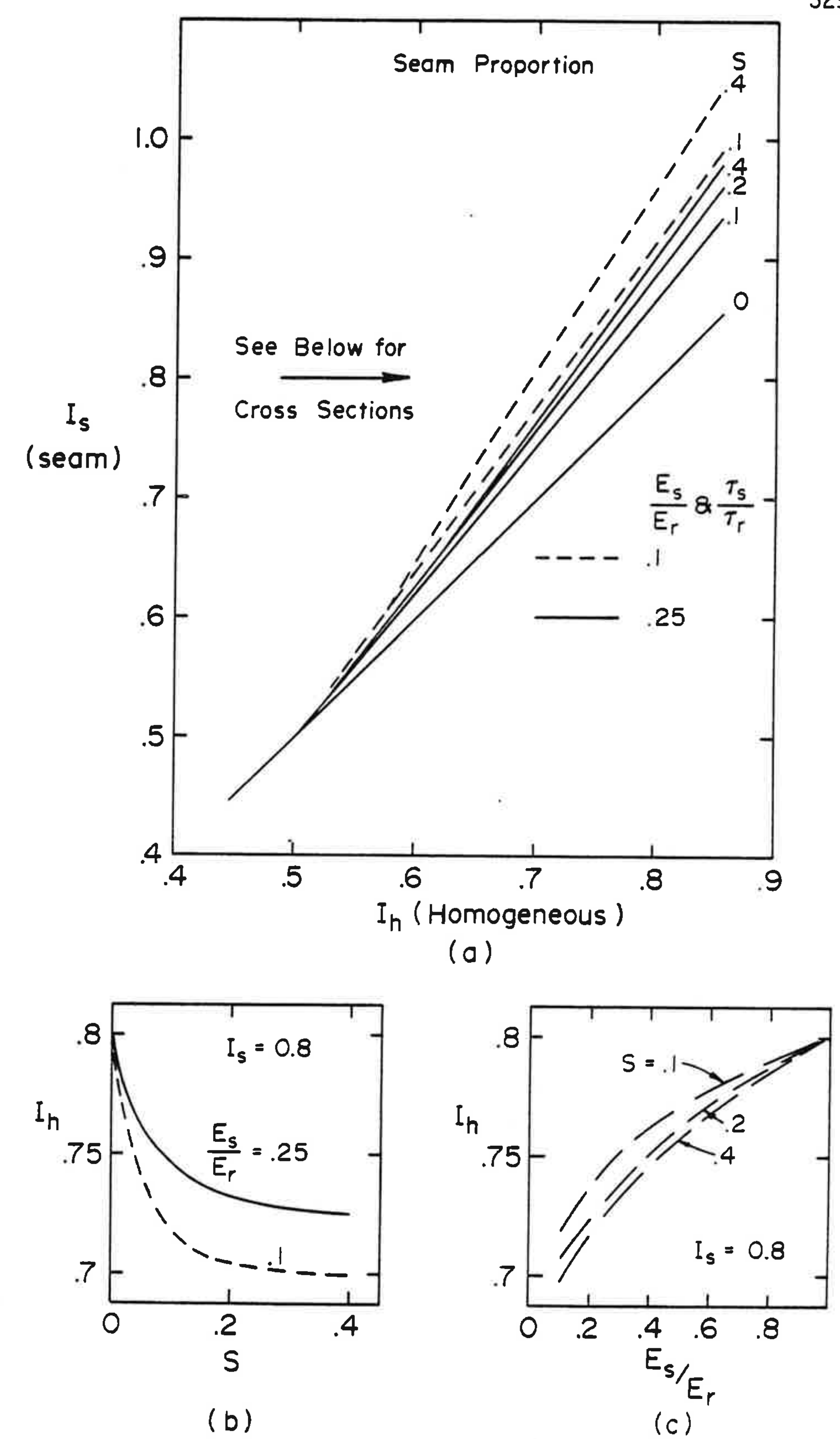


FIGURE C.6 DESIGN CHARTS TO ACCOUNT FOR PRESENCE OF SEAMS
$L/D = 2$, $E_p/E_r = 10$, $E_b/E_r = 1$

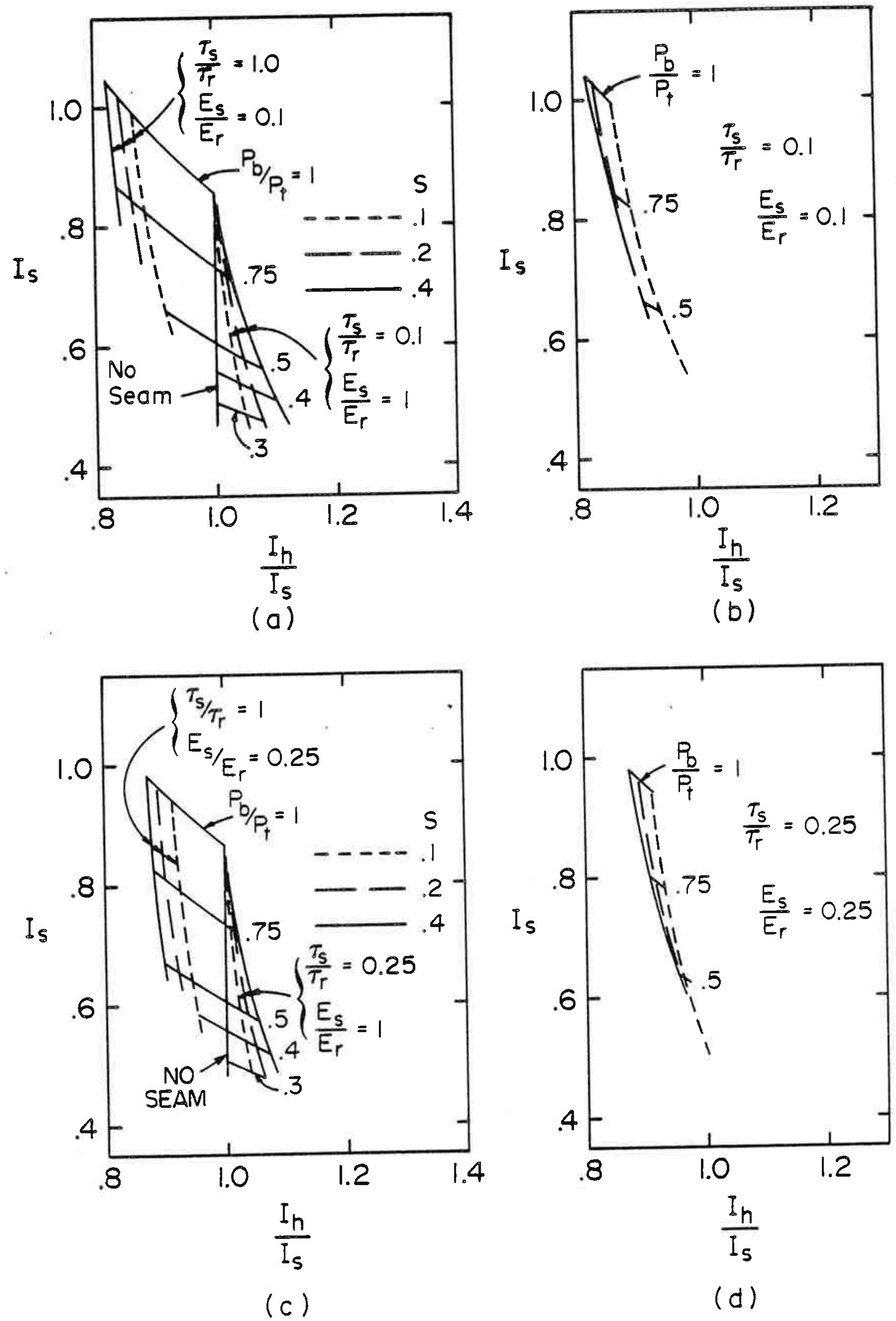


FIGURE C.7 DESIGN CHARTS TO ACCOUNT FOR PRESENCE OF SEAMS
$L/D = 2$, $E_p/E_r = 10$, $E_b/E_r = 1$

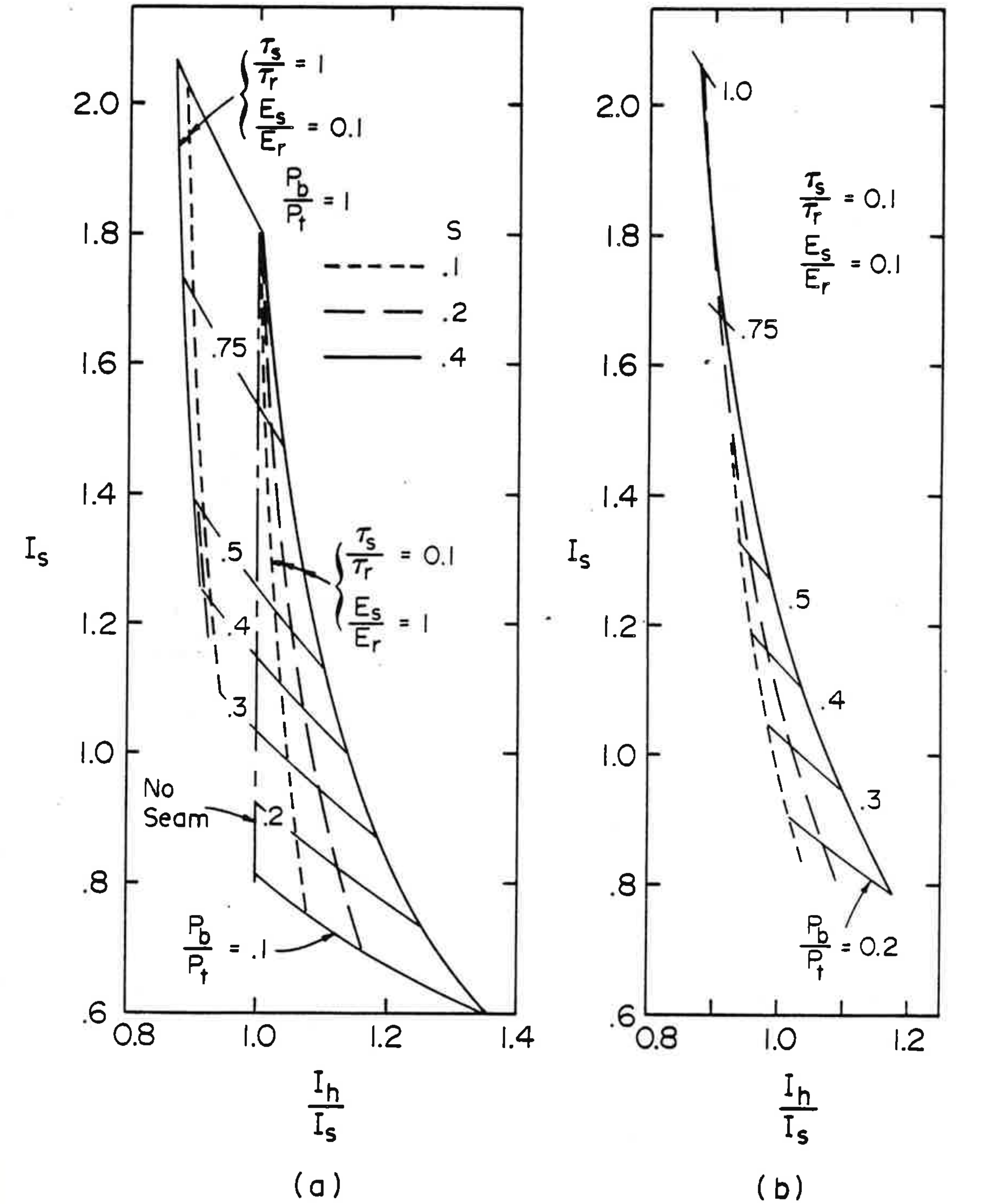
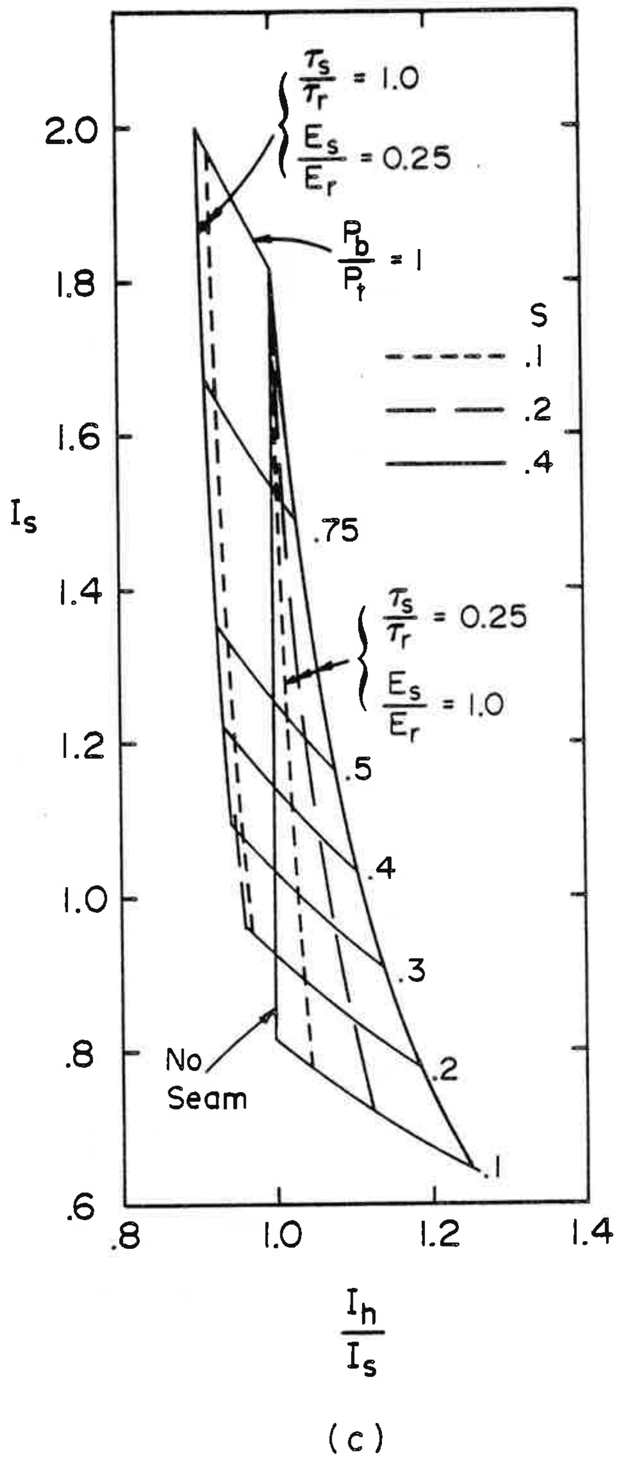


