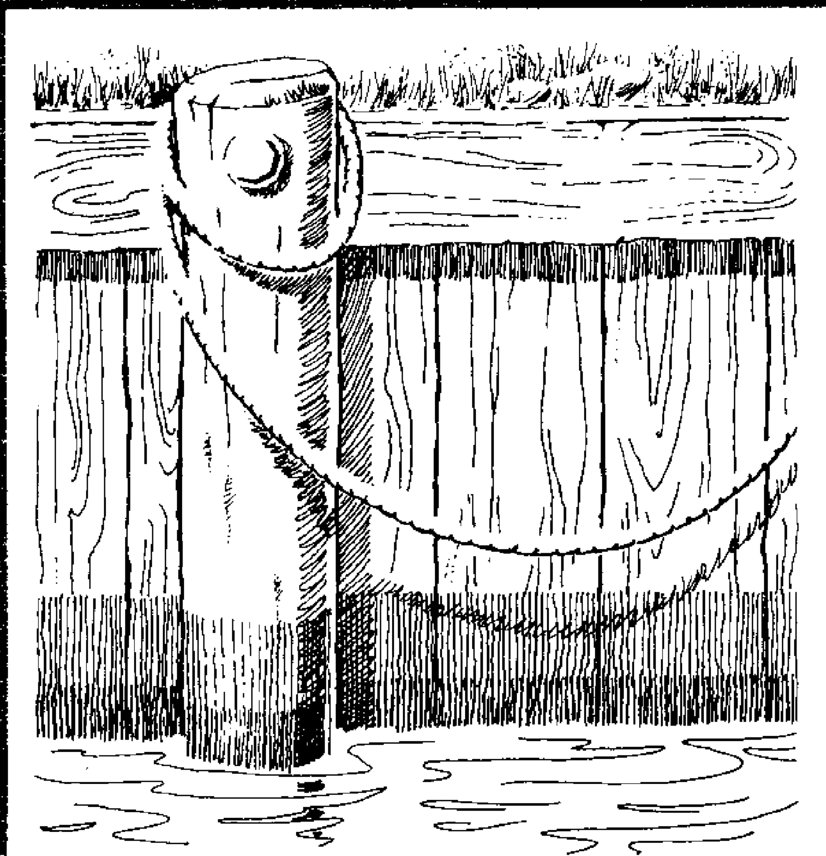

BULKHEADS

NEW YORK SEA GRANT INSTITUTE

This manual is part of the Coastal Structures Handbook Series. The series is being prepared for the New York Sea Grant Institute by the Geotechnical Engineering group at Cornell University, coordinated by Fred H. Kulhawy.

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COASTAL STRUCTURES HANDBOOK SERIES
BULKHEADS

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Ithaca, New York

1982

This research was sponsored by the New York Sea Grant Institute under a grant (NA81AA-D-00027) from the Office of Sea Grant, National Oceanic and Atmospheric Administration (NOAA), US Department of Commerce.

PREFACE

The analysis, design and construction of coastal structures is of great concern to a broad cross-section of the population living near major fresh and salt water bodies. Realizing this concern, the New York Sea Grant Institute instituted a project to develop a manual to assist a variety of user groups in addressing the problems associated with the development of coastal structures and coastal facilities. Although the engineering community will find the manual to be of use, the focus of this manual has been to develop a simplified user's guide which focuses on the analysis, design and construction of coastal structures. The emphasis has been on understanding the structures and their behavior, minimizing higher level mathematics, and presenting design charts and design examples for smaller scale structures, typical of those of importance to a small community and the individual homeowner. Large scale developments should be handled by design professionals with expertise in the field.

This project was initiated in late 1977 by the New York Sea Grant Institute and has been developed by the School of Civil and Environmental Engineering at Cornell University. The project was initiated by Drs. Fred H. Kulhawy and Dwight A. Sangrey. Dr. Sangrey left Cornell before much progress was made, and subsequent work has been supervised by Drs. Fred H. Kulhawy and Philip L.-F. Liu.

Under the auspices of this project, the following reports have been prepared and submitted to New York Sea Grant:

1. "Coastal Construction Materials", November 1979, by Walter D. Hubbell and Fred H. Kulhawy

2. "Environmental Loads in Coastal Construction", November 1979, by Walter D. Hubbell and Fred H. Kulhawy

3. "Analysis, Design and Construction of Pile Foundations in the Coastal Environment", April 1981, by Francis K.-P. Cheung and Fred H. Kulhawy

4. "Breakwaters, Jetties and Groins: A Design Guide", March 1982, by Laurie A. Ehrlich and Fred H. Kulhawy

5. "Analysis, Design and Construction of Bulkheads in the Coastal Environment", May 1982, by Thomas M. Saczynski and Fred H. Kulhawy

Additional reports to be completed in the near future include:

a. Boat Ramps

b. Docks, Piers and Wharves

Further topics to complete the manual should be initiated prior to the end of 1982.

ABSTRACT

The extensive employment of bulkheads in the coastal environment represents considerable capital expenditure. In many instances these bulkheads are constructed with little consideration for pertinent soil properties, soil-structure behavior or fabrication procedure. This work is intended to describe the complex behavior of these systems, to provide a rational and simplified design approach and to discuss other pertinent design and construction aspects.

Based upon the evidence disclosed by the literature, a particular design method was selected and a computer program was coded. The Free Earth Support method, as modified by Rowe, was used as the basis for a procedure to design anchored or cantilever bulkheads in sand or clay. The program was then modified so that parametric studies could be conducted and the results could be incorporated into simplified design charts. The reliability of the chosen design method and resulting design curves were tested by probabilistic methods.

Other design considerations, such as external loading, cost effectiveness, and component design and dimensioning, are elaborated upon. Examples are given which illustrate the use of the Free Earth Support method, as modified by Rowe, and the simplified method developed. Construction procedures and their impact upon wall performance are also discussed.

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LIST OF SYMBOLS

<u>English</u>

A	cross sectional area
b	base dimension
c	cohesion
c'	cohesion, factored
C_D	modifying coefficient, depth
C_P	modifying coefficient, tie-rod pull
C_m	modifying coefficient, bending moment
D	penetration depth
D'	dimensionless penetration depth
d	diameter
E	elastic modulus
F_p	actual stress
FS	factor of safety
f_b	allowable bending stress
f_c	allowable compressive stress, also tie-rod reduction factor
$f_{c\perp}$	allowable compressive stress, perpendicular to grain
f_p	allowable bearing stress
f_t	allowable tensile stress
f_v	allowable shear stress
f_y	yield stress
G_S	specific gravity

H	free standing wall height
H_A	anchor level
H_D	sheet pile length (H+D)
H_{HW}	high water level
H_W	low water level
h	height dimension
I	moment of inertia
K	horizontal stress coefficient
K_o	at rest stress coefficient
K_a	active stress coefficient
K_p	passive stress coefficient
L	distance between anchorage and passive stress resultant
LF	load factor
l	length dimension
M	bending moment
M'	dimensionless bending moment
N	1/2 (short dimension of base plate minus hole dimension)
P	tie-rod pull (force per unit length of wall)
P'	dimensionless tie-rod pull
P_f	probability of failure
P_H	horizontal force resultant of surcharge load
p	allowable load in withdrawal (force per unit length)
Q_L	line load (force per unit length)
Q_P	point load (force)
q	evenly distributed surcharge load (force per unit area)

R	reliability
R_D	loading ratio, depth
R_M	loading ratio, moment
R_P	loading ratio, tie-rod pull
r_d	moment reduction factor
r_t	moment reduction factor, unyielding anchorages
S	section modulus
S_x	standard deviation, independent variables
S_y	standard deviation, dependent variables
SM	safety margin
T	tie-rod load (force)
t	thickness dimension
V	shear (force)
W_r	resistance to withdrawal (force)
w	width; also driving width of a pile
x	independent variable
y	dependent variable

Greek Letters

α	relative wall height (H/H_D)
β	relative anchor level (H_A/H_D)
γ_i	unit weight of i^{th} soil layer
Δ	deflection
δ	soil-structure interface strength (degrees)
θ	angle of inclination

LIST OF SYMBOLS (Continued)

σ_H	horizontal stress
σ_v	vertical stress
τ	constant, M/H_D^3
ρ	pile flexibility, EI/H_D^4
ϕ	angle of internal friction of soil (degrees)
ψ	flexibility characteristic
ω	angle of backfill slope

LIST OF CONVERSION FACTORS

To Convert From	To	Multiply By
inch	mm	25.4
feet	meter	0.3048
lb	Newton	4.448
lb/in^2	kN/m^2	6.895
lb/in^2	MPa	0.006895
lb/ft^3	kN/m^3	0.1571
lb-ft	N-m	1.356
lb/ft^2	N/m^2	0.0478

CHAPTER 1

INTRODUCTION

Waterfront use has always posed a very basic problem: access to waterborne vehicles from the shore. The bulkhead has been extensively employed as the solution to this problem. The casual observer may conclude that the installation of these critical structures is a simple process. In reality, the only simple aspect of bulkheads is their geometry. The actual design, construction and behavior of these soil-structure systems is complex. Simplified approaches have often resulted in either overly conservative design or failure, both to the detriment of the owner. A rational approach is required which incorporates an understanding of bulkhead behavior, a sound computational procedure and good construction practices. The objective of this work is to provide such an approach, emphasizing a simplified design chart format.

Application of the approach suggested herein is intended for bulkhead sites where shore activity is relatively light, such as private residences and marinas. Sufficient flexibility does exist, however, to permit use over a broad spectrum of loading and soil conditions. Discretion is always incumbent upon the designer, especially where bulkhead heights exceed 15 feet (4.57 m), soil conditions are complex, heavy loads are anticipated or environmental conditions are severe.

1.1. Statement of the Problem

Bulkheads are flexible soil retaining walls which derive their stability from the structural members and the strength of the soil. The soil, as well as providing stability, creates loads upon the system which must be resisted. Figure 1-1 illustrates the configuration of the basic anchored bulkhead.

The principal component of the system is the sheet pile. Horizontal stresses exerted by the soil on the backfill side of the wall tend to move the piles outward. This outward movement is resisted by that portion of the wall embedded in the subgrade. If the penetration of the toe into the subgrade is not sufficient, failure will result whereby the toe "kicks out."

The horizontal stresses acting on the pile cause bending, making the pile function as a beam. Therefore pile design is twofold: the pile must be long enough to resist toe failure and it must be stout enough to resist flexural stresses induced by bending.

The sheet piles are tied together by wales. These members are designed to resist bending and are fastened to the piles by bolts or nails. At various points the wales will require splices which must resist the same loads as the wales.

The resistance to outward movement of the wall may be enhanced by employing a tie-rod and anchorage. Since a portion of the horizontal load is transmitted to the anchorage through the tie-rod, the tie-rod must be suitably designed. The anchorage must also be adequately dimensioned and properly positioned. If the anchorage is too close to the wall, it will be located within the failing soil mass, or failure wedge, and will be of no use.

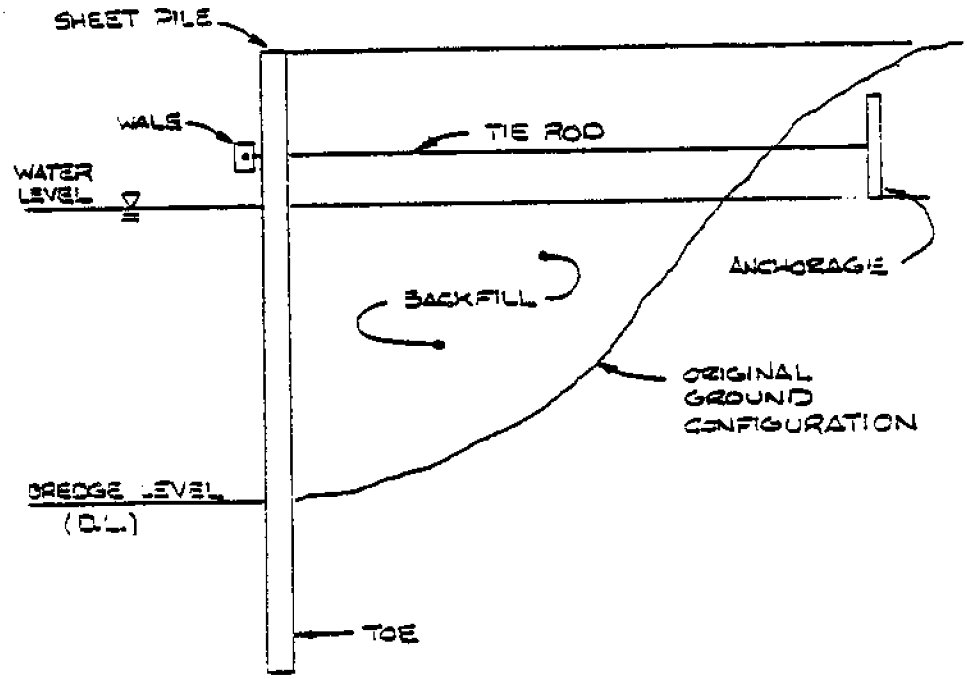


Figure 1-1. Anchored wall

1.1.1. Sheet Piles

Sheet piles are usually made of steel, concrete, or pressure treated wood. Other materials may be used as well, such as aluminum and asbestos.

Wooden sheet piles are generally a foot wide and vary in length and thickness to suit design conditions. An interlocking system, such as tongue-and-groove, is built into the pile as shown in Figure 1-2.

The configuration of steel and concrete sheet piles varies considerably. The choice of the appropriate section is a matter of computing the required engineering properties. Steel and concrete sheet piles also have interlocking devices, such as ball-and-socket connections shown in Figure 1-3 for steel. Concrete sheet pile interlocking is normally tongue-and-groove.

1.1.2. Bulkhead Types

The anchored bulkhead described earlier may be altered to produce another bulkhead type. The most basic variation is to remove the anchorage and tie-rod, creating a cantilevered wall (Figure 1-4). This variation may prove to be economical where relatively low walls are installed. In such cases, the additional penetration depth required to compensate for the lack of anchorage may very well be less costly than the anchorage.

A smooth- or flush-faced bulkhead may be designed by locating the wale on the backfill side of the wall. Although this may enhance boat docking to some extent, it requires more fasteners than the wale on the dredge side of the wall.

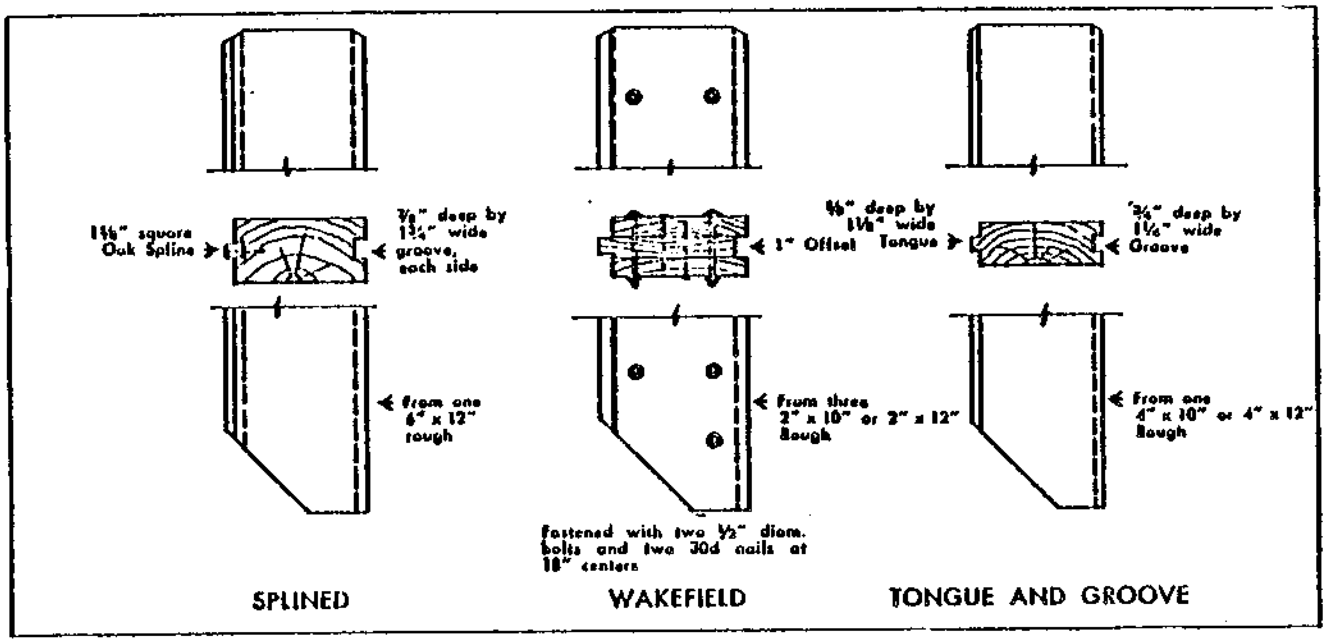


Figure 1-2. Typical timber piles (AWPI, 1970, p. 3)

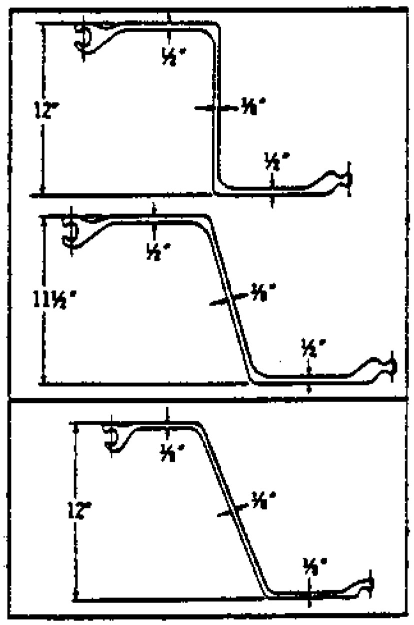


Figure 1-3. Typical ball and socket (United States Steel, 1975, facing p. 1)

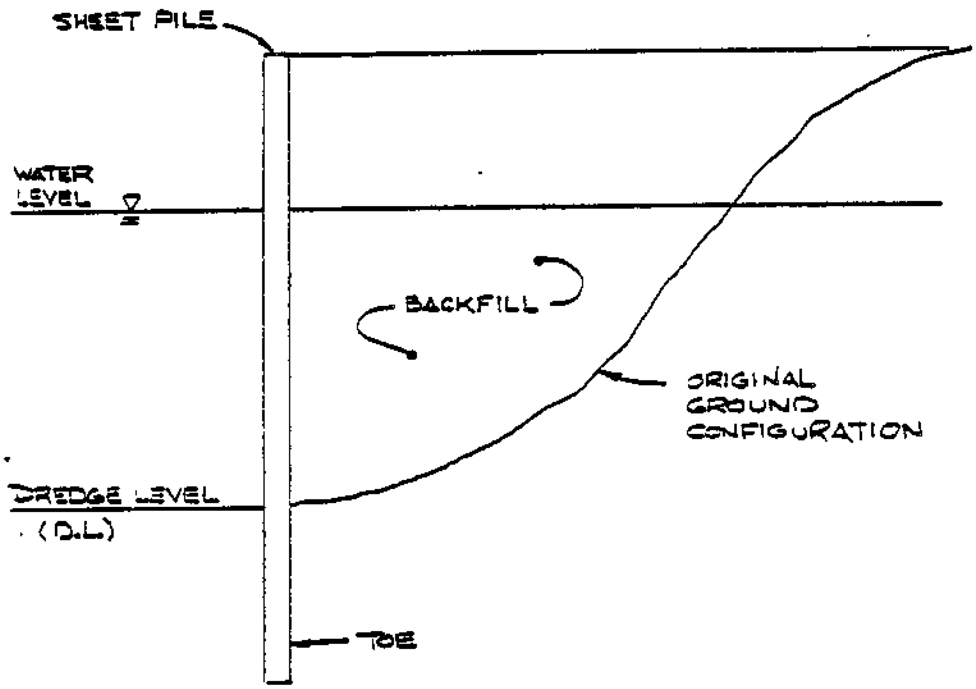


Figure 1-4. Cantilevered wall

The navy bulkhead is another variation of the anchored wall. These walls incorporate the use of 8 in (203 mm) diameter fender piles located in front of the sheet piles, as shown in Figure 1-5. The presence of the fender pile adds considerable rigidity to the system. This is warranted only for relatively high walls or for locations where there will be large external loads. Otherwise, the presence of the fender piles is not required.

Bulkhead types may also be categorized by construction sequence, i.e., a bulkhead may be a fill type or a dredge type. The sequence for a fill type is: drive the piles, install tie-rod and anchorage, then backfill. The sequence for a dredge type is: drive the piles, install the tie-rod and anchorage, backfill, then dredge in front of the wall to the desired depth. A consequence of construction sequence is the resulting stress distribution. Some advantage may be realized where dredge bulkheads are required as the soil behavior tends to be beneficial.

1.1.3. Soils

One of the most critical aspects of the bulkhead site is the type of soil present. In a very general sense, there are two types of soils that the designer must contend with: cohesionless soils, which can be referred to as sand, and cohesive soils, which can be referred to as clay. The behavior of sands is reasonably predictable and reliable designs may be rendered with minimal complications. Clays, on the other hand, are complex soils. Their strength varies considerably from point to point and their behavior depends upon a wide range of conditions, such as mineralogy, soil structure and stress history.