FIGURE C.8a,b DESIGN CHARTS TO ACCOUNT FOR PRESENCE OF SEAMS
$L/D = 10$, $E_p/E_r = 10$, $E_b/E_r = 1$



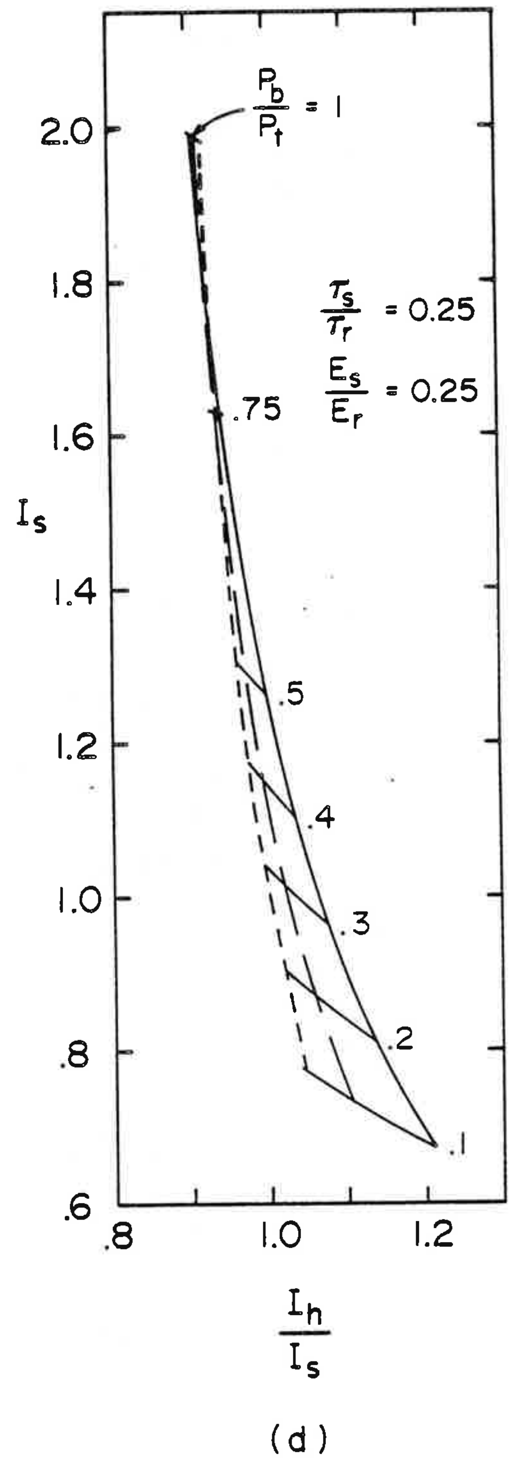
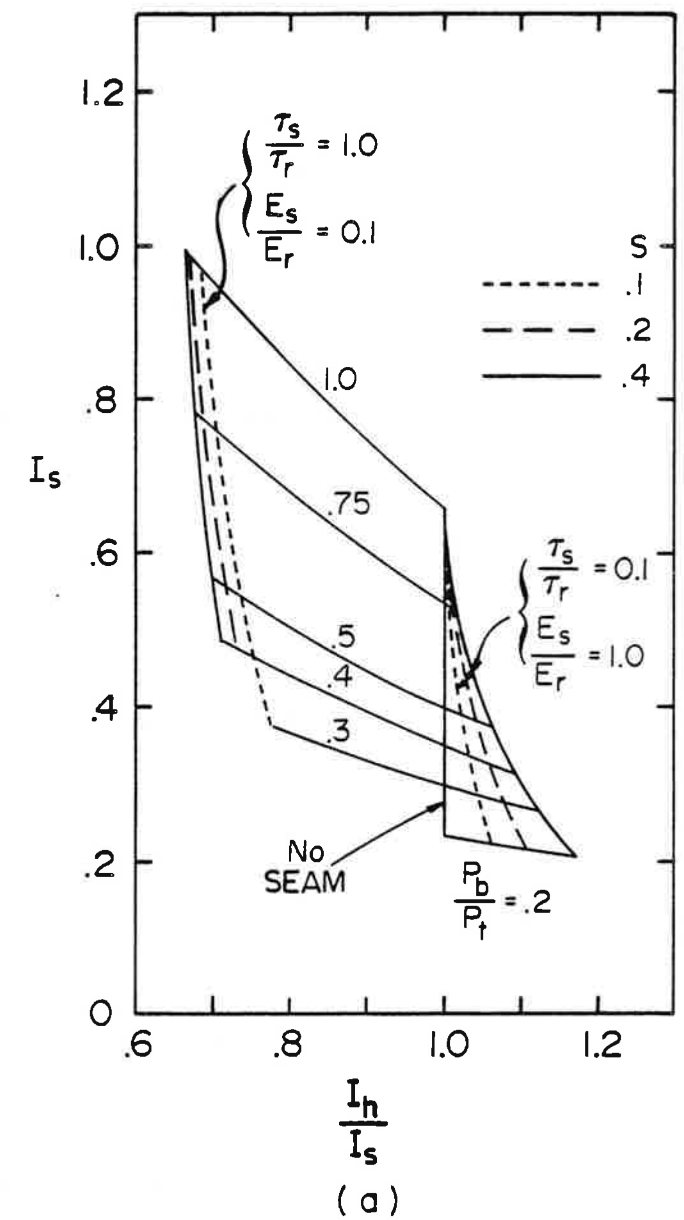


FIGURE C.8c,d DESIGN CHARTS TO ACCOUNT FOR PRESENCE OF SEAMS
L/D = 10, E_p/E_r = 10, E_b/E_r = 1



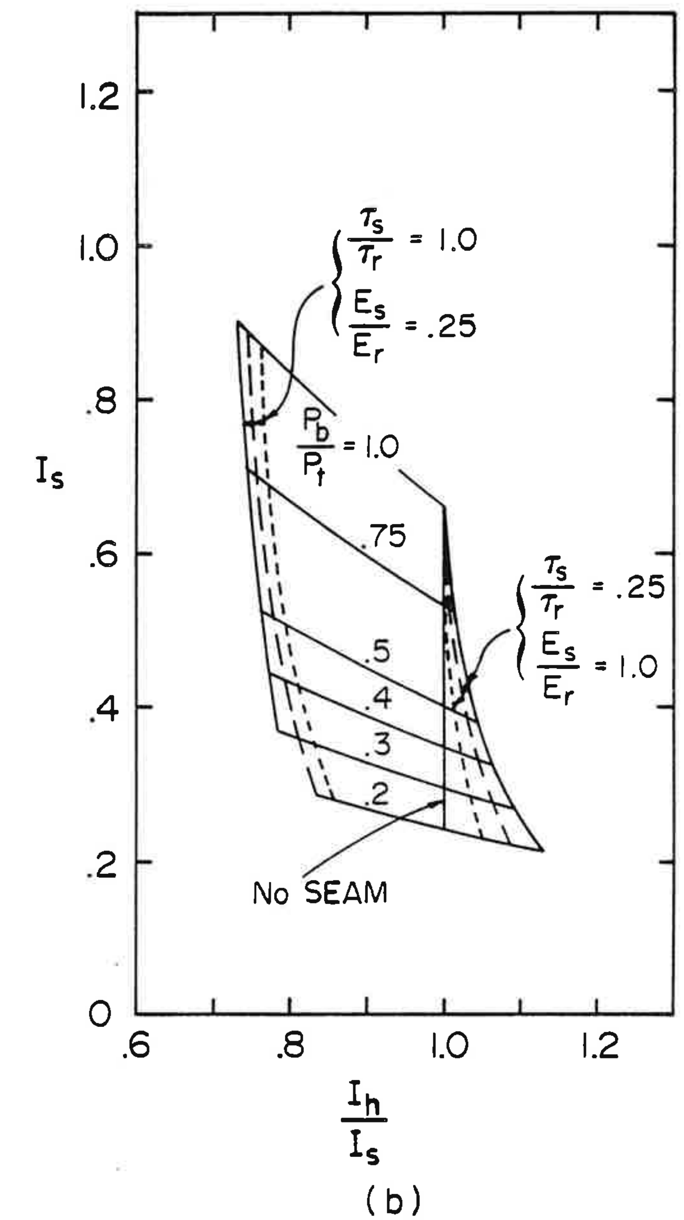


FIGURE C.9 DESIGN CHARTS TO ACCOUNT FOR PRESENCE OF SEAMS
L/D = 10, E_p/E_r = 100, E_b/E_r = 1

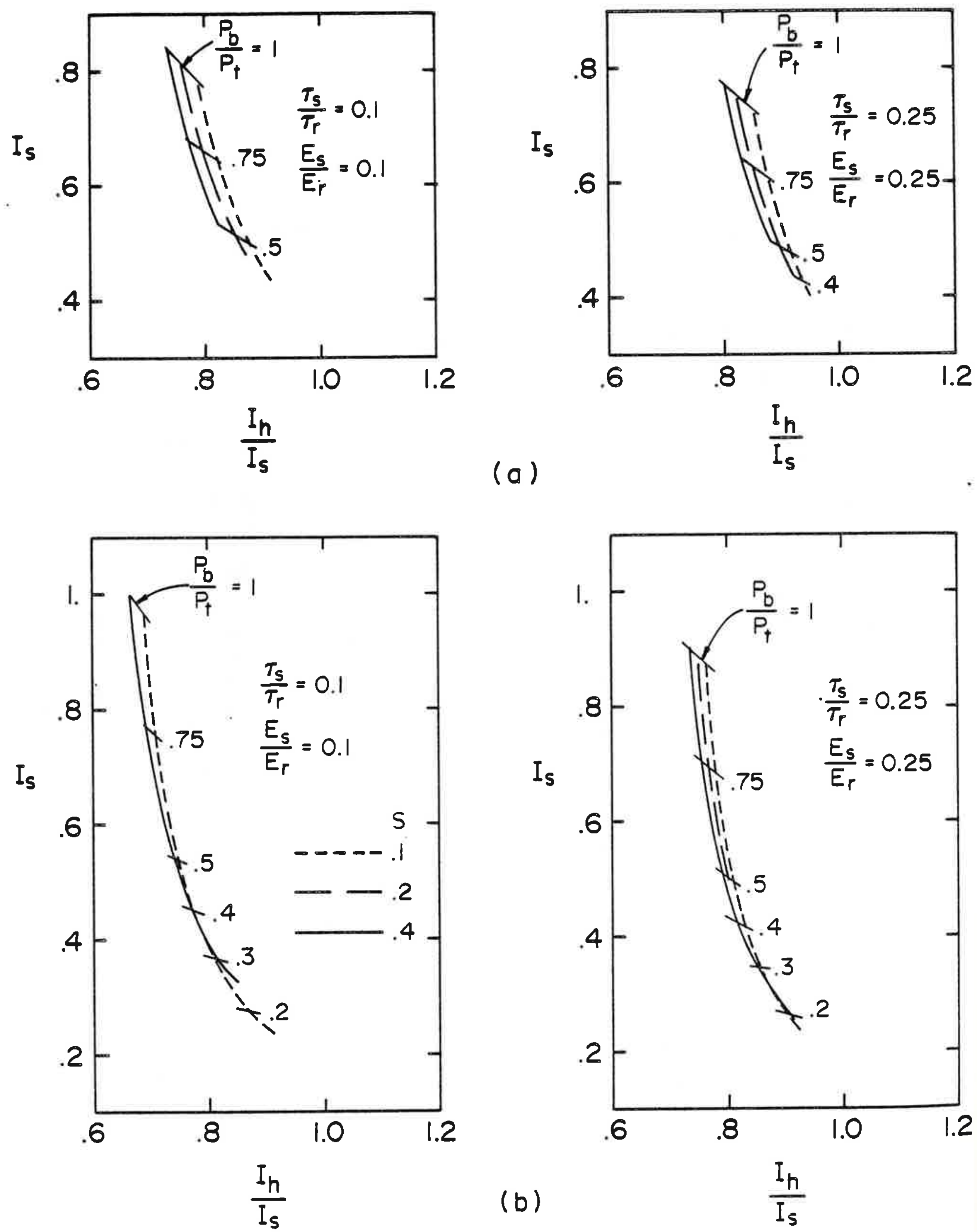


FIGURE C.10 DESIGN CHARTS TO ACCOUNT FOR PRESENCE OF SEAMS
E_p/E_r = 100, (a) L/D = 2, (b) L/D = 10

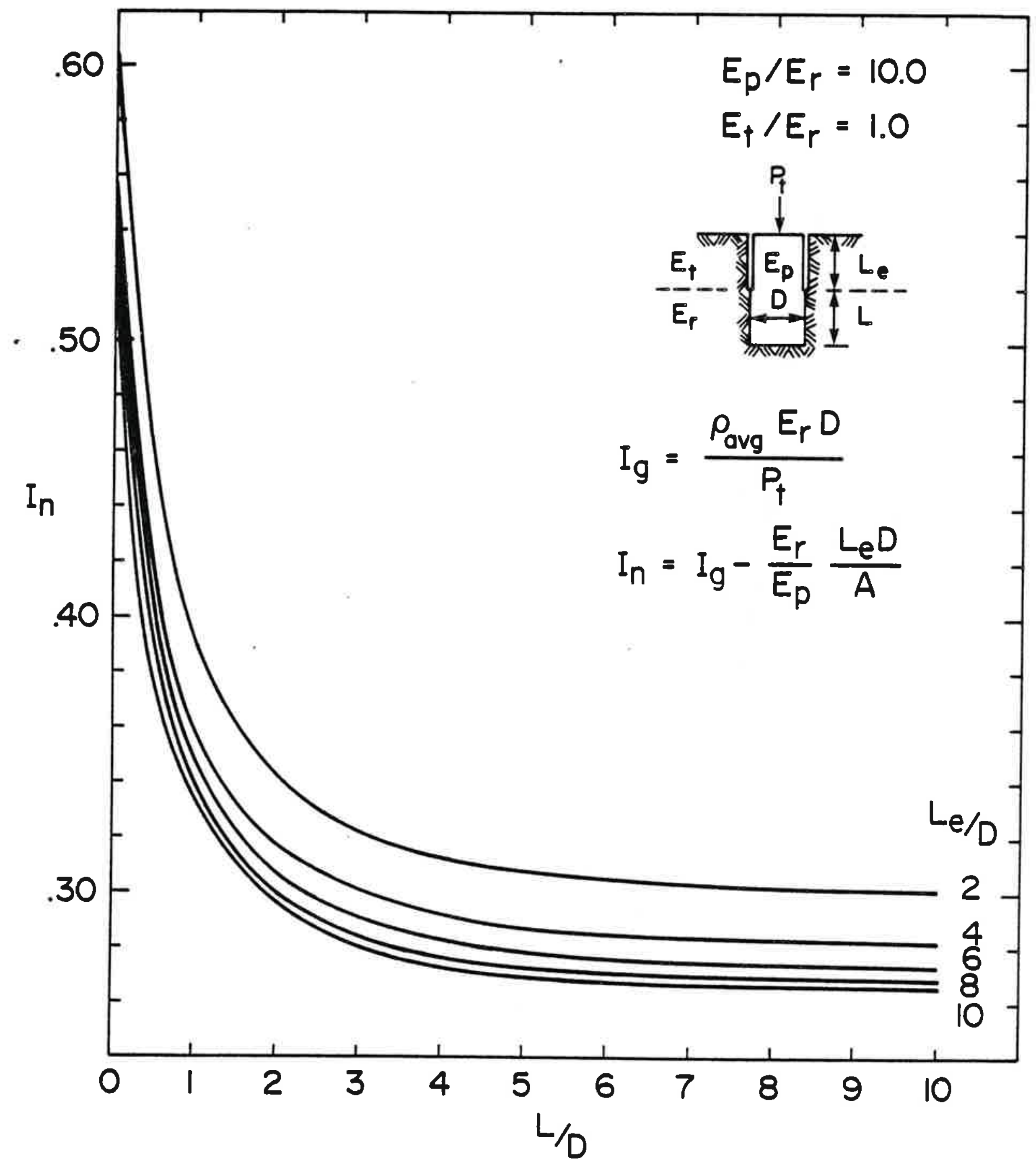


FIGURE C.11 NET SETTLEMENT INFLUENCE FACTORS FOR A RECESSED, COMPLETE SOCKETED PILE

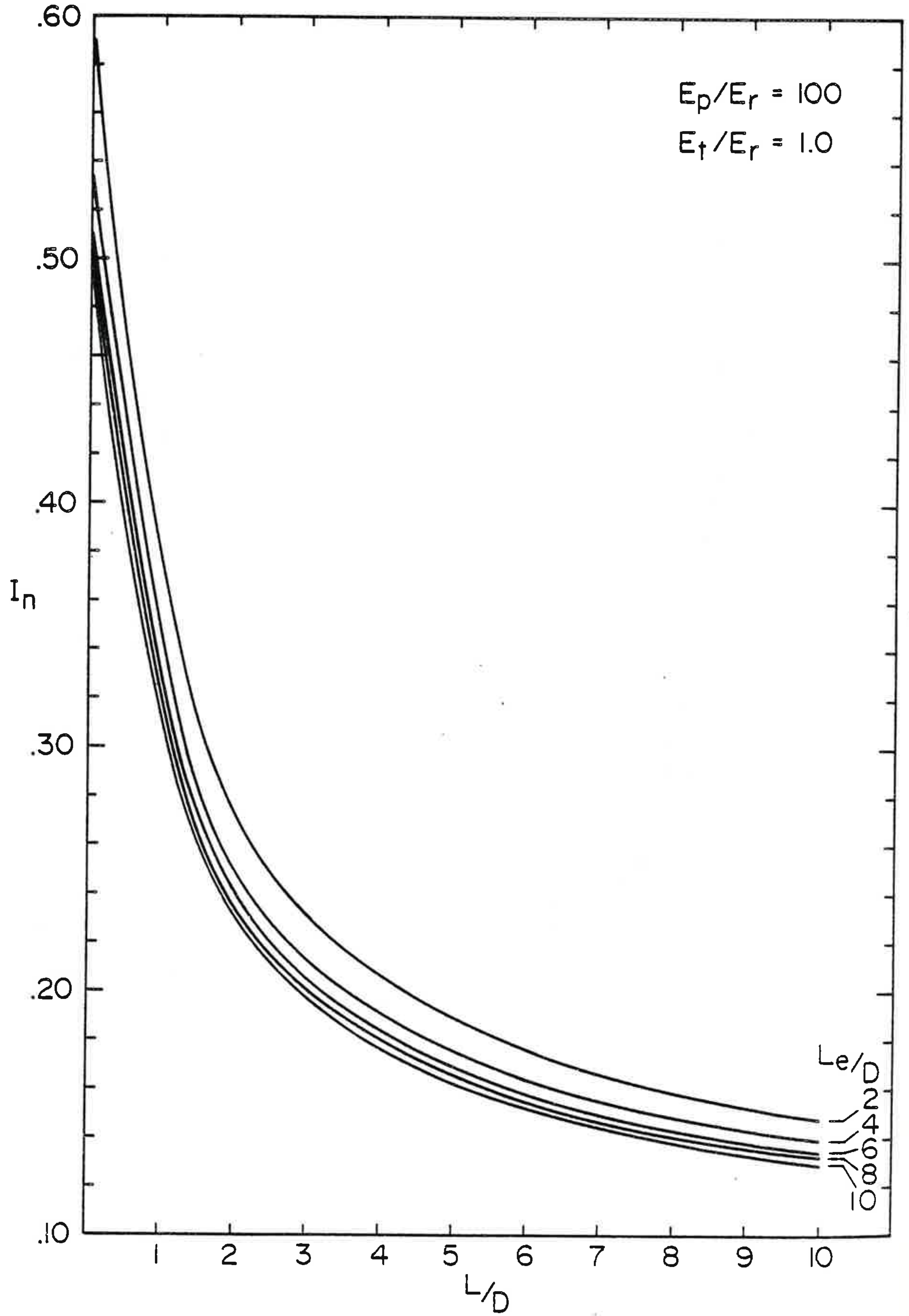


FIGURE C.12 NET SETTLEMENT INFLUENCE FACTORS FOR A RECESSED, COMPLETE SOCKETED PILE

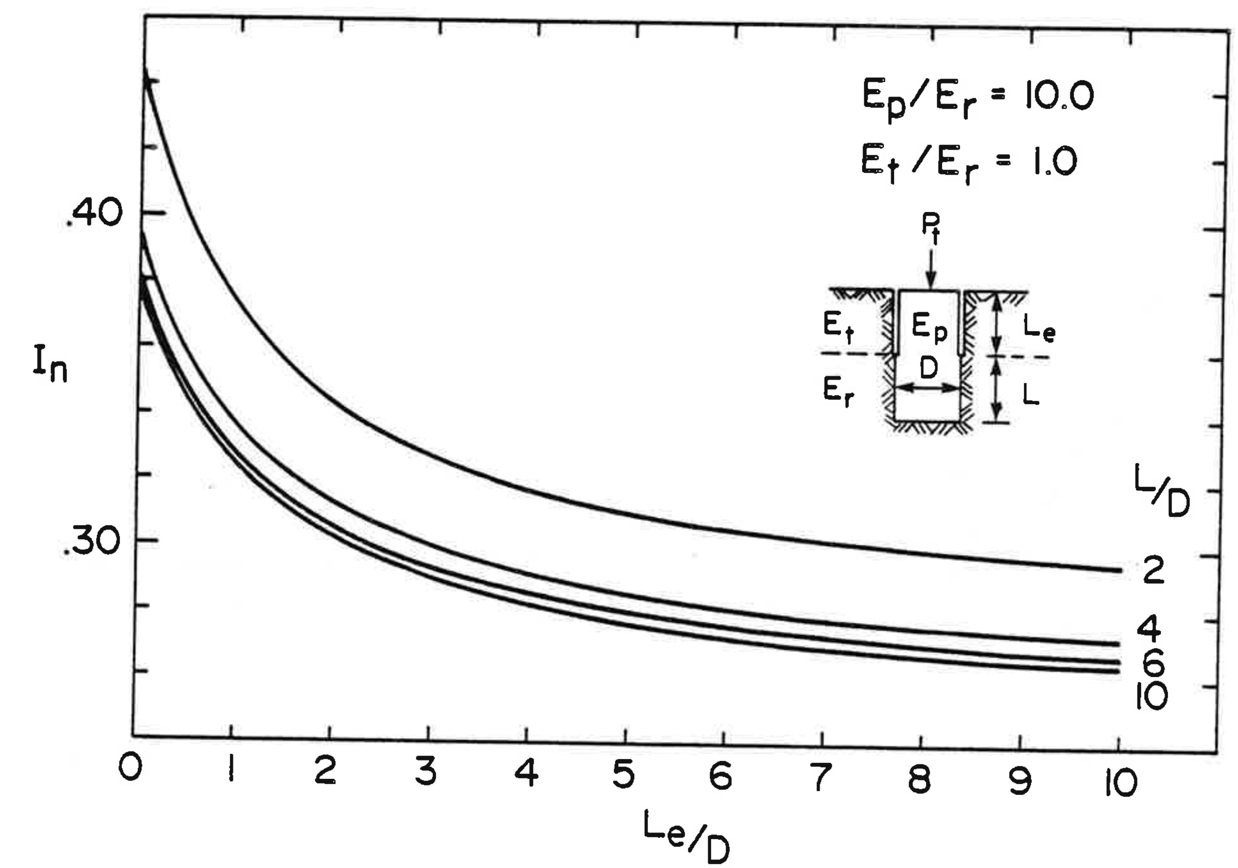


FIGURE C.13 NET SETTLEMENT INFLUENCE FACTORS FOR A RECESSED, COMPLETE SOCKETED PILE

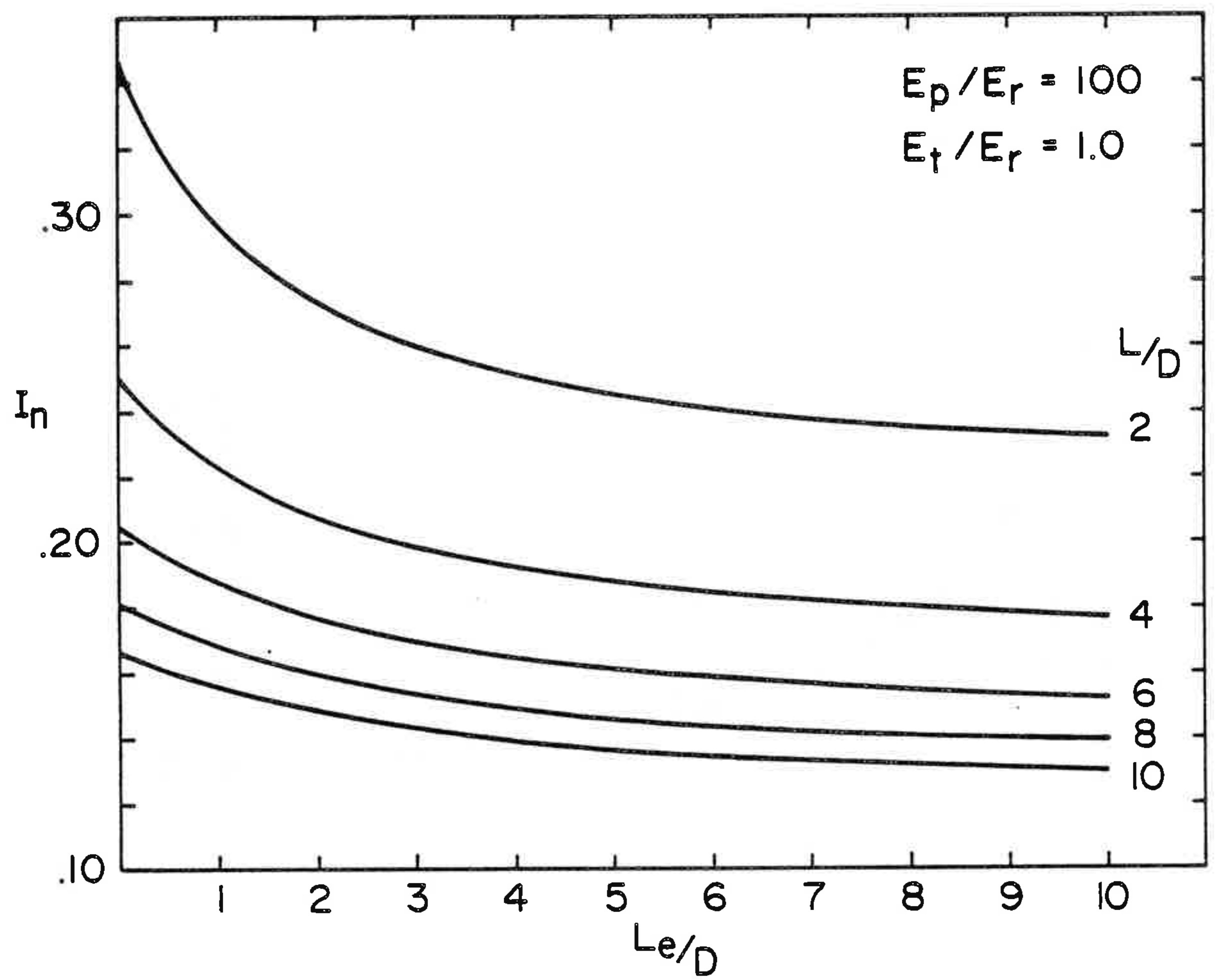
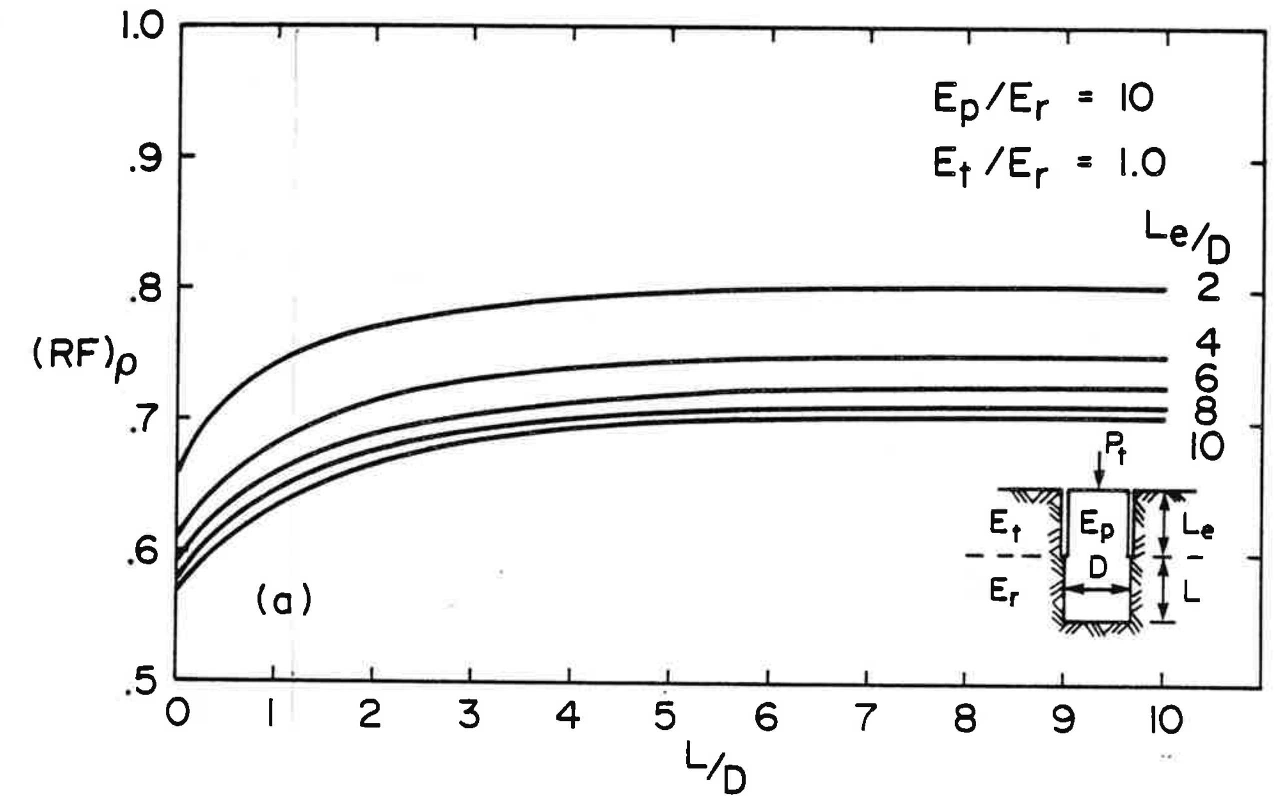