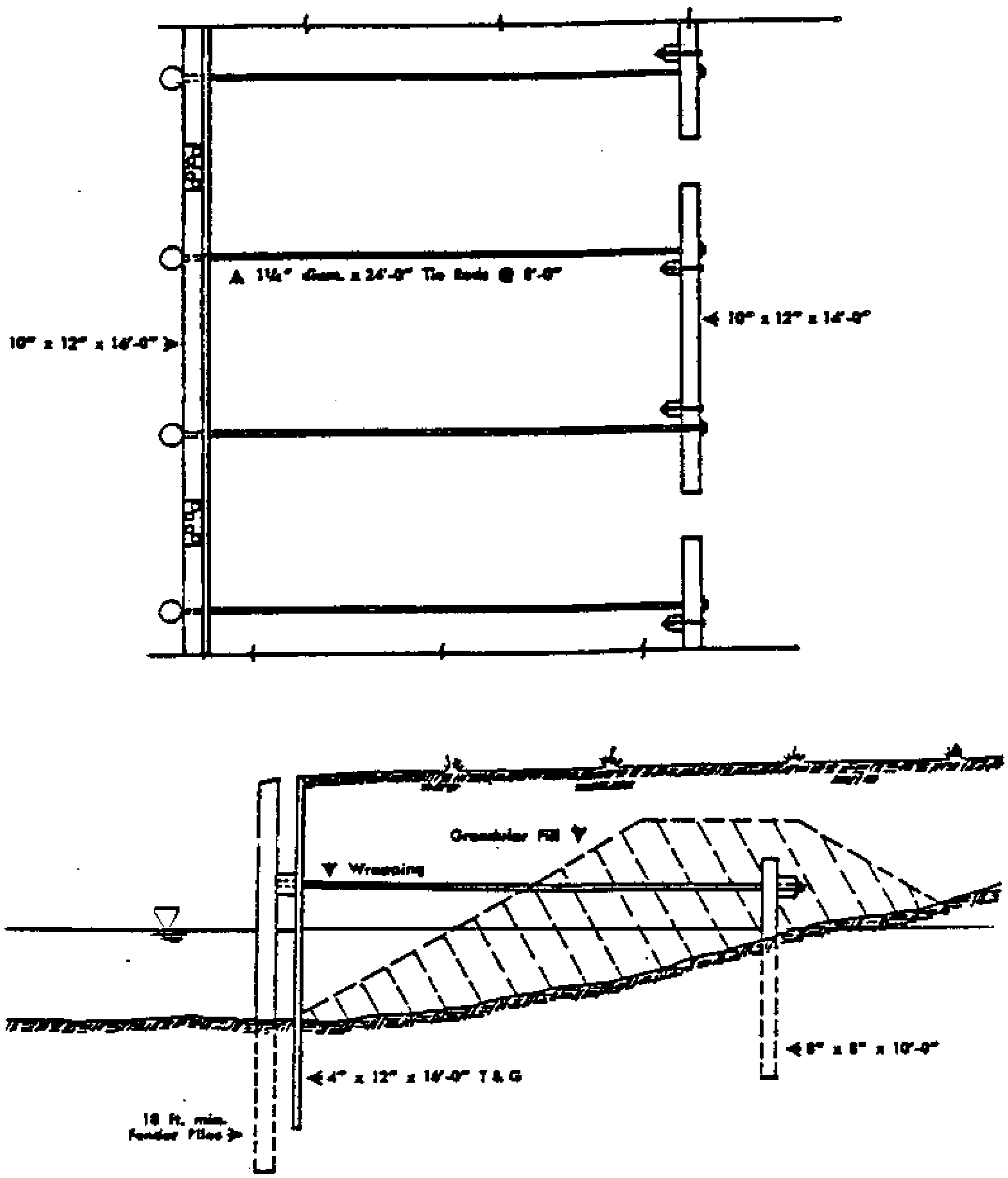


Figure 1-5. Navy bulkhead (AWPI, 1970, p. 3)

The presence of sand in the majority of bulkhead sites in New York State suggests that the design of most bulkheads may proceed in a straightforward manner. The less fortunate designer who must deal with clay is advised to use a cautious approach when attempting to determine the characteristics of the soil. A more detailed discussion regarding site and soil characterization may be found in textbooks (e.g., Wu, 1976).

1.2. Approach to the Solution

The key element in the design of bulkheads is a sound computational procedure. Such a procedure depends largely upon the adequacy of the mathematical model chosen to represent the behavior of the system. An examination of prior investigations of bulkhead behavior not only reveals weak and strong points of the various models, it also provides valuable insights as to the behavior itself. The valid aspects of the various approaches may then be incorporated, while questionable assumptions and details may be disregarded. A sound design procedure will be the result. This is the objective of the next chapter: to examine previous work, glean the useful facts, and formulate a computational approach.

Unfortunately, existing bulkhead design methods are cumbersome. Obviously, a simplified version of the most valid method is desirable. A simplified design procedure is therefore the major goal of this work. The third chapter explains such a simplified method and the means used to compose it. The fourth chapter explains the recommended design procedures.

Although the pile and tie-rod dimensions are the most difficult parameters to design, there are other considerations. Location and design of the anchorage, design of wales, splices and fasteners, external loadings, environmental factors, and the properties of the structural components are discussed in the fifth chapter. Other topics concerning the construction of bulkheads are contained in the sixth chapter.

The seventh chapter is a qualitative treatment of the reliability of bulkhead design. It explores the probability of failure in penetration depth, tie-rod pull, and moment of a hypothetical anchored wall. The design deals with sand and clay subgrades and lends credence to the statement that clay subgrades pose more difficult problems than sand subgrades.

Examples are provided in the appendices to illustrate each portion of the design procedure.

1.3. Summary

The problem to be solved by the bulkhead designer is to compute the dimensions of sheet piling so that the toe is driven to an adequate depth and the section is large enough to withstand bending stresses. If the designer opts for an anchorage and tie-rod, these must also be properly designed.

Herein, a procedure is developed in detail for the design of bulkheads.

CHAPTER 2

EVALUATION OF SOIL STRESSES AND THE DEVELOPMENT OF BULKHEAD DESIGN

Prior to the turn of the century, bulkhead design was governed by classical approaches or merely by rules of thumb. As worldwide commerce increased, the demand for port and harbor facilities also increased. To accommodate this demand, sites had to be utilized which required bulkheads with greater dimensions than previously necessary. The larger dimensions invalidated rules of thumb and rendered the classical approaches obsolete because of economics. A state of the art evolved for bulkhead design as a result of the continuing attempt to understand the complex behavior of these structures.

Each investigation and explanation of bulkhead behavior required simplifying assumptions so that the complexities of horizontal soil stress distribution could be dealt with. An examination of the various thoughts on bulkheads serves to determine the adequacy of the underlying assumptions, to highlight valid contributions which should be incorporated into a design scheme, and to give an overall concept of the true nature of bulkheads.

2.1. Soil Strength and Horizontal Stresses

The computation of stresses in fluids is relatively simple. Consider for example a vat of water as in Figure 2-1a. The stresses at point A are determined from the height of the water above A, h,

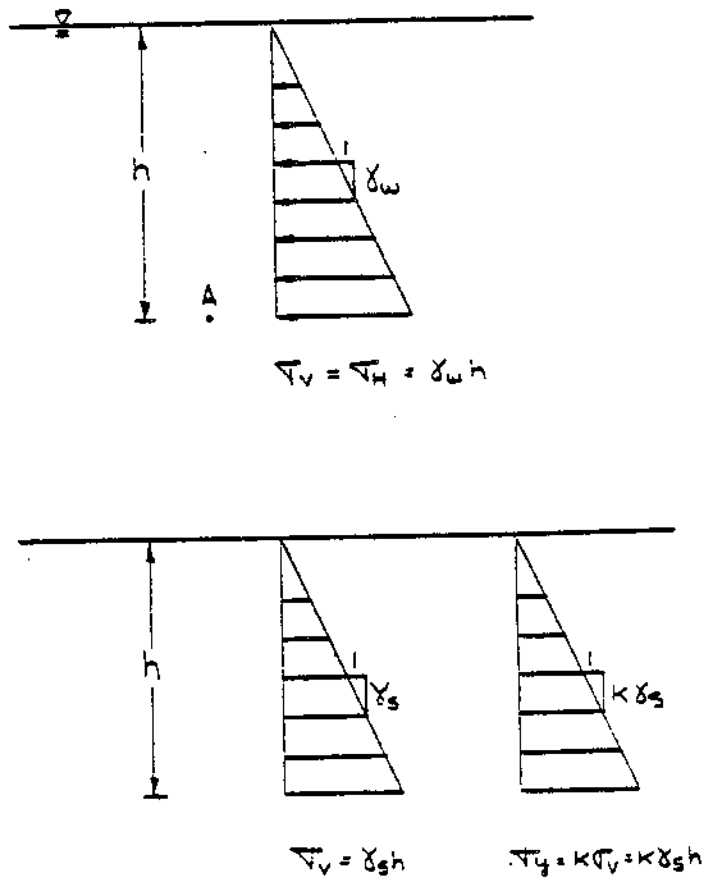


Figure 2-1. Horizontal and vertical stresses

and the unit weight of the water, γ_w. The vertical stress is σ_v. Since the water has no shear strength, the horizontal stress, σ_H, is equal to the vertical stress.

Soil stresses are more complicated to determine because the soil does possess shear strength. Therefore, the stresses in a soil mass at point B in Figure 2-1b are given by: $\sigma_v = \gamma_s h$, where γ_s is the unit weight of the soil, and $\sigma_H = K\sigma_v$, where K is a horizontal soil stress coefficient.

To illustrate the concept of horizontal soil stress coefficient, consider an infinitely rigid, infinitely thin wall retaining an adjacent mass of soil of height H, as shown in Figure 2-2a. The magnitude of the coefficient K depends on the amount of deflection, Δ, with respect to the wall height, H. With no wall deflection, the soil is said to be at rest and the coefficient is designated as K_o. As the wall is deflected away from the soil mass, the stress exerted reduces to a lower equilibrium state, known as the active state. The active stress coefficient is designated as K_a. If the wall is deflected into the soil mass, the stress exerted by the soil increases until the soil reaches an upper equilibrium state, known as the passive state. The passive stress coefficient is denoted by K_p.

Tests performed by Terzaghi (1954) revealed that minimum deflections are required to reach the limiting active and passive states. As suggested by Figure 2-2b, relatively small deflections are needed to reach the full active state and relatively large deflections are needed to reach the full passive state. Also indicated in the figure is that the net change in stresses is much greater for the passive

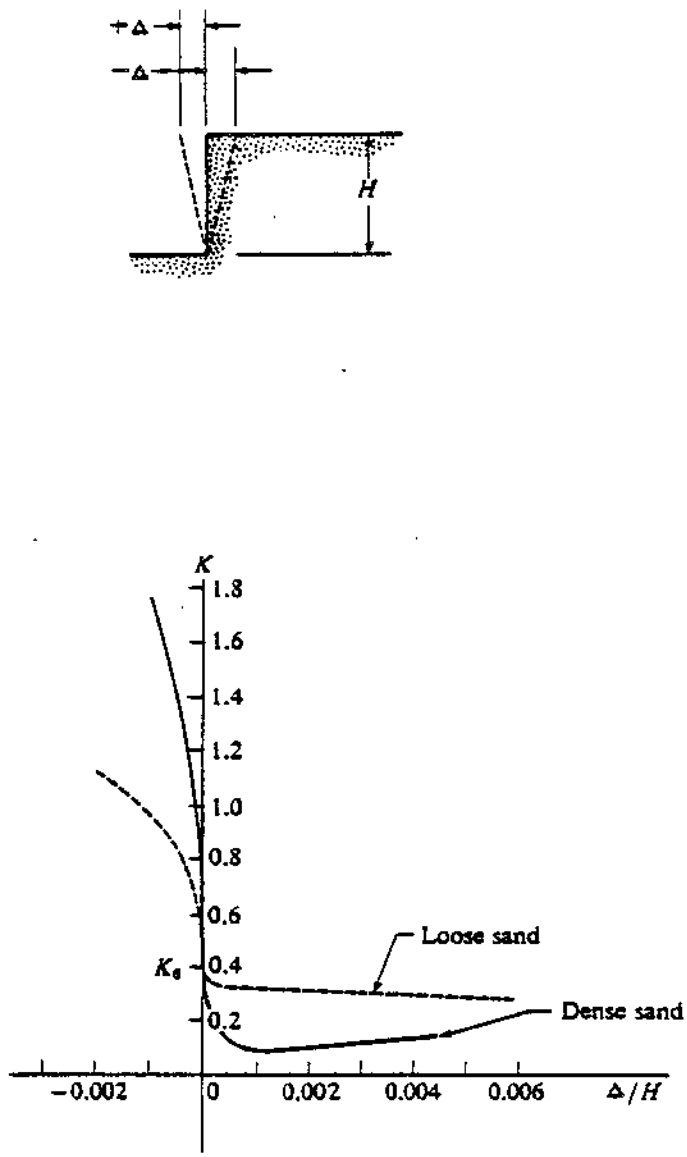


Figure 2-2. Horizontal stress coefficient as a function of deflection (Terzaghi, 1954, p. 1243)

case than for the active case for the same magnitude of deflection.

The soil stress coefficient depends upon the shear strength of the soil as well as the relative deflection of the wall. Shear strength is defined in terms of the Mohr-Coulomb failure criterion as

$$\tau = c + \sigma \tan \phi \qquad (2\text{-}1)$$

in which: τ = shear strength, c = soil cohesion, ϕ = the angle of internal friction, and σ = normal stress on the failure plane. Figure 2-3 illustrates this concept, which shows increasing strength with increasing normal stress.

For the purpose of this work, shear strength will be in terms either c or ϕ. Sand, silt and gravel are assumed to possess only frictional strength, so that $c = 0$. This applies to any combination of these granular soils. Clay soils are more complex, demonstrating different properties for short- and long-term behavior. When a cohesive soil is rapidly loaded to failure, water pressure in the pores is not allowed to drain and the soil exhibits cohesive strength only. If the pore water is allowed to dissipate as the soil is loaded to failure, it will exhibit frictional strength and may be assumed to maintain none of its cohesion. Therefore, the short-term strength of clays is represented by the undrained strength where $\phi = 0$, and the long-term strength is represented by the drained strength where $c = 0$. The drained and undrained strengths vary over a wide range.

The horizontal stress coefficients for soils with friction, including the drained case for clays, depend upon the angle of internal friction, ϕ, the angle of wall friction (i.e., strength of wall-soil

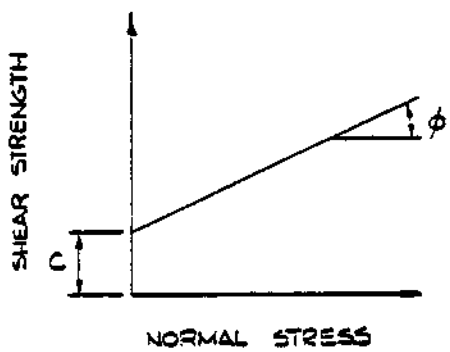


Figure 2-3. Mohr-Coulomb failure criterion

interface), δ, and the angle of inclination, ω, of the backfill with respect to the horizontal. The active stress coefficient, K_a, is given by

$$K_a = \frac{\cos^2 \phi}{\{1 + [\frac{\sin(\phi+\delta)\ \sin(\phi-\omega)}{\cos\delta\ \cos\omega}]^{1/2}\}^2} \qquad (2\text{-}2)$$

The passive stress coefficient, K_p, is given by

$$K_p = \frac{\cos^2 \phi}{\{1 - [\frac{\sin(\phi+\delta)\ \sin(\phi+\omega)}{\cos\delta\ \cos\omega}]^{1/2}\}^2} \qquad (2\text{-}3)$$

The angle of wall friction is often taken as

$$\delta = \frac{2}{3}\phi \qquad (2\text{-}4)$$

for wood and steel walls (Rowe, 1952). Further discussion of the wall-soil interface appears later in this section.

The active and passive stresses, P_a and P_p, may be computed using Rankine's formulation for frictionless soils,

$$P_a = \gamma_s h - 2c \qquad (2\text{-}5)$$

$$P_p = \gamma_s h + 2c \qquad (2\text{-}6)$$

when dealing with the undrained strength of clay.

If the length of the previously described hypothetical wall (Figure 2-2) is increased so that it penetrates into the subgrade to a depth, D, the wall deflection will produce an active state on one side and a passive state on the other. If D is sufficiently large, static equilibrium exists as the horizontal forces exerted on the active side are balanced by the horizontal forces on the passive side. A cantilevered bulkhead is thus established as in Figure 2-4a. The depth of penetration required below the dredge level to achieve equilibrium can be decreased by employing a tie-rod and anchoring system near the top of the wall as in Figure 2-4b. An anchored bulkhead is thus established.

With a known or assumed stress distribution, the depth of penetration, tie-rod load, and bending moment in the wall may be computed. By examining the evolution of bulkhead design, scrutiny of the underlying assumptions of each approach is possible. As the evidence produced by each investigation is accumulated and evaluated, it becomes clear which assumptions are valid and which aspects of a procedure are worthy of retention. These are the components of the design procedure which will result in the most representative calculations of depth, tie-rod load and bending moment.

With these concepts in mind, an examination of the evolution of bulkhead design follows.

2.2. Classical Theories

2.2.1. Fixed Earth Support

The Fixed Earth Support method, one of the classical approaches, relies on the premise that the toe of the wall does not move. With this assumption, the wall may be considered as a cantilevered beam above the point of fixity, permitting the assumption of a reaction at the point of fixity, F, as shown in Figure 2-5a. The third assumption is that the passive stress resultant is applied at a depth 0.8D.

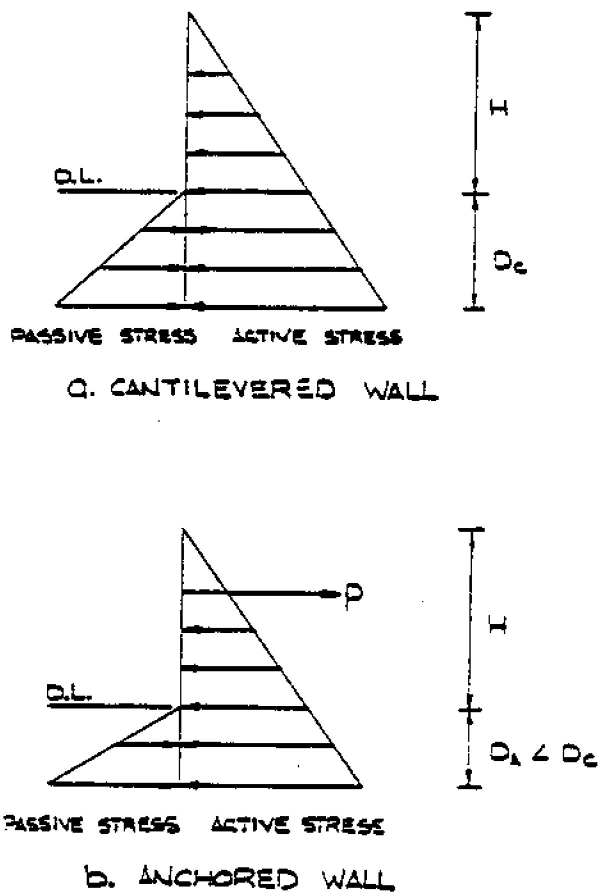


Figure 2-4. Stress distributions

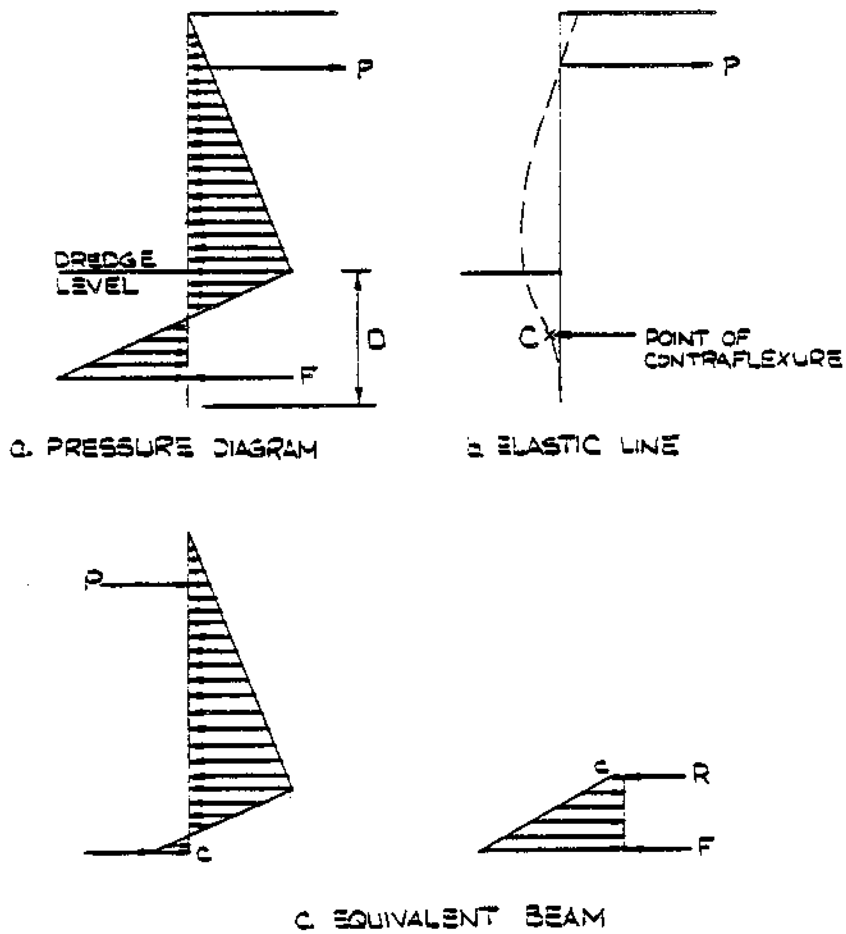


Figure 2-5. Fixed Earth Support assumptions

One way to analyze this case is to assume a depth of penetration, D, and compute the deflections of the wall based upon simple beam theory. If the deflection is not zero at 0.8D, another trial depth is attempted and deflections are recomputed. This process continues until a depth of penetration is achieved where the deflection computed at 0.8D is zero. This is the elastic line approach (Figure 2-5b).

Another approach simplifies the computations by assuming a hinge at the point of contraflexure, C, in Figure 2-5b. This permits the wall to be analyzed as two equivalent beams. The upper portion is treated as a simply supported beam with reactions at the tie-rod level and point of contraflexure, as shown in Figure 2-5c. The resultant forces are summed about the tie-rod level.

The active and passive stress coefficients suggested by Tschebotarioff (1951) are given by:

$$K_a = \tan^2 (45 - \phi/2) \text{ and} \tag{2-7}$$

$$K_p = I/K_a \tag{2-8}$$

Aside from the cumbersome numerical procedures involved, the Fixed Earth Support method has serious shortcomings that stem from the assumptions. Model tests have shown that deflections at the toe always occur (Rowe, 1952), thereby invalidating the premise that the wall may always be treated as a cantilever. Fixed Earth Support assumptions are good only for limited applications where toe deflections are relatively small.

2.3.2. Free Earth Support (FES)

This other classical method assumes that the toe of the wall is free to move, thereby enabling the full passive stress to develop along the pile below the dredge line. At the time of toe failure, the Free Earth Support (FES) stress distribution shown in Figure 2-6 can be computed using Coulomb's definitions for active and passive stresses.

Experiments have shown that the stress distribution for inadequate penetration is accurately described by the FES values (Rowe, 1952). This means that the minimum penetration depth where failure is imminent may be computed. The penetration is then adjusted so that the minimum depth is exceeded and a margin of safety is realized.

For penetration less than the required minimum depth, equilibrium is not achieved and the wall rotates as a rigid body. For penetration exceeding the minimum value, rigid body movement no longer occurs and the stresses are redistributed because of the flexibility of the wall. This redistribution causes the computation of bending moments, based upon FES assumptions, to be overly conservative and thereby uneconomical. In spite of this inaccuracy, it still remains a useful procedure for computing penetration depths, although an alternative procedure for calculating bending moments and tie-rod loads is warranted.