FIGURE C.14 NET SETTLEMENT INFLUENCE FACTORS FOR A RECESSED, COMPLETE SOCKETED PILE



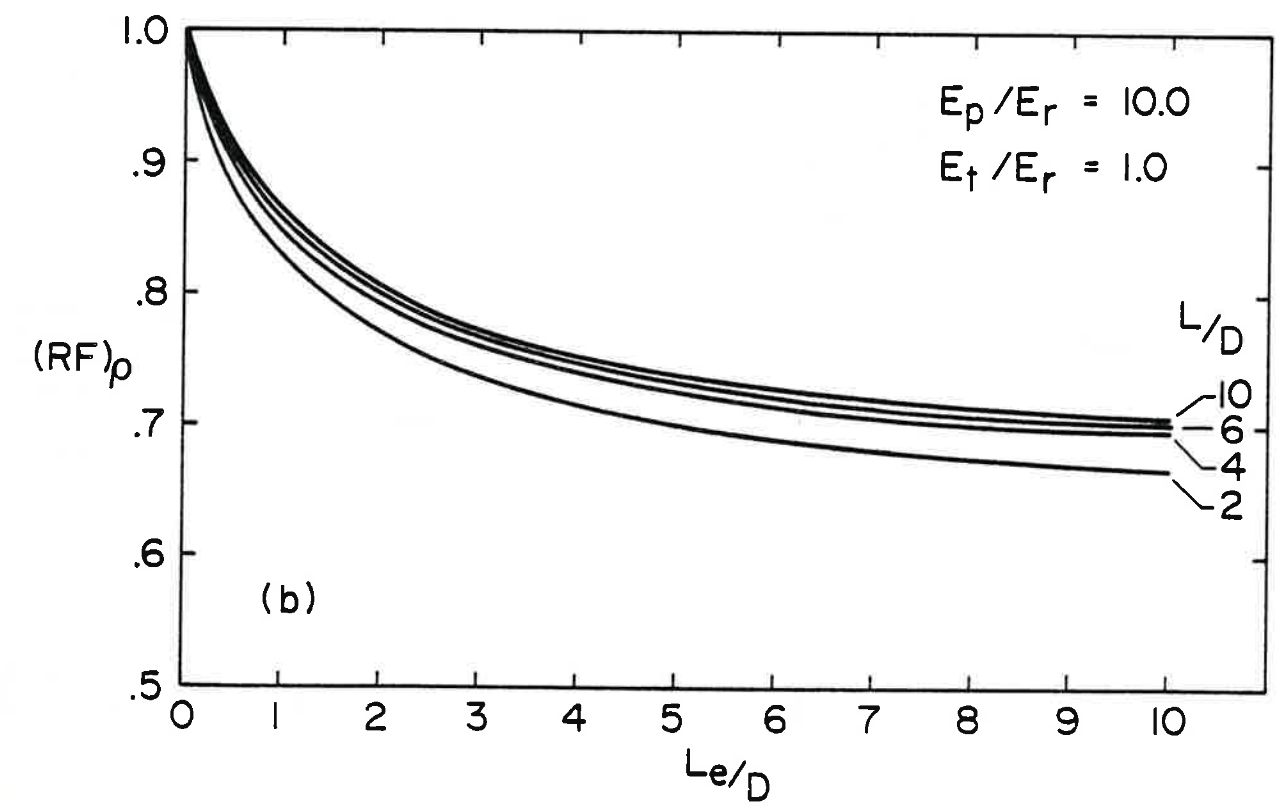


FIGURE C.15 REDUCTION FACTORS FOR A RECESSED SOCKET

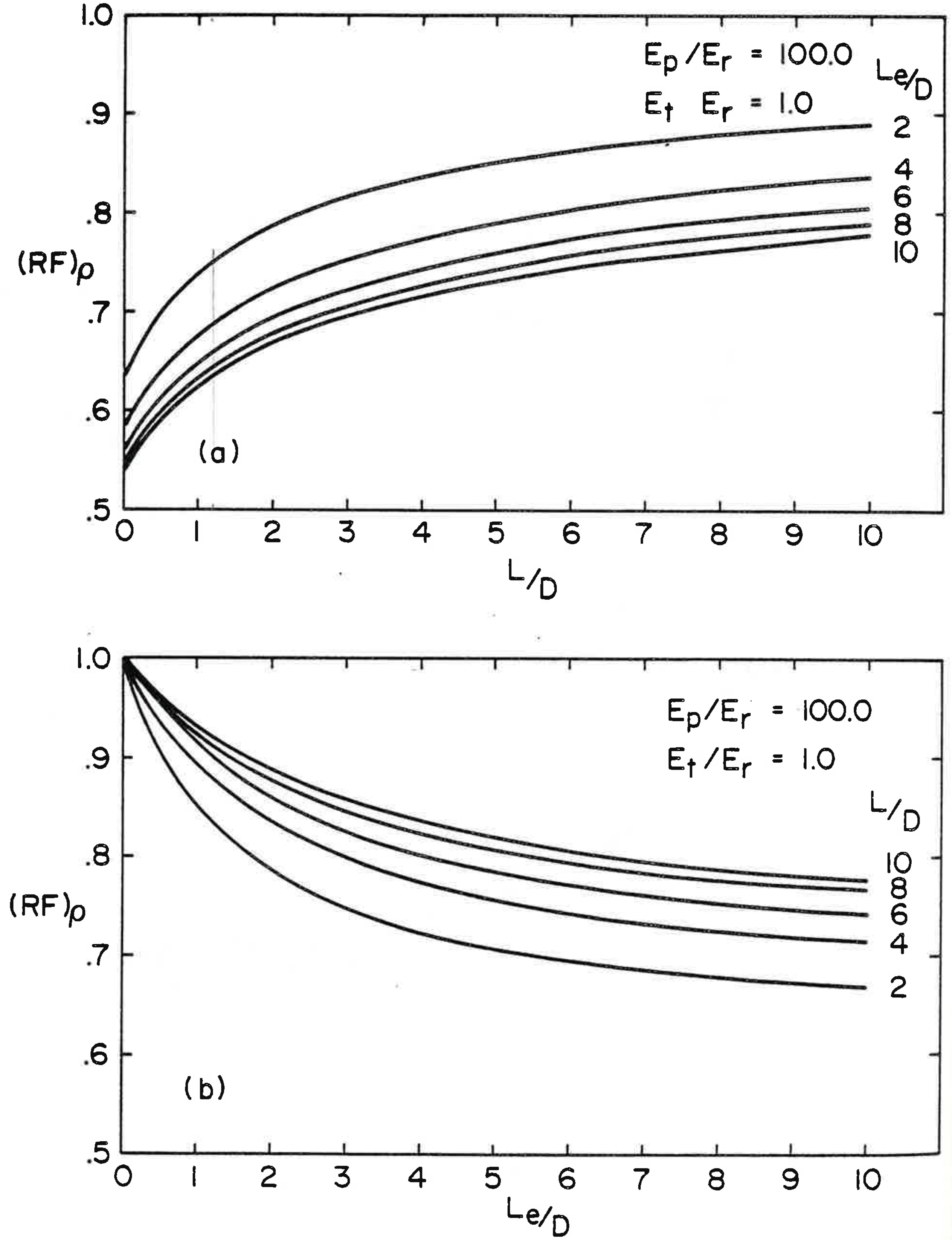


FIGURE C.16a,b REDUCTION FACTORS FOR A RECESSED SOCKET

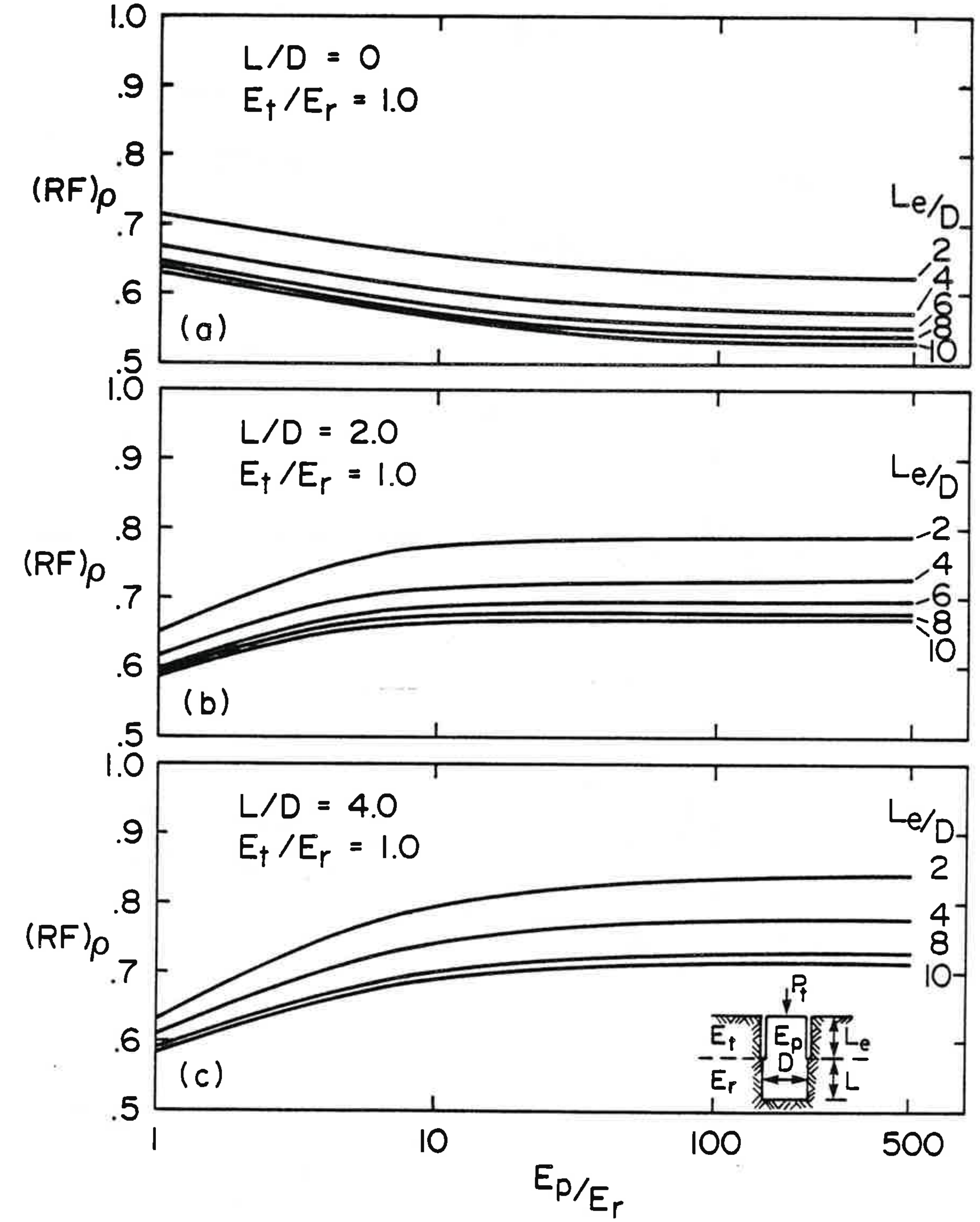


FIGURE C.17a,b,c REDUCTION FACTORS FOR A RECESSED SOCKET

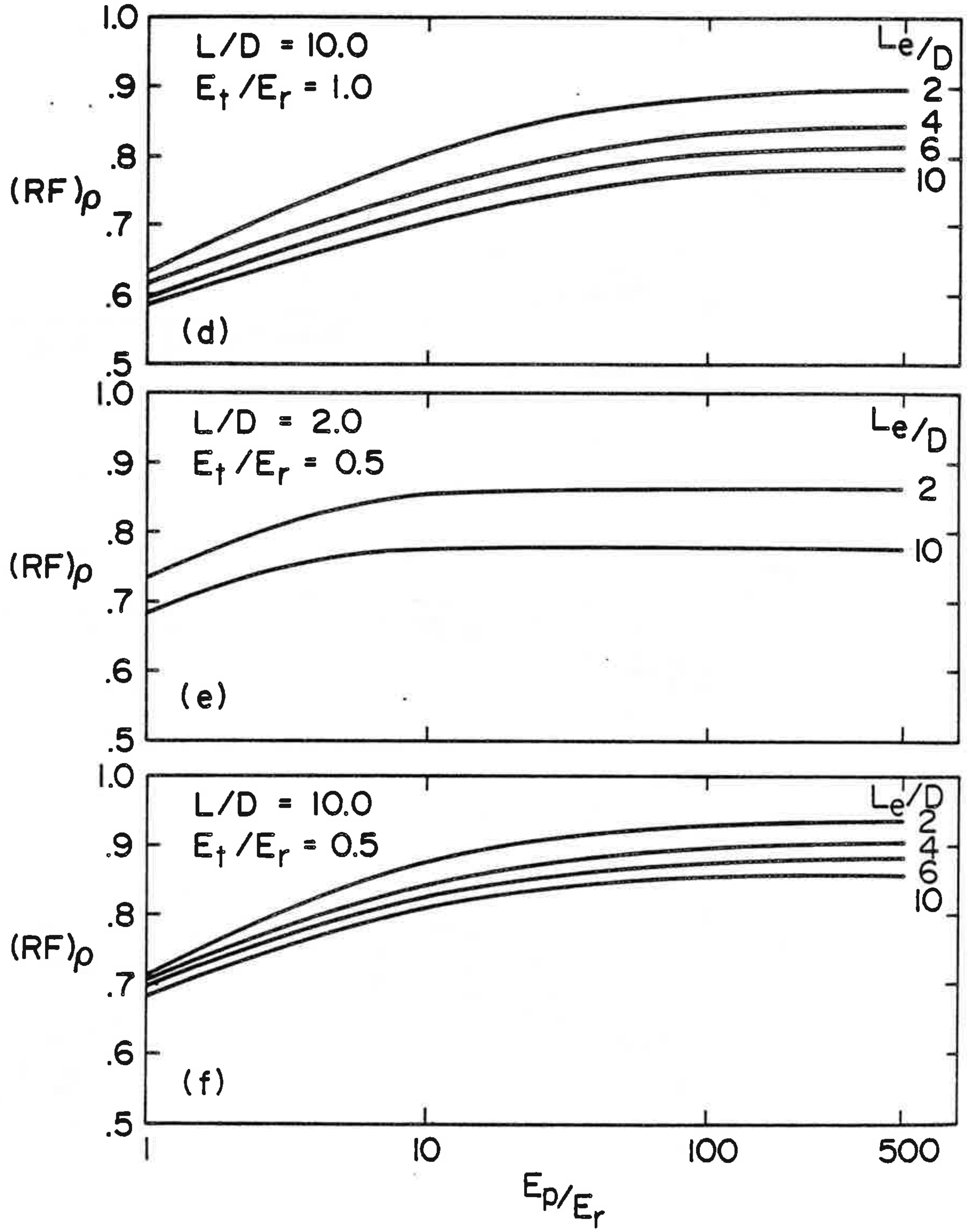


FIGURE C.17d,e,f REDUCTION FACTORS FOR A RECESSED SOCKET

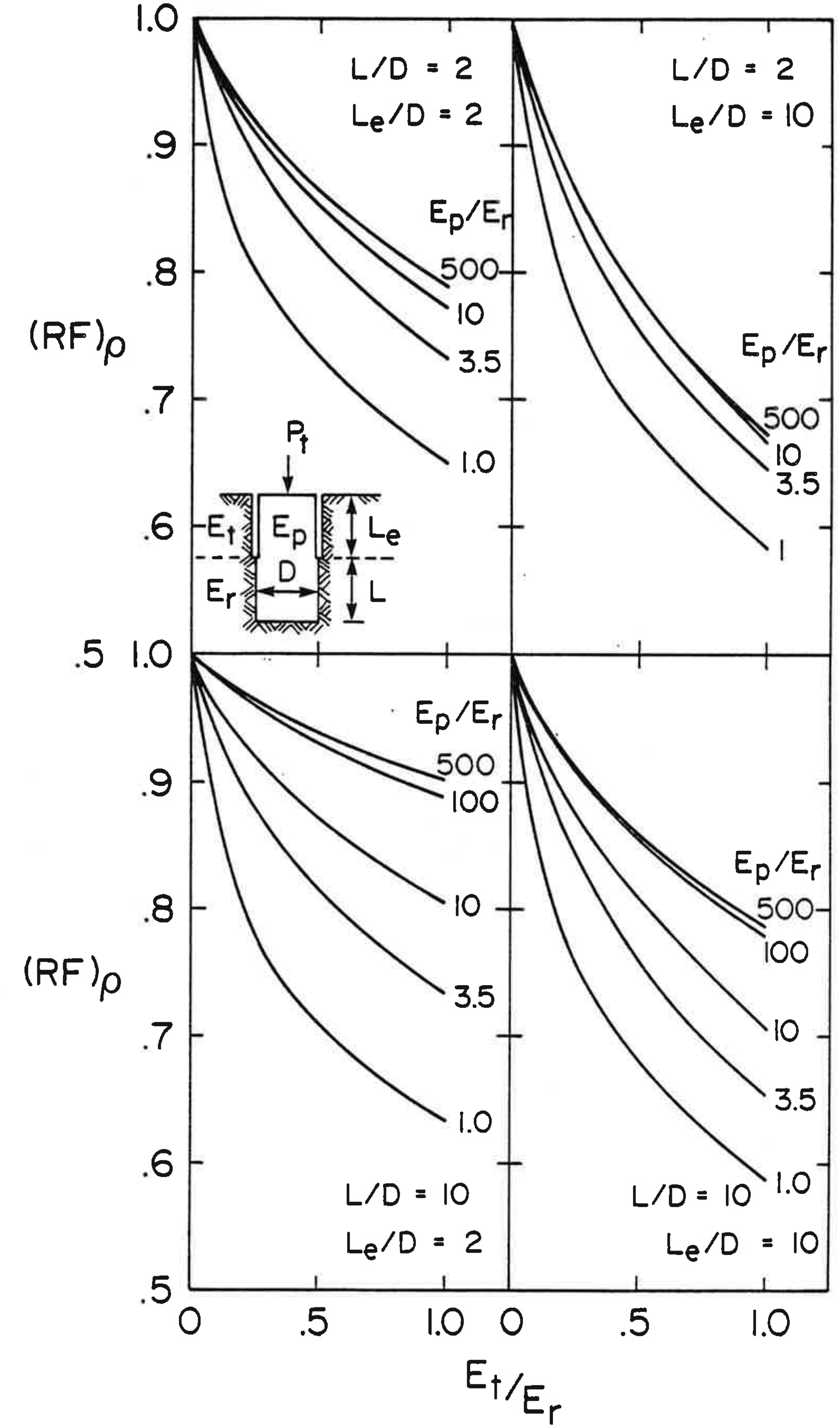


FIGURE C.18 VARIATION IN SETTLEMENT REDUCTION FACTOR WITH MODULUS RATIO

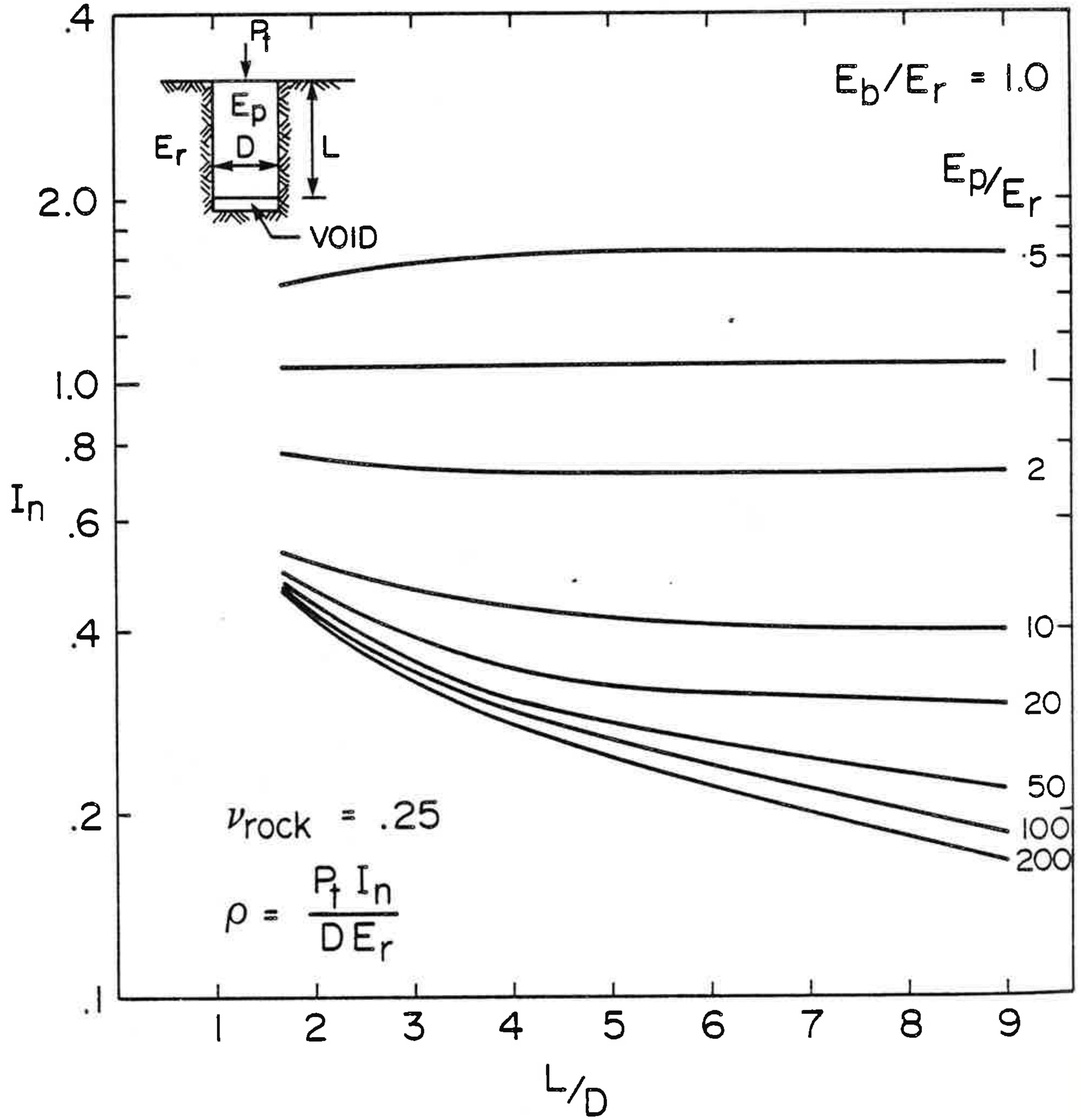


FIGURE C.19 SETTLEMENT INFLUENCE FACTORS FOR A SIDESHEAR (ONLY) SOCKETED PILE (MODIFIED FROM PELLS AND TURNER, 1979)

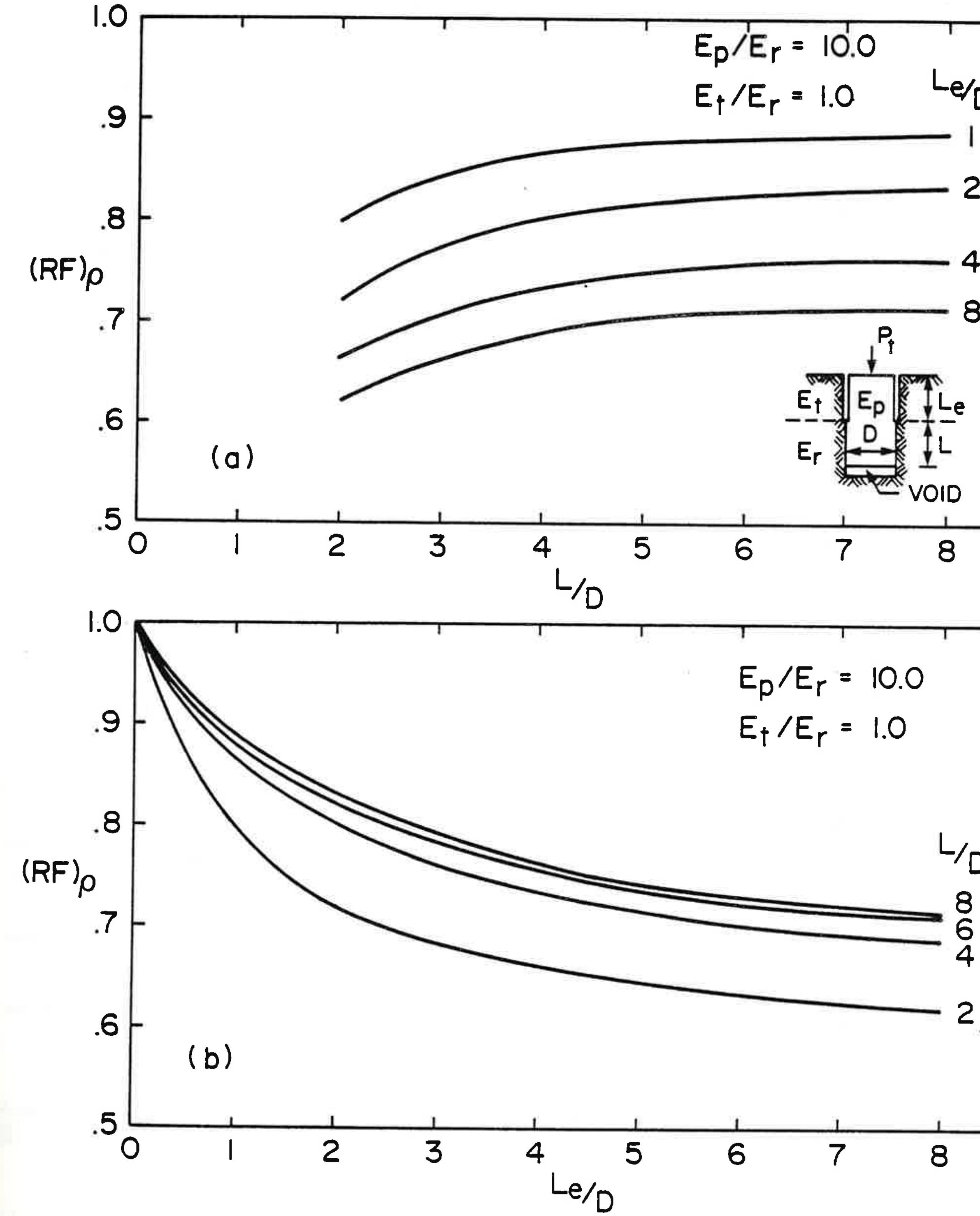


FIGURE C.20 SETTLEMENT REDUCTION FACTORS FOR A RECESSED, SIDESHEAR (ONLY) SOCKETED PILE (MODIFIED FROM PELLS AND TURNER, 1979)

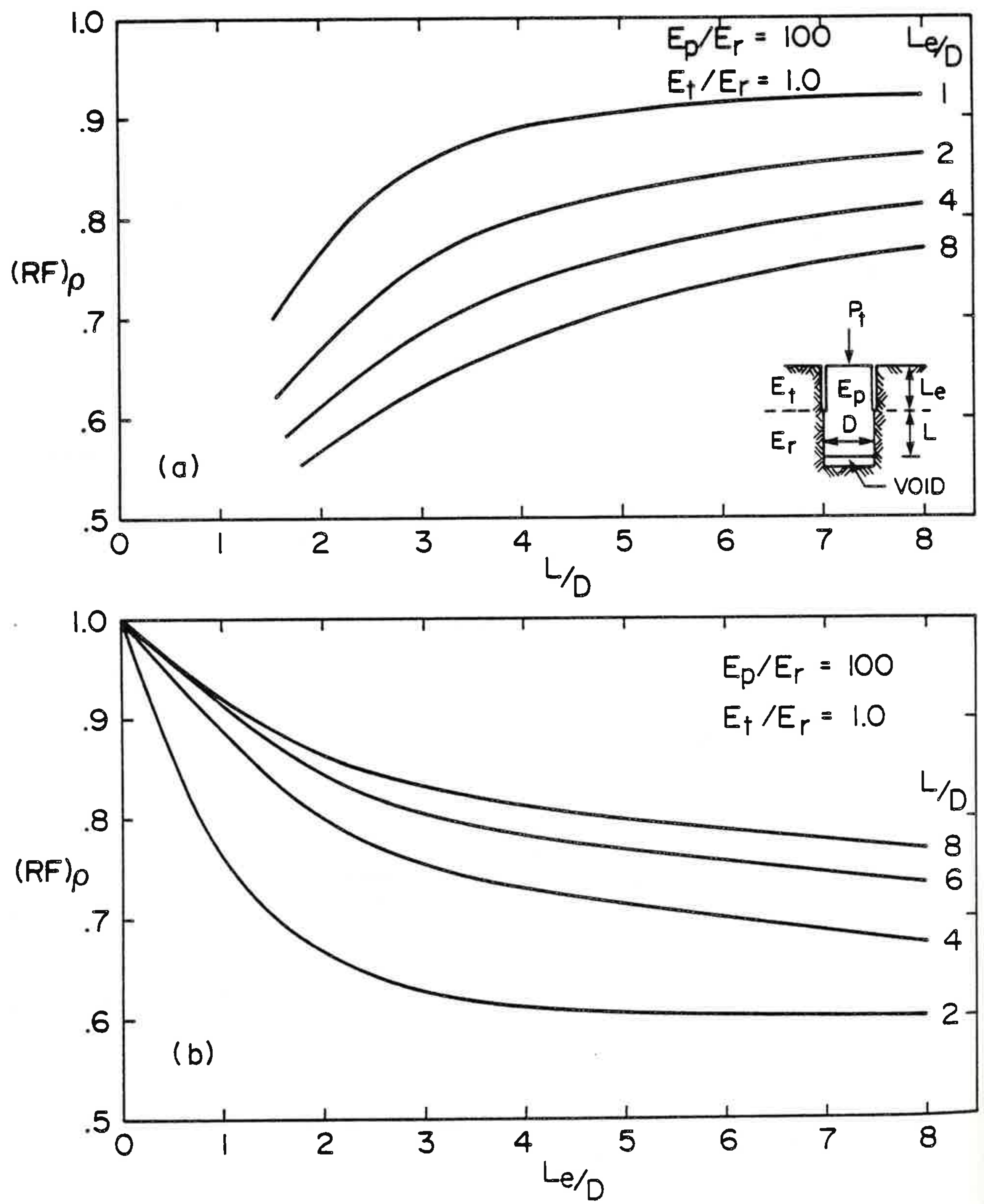


FIGURE C.21 SETTLEMENT REDUCTION FACTORS FOR A RECESSED, SIDESHEAR (ONLY) SOCKETED PILE (MODIFIED FROM PELLS AND TURNER, 1979)