2.3. Danish Rules

In spite of the rational approaches provided by the classical methods, quay walls in Denmark around 1900 were built with the guidance that "dimensions appear to be reasonable" (Tschebotarioff, 1951). Increased commerce at this time led to the demand for higher walls,

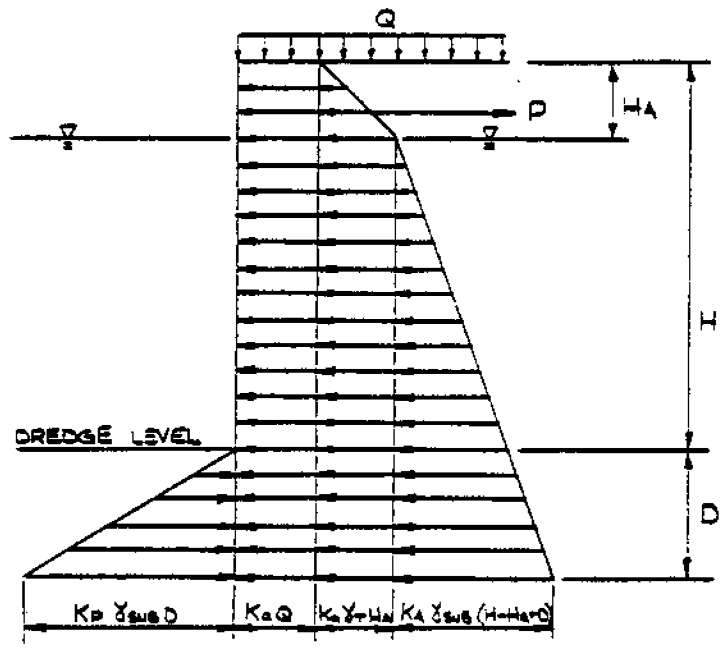


Figure 2-6. Free Earth Support assumptions

which in turn necessitated more stringent design procedures. Use of the Coulomb procedure to check timber walls already built showed that the stresses in these walls were three to four times higher than allowable stresses for timber. Since the walls had withstood the test of time with no apparent malfunction, it was surmised that the actual stresses were substantially less than the stresses predicted from the Coulomb method. With this deviation in mind, the Danish engineers Christiani and Nielsen designed the Åalborg Pier in 1906. This was considered a daring undertaking, not only because the pier was underdesigned with respect to Coulomb guidelines, but also because it was made of reinforced concrete and not timber. Although the design has often been criticized for lack of conservatism, the structure has stood for decades (Tschebotarioff, 1951).

One reason for the pier not failing is the presence of piles driven through the backfill into the subgrade. These piles transfer any surcharge load to below the subgrade so that this load does not add to the horizontal soil stresses already acting on the wall. Another more significant reason is a redistribution of stresses because of soil arching. As the wall deflected horizontally, the fill deformed so that an arch of soil formed between the tie-rod and dredge levels. The arch then carried part of the horizontal load imposed by the fill. This arching concept formed the basis for a set of design procedures called the Danish Rules.

The stress diagram for this formulation appears in Figure 2-7. The Free Earth Support stress is reduced by an amount defined by the parabola with amplitude, q, such that:

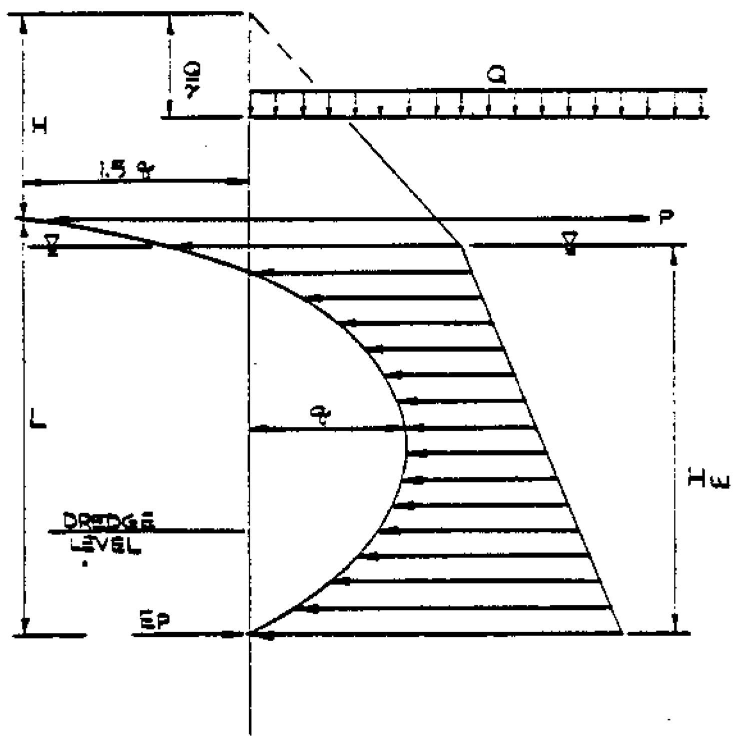


Figure 2-7. Danish Rules assumptions

$$q = \frac{k\ (4 + 10\ h/L)}{5 + 10\ h/L}\ P_m \tag{2-9}$$

and

$$k = \frac{1}{1 + \frac{0.1}{\sin\phi}\ (\frac{1+n\ Ea}{L\sigma})} \tag{2-10}$$

in which: h = distance from the tie-rod to the top of backfill, n = the ratio of bending moments at the tie-rod and at the dredge level, E = the elastic modulus of the sheet pile, a = the wall thickness, P_m = an assumed distributed load, and σ = the allowable bending stress of the wall.

The depth of penetration is taken as 3 to 3.5 times the distance H_w and then multiplied by a safety factor.

Although the Danish Rules have produced successful bulkheads, this approach is not recommended as it lacks rigorous analytical or experimental substantiation. However, the rules demonstrated the validity of using reduced stresses acting on the wall.

2.4. Limit Equilibrium Approaches

A method for solving soil stress problems based upon rupture theory was devised by Hansen (1953). The underlying principle of this approach is that a soil mass in a state of failure takes on a specific geometry, i.e., a specific figure of rupture (Figure 2-8). When the figure is established, Kötter's equation is used to compute soil stresses and the kimetatics are computed as shown in Figure 2-9. By varying certain dimensions, critical rupture figures can be determined. The design of the structure can then be completed by using the forces and moments stemming from the critical conditions.

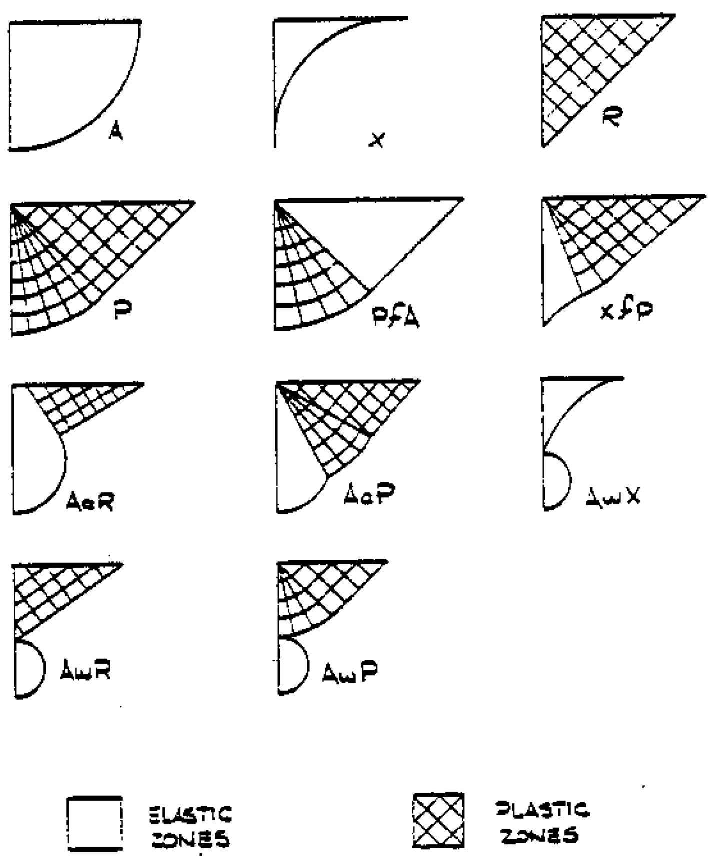


Figure 2-8. Rupture figures (Hansen, 1953, pp. 73-79)

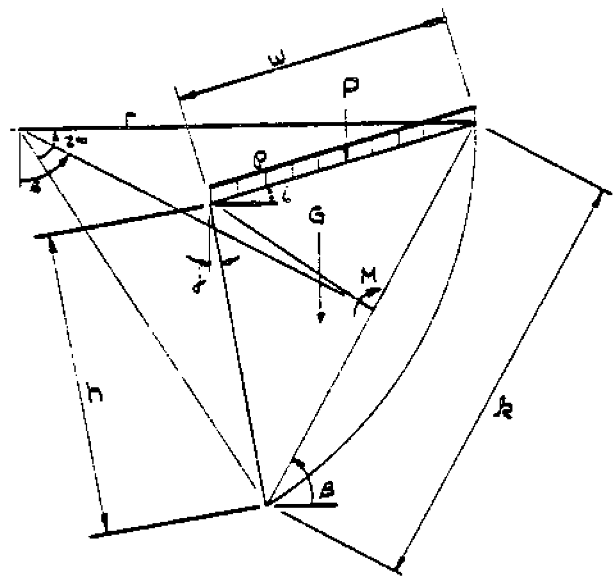


Figure 2-9. Kinematics of a rupture figure (Hansen, 1953, p. 104)

Brinch Hansen's approach appears attractive in that it enables the designer to obtain a true concept of the forces involved which tend to produce a particular mode of failure. Use of Kötter's equation in computing the stresses of soils in a plastic state is quite valid and enhances the accuracy of the computations. In spite of these benefits, the procedure is very tedious because many iterations are necessary to arrive at a satisfactory solution and Kötter's equation is very cumbersome.

2.5. Studies by Tschebotarioff

Large-scale model tests of bulkheads were conducted by Tschebotarioff at Princeton (1948) to corroborate or refute earlier concepts of bulkhead behavior. Tests were performed with three objectives in mind: reducing stresses acting on the wall from a fluid clay backfill; determine the effects of consolidation upon the magnitude of stresses exerted on the wall and observe the phenomenon of arching; investigate the distribution of stresses acting upon the wall.

The placement of dredge spoil as backfill is common practice as it greatly reduces the amount of fill required from a borrow area. There is an obvious advantage to this practice, but there are two significant disadvantages. Fluid clay has such a high water content that it behaves as a fluid, i.e., it has very little shear strength and the horizontal stresses are much higher than those from normal backfill. Also, the fluid clay must consolidate prior to any operations on its surface, such as construction of buildings. The studies involving fluid clay backfills are thus noteworthy.

An important consideration in these tests is the range of soils used. The angle of internal friction of the sands studied range between 32° and 36°, indicating that the sands were in the loose to medium dense range. The clay used, except for the fluid clay backfill, showed a cohesion of 300 psf (14.4 Pa) and an angle of internal friction of 17°, determined from consolidated-undrained shear tests. A mixture of sand and clay was produced with a resulting angle of internal friction of 32°.

Tests were conducted to determine the means required to minimize the horizontal stresses exerted by a fluid clay backfill. It was found that a sand dike placed at its natural angle of repose, shown as line 6-6 in Figure 2-10, was fully effective in reducing the stresses exerted by the fluid clay fill, i.e., the stresses were the same as if the entire fill was composed of sand. The same results were found when a sand blanket was placed whose width was equal to the wall height, as shown by line 8-8. A sand blanket whose width was 50 percent of the wall height, as shown by line 9-9, was 50 percent effective. A blanket width of 10 percent of the wall height was found to have no effect.

The presence of the sand dike or sand blanket did not enhance the rate of consolidation, but prefabricated cylindrical drains did. Vertical drains were acceptable, but were difficult to place because of construction impediments. Horizontal drains, on the other hand, were conceived as shown in Figure 2-11. It was felt that, although such drains would be expensive, they would be practical and would accelerate consolidation.

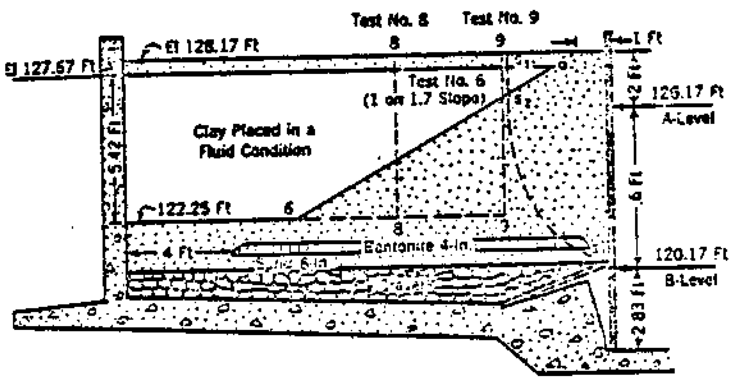


Figure 2-10. Test apparatus (Tschebotarioff, 1949, p. 25)

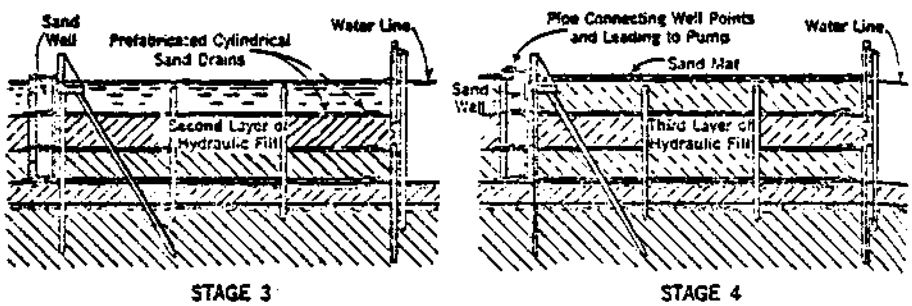


Figure 2-11. Sand drains to accelerate consolidation (Tschebotarioff, 1949, p. 28)

A major assumption of the Danish Rules is that an arch of soil forms between the tie-rod and dredge level which reduces the horizontal stresses acting upon the wall, as suggested by Figure 2-8. Tschebotarioff felt that this arching phenomenon warranted closer scrutiny. He made a distinction between dredge and fill bulkheads based upon his observations of arching.

For an arch of sand to form, a stable "abutment" must first be present. Then, as the wall deflects between the tie-rod and dredge level, an arch forms between these two abutments. For fill bulkheads, this abutment is present at the dredge level, but is lacking at the tie-rod until the fill is raised beyond that level. As the fill is placed, the wall deflects and no arch may form without the second abutment. Dredge bulkheads, on the other hand, allow the formation of an arch when the material in front of the wall is removed. When the two abutments are present, the dredging operation causes wall deflections between the tie-rod and final dredge level, and an arch forms. However, the arch is unstable as additional tie-rod yield causes it to break down.

A recommended design procedure evolved after the third set of tests. The approach suggested was a simplified equivalent beam procedure where a hinge is assumed to be located at the dredge level. For bulkheads in a subgrade of clean sand, the depth of penetration is taken to be 43 percent of the wall height, H, based upon limited test results. The factor of safety against toe failure was said to be at least 2.0. The active stress was computed from:

$$P_a = K_a \gamma_s H, \text{ where} \tag{2-11}$$

$$K_a = (1 - \frac{a}{f' H}) \; 0.33 \; f''', \tag{2-12}$$

in which: a = the height of soil above the tie-rod, $f' = 3.5$ and $f''' = 0.9$, based upon limited test results. Bending moments can be computed from the stress diagram (Figure 2-12). Tie-rod pulls should be designed for overstressing by dividing computed loads by the expression:

$$(1 - \frac{a}{f' H}) \; f'' \tag{2-13}$$

The term $f'' = 1.0$ for known subgrade materials and should be decreased for uncertainties in the subgrade.

A further observation made with respect to vibrating the backfill was that it increased the bending moments by 60 percent; similar vibration of the soil in front of the bulkhead tended to reduce the bending moments.

The tests at Princeton did not establish any valid relationship between the shear strength of clay and lateral stresses. This lack of correlation was interpreted to signify that once a safe depth of penetration was established, horizontal stresses in clay are a problem of deflection, not of rupture.

Since the range of soils tested was limited to a narrow band, the empirically derived formulas for bulkhead design are valid only for that range. As soils vary beyond the test range, their stress distributions must also vary, especially for clays. A more comprehensive design procedure is needed which encompasses a broader spectrum of soil conditions.

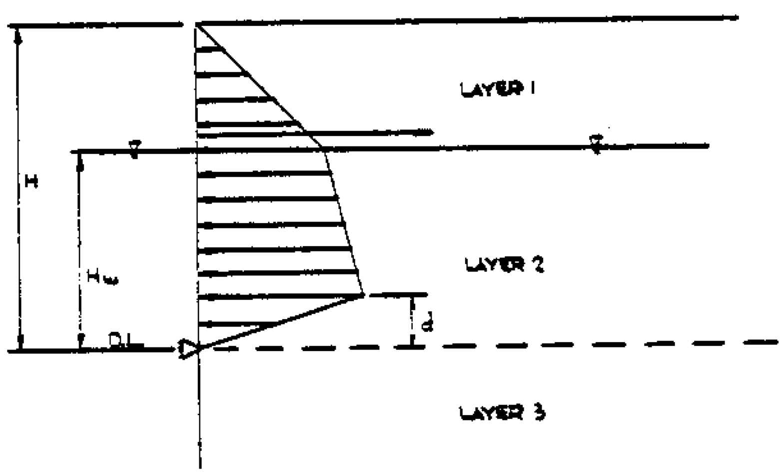


Figure 2-12. Design assumptions (Tschebotarioff, 1951, p. 561)

2.6. Studies by Rowe

Rowe contributed significantly to the understanding of bulkhead behavior (Rowe, 1951, 1952, 1955, 1956, 1957). His work began by observing the performance of scale model bulkheads in cohesionless soils where he focused upon the effects of sheet pile flexibility and soil stiffness. Based upon his findings, he formulated a bulkhead design procedure. He then developed a theoretical and analytical model where bulkhead behavior could be described as a beam on an elastic foundation. Several years later he performed further tests on walls in a cohesive subgrade, coupled these data with his previously developed analytical model, and recommended a procedure for the design of walls in clay. In subsequent work, he compared designs based upon his recommended procedures with Hansen's approach. Rowe's work was extensive well-documented, and it provided an insight that is very helpful in understanding bulkhead behavior.

2.6.1. Anchored Walls in Sand

Rowe felt that variations in the distribution of stress acting upon sheet pile walls resulted from variations in surcharge, tie-rod level, anchor yield, dredge level, pile flexibility and soil stiffness. To determine such effects, he instituted two series of stress tests and one series of flexibility tests (Rowe, 1952).

The stress tests were conducted on a 3 ft-6 in (1.07 m) high model wall, as shown in Figure 2-13a. The sequencing of these tests is shown in Figure 2-13b. Stress measurements were made directly by stress gauges, and bending induced strains were measured by strain

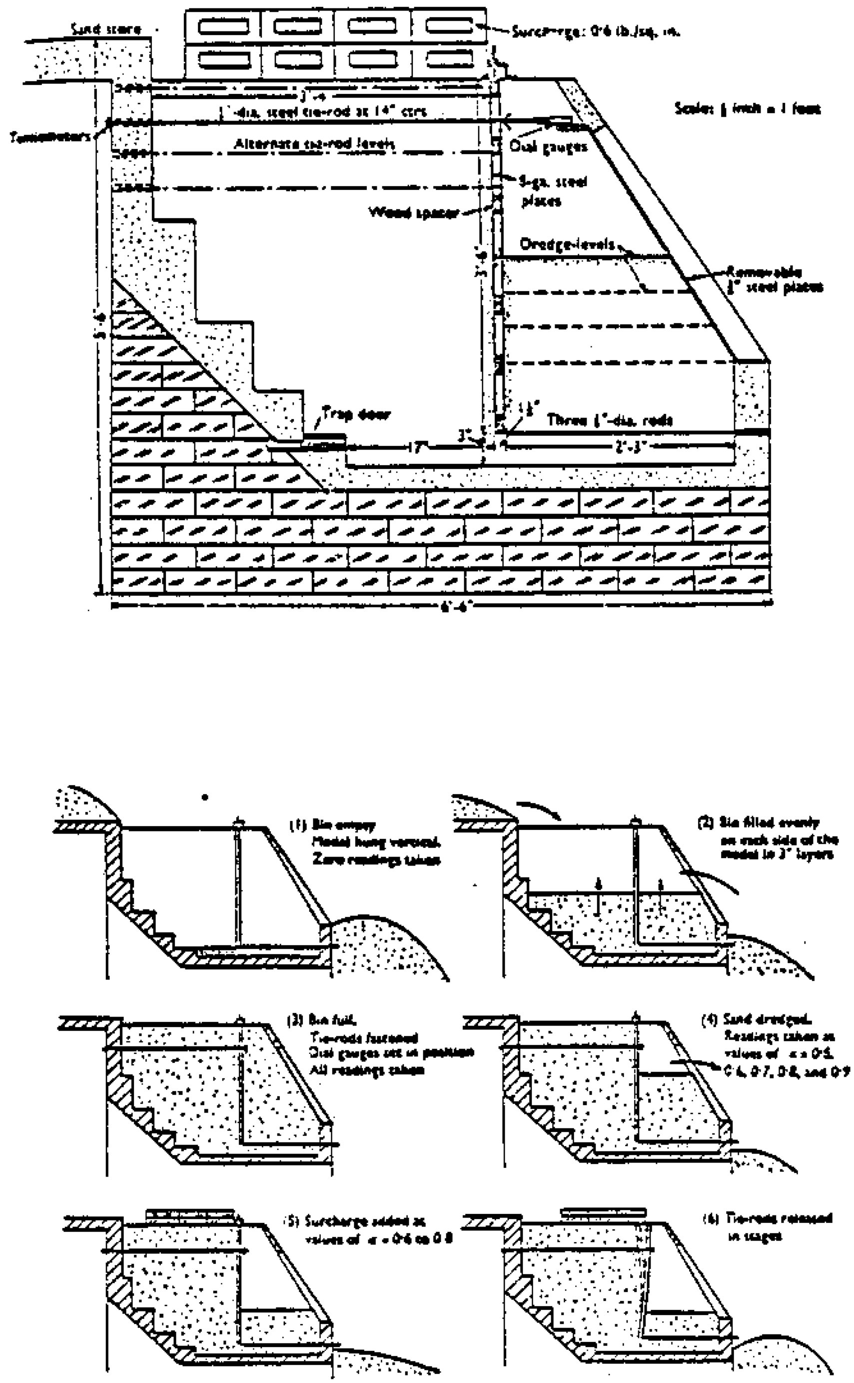


Figure 2-13. Stress tests

gauges. The only soil used in the stress tests was dry sand in a loose state.

The flexibility tests were conducted in the apparatus shown in Figure 2-14. The properties of the different piles used are given in Table 2-1. Different soils were used, each with a different angle of internal friction and dry unit weight. Each soil was tested in the loose state, with relative density equal to 0 percent, and in the dense state, with relative density equal to 100 percent. The soil properties are summarized in Table 2-2.