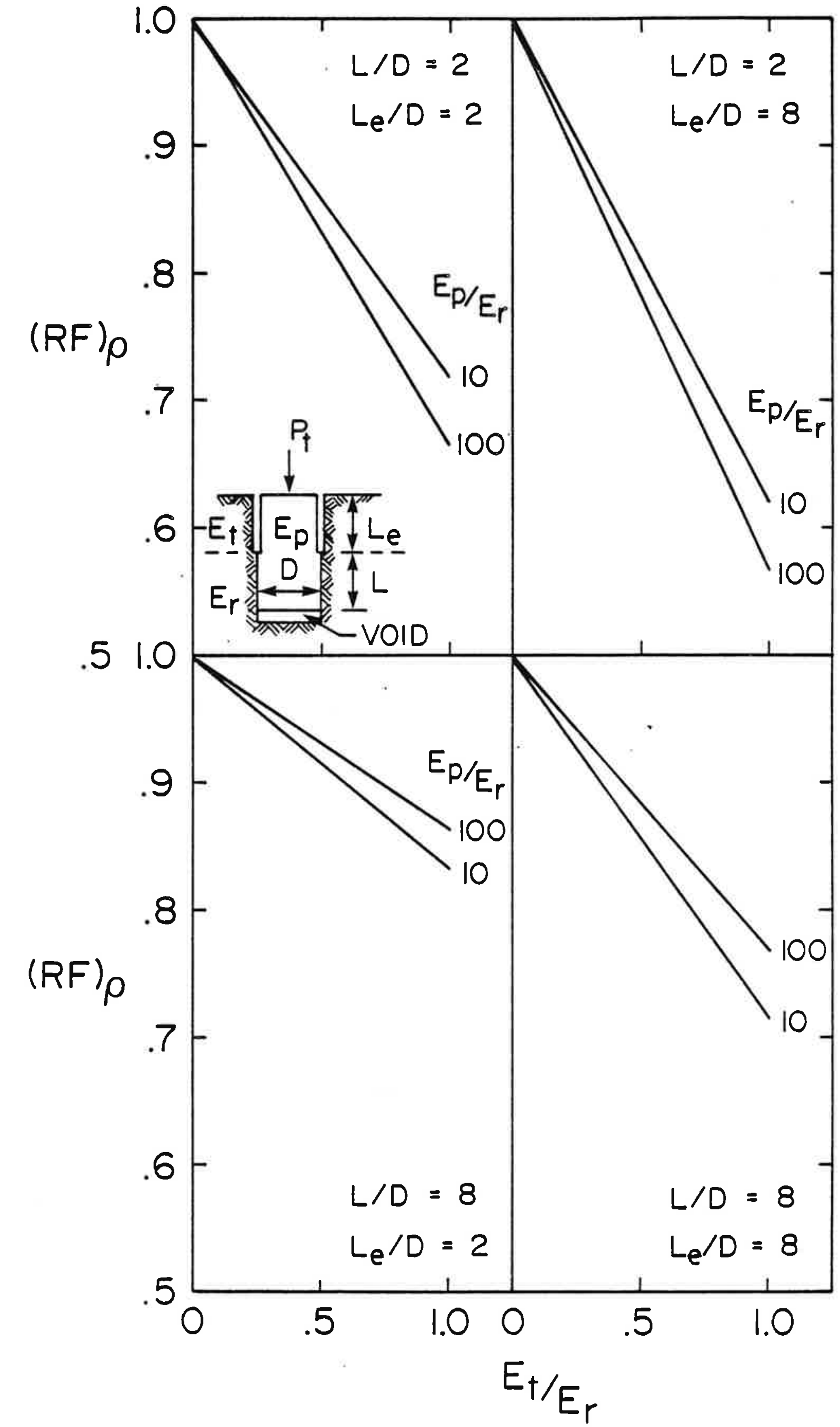


FIGURE C.22 VARIATION IN SETTLEMENT REDUCTION FACTOR WITH MODULUS RATIO E_t/E_r (DEDUCED FROM PELLS AND TURNER, 1979)

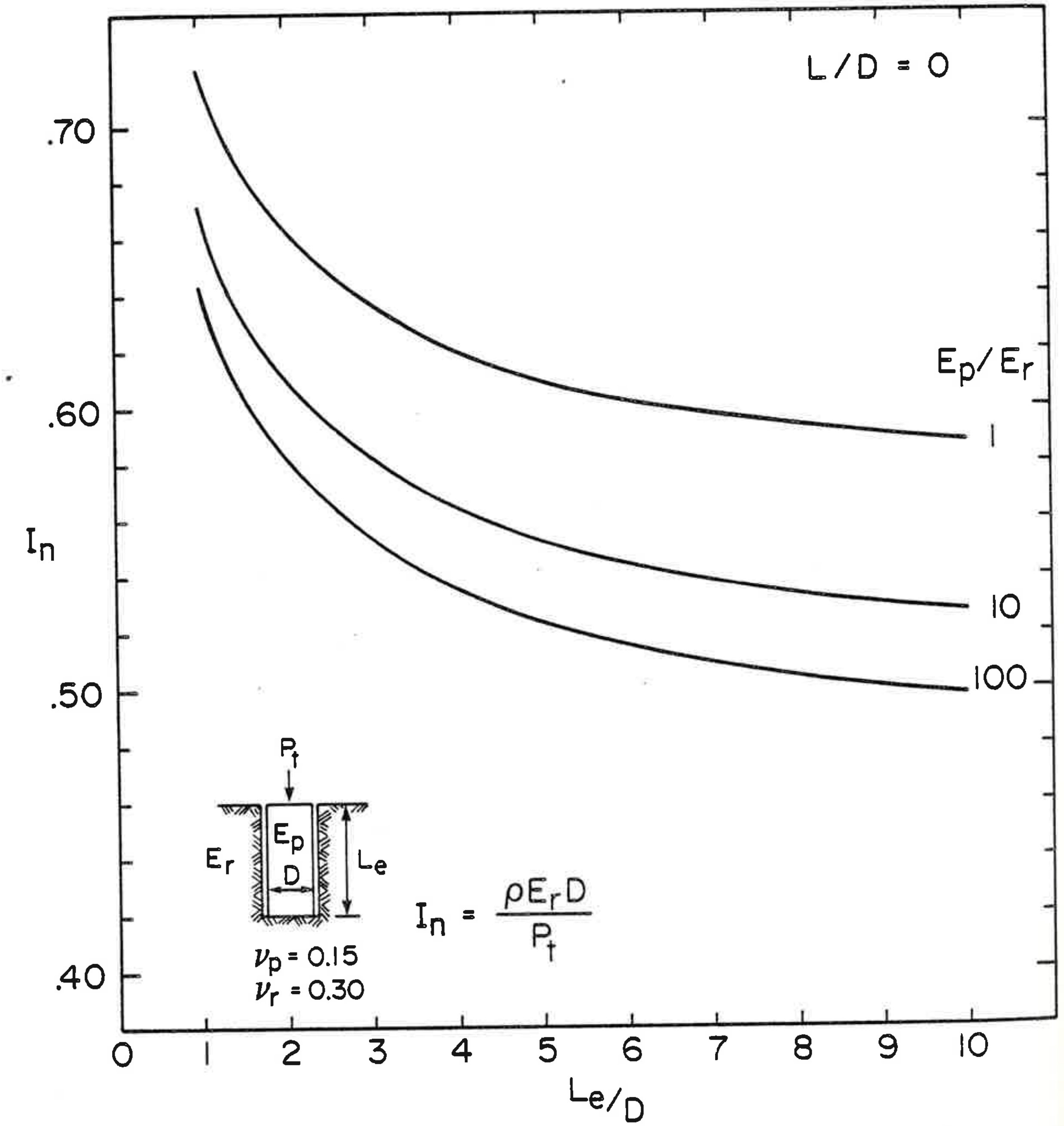


FIGURE C.23 NET SETTLEMENT INFLUENCE FACTORS FOR AN ENDBEARING (ONLY) SOCKETED PILE

APPENDIX D

BACKFIGURED SIDESHEAR RESISTANCE AND

ROCK MASS MODULUS FROM FIELD LOAD TESTS

TABLE D1

A(I) COMPRESSION SOCKETS (DIAMETERS > 350 mm) DIRECT DATA

ROCK TYPE	SOCKET DESIGNATION AND TYPE	SOCKET L (m)	DESCRIPTION D (m)	L/D	ROCK STRENGTH q_u (MPa)	MEASURED AVG. PEAK SIDESHEAR RESISTANCE, τ_f (MPa)	τ_f/q_u (α)	τ_{re}/τ_f	SITE LOCATION	REFERENCE	COMMENTS
Sandstone	D2-S	.90	.71	1.3	6(avg)	.65	.11	1.0	Sydney, N.S.W. Australia	Pells et al. (1980)	
Sandstone	East-C	1.4	.61	2.3	9.3(avg)	>1.93	>.21	NTF	Farmington,	Glos & Briggs	(R)
	West-C	1.47	.61	2.4	8.4(avg)	>2.12	>.25	NTF	N.M.,U.S.A.	(1983)	(R)
MW-Mud-	F1-S	1.0	1.2	.833	3.06	1.05	.34	-	Melbourne,	Johnston &	
stone	F2-S	1.0	1.2	.833	1.93	0.94	.49	-	Vic,Australia	Donald(1979)*	
HW-Mud-stone	S1-S	1.52	.66	2.3	.83	>.56(est .660)	>.67 (est .8)	-	Melbourne, Vic,Australia	Williams (1980)	
	S3-S	2.51	1.17	2.15	.55	.51	.92	1.0			(R)
	S5-S	2.59	1.12	2.31	.59	.485	.82	.99			
	S14-S	.87	.395	2.2	.58	.50	.86	.75			(R)
	S15-S	.87	.395	2.2	.58	.41	.70	.45			(R)
	S16-S	.87	.395	2.2	.58	>.36	>.63	NTF			(R)
MW-Mud	M1-S	2.0	1.22	1.6	2.46	.60	.24	1.0	Melbourne,	Williams (1980)	(B)
stone	M2-S	2.0	1.3	1.5	2.30	.64	.28	.98	Vic,Australia		(R)
	M3-S	2.0	1.23	1.6	2.30	.71	.31	1.0			
	M4-S	2.0	1.35	1.5	2.34	.62	.26	.95			(R&B)
	M8-C	1.8	.66	2.73	2.0	.967	.48	.83			Failure in side-shear
	M9-C	4.2	.66	6.36	2.3	.89	.39	1.0			"
	M10-C	7.8	.66	11.82	3.4	>.55	>.17	NTF			
SW-Mudstone	ES-S	1.52	1.09	1.4	7.2	>.86	>.12	NTF	Melbourne,	Williams (1980)	~60 joints/m
MW-SW Mudstone	WG/303-S	2.0	1.58	1.27	3.49	.85	.24	.95	Vic,Australia		

- S Sideshear socketed pile
- C Complete socketed pile
SW - Slightly weathered
MW - Moderately weathered
HW - Highly weathered

(R) denotes a roughened (R4, Pells et al., 1980) socket
(B) denotes a socket constructed and/or cast under bentonite
NTF denotes a load test not carried to failure
* data obtained from Williams and Pells (1981)
† data obtained from Horvath (1978)
τ_{re} measured average residual sideshear resistance

A(I) COMPRESSION SOCKETS (DIAMETERS > 350 mm) DIRECT DATA

ROCK TYPE	SOCKET DESIGNATION AND TYPE	SOCKET L (m)	DESCRIPTION D (m)	L/D	ROCK STRENGTH q_u (MPa)	MEASURED AVG. PEAK SIDESHEAR RESISTANCE, τ_f (MPa)	τ_f/q_u (α)	τ_{re}/τ_f	SITE LOCATION	REFERENCE	COMMENTS
Shale	1-C	2.9	.61	4.8	62-110	>.932	>.008-.015	NTF	Ottawa, Can.	Vogan (1977)[†]	
Shale	P1-S	1.37	.71	1.9	6.75(avg)	>1.54	>.23	NTF	Burlington, Ont., Canada	Horvath et al. (1983)	
	P2-C	1.37	.71	1.9	6.75(avg)	>1.45	>.22	NTF			
	P3-S	1.37	.71	1.9	6.75(avg)	2.13	.32	.96			(R)
	P4-C	1.37	.71	1.9	6.75(avg)	>1.8	.27	NTF			(R)
	P5-C	1.37	.71	1.9	6.75(avg)	>1.59	.24	NTF			
	P6B-S	1.37	.71	1.9	6.75(avg)	>2.48	>.37	NTF			(R)
Shale	2-S	1.0	.4	2.5	30(avg)	>2.69	>.08	-	Redfern, NSW Australia	Ground Test Pty. Ltd. (1976)*	- failure of conc. pile cap
Shale	1	3.35	.685	4.9	3.06	>1.1(@32mmdispl)	>.36	NTF	Perth, W.A., Australia	Millar (1976)*	(B)
	TP1	5.15	.66	7.8	.49(avg)	>.30(@30mmdispl)	>.61	NTF			
	TP2	8.9	.787	11.3	2.68	>.5(@4mm)est .72	.27	NTF			
Shale	1-S	.91	.635	1.43	15.2(avg)	>1.0	>.067	NTF	Toronto, Ont. Canada	MTC (1968)	see Horvath et al. (1980)
Shale	1-S	.9	.48	1.9	35-62	>3.64	>.09	NTF	Ottawa, Can.	Seychuk (1970)	
Shale	P3-S	.7	.45	1.56	34	2.5	.07	-	Sydney, NSW, Australia	Thorne (1980)	
	P6-S	1.3	.90	1.44	21	>1.1	>.05	NTF			
Claystone	1-C	2.0	.75	2.7	5.5(avg)	<1.3 assuming all load taken by side resistance	<.24	-	Eraring Power Stn., NSW, Australia	Elec. Comm. of NSW (undated)	base of socket suspected of containing debris
Siltstone	A1-C	4.9	1.2	4.1	1.0	>.28	>.28	NTF	Channel Is. N. Terr. Aust.	Sales (1983)	
Chalk	1-S	8.0	1.05	7.6	1.036(avg)	>.190	>.18	NTF	Littlebrook, U.K.	Mallard & Ballantyne(1976)	see also Buttling (1976)(B)

A(I) COMPRESSION SOCKETS (DIAMETERS > 350 mm) DIRECT DATA

ROCK TYPE	SOCKET DESIGNATION AND TYPE	SOCKET L (m)	DESCRIPTION D (m)	L/D	ROCK STRENGTH q_u (MPa)	MEASURED AVG. PEAK SIDESHEAR RESISTANCE, τ_f (MPa)	τ_f/q_u (α)	τ_{re}/τ_f	SITE LOCATION	REFERENCE	COMMENTS
HW-Diabase	X-S	12	.615	19.5	.412(avg)	.122	.26	~1.0	Johannesburg, S.A.	Webb (1976)	
Andesite	1-S	.56	.46	1.2	10.4	1.1	.105	~1.0	Canada	Rosenberg & Journeaux(1976)	-rock is fractured and sheared
Schist	-S	9.45	.61	16.2	40	>>.37	>>.01	NTF	Philadelphia, PA, U.S.A.	Koutsoftas (1981)	
Limestone	3.83	10.7	.762	14	3.83	>.38	>.10	NTF	Miami, Fla., U.S.A.	Gupton et al. (1982)	
Limestone	6.5	5.49	.762	7.2	6.5	>.78	>.12	NTF	Singer Is., Fla., U.S.A.	Gupton et al. (1982)	

A(II) COMPRESSION SOCKETS (DIAMETERS < 350 mm) DIRECT DATA

ROCK TYPE	SOCKET DESIGNATION AND TYPE	SOCKET DESCRIPTION L (m)	D (m)	L/D	ROCK STRENGTH q_u (MPa)	MEASURED AVG. PEAK SIDESHEAR RESISTANCE, τ_f (MPa)	τ_f/q_u (α)	τ_{re}/τ_f	SITE LOCATION	REFERENCE	COMMENTS
Sandstone	A1-S	.96	.075	12.8	6.0(avg)	.82	.14	1.0	Sydney, N.S.W. Australia	Pells et al. (1980)	
	A2-S	.92	.21	4.4		1.12	.19	.91			
	A3-S	.40	.315	1.3		1.41	.24	1.0			
	A4-S	1.37	.210	6.5		.89	.15	.92			
	A5-S	.518	.210	2.5		.81	.14	.80			
	B1-S	.46	.16	2.9		.94	.16	.44			
	B2-S	.45	.16	2.8		1.16	.19	.65			
	B3-S	.52	.315	1.7		.89	.15	.89			
	B4-S	.33	.255	1.3		1.65	.28	.67			
	B5-S	.62	.16	3.9		1.13	.19	.53			
	C3-S	.45	.31	1.45		.48	.08	1.0			auger smear on wall
	C4-S	.60	.21	2.9		1.20	.20	.88			
	C5-S	.70	.21	3.3		1.17	.20	.86			
	D1-S	1.3	.29	4.5		.32	.05	1.0			(B)
	E3-S	1.27	.29	1.6		.68	.11	1.0			
	X1-S	.184	.160	1.2	14.0(avg)	2.3	.16	.61			
	X2-S	.160	.064	2.5		2.6	.19	.65			
	X4-S	.130	.084	1.5		3.46	.25	.85			
	X5-S	.330	.084	3.9		2.43	.17	.50			
	X6-S	.385	.084	4.6		2.59	.19	.61			
	X9-S	.255	.091	2.8		.15	.01	1.0			(B)
	X7-S	.112	.16	.75		5.22	.37	.44			
	X8-S	.160	.16	1.0		2.69	.19	.29			
	C2-S	.340	.210	1.6	30.0(avg)	4.75	.16	.59			