2.6.1.1. Conclusions Based Upon the Stress Tests

The first series of stress tests demonstrated that the initial stress distribution deviated from Coulomb's FES predictions. As the dredging continued, however, the stress distribution eventually reached the free earth values when toe failure occurred. Prior to failure, stress increases developed above the tie-rod and decreases developed below, i.e., arching occurred. The stress reduction, because of arching, was substantially less than that predicted by the Danish Rules. The first series of tests also showed that a considerable shear force developed at the toe which tended to resist outward movement.

The second series of stress tests incorporated controlled anchor yield while the first series permitted none. The placement of various surcharge loads was another added feature. This series showed that arch instability resulted with anchor yield or additional dredging and that the stress distribution developed was in accordance with Free Earth Support predictions. The amount of yield necessary for the

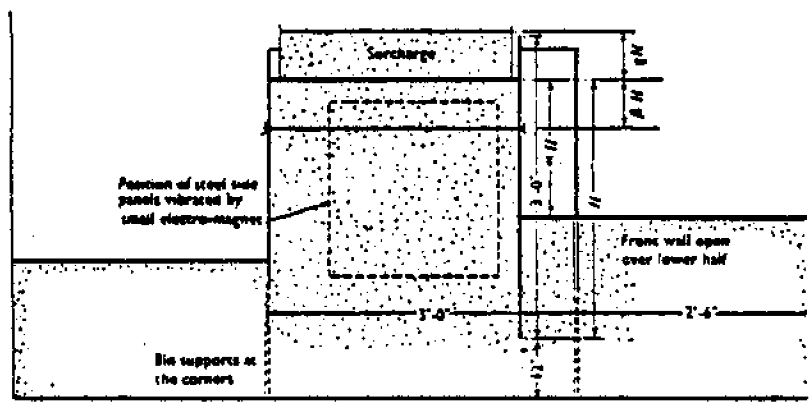


Figure 2-14. Apparatus for flexibility tests (Rowe, 1952, p. 38)

Table 2-1. Pile characteristics (Rowe, 1952)

Material	Plate Thickness in	(mm)	Pile Length in	(m)	Flexibility log ρ	Test
Steel	0.330	(8.38)	42	(1.07)	-3.32	Stress Tests
Steel	0.164	(4.19)	36	(0.91)	-3.18	"
			32	(0.81)	-3.38	"
			30	(0.76)	-3.49	"
			28	(0.71)	-3.61	"
			26	(0.66)	-3.74	"
Steel	0.109	(2.77)	36	(0.91)	-2.52	Flexibility Tests
			31.5	(0.80)	-2.74	"
			27.5	(0.70)	-2.98	"
			24	(0.61)	-3.22	"
			21	(0.53)	-3.45	"
Aluminum	0.083	(2.11)	29	(0.74)	-2.07	"
			26	(0.66)	-2.26	"
			23	(0.58)	-2.48	"
			20	(0.51)	-2.72	"

Table 2-2. Soil properties (Rowe, 1952)

	Loose State: $D_r = 0\%$				Dense State: $D_r = 100\%$			
	Dry Unit Wt.		Angle of Int. Fric. (Degrees)	Void Ratio	Dry Unit Wt.		Angle of Int. Fric. (Degrees)	Void Ratio
	$\frac{lb}{ft^3}$	$(\frac{kN}{m^3})$			$\frac{lb}{ft^3}$	$(\frac{kN}{m^3})$		
Sand	90	(14.1)	30	0.78	100	(15.7)	41	0.53
Dorset Pea Gravel	98	(15.4)	30	0.74	110	(17.3)	37	0.49
Whinstone Chips	82	(12.9)	39	1.06	96	(15.1)	50	0.74
Ashes	40	(6.28)	40	1.76	56	(8.79)	50	0.95

complete breakdown of arching was equal to $H_D/1000$. Rowe stated that the amount of yield one could expect in the field is between $H_D/930$ and $H_D/360$. In other words, arching is not a stable state under normal conditions.

The active stresses acting upon the model walls were found to agree closely with Tschebotarioff's predictions. Bending moments, however, were at times found to be as much as twice as high. Rowe surmised that this discrepancy could be resolved by observing the effects of varying the pile flexibility. This was the objective of the flexibility tests.

2.6.1.2. Conclusions Based Upon the Flexibility Tests

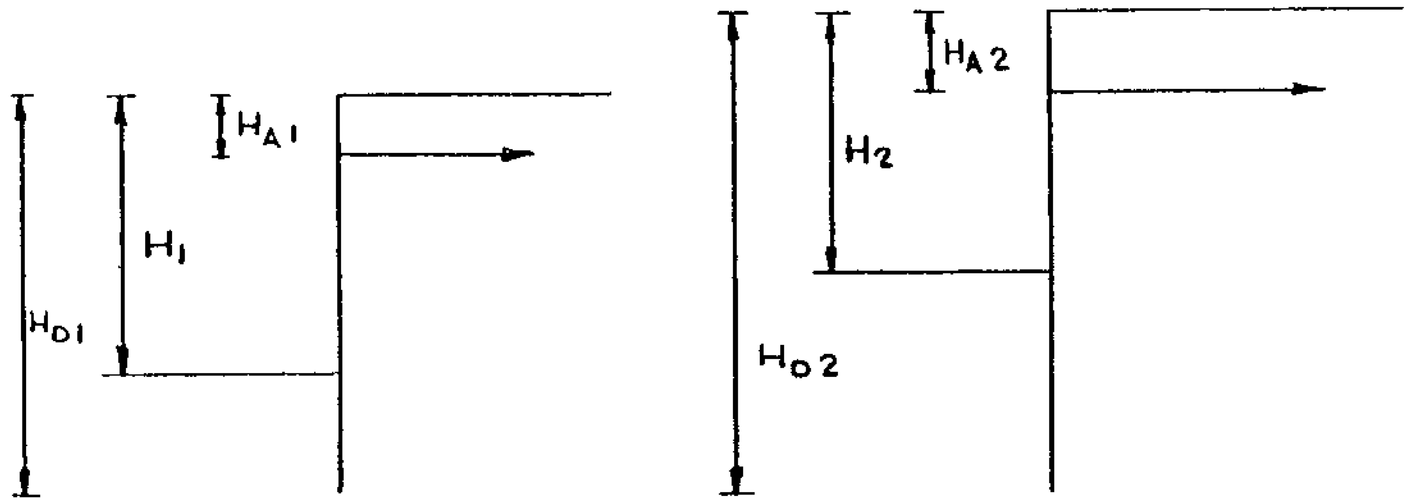
Rowe determined that prototype walls must behave in the same manner as the model walls if the conditions of similitude are maintained. The most important aspects of these conditions shown by the tests are two ratios. The first proportionality states that bending moment, M, and pile length, H_D, are related by the constant τ, such that

$$\tau = \frac{M}{H_D^3} \tag{2-14}$$

The second states that the pile length, elastic modulus of the pile and moment of inertia of the pile are related by the pile flexibility number, ρ, such that:

$$\rho = \frac{H_D^4}{EI} \tag{2-15}$$

He then concluded that the behavior of prototype and model walls must be similar if their relative wall heights, α (Figure 2-15) are equal,



$H_1 = 3, \quad H_{D1} = 5, \quad H_{A1} = 1$

$H_2 = 6, \quad H_{D2} = 10, \quad H_{A2} = 2$

$$\alpha = \frac{H_1}{H_{D1}} = \frac{H_2}{H_{D2}} = 0.6$$

$$\beta = \frac{H_{A1}}{H_{D1}} = \frac{H_{A2}}{H_{D2}} = 0.2$$

Figure 2-15. Relative wall height and relative tie rod level and relative tie-rod levels, β, are equal, where

$$\alpha = \frac{H}{H_D} \tag{2-16}$$

and

$$\beta = \frac{H_A}{H_D} \tag{2-17}$$

It was determined that pile flexibility had a major effect upon stress distribution and bending moment. As demonstrated in Figure 2-16a, a more flexible pile permits larger deflections, Δ, at the dredge level relative to the deflections at the toe. The larger deflection causes a greater amount of passive stress to be mobilized at that point. Consequently, the passive stress resultants occur closer to the dredge level with more flexible piles, as shown in Figure 2-16b. The influence of pile flexibility in dense subgrades is similar, but with a more pronounced effect as the passive stress resultant was located even closer to the dredge level.

The flexibility tests also indicated that tie-rod loads differ from the Free Earth Support values, depending upon relative tie-rod height, β, relative wall height, α, and pile flexibility. It was also shown that tie-rod loads could be increased by as much as 50 percent because of differential tie-rod yield and anchor settlement, i.e., adjacent tie-rods may deflect unevenly, thus causing one tie-rod to take more of the load.

2.6.1.3. Design Procedure for Anchored Walls in Sand

As well as providing a sound qualitative description of bulkhead behavior, Rowe's observations and conclusions served as a basis for

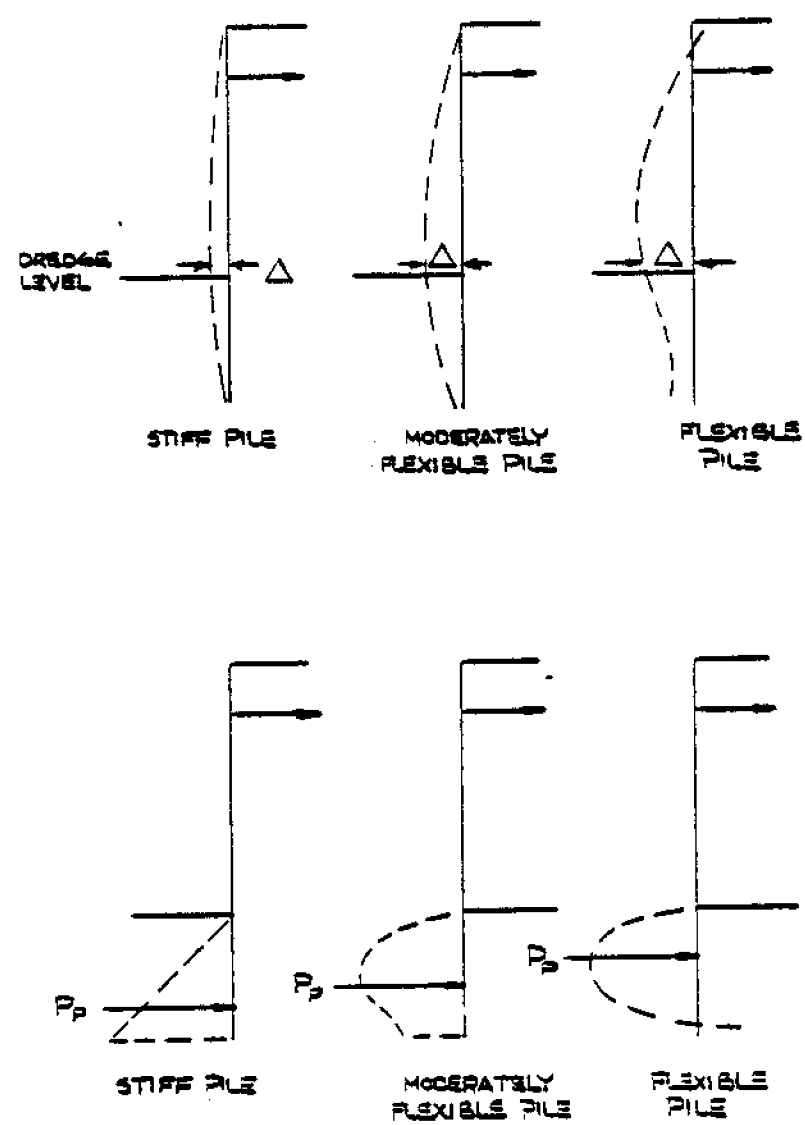


Figure 2-16. Effects of pile flexibility on pile deflections and passive stress

computing penetration depths, bending moments, and tie-rod loads. Since much of Rowe's observations were reported in terms of deviations from FES values, it is not surprising then to find that his recommended design procedure begins by computing the FES values. These values are modified by employing factors derived from the tests, the factors depending upon relative wall height, relative tie-rod level, pile flexibility and the relative density of the subgrade.

It has been suggested that once a safe penetration depth has been achieved, bulkhead design is a matter of deflection (Rowe, 1952; Tschebotarioff, 1948). Rowe's work clearly established that the stress distribution acting upon the wall at the time of toe failure was accurately described by the FES method. Hence, the FES method can be used to compute a safe penetration when safety factors are applied to the loads. Once the penetration depth is computed, its maximum bending moment and tie-rod loads are computed using the FES stress distribution. The safety factor used for the penetration computation is not used for the moment and tie-rod computations.

The FES bending moment is used to determine the design bending moment by incorporating a reduction factor, r_d, chosen from Figure 2-17a. The reduction factor is read directly from the figure for the appropriate relative wall height, α, and subgrade relative density. The reduction factor is chosen for several values of pile flexibility, ρ.

For the conditions of similitude to be obeyed, the maximum bending moment is converted to

$$\tau_{max} = \frac{M_{max}}{H_D} \qquad (2\text{-}18)$$

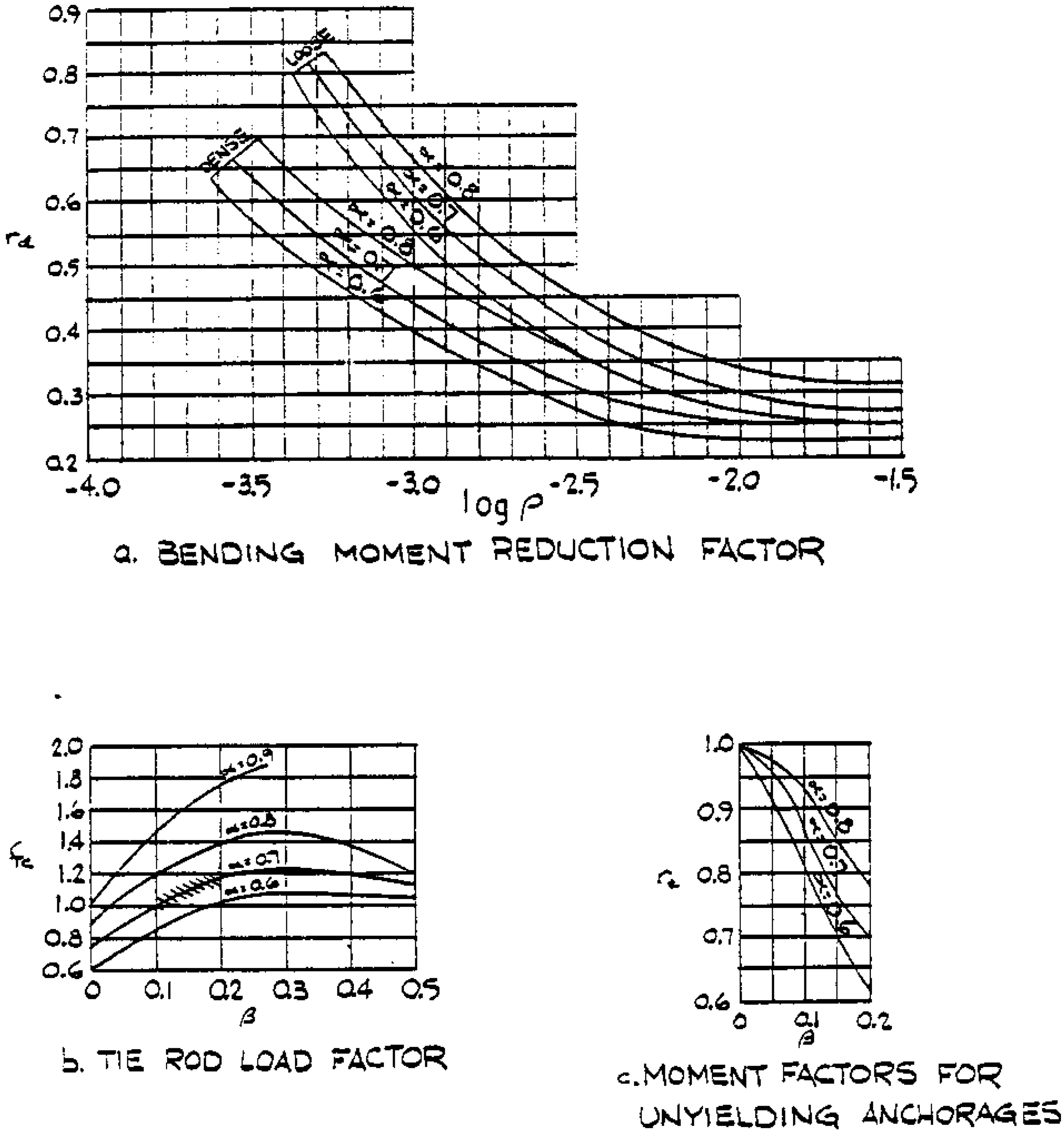


Figure 2-17. Tie rod and bending moment factors, sand (after Rowe, 1952, p. 45; 1956, p. 308)

where M_{max} is the maximum bending moment in inch-pounds. An operating curve is then developed as shown in Figure 2-18 where

$$\tau_{op} = \tau_{max} \cdot r_d \tag{2-19}$$

and r_d = the reduction factor for that particular value of log ρ. A structural curve is then developed for each value of ρ with

$$\tau_{STR} = \frac{\psi}{(H_D \rho^2)^{1/3}} \tag{2-20}$$

and

$$\psi = \frac{f_b}{(EI)^{2/3}} \tag{2-21}$$

where ψ = flexibility characteristic, f_b = allowable bending stress, S = section modulus, E = elastic modulus of the pile material, and I = moment of inertia. The intersection of the operating and structural curves gives the solution in terms of τ. The design bending moment then may be computed by using Eq. 2-18.

The tie-rod load is more simply computed by multiplying the FES value by the tie-rod load factor, f_c, found in Figure 2-17b. The factor, f_c, is read directly for the appropriate values of α and β.

For dredge type bulkheads with unyielding anchorages, additional reductions in bending moment may be computed by using Figure 2-17c. The reduction factor, r_t, is read for appropriate values of α and β.

The FES method and Rowe reduction methods are quite lengthy procedures. They are described in greater detail in a later section. Design examples may be found in the Appendices.

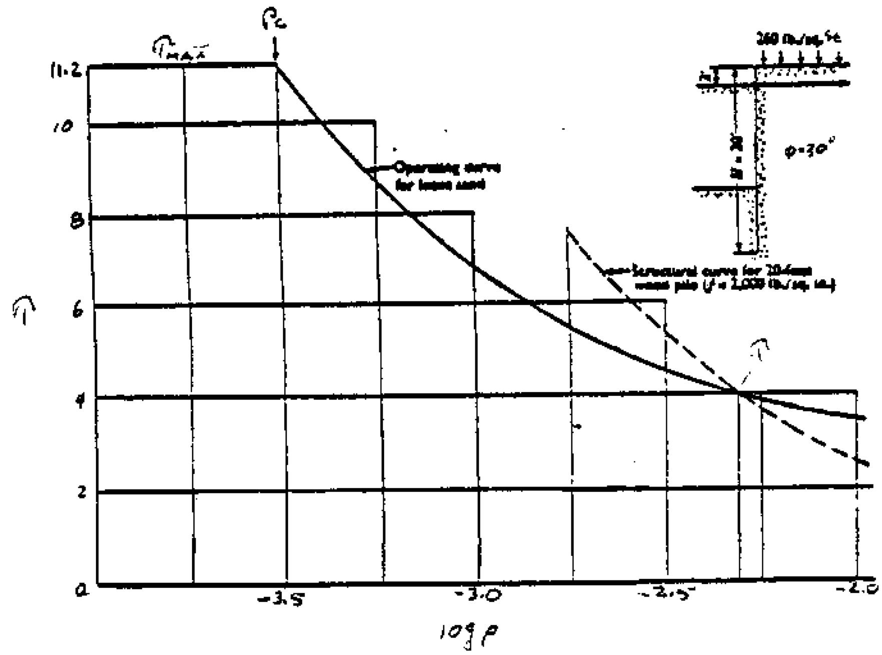


Figure 2-18. Typical operating and structural curves (Rowe, 1952, p. 54)

2.6.2. Comments by Terzaghi

Terzaghi reviewed the works of Tschebotarioff and Rowe shortly after Rowe's scale model test results were published (Terzaghi, 1954). He stated that Tschebotarioff was in error to suggest that the Fixed Earth Support method be used for all calculations since the fixity of the pile toe ranged between fully free and fully fixed, depending upon pile flexibility and the relative density of the subgrade material. He agreed with Rowe that soil stresses can be computed based upon Coulomb's formulation, the maximum bending moment can be found using the Free Earth Support method, and a reduction should be applied to the maximum moment, depending upon pile flexibility and subgrade relative density.

In this work Terzaghi also suggested the scope of exploration required for bulkheads. He recommended standard penetration tests and laboratory tests for sands. For clays, he recommended undisturbed sampling for laboratory tests in addition to vane shear tests. The exploration should also be of such an extent that it reveals soft soils beneath the pile tip which could cause excessive settlement and slope failures of submerged soils in front of the bulkhead which could undermine the stability of the toe.

2.6.3. Theoretical Analysis

Rowe performed a theoretical analysis of sheet pile walls by modeling the wall as a beam on an elastic foundation. The differential equation which governs the model behavior is

$$EI \frac{d^4y}{dx^4} - kv = 0 \tag{2-22}$$

in which: E = elastic modulus of the beam (pile), I = moment of inertia, y = axis in the direction of beam deflections, v = magnitude of beam deflections, x = axis of the long dimension of the beam, and k = subgrade modulus in stress units (Rowe, 1955).

For a subgrade modulus that increases linearly with depth, the differential equation must be solved by series. The resulting polynomial for Rowe's solution was of the 30th order, a very cumbersome expression. Nevertheless, he proceeded to compute deflections and bending moments for walls in sand and in clay.

A comparison was made between the results of the theoretical analysis of anchored walls in sand and the observations made on the tests of model walls. The comparison showed very good agreement, except for very stiff walls in dense sand. This apparent discrepancy is not important since, it is pointed out by Rowe, the stiffness of the walls in the anomolous case was beyond the range normally encountered in the field.

The theoretical analysis is too unwieldly to use as a design tool, but the agreement with the experimental evidence of walls in sands suggests that it may be useful in providing information about walls in clay.

2.6.4. Anchored Walls in Clay

Rowe approached the problem of a wall in clay as a beam on an elastic foundation (1957). He stated that the subgrade modulus could be related to its cohesion in terms of Skempton's stability number (1945),

$$S_t = \frac{c}{\gamma_s h+q} \sqrt{1 + \frac{c}{c_w}} \qquad (2\text{-}23)$$

in which: c = cohesion in the subgrade, c_w = adhesion on the wall, h = overburden stress of the fill, and q = surcharge. He also noted that the term $\sqrt{1 + \frac{c}{c_w}}$ could be taken as 1.25 in most cases.