A(II) COMPRESSION SOCKETS (DIAMETERS < 350 mm) DIRECT DATA

ROCK TYPE	SOCKET DESIGNATION AND TYPE	SOCKET DESCRIPTION L (m)	D (m)	L/D	ROCK STRENGTH q_u (MPa)	MEASURED AVG. PEAK SIDESHEAR RESISTANCE, τ_f (MPa)	τ_f/q_u (α)	τ_{re}/τ_f	SITE LOCATION	REFERENCE	COMMENTS
Sandstone	1-S	.97	.267	3.6	27.6	3.87	.14	NTF	Sydney, N.S.W. Australia	MacKenzie (1969)	
Sandstone	1-S	.64	.245	2.6	12-24	.9	.04-.08	.95	Lennox Is., P.E.I., Canada	Vogan (1977)	see also Horvath (1978)
	2-S	.76	.245	3.1		.83	.03-.07	.80			
	3-S	.95	.245	3.9		1.25	.05-.10	.83			
HW-Mudstone	S12-S	.90	.335	2.7	.59	.41	.71	.63	Melbourne, Vic, Australia	Williams (1980)	

A(II) COMPRESSION SOCKETS (DIAMETERS < 350 mm) DIRECT DATA

ROCK TYPE	SOCKET DESIGNATION AND TYPE	SOCKET DESCRIPTION L (m)	D (m)	L/D	ROCK STRENGTH q_u (MPa)	MEASURED AVG. PEAK SIDESHEAR RESISTANCE, τ_f (MPa)	τ_f/q_u (α)	τ_{re}/τ_f	SITE LOCATION	REFERENCE	COMMENTS f'_c (MPa)	τ_f/f'_c
Limestone	7-C	.102	.108	.94	74 (avg)	4.43	.06	1.0	Ottawa, Ont. Canada	Gibson (1973)	26.9	.16
	10-C	.152	.108	1.41		3.92	.05	1.0			28.2	.14
	8-C	.203	.108	1.88		1.72	.02	1.0			28.2	.06
	9-C	.305	.108	2.82		2.28	.031	1.0			28.2	.08
	1-C	.127	.159	.80		2.12	.029	.89			35.2	.06
	13-S	.152	.159	.96		3.31	.044	1.0			41.4	.08
	5-C	.165	.159	1.04		3.69	.05	1.0			24.1	.15
	11-S	.178	.159	1.12		1.62	.022	~1.0			29	.06
	2-C	.229	.159	1.44		>4.52	.061	NTF			35.2	.13(R)
	14-C	.229	.159	1.44		1.55	.021	1.0			26.2	.06
	4-C	.305	.159	1.92		2.98	.04	1.0			35.9	.08
	15-C	.305	.159	1.92		1.77	.024	1.0			26.2	.07
	3-C	.432	.159	2.72		2.14	.029	1.0			35.9	.06
	6-C	.457	.159	2.87		2.79	.038	NTF			26.9	.10
	12-C	.508	.159	3.2		3.35	.045	NTF			41.4	.08
	16-C	.229	.229	1.0		1.13	.015	~1.0			38.6	.03
	17-S	.216	.229	.94		2.88	.039	1.0			40.7	.07

A(III) SOCKET PULL-OUT TESTS (DIAMETER > 350 mm) DIRECT DATA

ROCK TYPE	SOCKET DESIGNATION AND TYPE	SOCKET DESCRIPTION L (m)	D (m)	L/D	ROCK STRENGTH q_u (MPa)	MEASURED AVG. PEAK SIDESHEAR RESISTANCE, τ_f (MPa)	τ_f/q_u (α)	τ_{re}/τ_f	SITE LOCATION	REFERENCE	COMMENTS
Sandstone	1A-S	1.08	.471	2.3	2.5(avg)	>.53	>.21	NTF	E. Transvaal, South Africa	Webb & Davies (1980)	
	1B-S	1.75	.45	3.9		.73	.28	1.0			
	1C-S	2.77	.45	6.2		>.68	>.27	NTF			
	2B-S	.90	.45	2.0		.63	.25	1.0			
	2C-S	1.3	.536	2.4		>.48	>.19	NTF			
	2D-S	.50	.45	1.1		.42	.17	1.0			
	2F-S	1.67	.436	3.8		.91	.36	.91			
	3A-S	.60	.45	1.3	18.0(avg)	.59	.03	.8			
	3B-S	.80	.45	1.8		>3.18	>.18	NTF			
	3C-S	1.6	.45	3.6		>2.26	>.13	NTF			
Shale	1-S	6.0	.61	9.8	.48	>.310	>.65	NTF	Brookfield, N.S., Canada	Matich & Kozicki(1967)	(R)

A(IV) SOCKET PULL-OUT TESTS (DIAMETER < 350 mm) DIRECT DATA

ROCK TYPE	SOCKET DESIGNATION AND TYPE	L (m)	D (m)	L/D	q_u (MPa)	τ_f (MPa)	τ_f/q_u	τ_{re}/τ_f	SITE LOCATION	REFERENCE
Shale	1-S	.90	.203	4.4	20.7	>2.1	>.10	NTF	Canada	Rosenberg & Journeaux(1976)

TABLE D2

B(I) COMPRESSION SOCKETS (DIAMETER > 350 mm) INDIRECT DATA

ROCK TYPE	SOCKET DESIGNATION AND TYPE	SOCKET L (m)	DESCRIPTION D (m)	L/D	ROCK STRENGTH q_u (MPa)	MEASURED AVG. PEAK SIDESHEAR RESISTANCE, τ_f (MPa)	τ_f/q_u (α)	τ_{re}/τ_f	SITE LOCATION	REFERENCE	COMMENTS
Sandstone	1-S				not measured	>1.075	-	NTF	San Francisco, CA, U.S.A.	Moore (1964)	
Shale	1-S	3.36	.917	3.66	not measured	.24	-	~.96	U.K.	Thorburn (1966)	
	2-C	1.22	1.22	1.0	9.4-12.4(from cube tests)	est >.75	>.06	-		Thorburn (1966)*	
Shale	1-S	.70	.838	.80	not given	2.46	-	NTF	Toronto,Can.	Trow (1972)†	
Shale	4-S	.90	.46	1.96	not measured	.628@12.12mm	-	-	Allegheny Co. PA, U.S.A.	Spanovich & Garvin (1979)	Failure defined as exceeding a specific settlement (eg. ρ > .029 mm/KN)[g]
	9-S	.90	.61	1.48		.40@10.3mm	-	-			
	3-S	.90	.76	1.2		1.322@10.3mm	-	-			
	7-S	1.5	.76	2.46		.352@10.5mm	-	-			
	2-S	1.5	.89	1.97		.621@19.71mm	-	-			
Shale	1-C	.95	.50	1.9	not given	.22	-	-	Prague, Czechoslovakia	Nemec(1979)	-clayey-sandy shales, bedded in slabs
	2-C	.825	.49	1.68		.38	-	-			
	3-C	.90	.50	1.8		.72	-	-			
Shale	1-C	.95	.50	1.9	not given	.21	-	-	Prague, Czechoslovakia	Nemec(1979)	-silty, finely micaceous shales
	2-C	.95	.50	1.9		.30	-	-			
	3-C	.95	.50	1.9		.26	-	-			
	4-C	.95	.50	1.9		.27	-	-			
	5-C	.95	.50	1.9		.86	-	-			
	6-C	.95	.50	1.9		.42	-	-			
Shale	1-C	.7	.8	.875	not given	2.5	-	NTF	Toronto, Ont.,Canada	Horvath et al.,(1980)	
Clay Shale	DT1-C	1.37	.89	1.53	S_u=.31	.28	τ_f/S_u=.90	-	Dallas, Texas	Aurora & Reese (1977)	S_u-static cone test
	MT1-C	1.35	.74	1.82	S_u=.71	.37	=.52	-	Montopolis, Texas,U.S.A.		S_u-u.u. triaxial tests (B)
	MT2-C	1.35	.79	1.71		.41	=.58	-			
	MT3-C	1.52	.75	2.03		.69	=.97	-			

B(I) COMPRESSION SOCKETS (DIAMETER > 350 mm) INDIRECT DATA

ROCK TYPE	SOCKET DESIGNATION AND TYPE	SOCKET L (m)	DESCRIPTION D (m)	L/D	ROCK STRENGTH q_u (MPa)	MEASURED AVG. PEAK SIDESHEAR RESISTANCE, τ_f (MPa)	τ_f/q_u (α)	τ_{re}/τ_f	SITE LOCATION	REFERENCE	COMMENTS
Siltstone	1-C	.91	.77	1.18	.083-40.7	>.310	-	NTF	Calgary,Alta. Canada	Bertok & Berezowski (1983)	
Chalk	1-C	10.0	1.05	9.0	not measured	.205	-	-	Erith, U.K.	Buttling (1976)	
	2-C	10.0	.90	11.0	not measured	.120	-	-	Dover, U.K.	Buttling (1976)	
Chalk	1-C	6.1	.406	15	not given	.148	-	-	Norwich, U.K.	Lord (1976)	
Marl	TP4	3.3	.6(est)	5.5	not given	.710	-	NTF	Redcar,U.K.	Jorden & Dobie† (1976)	
Marl	A	6.37	.74	8.61	S_u=.4	.21@6mm	τ_f/S_u=.53	-	Kilroot, N. Ireland	Leach et al.* (1976)	c'=39kPa,ϕ'=28°
	B	8.98	.74	12.14	S_u=.4	.12@>14mm	=.26	-			c'=12kPa,ϕ'=27°
Limestone	1-C	1.53	.76	2.01	not measured	>1.63	-	NTF	St. Louis, Missouri,USA	Jackson et al. (1973)	
Limestone	1-C	3.0	.61	4.9	not given	>2.07	-	NTF	Buffalo,N.Y. U.S.A.	D'Appolonia† (1967)	
Limestone	1-C	2.72	1.83	1.5	not given	>1.64	-	NTF	Chicago, Ill. U.S.A.	Bushell & Baker (1982)	
Limestone	1-C	2.91	.76	3.83	10.92(avg)	<.90	<.08	~1.0	Dade Co.,Fla. U.S.A.	Kaderabek (1982)	side resistance not determined directly
Biotite-Gneiss	1-S	3.3	1.1	3.0	not given	>.96	-	NTF	not given	Costa Nunnes & Fernandes(1981)	

B(II) COMPRESSION SOCKETS (DIAMETERS < 350 mm) INDIRECT DATA

ROCK TYPE	SOCKET DESIGNATION AND TYPE	SOCKET DESCRIPTION L (m)	D (m)	L/D	ROCK STRENGTH q_u (MPa)	MEASURED AVG. PEAK SIDESHEAR RESISTANCE, τ_f (MPa)	τ_f/q_u (α)	τ_{re}/τ_f	SITE LOCATION	REFERENCE	COMMENTS
Shale	1-C	.47	.30	1.57	not given	.20	-	-	Prague, Czecho-slovakia	Nemec (1979)	-clayey, thinly bedded shales
	2-C	.47	.30	1.57		.25	-	-			
	3-C	.49	.30	1.63		.36	-	-			
	4-C	.49	.30	1.63		.36	-	-			
	5-C	.48	.30	1.6		.45	-	-			
	6-C	.48	.30	1.6		1.35	-	-			
Slate	1-S	.64	.23	2.8	not given	>1.885	-	NTF	Come-by-Chance, Nfld.,Canada	Vogan (1977)†	
	2-S	1.05	.23	4.6		>1.167	-	NTF			
	3-S	1.13	.23	4.9		>1.07	-	NTF			
	4-S	.98	.23	4.3		> .47	-	NTF			
	5-S	1.62	.23	7.1		>1.07	-	NTF			

B(III) SOCKET PULL-OUT TESTS (DIAMETERS > 350 mm)

Mudstone	East-S	1.0	.9	1.1	1.09(avg)	.12@3mm	.11	NTF	Pt. Elizabeth, S. Africa	Wilson (1976)	
	West-S	1.0	.9	1.1	1.09(avg)	.18@12mm	.17	NTF			
Shale	1-S	1.0	.76	1.32	not given	1.3	-	1.0	Whitby,Ont.Can.	Horvath et al. (1980)	
Chalk	1-S	1.52	.762	2.0	not given	.242	-	-	Grays, Essex, U.K.	Palmer (1977)	
	2-S	1.52	.762	2.0	not given	.181	-	-			

B(IV) SOCKET PULL-OUT TESTS (DIAMETERS 150-350 mm)

Mica Schist	1-S	.8	.305	2.6	not given	2.66	-	NTF	New York, N.Y. U.S.A.	ENR (1937)†	

B(V) ANCHORS (DIAMETERS < 150 mm)

ROCK TYPE	SOCKET DESIGNATION AND TYPE	SOCKET DESCRIPTION L (m)	D (m)	L/D	ROCK STRENGTH q_u (MPa)	MEASURED AVG. PEAK SIDESHEAR RESISTANCE, τ_f (MPa)	τ_f/q_u (α)	τ_{re}/τ_f	SITE LOCATION	REFERENCE	COMMENTS
Mudstone	1-S	14.0	.10	140	1.09	>.31	>.28	NTF	Pt. Elizabeth, S. Africa	Wilson (1976)	
Shale	1-S	.60	.076	7.9	not given	.345	-	.70	Hamilton, Ont.,Canada	MTC (1964)†	
	2-S	.90	.076	11.8		.345	-	~1.0			
	3-S	1.2	.076	15.8		.443	-	~1.0			
Chalk	-	3-9	.076-.102	30-120	.995-1.07	.215-.107	-	-	Reading & Ramsgate,U.K.	Littlejohn (1970)†	