Incorporating Terzaghi's work in determining subgrade moduli (Terzaghi, 1955), Rowe developed a relationship using the subgrade modulus, subgrade compressibility and stability number. The beam on elastic foundation analysis proceeded with variations of pile flexibility and stability number. Theoretical bending moments were compared with FES values and the percent reduction was plotted versus log ρ.

A series of scale model tests was performed which defined the limits of applicability of the theoretical analysis. The tests also substantiated the accuracy of the analysis. Correlating the theoretical and experimental data, Rowe presented three figures for the amount of reduction allowed as a function of stability number, which are shown in Figure 2-19. The figures represent pile flexibilities which will give three points on an operating curve. The flexibilities represented are: maximum stiffness for log ρ = -3.1, minimum stiffness for log ρ = -2.0, and a typical working stiffness for log ρ = -2.6.

Operating and structural curves are generated in the same manner as for anchored walls in sand. Once the design flexibility is determined, Figure 2-19b is used to find the required tie-rod load factor, using the stability number of the subgrade and design log ρ of the wall. A detailed procedure is found in a later chapter.

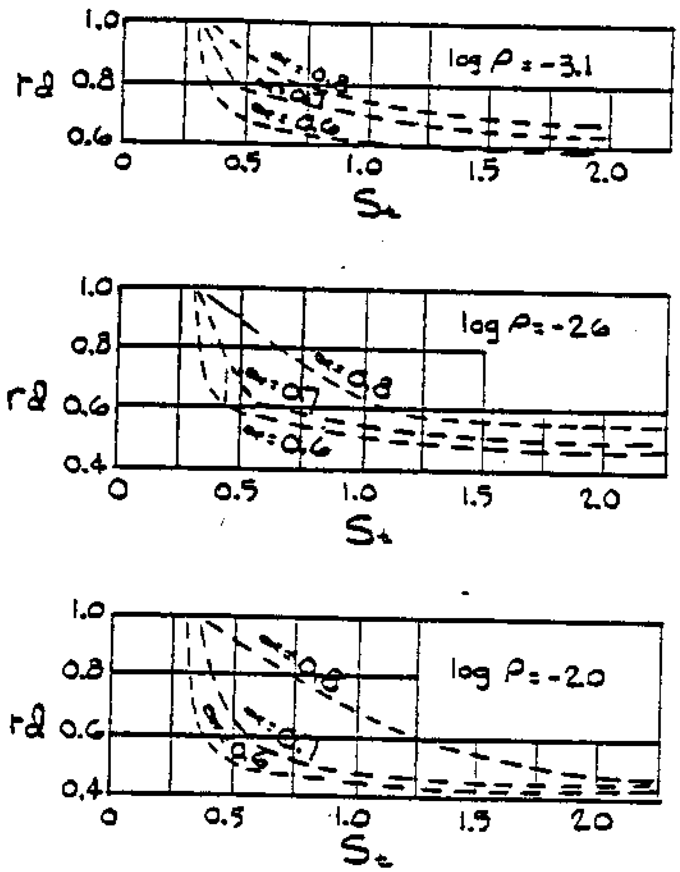


Figure 2-19. Tie-rod and bending moment factors, clay (after Rowe, 1957, p. 642)

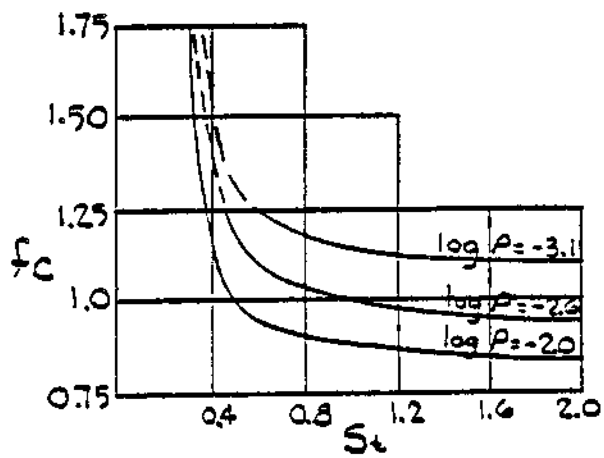


Figure 2-19. Continued

2.6.5. Comparison with Limit Equilibrium Approach

Rowe computed bulkhead designs based upon Hansen's limit equilibrium analysis and compared these to the results of the scale model tests in sand (1956). In general, the limit equilibrium and model test results were in close agreement.

In addition to corroborating the moment reduction method, this comparison led to other observations that enhanced bulkhead design. One such observation was that the most economical designs resulted where the relative wall height, α, was approximately 0.73 and the relative tie-rod location, β, was approximately 0.20. The finding that tie-rod loads should be factored within a range between 0.88 and 1.25 was also a consequence of this comparison and is reflected in Figure 2-17b. And, based upon this work, it was clearly shown that with sufficient penetration, bulkhead design becomes a problem of deformation, not ultimate collapse.

2.6.6. Cantilevered Walls in Sand

One of Rowe's earlier works dealt with cantilevered walls in sand (1951). His studies proceeded in a manner similar to the anchored wall studies. A series of tests were conducted that compared the amount of moment reduction from the FES method depending upon relative wall height, α, pile flexibility, ρ, and relative density of the subgrade. The reduction curves shown in Figure 2-20 resulted from these studies.

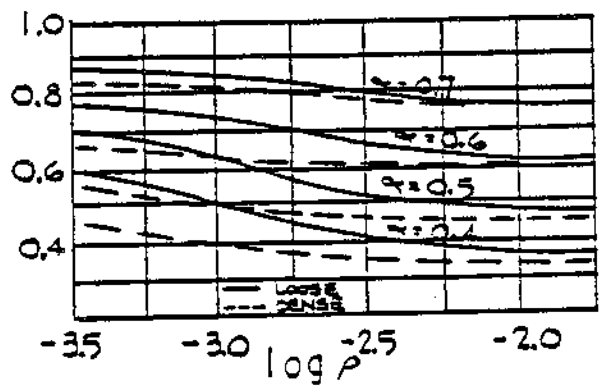


Figure 2-20. Bending moment factors, cantilever walls in sand (after Rowe, 1951, p. 319)

2.7. Numerical Methods Analyses

The rapid development of the digital computer enhanced the viability of the finite element method of analysis (FEM) to a great extent. This method has been extremely valuable in describing the complex phenomena of soil-structure interaction. The finite element method has been applied to assess many soil stress problems.

One such application was an analysis of the Port Allen and Old River locks. Clough and Duncan developed an incremental finite element analysis with nonlinear, stress dependent, inelastic soil stress-strain behavior (1969). The analysis was accurate in predicting the behavior of these U-shaped, reinforced concrete structures as was shown by comparisons with the extensive instrumentation which was installed to monitor the locks.

An investigation of the behavior of high anchored bulkheads in Norway was reported by Bjerrum, Clausen and Duncan (1972). The bulkheads were instrumented with strain gauges and inclinometers were installed in the adjacent soil. A finite element analysis of the bulkheads was conducted using a modified version of the Port Allen computer program. Comparison of the FEM results with the instrumentation data and Rowe reduction method showed good agreement.

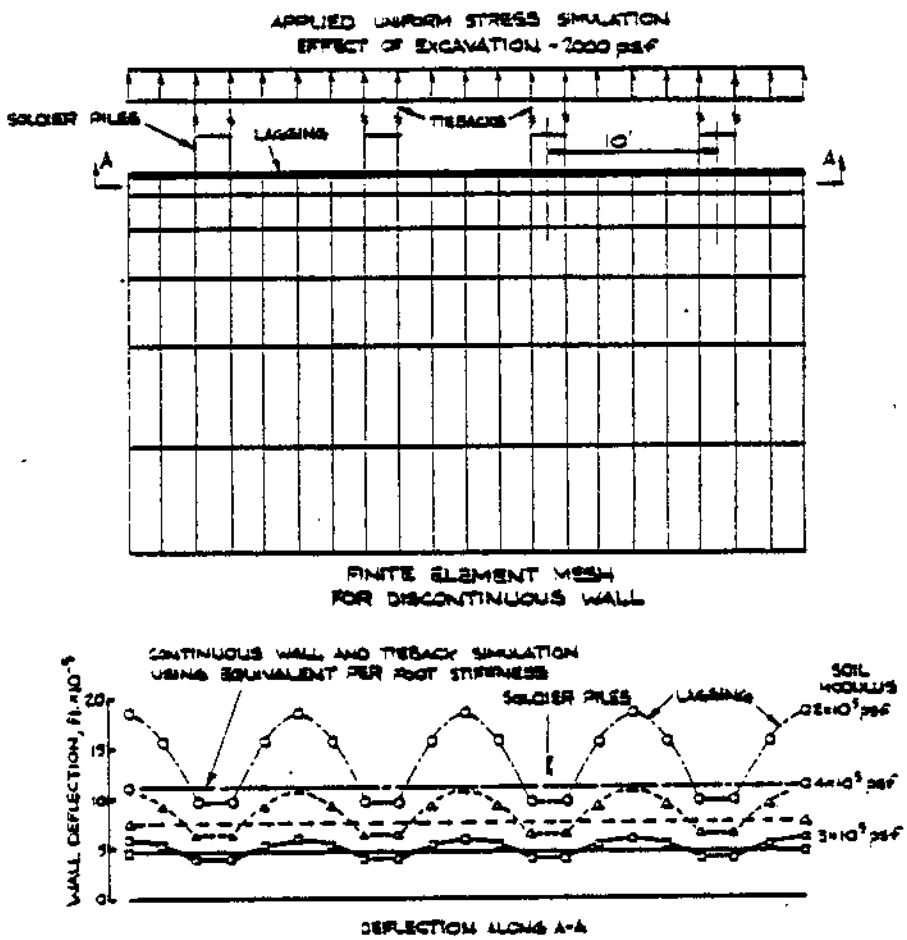
Finite element analysis has also been a tool for examining the behavior of tie back excavations. Although this behavior is somewhat different from bulkhead behavior in that anchors are employed at multiple levels and are basically unyielding, some observations can be applied to bulkheads on a qualitative basis.

In a study by Tsui (1974), discontinuous wall behavior was examined. A soldier pile and lagging wall, or Berlin wall, was first analyzed by FEM as a continuous, planar wall, then as a discontinuous wall. An equivalent planar wall was developed by distributing the stiffness of the soldier piles across the spacing between adjacent piles. The discontinuous wall was modeled by stimulating the ties as spring supports, applying a soil stress of 1 tsf (96.2 N/m^2) and varying the soil modulus as 100 tsf (9.61 kN/m^2), 200 tsf (19.2 kN/m^2), and 400 tsf (38.5 kN/m^2). Comparisons of these two models (Figure 2-21) show that deflections in the lagging were 70 percent greater for the planar wall in soft soil, and 27 percent greater in stiffer soil. The Berlin wall behavior is analogous to the behavior of navy bulkheads where the 8 in (0.2 m) fender piles are similar to the soldier piles as they represent great increases in stiffness at discrete points along the wall. The navy bulkhead problem will be addressed later in this work.

2.8. Soil-Structure Interface Strength

The strength of the soil-structure interface is an important aspect of bulkhead behavior, as suggested by the Coulomb formulation for active and passive soil stress coefficients (Eqs. 2-2 and 2-3). The interface strength, δ, was suggested by Rowe to be taken as $2/3\phi$ for steel and timber sheet piles (Rowe, 1952). This recommendation was made without the corroboration of significant test results.

A more recent study has, however, addressed interface strength more comprehensively. Kulhawy and Peterson (1979) conducted tests using concrete blocks with four variations in roughness, three relative



Note: 1 ft = 0.305 m; 1 psf = 0.0478 N/m^2

Figure 2-21. Results of finite element analysis of discontinuous walls (Tsui, 1974, p. 3-7-2) densities for each of two soil types, and three normal stresses. The tests were performed in a direct shear device.

It was pointed out that the causitive aspect of the interaction lay in the relative roughness of the structural face with respect to the roughness of the soil, i.e., large soil particles and small asperities in the wall allow the soil particles to skid across the wall, while small or large particles acting along a wall with high amplitude, small wavelength asperities tend to develop more friction.

The implications of the tests as they concern bulkheads are that: for precast concrete sheet piles, δ can be taken as 0.9ϕ; for steel and timber sheet piles, other data must be consulted, although the principles of relative roughness hold true.

Peterson et al. (1976), summarized test conditions and results of investigations of skin friction. Of particular interest are the ratios of δ/ϕ for steel and for wood, with the direction of frictional resistance parallel to the grain. These values are summarized in Table 2-3. Also of interest are values of δ that were determined, but without reference to ϕ. These are also shown in Table 2-3.

The significance of the summarized skin friction data is that the value suggested by Rowe, $\delta = 2/3\phi$, is a reasonable value to use; it seems overly conservative in the case of wood sheet piles. However, the sample size of only eight values for wood is too small to be used for application to other design situations. In the case of steel, it can be seen that the mean value for ϕ is 37.2 degrees. This value obviously precludes granular soils in the loose state, which tend to show lower ratios of δ/ϕ (Peterson et al., 1976). Here Rowe's suggestion again appears reasonable.

Table 2-3. Summary of skin friction data (summarized from Peterson, et al., 1976)

No. of		Angle of Interval Friction, ϕ		Angle of Skin Friction, δ		δ/ϕ	
Values	Material	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
18	Steel	37.2	4.93	28.4	5.27	0.758	0.129
31	Steel			25.6	2.64		
8	Wood	37.2	4.93	32.8	3.42	0.865	0.06

The conservatism resulting from using $\delta = 2/3\phi$, in lieu of 0.8 , is reflected in Table 2-4. It can be seen that the conservatism results in small increases in the active case, a 17 percent increase in the passive case for loose soils, and a 54 percent increase for dense soils. With the exception of dense soils, the conservatism does not appear to be substantial. In the case of dense soils, penetration depths are already substantially less than those for loose soils. Thus, the conservatism results in only slight increases in depth when compared to depths computed using the less conservative assumption.

2.9. Summary

Tracing the evolution of thought that governs bulkhead design serves two purposes: it provides an understanding of the complex interaction of the soil and the flexible retaining wall, and it presents rationale for choosing the optimum design procedure.

Although conservative, the classical methods provided rational approaches to design. Both methods assumed a linear stress distribution, but made contrary assumptions with respect to fixity at the toe of the pile. Later approaches assumed nonlinear pressure distribution. The Danish Rules allowed for reduced wall stresses because of arching of the soil between the anchor and dredge levels.

Large scale model tests performed by Tschebotarioff revealed that arching was unstable in bulkheads with yielding anchorages and that reductions of wall loads were because of stress distributions that deviated from the classical assumptions. His test results also suggested that high wall stresses from fluid clay backfill could be alleviated by using sand blankets or dikes adjacent to the wall.

Table 2-4. Soil stress coefficients

ϕ	$\delta = 2/3\phi$	K_a	K_p	$\delta = 0.8\phi$	K_a	K_p
30	20	0.279	5.74	24	0.270	6.70
40	26.7	0.179	16.7	32	0.171	25.7

The extensive investigations by Rowe covered a broad spectrum of conditions and contributed significantly to the understanding of bulkhead behavior. His tests demonstrated that the stress distribution at the time of toe failure of a wall is accurately described using free earth support assumptions. The Free Earth Support value for depth of penetration therefore specifies the minimum depth for a factor of safety of 1.0. With increasing depths and increasing densities of subgrades, fixity approaches the Fixed Earth Support assumption (Terzaghi, 1955). Once a safe depth of penetration is established, Rowe determined that the deviation of loads from the Free Earth Support method is a function of subgrade strength and wall flexibility. A more applicable model than the simply supported beam was used to describe the soil-structure interaction, i.e., the beam on elastic foundation with a linearly varying subgrade modulus. Rowe compared his model test results to the results of other investigators. He found that Tschebotarioff's suggested method was valid only for the ranges of soil stiffness and pile flexibility that were tested at Princeton. Within this range, there was close agreement. Comments by Terzaghi indicated that he agreed with Rowe's findings. The approach using the theory of plasticity proposed by Hansen also produced designs very similar to those resulting from the Rowe method. Considering the difficulties in manipulating the complex equations and rupture figures of Hansen's method, the Rowe approach offers a very attractive alternative.

Rowe's study of bulkheads was then extended to walls in cohesive subgrades. A method was derived from this investigation whereby designs could be developed based upon the undrained strength of the soil.

The finite element method provides an accurate means to investigate the complex natures of soil-structure interaction and horizontal soil stresses. A proven FEM routine was used to evaluate a large bulkhead and the results compared favorably with instruments and strain gauges used to monitor the wall. The results also demonstrated good agreement with the Rowe method, thus adding more credence to the Rowe procedure.

An investigation of tied-back walls served to qualitatively model and explain the mechanics of a discontinuous wall. The behavior of the soldier pile and lagging system can be expected to be somewhat similar to the behavior of the 8 inch fender pile and sheet pile system of a navy bulkhead. A discussion of these implications appears later in this work.

CHAPTER 3

DEVELOPMENT OF A SIMPLIFIED DESIGN APPROACH

The discussion in the preceding chapter illustrated the variety of approaches to bulkhead design and showed that one approach is both reasonable and comprehensive. Therefore the Rowe method, which incorporates the Free Earth Support method, with modifications to bending moment and tie-rod load, is selected as the basis for a simplified design method.

In spite of its obvious merits, the Rowe method is somewhat more involved than the simpler methods. This, coupled with a lack of understanding of bulkhead behavior, will lead engineers who have not benefited from extensive training in soil mechanics to employ less complex methods. The results can range from overdesigned, uneconomical walls to inadequately designed walls. For these reasons, a simplified approach is developed herein where design curves are generated from data utilizing the Rowe method. These curves can then be employed in conjunction with simple manipulations of the pertinent parameters to develop bulkhead designs.

3.1. Computer Program

The development of a design curve requires a substantial number of data points for establishing a clear trend. To produce these data by using the Rowe method and hand calculations would be a formidable task and require a great deal of time. Use of the digital computer greatly diminishes the time necessary to produce a sufficient amount of data. A computer program was therefore developed that would yield bulkhead designs for cantilevered and anchored walls in sand and clay. The desired output consisted of penetration depths, tie-rod loads, and maximum bending moments for walls made of timber, A328 steel and A690 steel.

It was considered to be necessary that the program have the capability of dealing with any geometry (e.g., standing wall height, water level) and heterogeneous (multi-layered) soils with the assumption that each soil layer is isotropic and homogeneous. These arbitrary parameters define the problem and enter the program as input data. The parameters are (Figure 3-1):

H = standing wall height,

H_A = anchor level height,

H_W = low water level height,

γ_i = appropriate unit weight of ith soil layer,

ϕ_i = angle of internal friction of ith soil layer,

c_i = cohesion of ith soil layer, and

t_i = thickness of ith layer.

Since the Rowe method entails the use of curves, selected data points on the curves must be read in as data. The curves are factors to be applied against bending moments and tie-rod loads for anchored walls in sand (Figure 2-17), anchored walls in clay (Figure 2-19) and cantilevered walls in sand (Figure 2-20).

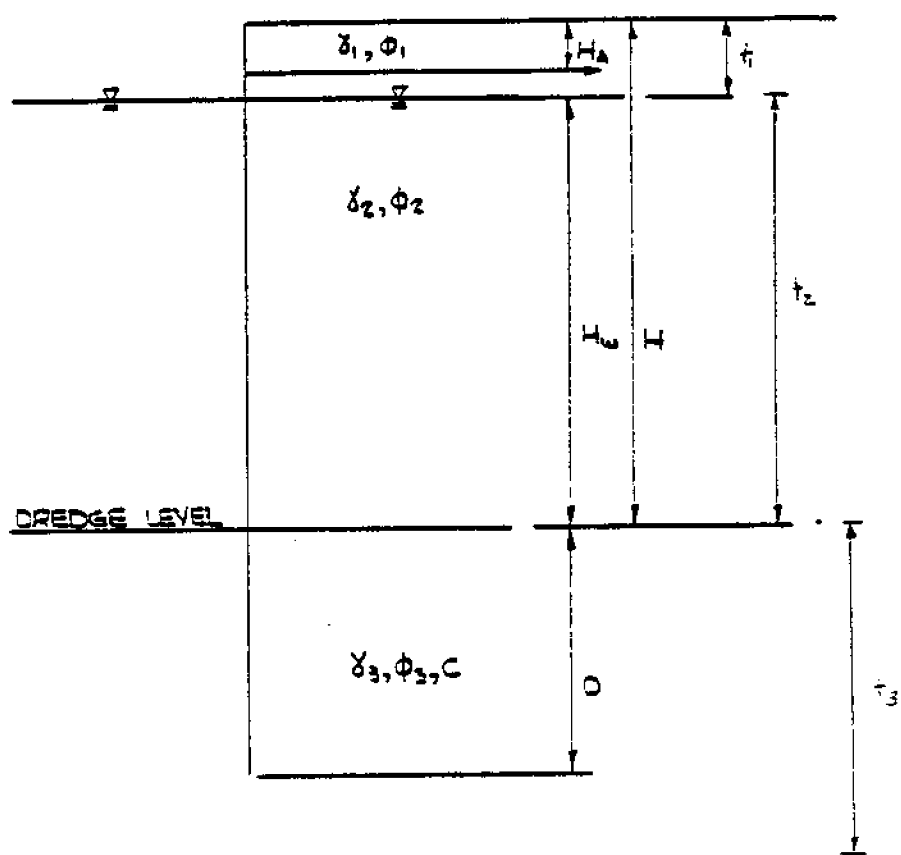


Figure 3-1. Input parameters for computer program

It was noted earlier that once a safe penetration depth is established, the problem becomes one of deflection. From Rowe's studies (1952), it was ascertained that the stress distribution at the time of toe failure is adequately described by Free Earth Support computations. Since this is the penetration depth at failure, a safety factor must be applied. Terzaghi suggested applying such a factor against the soil strength parameter (1954). Since shear strength of cohesionless soil varies with the tangent of the internal angle of friction:

$$\phi_f = \tan^{-1} \left(\frac{1}{FS} \tan\phi\right), \tag{3-1}$$

in which: ϕ_f = factored soil parameter, ϕ = unfactored soil parameter and FS = a safety factor.

It follows that the computer program should factor the soil strength parameter and find the appropriate depth of penetration by the Free Earth Support method. Then, tie-rod load and maximum bending moment can be computed based upon unfactored soil parameters and Free Earth Support pressure distributions. The Free Earth Support procedure is detailed in a later section.

The computer program must then choose the proper factors for bending moment and tie-rod loads. It must, therefore, "enter" the proper curve at the proper place by interpolating. Since it is unlikely that relative densities can be accurately established in the field and reduction curves only provide for "loose" and "dense" sands, the program must correlate relative density with the angle of internal friction. The routine must, therefore, arbitrarily select a friction angle of 30 degrees for loose sand and 40 degrees for dense sand. For intermediate values the routine must interpolate and, for values outside this range, it must assign the upper or lower bound as appropriate. This argument also holds for the stability number of clays.

Once the proper "graphs" are selected by the program, an operating curve must be generated whereby a reduction factor is chosen for the maximum bending moment depending upon the pile flexibility number, ρ. A structural curve is developed based upon the material properties of the member in question, its shape factor and flexibility number, ρ. The intersection of these curves is found and the design bending moment is computed. This process must be accomplished for wood piles, and steel piles fabricated from A328 steel and A690 steel. A similar, but less complicated, process must occur for the tie-rod loads. The Rowe method is demonstrated in detail in a later section.

3.1.1. Subroutines

The computer program developed for designing bulkheads was entitled "WALL" and consists of a main program and 12 subroutines. A description of the various functions follows.

The main program serves to input and display data, to regulate data sent to subroutines and to make decisions as to which subroutine is to be used.

Subroutine "FACTOR" is first called to apply a safety factor against the strength parameter, compute active and passive stress coefficients, and to keep track of the unfactored strength parameters and associated coefficients.

Subroutine "DEPTH" arranges soil layers sequentially by depth. In addition to those already input, it identifies the depths of the water level and dredge level as layers. If this causes duplicity, a logical statement is invoked and the redundancy is eliminated.

Subroutine "PARAM" maintains the proper association between soil layers and their respective soil properties. It also computes the submerged unit weight for soils below the water table.

Subroutine "FORCES" is used to compute horizontal soil stresses, resultant forces and moments based upon Free Earth Support calculations. The main program decides whether to use factored or unfactored soil stress coefficients. Moments are summed about the tie-rod for penetration computations. The main program controls an iterative process where the depth of penetration is increased or decreased until the sum of moments about the tie-rod is equal to zero. When the depth of penetration iterations are completed, the main program directs "FORCES" to compute stresses and forces based upon the design penetration and unfactored soil stress coefficients. Output is generated for the factored and unfactored cases. For verification purposes, the following parameters are displayed for respective layer depths: active and passive soil stress coefficients, unit weight, overburden stress, horizontal stress, resultant force and moment. Penetration depth is also displayed.

Subroutine "TIE" is called to compute moments about the point of application of the passive stress resultant. The moments are based upon resultant forces from unfactored soil parameters. This subroutine is bypassed for cantilevered walls. The tie-rod load is displayed as output.

Subroutine "MOM" locates the point of zero shear, then computes the maximum bending moment. Free Earth Support calculations are now complete and the point of zero shear and maximum bending moment are displayed.

Subroutine "ROWE" computes the bending moments and tie-rod loads used for design. It controls which reduction curves to use, i.e., anchored walls in sand or clay, or cantilevered walls in sand. No reductions are allowed for cantilevered walls in clay. In addition to selecting the proper curves, it serves to: interpolate between graphs, generate operating and structural curves, compute the design moment and tie-rod loads, and select the corresponding sections for wood members, A328 steel members and A690 steel members.

Subroutines "SAND," "CLAY," and "CANT" select the appropriate moment and tie-rod load factors based upon decisions made in the "ROWE" subroutine.

The intersection of operating and structural curves is accomplished by calling subroutine "POI." This subroutine solves for the point of intersection of two straight line segments that are defined by four points, two points from each curve. Linear approximation is adequate for anchored walls in sand because the curvature of the graph is spread over 25 points. A similar argument applies to cantilever walls in sand. For anchored walls in clay, however, only 3 points are given by the Rowe reduction curves, one each for:

$\log \rho = -3.1$ (stiff walls)

$\log \rho = -2.6$ (working stress zone), and

$\log \rho = -2.0$ (first yield)

This necessitates generating a curve with sufficient data points based upon a best fit of the 3 given points. A curve fitting algorithm is provided by subroutine "FIT" which performs a linear regression based upon bivariant log-normal distribution. The equation of the line of best fit is displayed along with the correlation coefficient, the original data points, corresponding fitted data points, and the difference between the original and the fitted point. For the purpose of generating an operating curve with sufficient points to use in the "POI" subroutine, the equation of the line of best fit is utilized to produce 24 line segments for selected values of pile flexibility.

A computer source list, sample output and User's Guide may be found in the Appendices.

3.2. Producing Data for Design Curves

Once the program was debugged, it was modified so that variations of input parameters would produce enough data of statistical significance for each case.

3.2.1. Case I: Anchored Walls in Sand

There were six curves generated for this case, each depending upon the relative density of fill with respect to the relative density of the subgrade. The free-standing wall height was varied for each combination of relative densities, the water level was varied for each wall height and the anchor level was varied for each height of water, i.e., $H = 5$ 10, 15 and 20 ft (1.50, 3.05, 4.57, and 6.10 m), $H_w = 0.6H$, $0.7H$ and $0.8H$, and $H_A = 0.9\ (H-H_w)$, $0.8\ (H-H_w)$, $0.7\ (H-H_w)$, $0.6\ (H-H_w)$, $0.5\ (H-H_w)$. The combinations of relative densities were:

Loose Fill/Loose Subgrade,

Loose Fill/Medium Subgrade,

Loose Fill/Dense Subgrade,

Medium Fill/Medium Subgrade,

Medium Fill/Dense Subgrade, and

Dense Fill/Dense Subgrade

The fill was considered to consist of one soil type which extended above and below the water level. The only property difference was in the unit weight. Above the water table, moist unit weight was assigned and below the water level, submerged unit weight was assigned. Unit weights were correlated with relative densities, which in turn were correlated to internal angles of friction. Table 3-1 lists these relationships. A total of 360 data points was generated for Case I.

3.2.2. Case II: Anchored Walls in Clay (Undrained)

There were 3 curves generated for Case II, each depending upon the ratio of the moist unit weight of fill times the standing wall height to the cohesion of the subgrade:

$$\frac{c}{\gamma_1 H} = 0.25, \tag{3-2.a}$$

$$\frac{c}{\gamma_1 H} = 0.30, \text{ and} \tag{3-2.b}$$

$$\frac{c}{\gamma_1 H} = 0.35. \tag{3-2.c}$$

Table 3-1. Relationship of soil properties (sand)

ϕ	Relative Density	γ_{moist}	γ_{sat}
30°	Loose	100 pcf (15.7 kN/m^3)	120 pcf (18.8 kN/m^3)
35°	Medium	105 pcf (16.5 kN/m^3)	125 pcf (19.6 kN/m^3)
40°	Dense	110 pcf (17.2 kN/m^3)	130 pcf (20.4 kN/m^3)

Note: $\gamma_{sub} = \gamma_{sat} - \gamma_{water}$. Use γ_{sub} for the actual analysis.

These relationships produced stability numbers between 0.40 and 0.70. Stability numbers greater than 0.70 produce results with very small depths of penetration and very low bending moments and tie-rod loads; the long-term (drained) condition will prevail under these circumstances. Stability numbers less than 0.40 will produce no data since, using the factored cohesion parameter, the stability number is less than 0.25 and walls cannot stand for any depth of penetration with such low stability numbers.

Sand backfill was assumed to be present from the dredge level to the top of the wall. Also assumed was that the sand backfill was in the loose state as it is generally not compacted with the bulkhead in place. Cohesive material above the dredge level produces low stresses for the undrained case since Rankine distribution prevails (Mana, 1978). In cases where cohesion is present above the dredge level, the drained condition will prevail.

The relationship establishing the density of the subgrade is given by:

$$\begin{aligned} \gamma_3 &= 110 + \frac{c}{200} \ \frac{\text{lb}}{\text{ft}^3} \\ &= 17.2 + \frac{c}{31.3} \ \frac{\text{kN}}{\text{m}^3} . \end{aligned} \qquad (3\text{-}3)$$

The relationship of densities for the fill material is the same as in Case I.

The wall heights, water level heights and anchor level heights were varied as in Case I so that 180 data points were generated.

3.2.3. Case III: Anchored Walls in Clay (Drained)

There were six curves generated for Case III, 3 curves for loose sand fill overlying a clay subgrade and 3 curves for homogeneous material. Relationships between the angle of internal friction and soil unit weight are shown in Table 3-2.

The wall heights, water level heights and anchor level heights were varied as before to give rise to 360 data points.

3.2.4. Cantilevered Walls

Case IV: Cantilevered Walls in Sand

Case V: Cantilevered Walls in Clay (Undrained)

Case VI: Cantilevered Walls in Clay (Drained)

The cases for cantilevered walls proceeded similarly to the anchored cases. The only difference was that, since there was no tie-rod, there could be no variation for anchor level. Consequently, there were five times fewer sets of data.

For all cohesionless cases, each set of data included the soil properties of each layer (K_a, γ, z), the tie-rod load (P) and the bending moment for A328 steel, A690 steel and wood (M_1, M_2, M_3). The depth of penetration was displayed as the depth to the bottom of the third layer (t_3). The anchor level was also included where appropriate.

For the cohesive cases, each data set included the same parameters as listed above, plus the factored and unfactored cohesions and stability numbers.

Table 3-2. Relationship between drained strength of clay and unit weights

ϕ	γ_{moist}	γ_{sat}
24°	94 pcf (14.7 kN/m^3)	114 pcf (17.9 kN/m^3)
26°	96 pcf (15.0 kN/m^3)	116 pcf (18.2 kN/m^3)
28°	98 pcf (15.4 kN/m^3)	118 pcf (18.5 kN/m^3)
30°	100 pcf (15.7 kN/m^3)	120 pcf (18.8 kN/m^3)

Note: $\gamma_{sub} = \gamma_{sat} - \gamma_{water}$. Use γ_{sub} for the actual analysis.

3.3. Manipulating the Data

The sets of data generated represented designs for a wide range of geometric and soil conditions. More than 1100 values now required rendering the data into a meaningful and usable format. The approach was to find the mathematical relationships between the loading conditions and the resulting penetration depths, maximum bending moments and tie-rod loads. The mathematical functions to be formulated required simplicity, wide ranges of applicability and clearly established correlations.

3.3.1. The Normalized Parameters

Normalized parameters were sought as these would offer the most general format for design curves. Loading parameters were nondimensionalized in terms of the known geometric and soil parameters. Since Free Earth Support calculations are the basis for the Rowe method and involve unit weight times some length cubed, a combination involving the unit weight and thickness cubed of each layer was used as a basis for establishing relationships. Each of the three layers contributes to the loading and resulting design parameters, but the thickness of the third layer is initially unknown as this is the depth of penetration. The ratio, R, was therefore formulated as

$$R = \frac{\gamma_1 t_1^3 + \gamma_2 t_2^3}{\gamma_3 H^3} \qquad (3\text{-}4)$$

The numerator represents the loads above the dredge level. The denominator normalizes the term utilizing the unit weight of the subgrade. Since the depth of penetration, D, is unknown, the standing wall height was considered the most pertinent variable with length units.

With R established as an independent variable, nondimensionalized dependent variables were chosen as $\frac{D}{H}$ = dimensionless depth, $\frac{P}{\gamma L^2}$ = dimensionless tie-rod load, and $\frac{M}{\gamma L^3}$ = dimensionless bending moment, where L = some parameter of length units, and γ = one of the 3 unit weights of the problem.

3.3.2. Testing the Relationships

Since plotting the dependent and independent variables by hand was a problem because of the amount of data, a curve-fitting technique was established utilizing linear regression analysis. This approach enables a curve of best fit to be established from a population of ordered pairs. The fit can then be tested from the Pearson product-moment correlation. For an ordered pair (x,y), in which: x = the independent variable and y = the dependent variable, a population of n ordered pairs can be analyzed with a resulting line of best fit. The following Gaussian elimination scheme defines the process:

$$\bar{x} = \frac{1}{n} \sum_{i=1}^{n} x_i, \qquad (\bar{x} = \text{mean}) \tag{3-5}$$

$$\bar{y} = \frac{1}{n} \sum_{i=1}^{n} y_i, \qquad (\bar{y} = \text{mean}) \tag{3-6}$$

$$S_x^2 = (\frac{1}{n} \sum_{i=1}^{n} x_i^2) - \bar{x}^2, \qquad (S_x^2 = \text{variance}) \tag{3-7}$$

$$S_y^2 = (\frac{1}{n} \sum_{i=1}^{n} y_i^2) - \bar{y}^2, \qquad (S_y^2 = \text{variance}) \tag{3-8}$$

$$S_x = \sqrt{\frac{S_x^2 n}{n-1}}, \qquad (S_x = \text{standard deviation}) \tag{3-9}$$

$$S_y = \sqrt{\frac{S_y^2 n}{n-1}}\ , \qquad (S_y = \text{standard deviation}) \tag{3-10}$$

$$m = \left(\frac{1}{n}\sum_{i=1}^{n} s_i \sum_{i=1}^{n} y_i - \sum_{i=1}^{n} x_i y_i\right) \div \left[\frac{1}{n}\left(\sum_{i=1}^{n} x_i\right)^2 - \sum_{i=1}^{n} x_i^2\right], \tag{3-11}$$

$$b = \bar{y} - m\bar{x}\ , \tag{3-12}$$

$$r = m\,\frac{S_x}{S_y} \tag{3-13}$$

in which: m = slope of the line of best fit, b = y-intercept of the line of best fit, and r = the correlation coefficient of the test. The correlation coefficient for a bivariant normal distribution will range from zero, for a distribution of absolutely no relationship, to $\pm$ 1.0, for a distribution whose ordered pairs are all located on the line of best fit.

Some situations required the best fit of a curved line to data. This was implemented using the natural logarithm of the variables, thus creating a bivariant log-normal distribution. The curve of best fit would then be described as:

$$\ln y = m \log x + b, \text{ or} \tag{3-13}$$

$$y = e^b x^m\ . \tag{3-14}$$

It became apparent that other parameters would need to be incorporated because low correlation coefficients resulted from the initial tests. Since penetration depth, tie-rod load and maximum moment vary with tie-rod height and water level height, it followed that these parameters be utilized as modifying factors. Penetration depth is also a modifying factor for tie-rod loads and bending moments.

Situations with cohesion in the subgrades required a somewhat different loading ratio, R, because of a different stress distribution, such that:

$$R = \frac{\gamma_1 t_1^3 + \gamma_2 t_2^3}{(5c - \gamma_1 t_1 - \gamma_2 t_2)H^2} = \frac{\gamma_1 t_1^3 + \gamma_2 t_2^3}{(4cr - \gamma_1 t_1 - \gamma_2 t_2)H^2} \qquad (3\text{-}15)$$

Modifying factors are applied in a similar manner as for cohesionless soils with the addition of the dimensionless stability number, S_t.

Testing for the curve of best fit proceeded whereby the combinations of factors for a modifying coefficient, C, were varied until the highest correlation coefficient resulted. For example, for penetration depth for anchored walls in sand

$$R_D = C_D \cdot R \qquad (3\text{-}15a)$$

$$C_D = \left(\frac{H_w}{H}\right)^2 \frac{H_A}{H - H_A}\,, \qquad (3\text{-}15b)$$

Modifying coefficients are similarly formulated for moments and tie-rod loads, and are subscripted as M and and P respectively. Modifying coefficients are summarized in Table 3-3.

When testing for best fit of these parameters, it was found that for the normalizing term, γL, the best fit resulted for:

Table 3-3. Modifying coefficients for curve fitting

Case	Depth C_D	Bending Moment C_M	Tie-rod Pull C_P
I	$\frac{(H_W)^2}{(H)^2} \frac{(H_A)}{(H-H_A)}$	$\frac{(D)}{(H)} \frac{(H_A)}{(H_W)}$	$\frac{(D)}{(H)} \frac{(H_A)}{(H_W)}$
II	$\frac{H_W}{(H-H_A)\ S_T}$	1.00	$\frac{H_A}{(D)\ (S_T)}$
III	$\frac{(H_W)^2}{(H)^2} \frac{(H_A)}{(H-H_A)}$	$\frac{(D)}{(H)} \frac{(H_A)}{(H_W)}$	$\frac{(D)}{(H)} \frac{(H_A)}{(H_W)}$
IV	1.00	$\frac{(H_W)}{(D)}$	n/a
V	1.00	$\frac{(S_T)^3\ (H_W)^3}{(H)}$	n/a
VI	1.00	$\frac{(H_W)}{(D)}$	n/a

$\gamma = \gamma_1$, for tie-rod pull, and

$\gamma = \gamma_3$, for bending moment, using L = distance from tie-rod to point of application of passive pressure =

$$(H - H_A + \frac{2}{3} D), \qquad (3\text{-}16)$$

The trials proceeded with the objective of attaining correlation coefficients of 0.90 or greater. This insured statistical significance of the relationship. Statistical significance does not necessarily imply engineering significance, that is, a correlation coefficient of 0.90 may still have an unacceptable deviation between the fitted value of a data point and the original value. Conversely, a lower correlation coefficient, say 0.75, may have a small deviation. For this reason, the correlation coefficient was used as a primary test value. If the value proved satisfactory, or improved values could not be attained, acceptance was based upon the percent difference between the fitted and original values.

Once the optimum fits were established, the data points and curves of best fit were plotted utilizing a COMPLOT DP plotter. For use as design charts, the curves were replotted without the data points.

It was apparent from examining the data that bending moments for A690 steel and wood members deviated from bending moments for A328 steel members in a consistent but negligible manner. It was therefore deemed appropriate to formulate a ratio of bending moments with those for A328 steel members as the basis. This was done by computer for anchored walls in sand and cantilevered walls in sand. A normal distribution of ratios for each case was rendered and a mean value and standard distribution were computed. The results are summarized in Table 3-4.

3.4. Summary

The curves of best fit are shown in Figures 3-2 through 3-16. The equations of the curves are governed by:

$$\frac{D}{H} = mR_D + b \qquad \text{(Penetration depth),} \tag{3-16}$$

$$\frac{M}{\gamma_3 L^3} = bR_M^m \qquad \text{(Bending moments), and} \tag{3-17}$$

$$\frac{P}{\gamma_1 L^2} = bR_P^m \qquad \text{(Tie-rod pull).} \tag{3-18}$$

The modifying coefficients of the curves are listed in Table 3-4, and the curve constants m and b are given in Table 3-5. The variability of the design curves is displayed in Table 3-6 in terms of the mean and standard deviation of percent difference. This parameter, percent difference, reflects the difference between the curve of best fit and the original data point after the ordinates have been dimensionalized, i.e., the parameters penetration depth, bending moment and tie-rod pull.

3.5. Conclusions

The data points in Figures 3-2 through 3-16 follow the specific trends indicated by the curves of best fit. The apparent scatter in some plots may be misleading as they seem to signify a large difference

Table 3-4. Bending moment ratios

Material Relationship	A690/A328	Wood/A328
Mean	0.90	0.94
Standard Deviation	0.05	0.03

Table 3-5. Summary of curve fitting

Case	Condition	Depth		Bending Moment		Tie-rod Pull	
		m	b	m	b	m	b
I	L/L	-0.787	0.538	0.315	0.274	1.27	2.17
	L/M	-0.704	0.358	0.400	0.405	1.35	7.44
	L/D	-0.605	0.252	0.441	0.530	1.39	20.1
	M/M	-0.576	0.366	0.416	0.354	1.35	4.84
	M/D	-0.485	0.252	0.448	0.452	1.41	14.7
	D/D	-0.368	0.247	0.459	0.390	1.41	9.79
II	$C/\gamma H = 0.25$	-0.681	0.976	3.44	76.3	0.120	0.677
	$C/\gamma H = 0.30$	-0.253	0.307	1.49	93.7	0.178	0.641
	$C/\gamma H = 0.35$	-0.172	0.175	0.960	118.0	0.233	0.473
III	Sand Fill						
	$\phi = 24$	-0.898	0.919	0.333	0.202	1.09	0.330
	$\phi = 26$	-0.854	0.752	0.313	0.218	1.18	0.678
	$\phi = 28$	-0.840	0.627	0.321	0.251	1.25	1.33
	Homogeneous Clay						
	$\phi = 24$	-1.17	1.42	0.304	0.224	1.11	0.685
	$\phi = 26$	-1.04	1.03	0.313	0.205	1.17	1.02
	$\phi = 28$	-0.865	0.735	0.312	0.261	1.25	1.60
IV	L/L	-0.492	1.42	-0.0365	0.103	n/a	n/a
	L/M	-0.407	1.03	-0.0538	0.133	n/a	n/a
	L/D	-0.295	0.753	-0.0788	0.149	n/a	n/a
	M/M	-0.299	1.02	-0.0474	0.117	n/a	n/a
	M/D	-0.219	0.739	-0.0678	0.136	n/a	n/a
	D/D	-0.176	0.728	-0.0541	0.122	n/a	n/a

Table 3-5. Continued

Case	Condition	Depth		Bending Moment		Tie-rod Pull	
		m	b	m	b	m	b
V	$C/\gamma H = 0.25$	-3.48	2.34	-0.508	0.0223	n/a	n/a
	$C/\gamma H = 0.30$	-1.32	0.816	-0.271	0.0402	n/a	n/a
	$C/\gamma H = 35$	-0.878	0.484	-0.211	0.0378	n/a	n/a
VI	Sand Fill						
	$\phi = 24$	-0.712	2.32	0.0829	0.0603	n/a	n/a
	$\phi = 26$	-0.597	1.93	0.0334	0.0728	n/a	n/a
	$\phi = 28$	-0.541	1.65	-0.0208	0.0911	n/a	n/a
	Homogeneous Clay						
	$\phi = 24$	-0.995	2.23	0.0803	0.0787	n/a	n/a
	$\phi = 26$	-0.749	1.89	0.156	0.110	n/a	n/a
	$\phi = 28$	-0.664	1.63	-0.0259	0.964	n/a	n/a

Table 3-6. Variability in design curves (percent difference)

Case	No. Values	Depth		Bending Moment		Tie-rod Pull	
		Mean	S. Dev.	Mean	S. Dev.	Mean	S. Dev.
I	360	0.09	3.01	0.55	10.5	0.24	7.03
II	180	0.20	5.46	0.47	9.28	-0.37	6.93
III	360	0.05	2.33	0.51	10.3	0.23	6.68
IV	72	0.01	0.60	0.02	3.14	n/a	n/a
V	36	-0.22	3.09	0.34	9.34	n/a	n/a
VI	72	0.00	0.72	0.02	1.95	n/a	n/a

between the curve and actual design values. The true significance of the variability of the data may be established by investigating the reliability of the design. This examination is conducted in a later chapter and it incorporates data contained in Table 3-6.

The presence of the data points in the figures tends to interfere with use of the curves as design aids. For this reason, the curves are presented in Chapter 4 without the data points. The equations of the curve best fit may be used in lieu of the curves by employing the curve constants listed in Table 3-5.

The design curves reflect the bending moments computed for A328 steel only, but they may still be used for computing moments for A690 steel and wood. As suggested by Table 3-3, the bending moments for A690 steel and wood are slightly less than for A328 steel. It is, therefore, slightly conservative to use values computed for A328 steel for the design of A690 steel or wood members.