TABLE D3 BACKFIGURED MODULUS (E_r) FROM FIELD TEST CASE HISTORIES

Rock Type	Socket Designation & Type	L (m)	D (m)	L_e/D	q_u (MPa)	τ_f (MPa)	Backfigured Modulus (E_r) By Authors (MPa)	This Study (MPa) (1)	(2)	(3)	Pile Modulus E_r/q_u	(E_p) (GPa)	Location & Data	Comments
1)Sandstone	A2-SSC	.92	.21	1.7	6(avg)	1.12	180	232	-	191*	31.8	35(est)	Sydney,N.S.W.	
	A3-SSC	.4	.32	0		1.41	230	266*	-	-	44.3		Australia	
	A4-SSC	1.37	.21	.71		.89	240	271	-	247*	41.2		Pells,Rowe &	
	A5-SSC	.518	.21	3.1		.81	380	394	-	308*	51.3		Turner (1980)	
	B1-SSC	.46	.16	3.3		.94	300	337	-	270*	45			
	B2-SSC	.45	.16	3.8		1.16	140	163	-	116*	19.3			-vertical clay filled
	B3-SSC	.52	.32	1.1		.89	320	325	-	235*	39.2			joint above socket B2
	B4-SSC	.33	.26	.4		1.65	800	660	-	591*	98.5			
	B5-SSC	.62	.16	1.4	6(avg)	1.13	140	160	-	130*	21.7			-socket B5 located in
	C2-SSC	.34	.21	6.7	30(avg)	4.75	1610	2712	2232*	1687	74.4			zone of several clay
	C4-SSC	.60	.21	1.0	6(avg)	1.20	130	149	136*	125	22.7			seams & crushed sand-
	C5-SSC	.70	.21	.5	6(avg)	1.17	90	106	-	97†	16.2	35		stone zones
														-authors note 'kinked'
														profile of socket C5
2)Sandstone	1A-SST	1.08	.471	1.1	2.5(avg)	.53	-	117	-	100*	40	45	E. Transvaal,	
	1B-SST	1.75	.45	1.1		.73	-	215	-	188*	75.2		South Africa	
	1C-SST	2.77	.45	1.1		>.68	-	323	-	303*	121.2		Webb & Davies	
	2C-SST	1.3	.536	.93		.48	-	115	-	100*	40		(1980)	
	2F-SST	1.67	.463	1.1	2.5(avg)	.91	-	246	-	202*	80.8			
	3B-SST	.8	.45	1.1	18(avg)	3.18	-	801	-	638*	35			
	3C-SST	1.6	.45	1.1	18(avg)	>2.26	-	1807	-	1532*	85	45		
3)Sandstone	East-SEC	1.39	.61	10(est)	9.26(avg)	>1.93	1723	1551	1163	979*	105.3	40	Farmington,	
	West-SEC	1.47	.61	10(est)	8.35(avg)	>2.12	1310	1278	944	801*	95.9	37	N.M., U.S.A.	
													Glos & Briggs	
													(1983)	

NOTES:
1. SSC - sideshear compression socket test
2. SST - sideshear tension socket test
3. SEC - sideshear & endbearing socket test
4. EBC - endbearing compression socket test
5. Case (1) No reduction factor (L_e = 0)
6. Case (2) Reduction factor (E_t/E_r^e = 0.5) applied based on L_e/D
7. Case (3) Reduction factor (E_t/E_r = 1.0) applied based on L_e/D
8. R - indicates rough socket (eg. SSCR)
9. * - indicates most probable value (used in regression analysis)

Rock Type	Socket Designation & Type	L (m)	D (m)	L_e/D	q_u (MPa)	τ_f (MPa)	Backfigured Modulus (E_r) By Authors (MPa)	This Study (MPa) (1)	(2)	(3)	Pile Modulus E_r/q_u	(E_p) (GPa)	Location & Data	Comments
4)Sandstone	1-SSC	.635	.245	6.1	12-24 (Range)	.90	-	149	-	100†		35	Lennox Is., P.E.I.	-q_u values deter-
	2-SSC	.965	.245	3.5	"	1.25	-	184	-	135†		35	Canada,Vogan	mined at an ad-
	3-SSC	.762	.245	3.7	"	.83	-	187	-	133†		35	(1977)	jacent site
														-some soft shale layers
5)Sandstone	2-SSC	.266	.965	0	27.6	3.85	-	1077*	-	-	39	35	Sydney, N.S.W.,	-alternate hard
													Australia,	and soft layers
													MacKenzie (1969)	of sandstone
6a)Mudstone	S1-SSC	1.52	.66	1.52	.83	.66(est)	259	330	-	306*	369	35	Melbourne, Vic.	-.3 m thick sand-
	S3-SSC	2.51	1.17	.85	.55	.51	586	473	-	406*	238		Australia,	stone layer en-
	S5-SSC	2.59	1.12	.89	.59	.485	562	561	-	501*	849		Williams (1980)	countered within
	S12-SSC	.9	.335	.30	.58	.41	161	141*	-	133	229		(series 'S' sockets	zone of influence
	S14-SSC	.87	.395	.25	.58	.50	90	88*	-	83	143		denote Stanley Ave.	of sockets S1,
	S15-SSC	.87	.395	.25	.58	.41	100	70*	-	66	114		site).	S3,S5
	S16-SSC	.885	.395	.25	.58	>.36	100	83*	-	78	134	35		
		L_e												
6b)Mudstone	S2-EBC	0	.6	0	.54	-	256*				474	35	Same as above	
	S4-EBC	0	1.0	0	.57	-	473*				830			
	S6-EBC	0	.1	0	.60	-	210*				350			
	S7-EBC	.25	.1	2.5	.44	-	143*				325			
	S7-EBC	2.0	.3	6.7	.65	-	184*				283			
	S10-EBC	1.0	.1	10	.75	-	74*				99			
	S11-EBC	1.0	.3	3.3	.67	-	165*				246			
	S13-EBC	1.52	.1	15.2	.57	-	98*				172			
	S19-EBC	.2	.1	2	.45	-	97*				216	35		
	S22-EBC	2.2	.1	22	.52	-	77*				148	35		

† Not included in regression analysis

Rock Type	Socket Designation & Type	L (m)	D (m)	L_e/D	q_u (MPa)	τ_f (MPa)	Backfigured Modulus (E_r) By Authors (MPa)	This Study (MPa) (1)	(2)	(3)	E_r/q_u	Pile Modulus (E_p) (GPa)	Location & Data	Comments
6c)Mudstone	M8-SEC	1.8	.66	.91	2.0	.967	610	557	513*	473	257	35	Melbourne,Vic.	
	M9-SEC	4.2	.66	.91	2.3	.892	970	1002	951*	845	367	35	Australia,	
	M10-SEC	7.8	.66	.91	3.4	>.549	840	864	803*	726	214	35	Williams (1980)	
6d)Mudstone	M1-SST	2.0	1.22	9.43	2.46	.60	344*				140	35	(series 'M'	
	M2-SST	2.0	1.3	8.85	2.3	.64	321*				140		sockets denote	
	M3-SST	2.0	1.23	9.35	2.3	.71	653*				284		Middleborough Rd.	
	M4-SST	2.0	1.35	8.52	2.34	.62	-				-	35	site)	
6e)Mudstone	M1-EBC	15.5	1.0	15.5	2.68	-	420*				157	35		-calculated using $\rho = \frac{P_t}{E_r D} I_n \mu_o$ where $I_n = 1.05$, μ_o = embedment factor for a rigid footing
	M2-EBC	15.5	1.0	15.5	2.45	-	576*				235			
	M3-EBC	15.5	1.0	15.5	2.45	-	866*				353			
	M4-EBC	15.5	1.0	15.5	2.68	-	630*				235			
	M6-EBC	1.8	.6	3	1.93	-	355*				184			
	M7-EBC	3.0	1.0	3	1.40	-	180*				129			
	M13-EBC	1.0	.1	10	2.98	-	714*				240			
	M14-EBC	2.0	.1	2	1.83	-	535*				292			
	M15-EBC	.5	.1	.5	2.27	-	573*				252			
	M16-EBC	.3	.1	.3	2.12	-	470*				222			
	M17-EBC	0	.1	0	3.09	-	1133†				367			
	M18-EBC	.2	.1	2	1.53	-	251*				164			
	M19-EBC	0	.3	0	1.14	-	1155†				1013			
	M20-EBC	.9	.3	3	2.19	-	1139†				520			
	M21-EBC	1.5	.3	5	1.97	-	339*				172	35		

† Not included in regression analysis.

Rock Type	Socket Designation & Type	L (m)	D (m)	L_e/D	q_u (MPa)	τ_f (MPa)	Backfigured Modulus (E_r) By Authors (MPa)	This Study (MPa) (1)	(2)	(3)	E_r/q_u	Pile Modulus (E_p) (GPa)	Location & Data	Comments
6f)Mudstone	WG303/2-SST	2 L_e	1.22	3.16	3.49	.85	1254				359	35	Melbourne,Vic., Australia	-also see Williams & Ervin (1980)
	WG303/2-EBC	10	1	10	4.28	-	1050				245	35	Williams (1980) (series 'WG' denotes Westgate	
6g)Mudstone	ES-SSC	1.52 L_e	1.09	1.47	7.2	.86	67	91	-	~67†	9.3	35	Freeway site) (series 'E' denotes Eastern	-Eastern Freeway site displayed highly fractured rock
	EB-EBC	5	.76	6.6	8.63	-	80	-	-	-†	9.3	35	Freeway site)	
7)Shale	P1-EBC	.8	.45		32(avg)	-	avg 550-770† (P1–P3)						Westmead,N.S.W.,	-authors note range of E_r (for sockets P_1,P_2,P_3) dependent upon E_p used (40 & 30 GPa)
	P2-SEC	.675	.45			-							Australia,	
	P3-SSC	.7	.45			2.5							Thorne (1980)	
	P4-EBC	1.4	.9				4000 †							
	P5-SEC	1.4	.9				2760 †							
	P6-SSC	1.3	.9			>1.22	2300 †							
8)Shale	P1-SSC	1.37	.71	.85	6.75(avg)	>1.54	350	426	397*	363	59	37	Burlington, Ont.	
	P2-SEC					>1.45	290	302	283*	262	42		Canada, Horvath	
	P3-SSCR					2.13	235	289	270*	249	40		(1982)	
	P4-SECR					>1.8	310	388	377*	351	56			
	P6-SSCR	1.37	.71	.85	6.75(avg)	>2.48	390	481	443*	415	66	37		
9)Shale	#4-SST	5.98	.61	3.6	.48	.31	-	110		99*	206	39	Brookfield, N.S., Canada, Matich & Kozicki (1967)	

† Not included in regression analysis.

Rock Type	Socket Designation & Type	L (m)	D (m)	L_e/D	q_u (MPa)	τ_f (MPa)	Backfigured Modulus (E_r) By Authors (MPa)	This Study (MPa) (1)	(2)	(3)	E_r/q_u	Pile Modulus (E_p) (GPa)	Location & Data	Comments
10)Shale	#1-SSC	.91	.635	0	15.2(avg)	>1.0	-	384*	-	-	25	45	Toronto, Ont., Canada, MTC 1968)	-see Horvath et al. (1980)
	#2-EBC	1.37	.584	2.35	"	-	-	331*	-	-	22	45		
	#3-SEC	1.04	.635	0	"	-	-	320*	-	-	21	45		
11)Shale	- -SST	.914	.203	2**	20.7	2.1	-	1120	-	881*	43	35	Canada, Rosenberg & Journeaux(1976)	
12)Siltstone	Axial-SEC	4.9	1.2	0	1.0	>.279	200	288*	-	-	288	38	Channel Is., N. Terr., Australia, Sales (1983)	-low RQD's over socket length (eg. approx. 4-16 fractures per meter)
13)Chalk	Little-brook'D' -SSC	8.0	1.05	0	1.036 (avg)	.190	-	128*	-	-	2.17	36	Littlebrook, U.K. Buttling (1976)	-also see Mallard & Ballantyne (1976)
14)Andesite	- -SSC	.56	.46	2**	10.4	1.1	-	628	-	428*	41	35	Canada, Rosenberg & Journeaux(1976)	-fractured & sheared andesite -shear zones with soft green chlorite

Notes: † embedment ratio used to calculate reduction factor due to socket recessment below rock surface
* most probable value of backfigured modulus based on information given by authors
** estimated value of embedment, since socket recessment in rock not explicitly given

TABLE D4 AVERAGE OF PRESUMED BEST VALUES OF BACKFIGURED MODULUS FOR TEST LOCATION AND ROCK STRENGTHS (values indicated by asterisk in Table 4.7)

Rock Type	$\bar{q}_u$ (MPa)	$\bar{\tau}_f$ (MPa)	No. of Tests	Mean and Standard Deviation of Backfigured Modulus ($\bar{E}_r$) By Authors (MPa)	This Study (MPa)	Pile Modulus ($\bar{E}_p$) Used in Calculations (GPa)	$\bar{E}_r/\bar{q}_u$	$\bar{\tau}_f/\bar{q}_u$	$\bar{E}_r/\bar{\tau}_f$	Location and Data
1)Sandstone	6	1.12(.25)	10	286 (199)	249(137)	35	41.5	.18	222	Sydney, N.S.W., Australia; Pells,
	30	4.75	1	1610	2232	35	74.4	.16	470	Rowe & Turner(1980)
2)Sandstone	2.5	.666(.17)	5	-	179(84)	45	71.6	.27	269	E. Transvaal, S. Africa; Webb &
	18	2.72(.65)	2	-	1085(632)	45	60	.15	399	Davies (1980)
3)Sandstone	8.8	2.03(.13)	2	1517(292)	890(126)	39	101	.23	438	Farmington, N.M., USA Glos & Briggs (1983)
4)Sandstone	18*	.993(.23)	3	-	122(20)†	35	7	.06	123	Lennox Is., P.E.I., Canada; Vogan (1977)
5)Sandstone	27.6	3.85	1	-	1077	35	39	.14	280	Sydney, N.S.W., Australia; MacKenzie (1969)
6a)Mudstone	.61 (.10)	.476(.10)	7	265(219) $\bar{x}$=198	228(176) (Stanley Ave-SSC)	35	374	.78	479	Melbourne, Vic., Australia; Williams (1980)
6b)Mudstone	.58 (.10)	-	10	178(120) SD=143	- (Stanley Ave-EBC)	35	307			
6c)Mudstone	2.6 (.74)	.802(.22)	3	807(182) $\bar{x}$=527	756(223) (Middleborough Rd-SEC)	35	291	.31	943	
6d)Mudstone	2.35 (.08)	.64(.05)	3	1439(185) SD=215	- (Middleborough Rd-SST)	35	187	.27	686	
6e)Mudstone	2.18 (.56)	-	15	622(321) [492(197)]	- (Middleborough Rd-EBC) (excludes values obtained from sockets M17,M19,M20)	35	226			
6f)Mudstone	3.89 (.56)	.85	2	1152(144)	- (Westgate Fwy.-EBC&SST)	35	296	.22	1355	

* midrange of 12-24 ( ) indicates standard deviation of mean † not included in regression analysis

Rock Type	$\bar{q}_u$ (MPa)	$\bar{\tau}_f$ (MPa)	No. of Tests	Mean and Standard Deviation of Backfigured Modulus (E_r) By Authors (MPa)	This Study (MPa)	Pile Modulus ($\bar{E}_p$) Used in Calculations (GPa)	$\bar{E}_r/\bar{q}_u$	$\bar{\tau}_f/\bar{q}_u$	$\bar{E}_r/\bar{\tau}_f$	Location and Data
6g)Mudstone	7.92 (1.01)	.86	2	74(10)[2] †	- (Eastern Fwy-EBC & SSC)	35	9.34	.11	123	Melbourne, Vic., Australia; Williams (1980)
7)Shale	32(5.9) 32(5.9)	2.5[3] >1.22[4]	3 3	550-700 † 3020(879) †	- -	30&40 30&40				Westmead, N.S.W., Australia; Thorne (1980)
8)Shale	6.75	1.88(.43)	5	315(59)	354(75)	37	52	.28	188	Burlington, Ont.,Can. Horvath (1982)
9)Shale	.48	.31	1	-	99	39	206	.65	319	Brookfield, N.S.,Can. Matich & Kozicki (1967)
10)Shale	15.2	>1.0	3	-	345(34)	45	23	.07	345	Toronto,Ont.,Canada MTC (1968)
11)Shale	20.7	2.1	1	-	881	35	43	.101	420	Canada, Rosenberg & Journeaux (1976)
12)Silt-stone	1.0	>.279	1	200	288	38	288	.279	1039	Channel Is., N. Terr. Australia,Sales(1983)
13)Chalk	1.04	.190	1	-	128	36	123	.18	673	Littlebrook, U.K., Buttling (1976)
14)Andesite	10.4	1.1	1	-	428	35	41.5	.11	389	Canada, Rosenberg & Journeaux (1976)

NOTES:
1 Based on average for M1, M2 and M3 only
2 Highly fractured mudstone
3 Small diameter tests
4 Large diameter tests