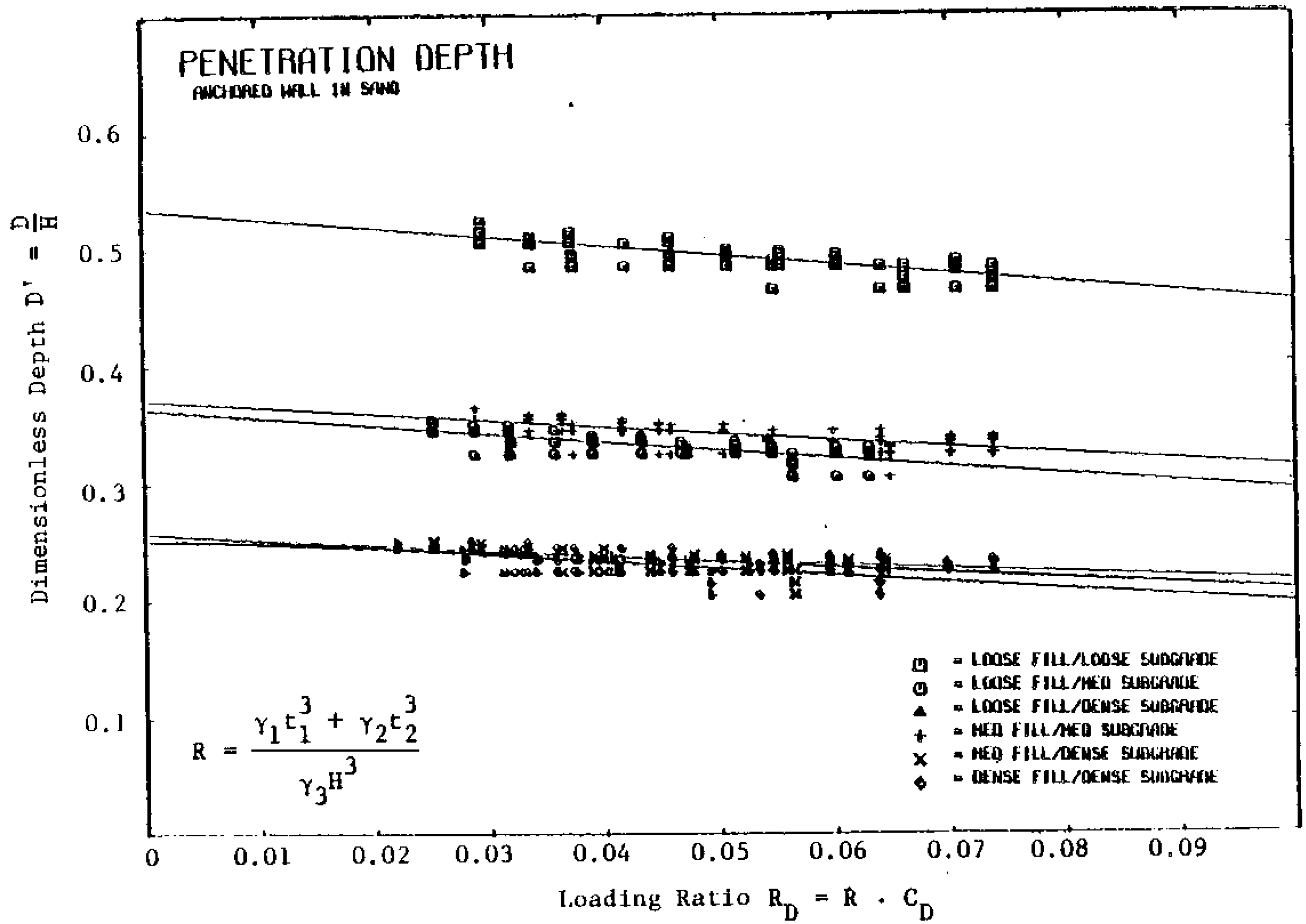


Figure 3-2. D' vs. R_D: sand

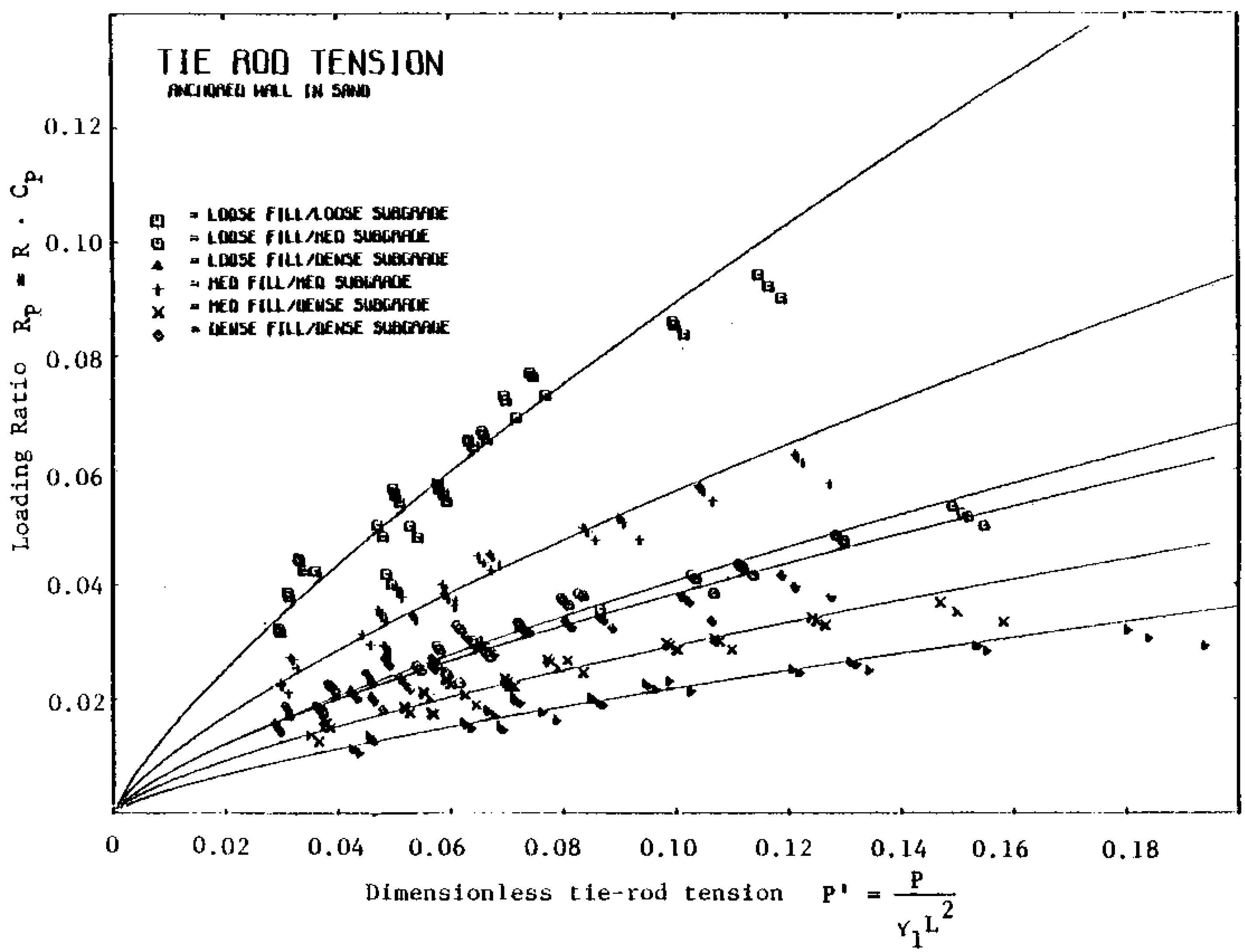


Figure 3-3. P' vs R_p: sand

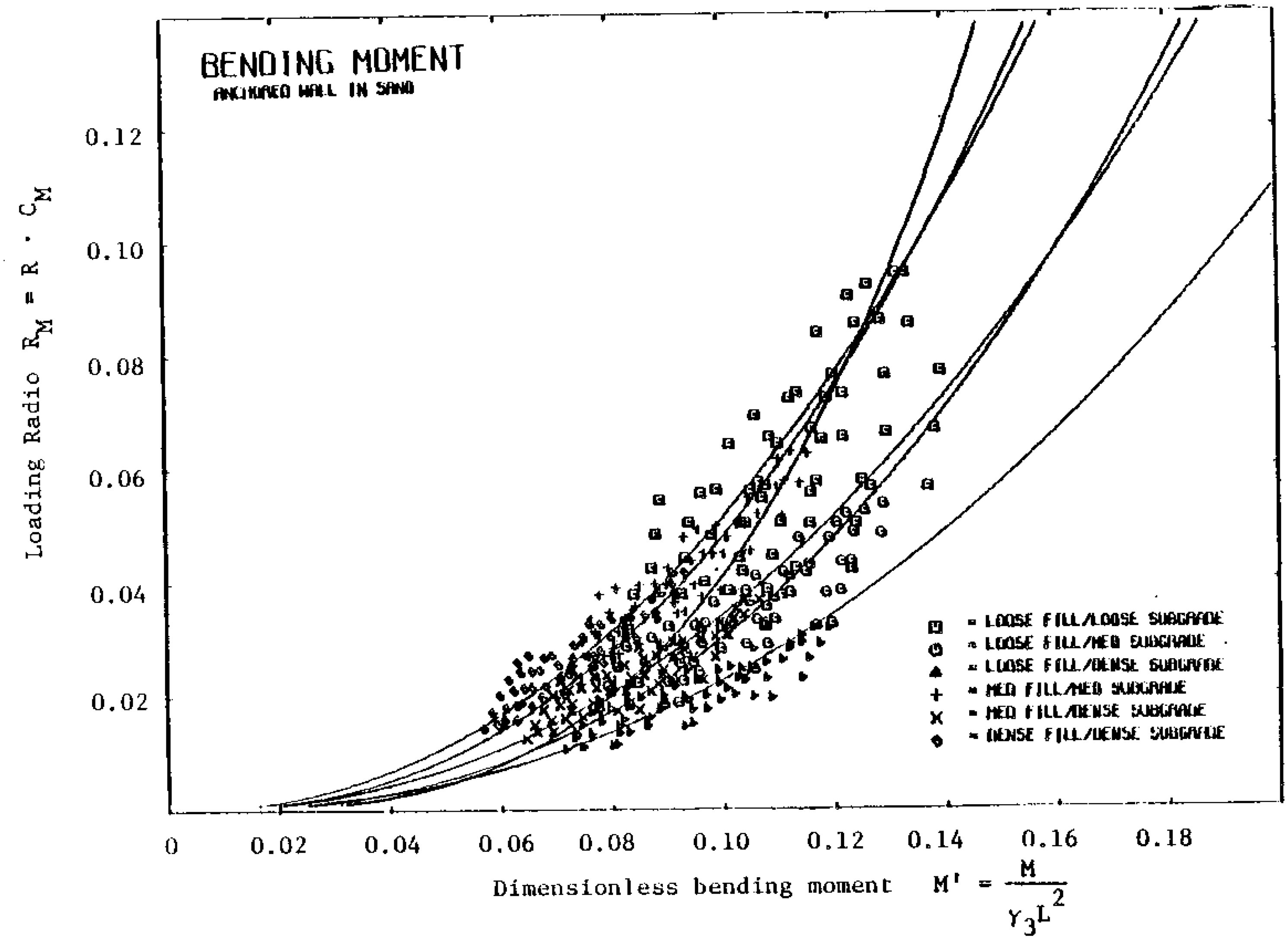


Figure 3-4. M' vs. R_M: sand

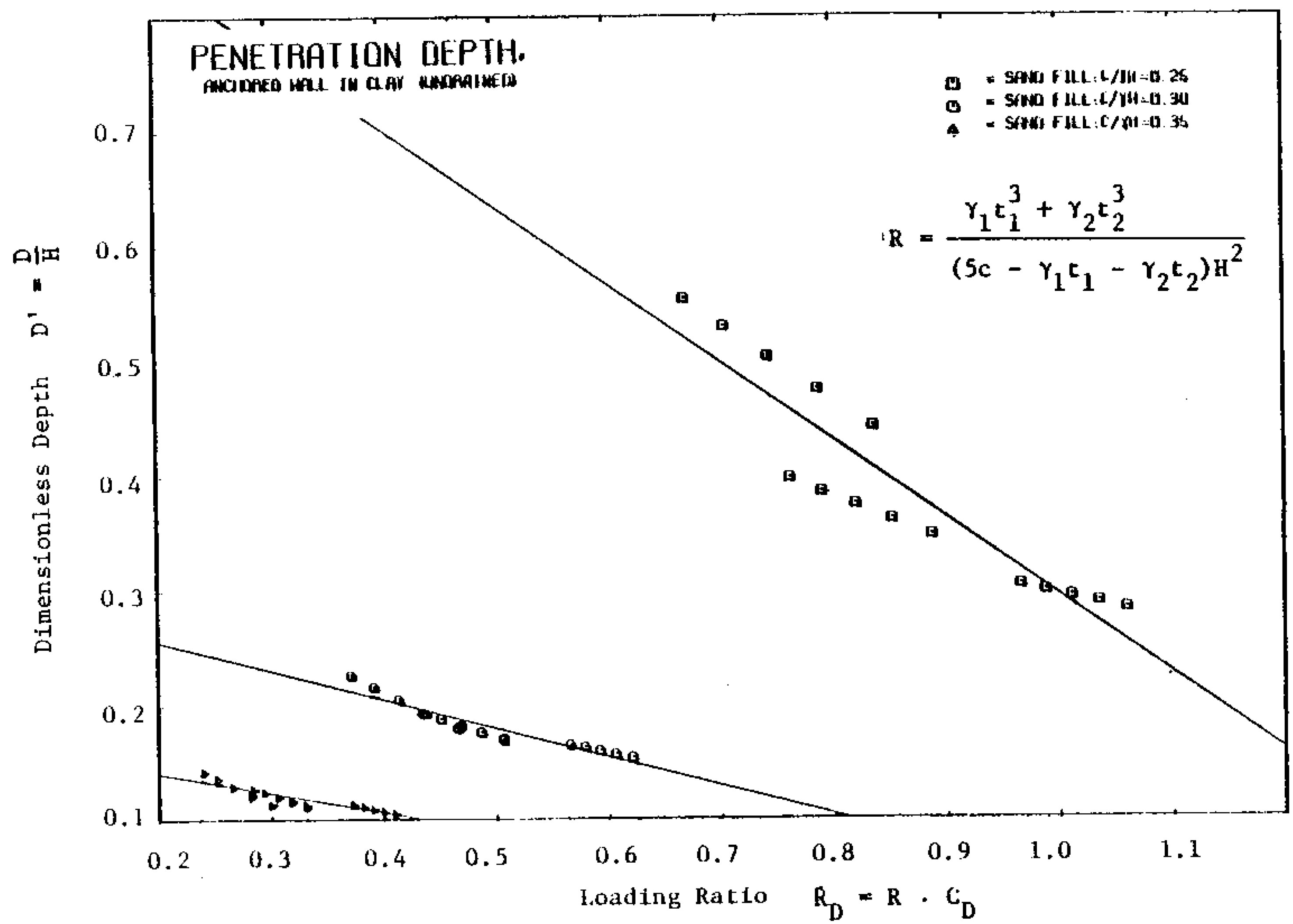


Figure 3-5. D' vs. R_D: clay (undrained)

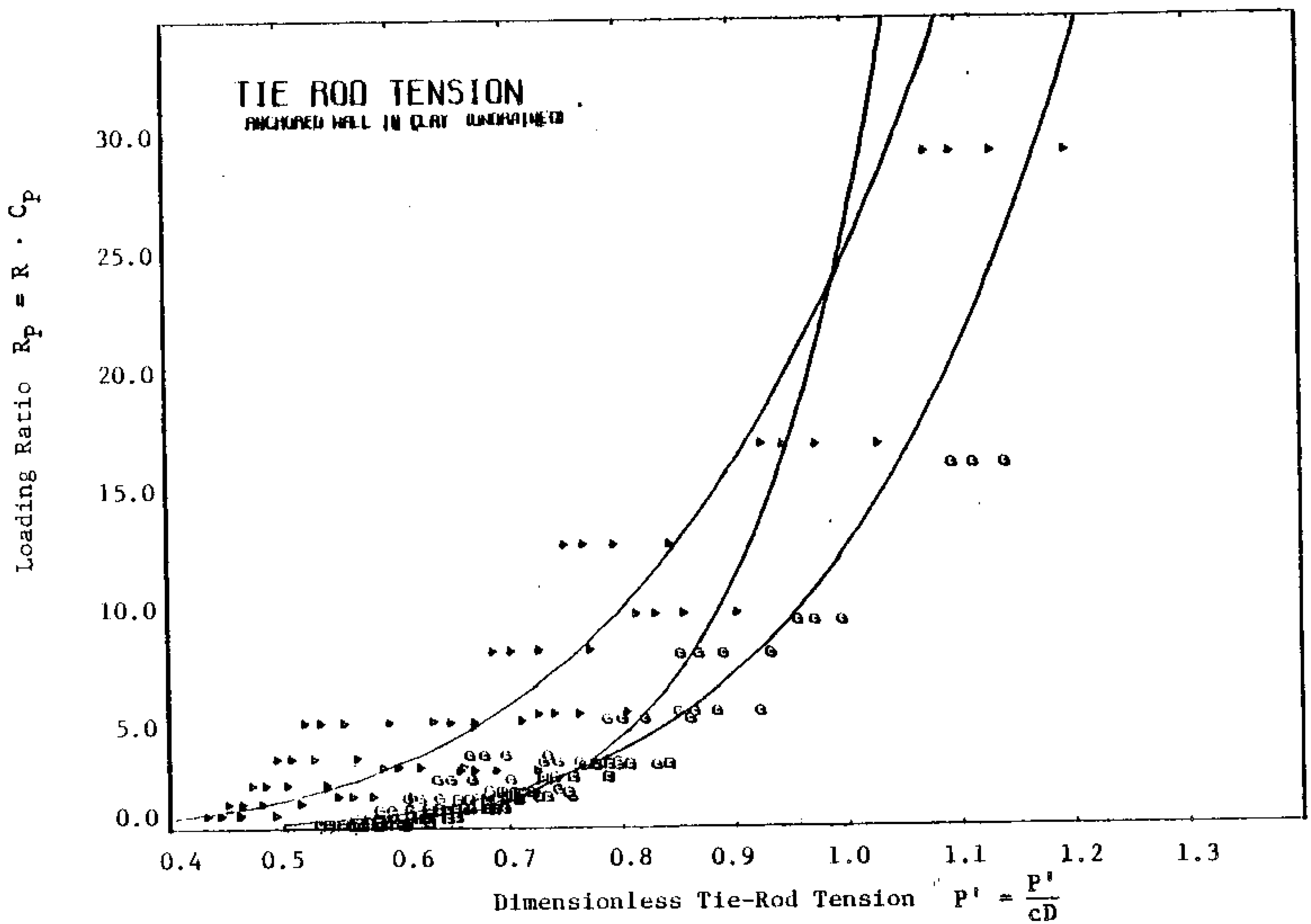


Figure 3-6. P' vs. R_p: clay (undrained)

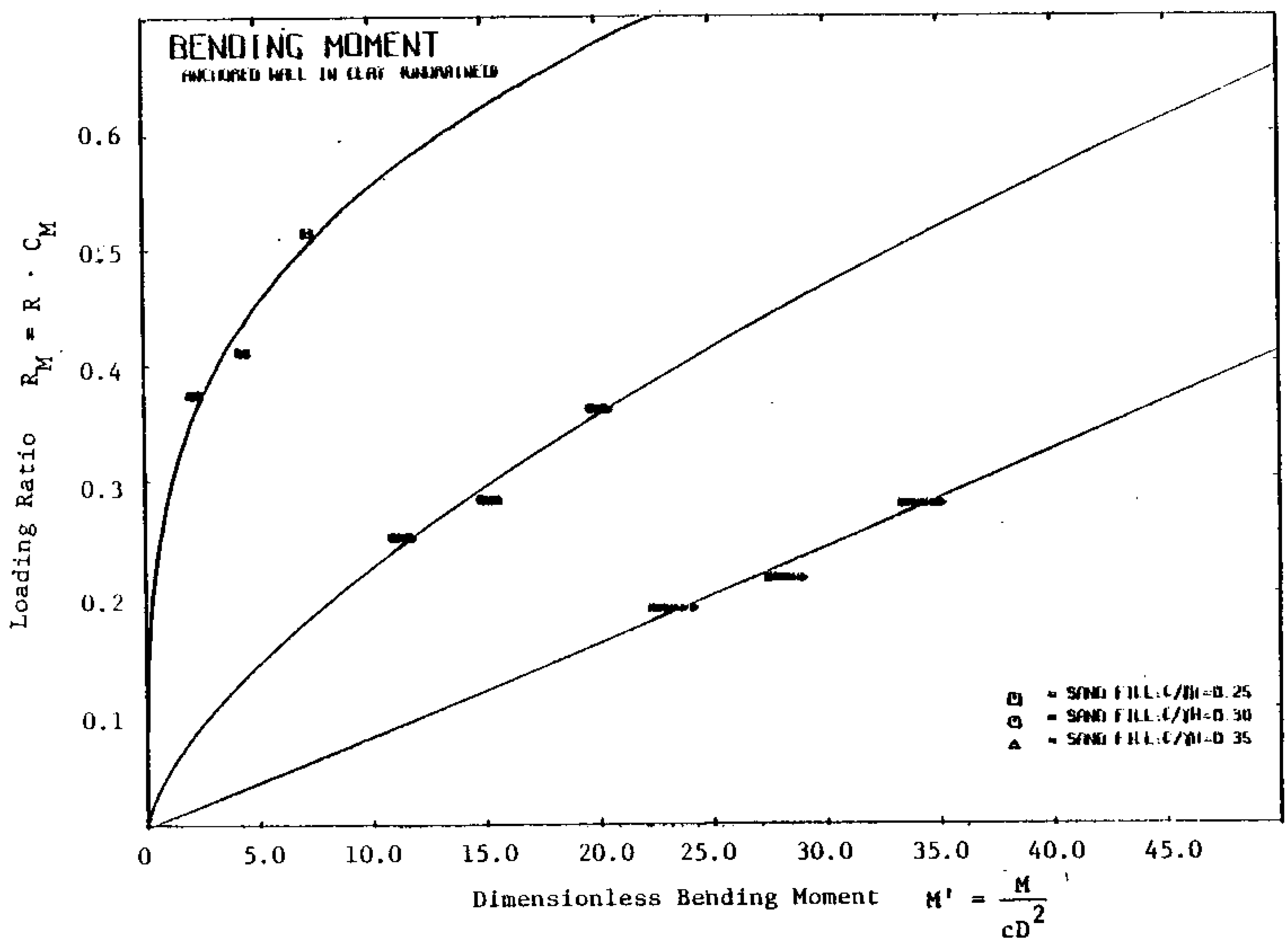


Figure 3-7. M' vs. R_M: clay (undrained)

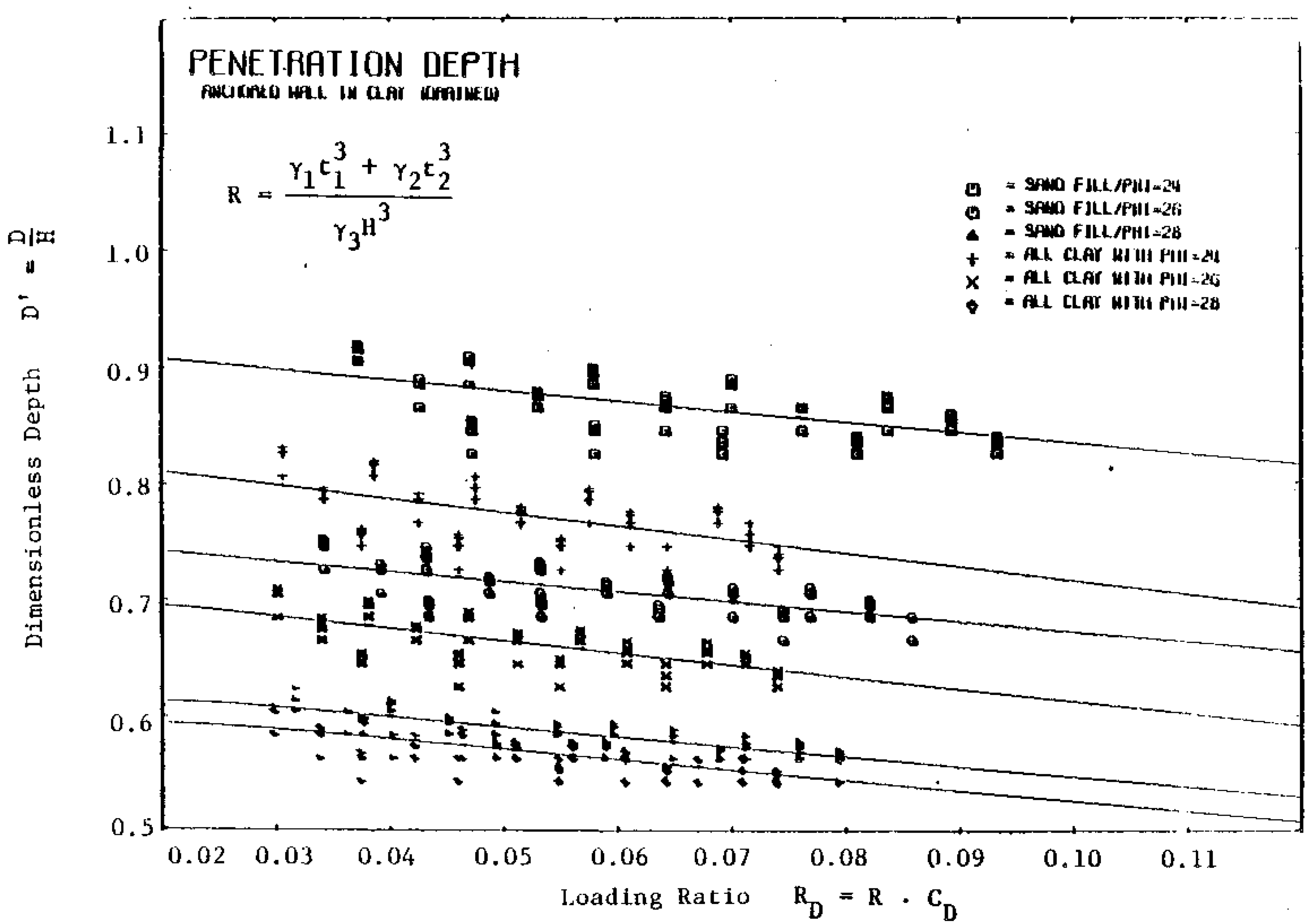


Figure 3-8. D' vs R_D: clay (drained)

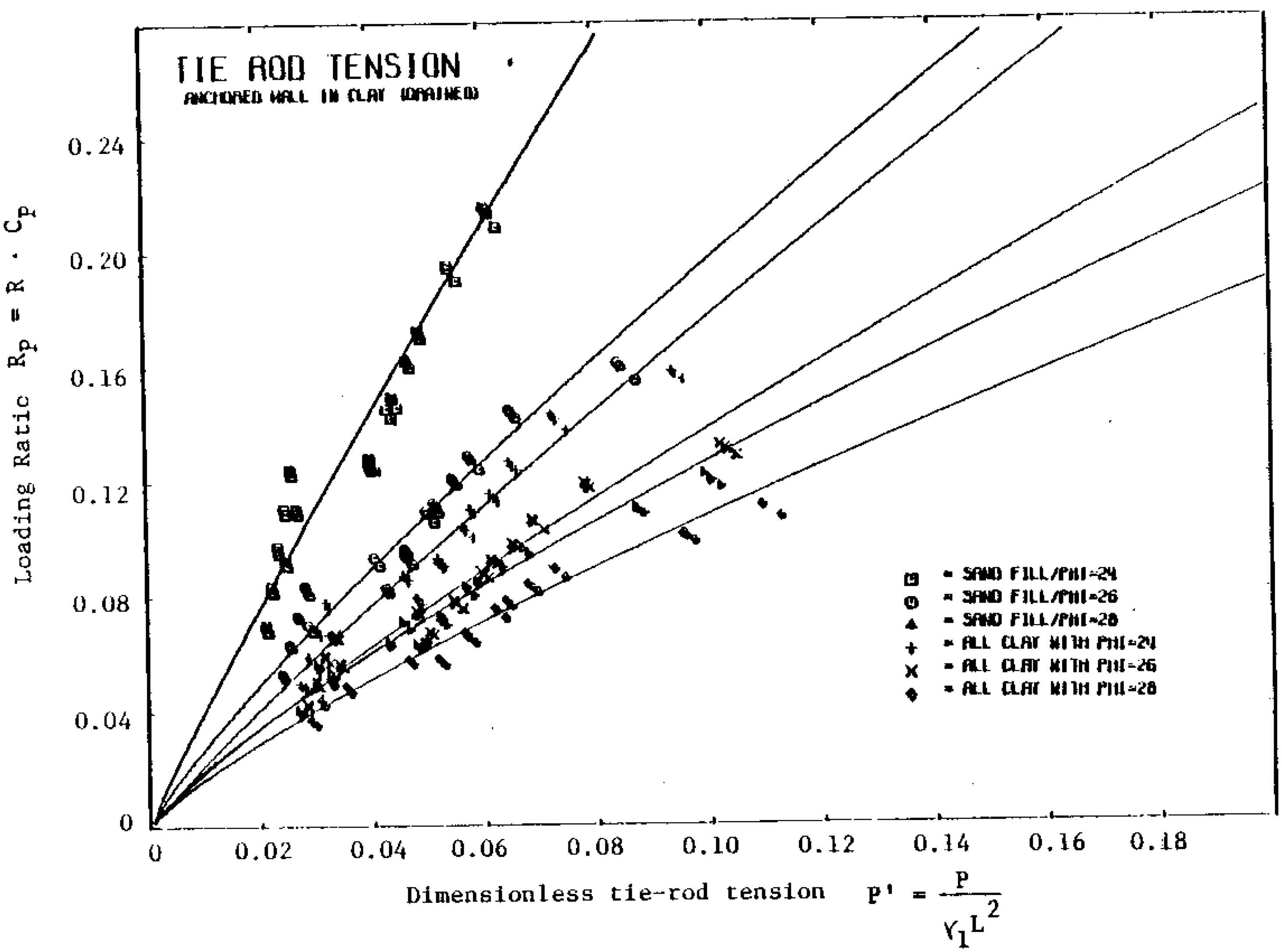


Figure 3-9. P' vs. R_p: clay (drained)

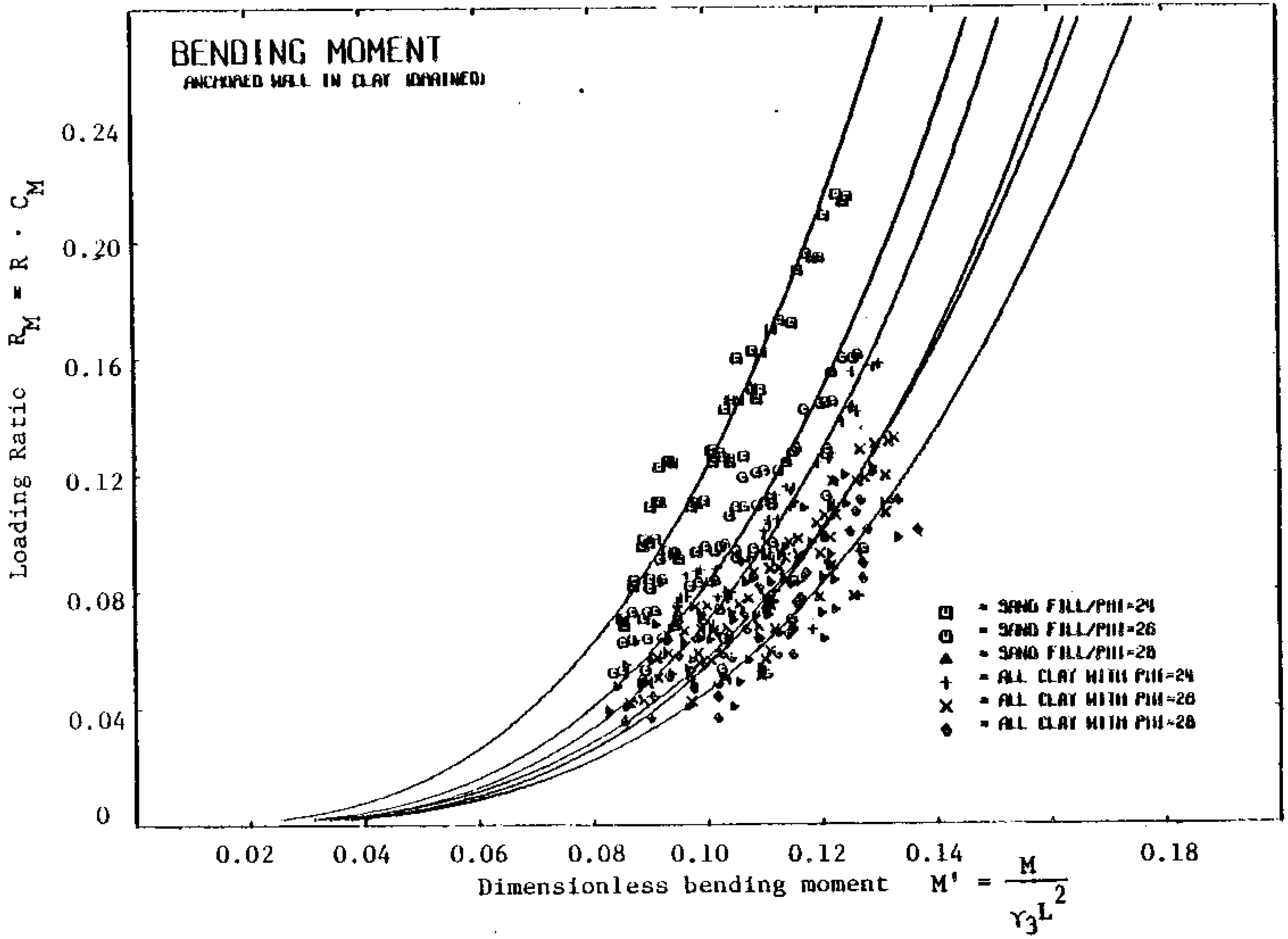


Figure 3-10. M' vs. R_M: clay (drained)

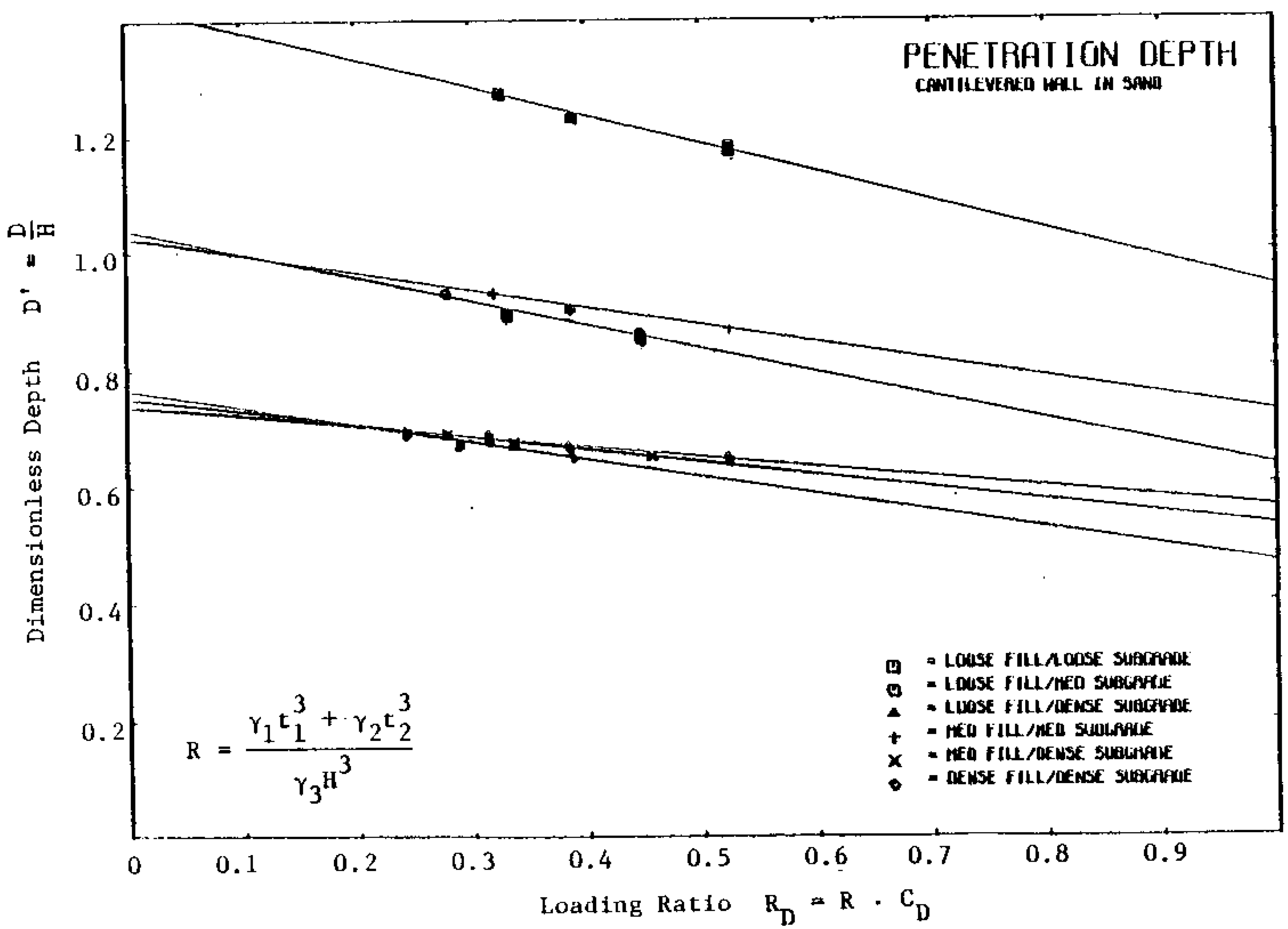


Figure 3-11. D' vs. R_D: sand

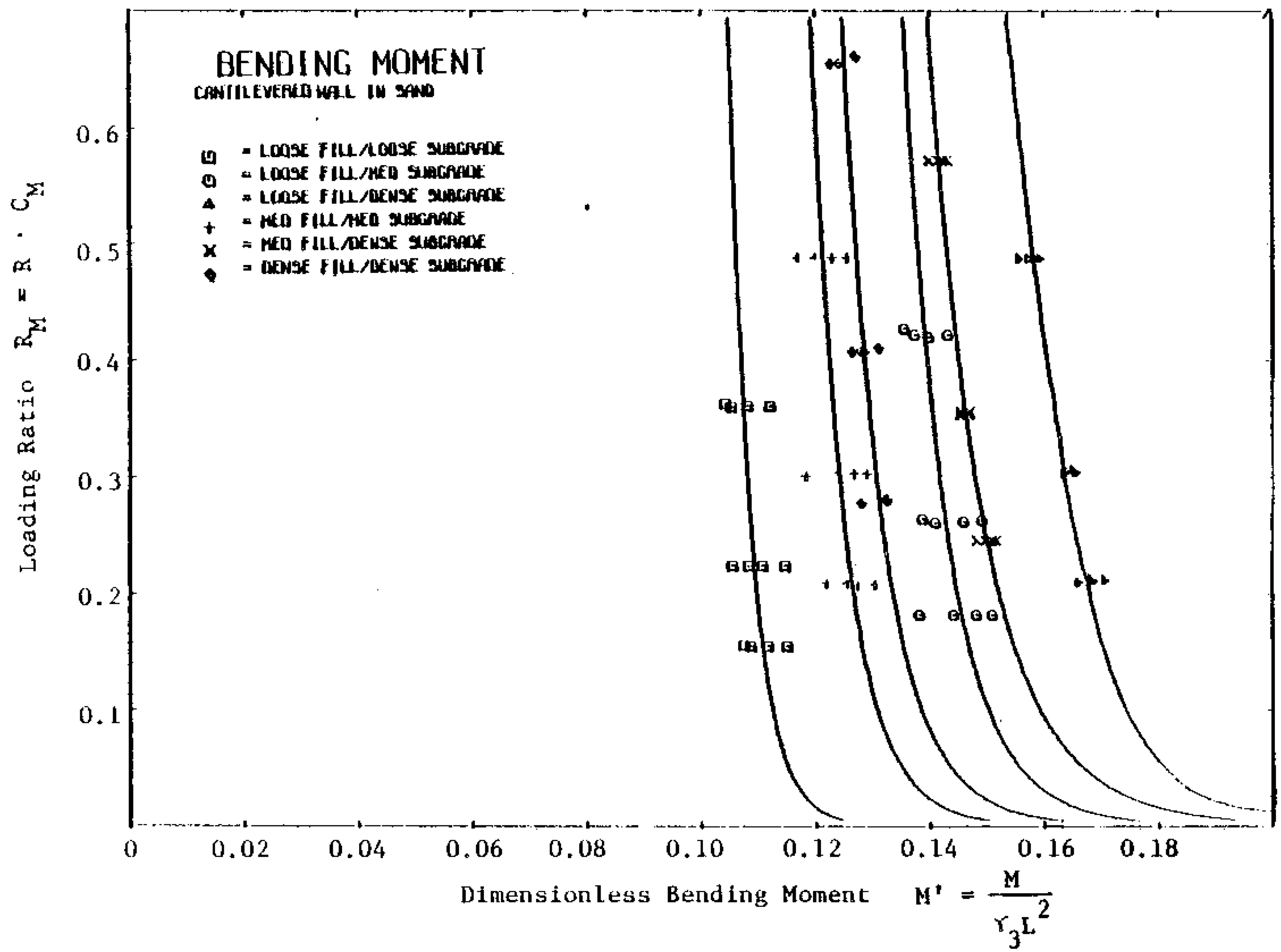


Figure 3-12. M' vs. R_M: sand

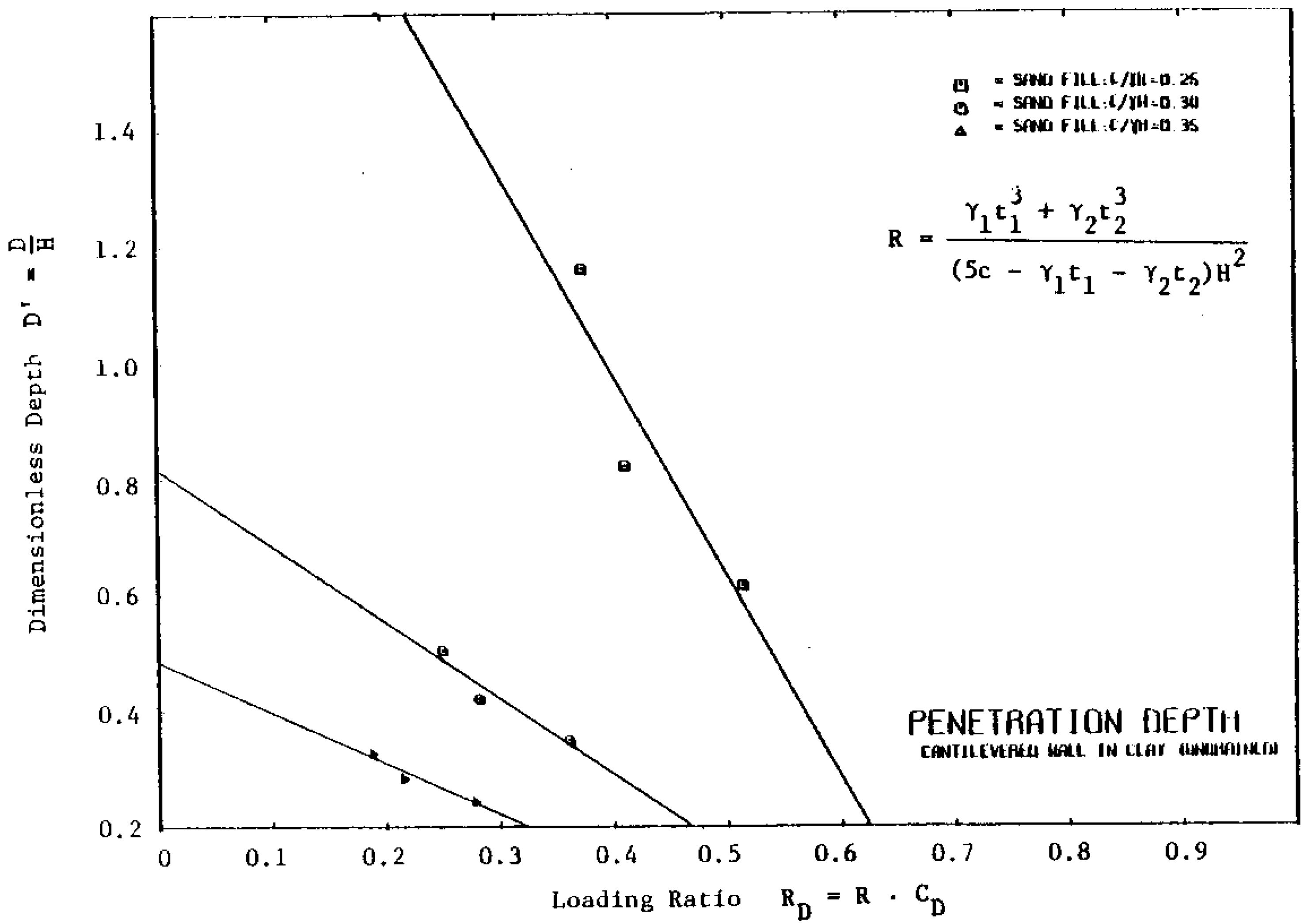


Figure 3-13. D' vs. R_D: clay (undrained)

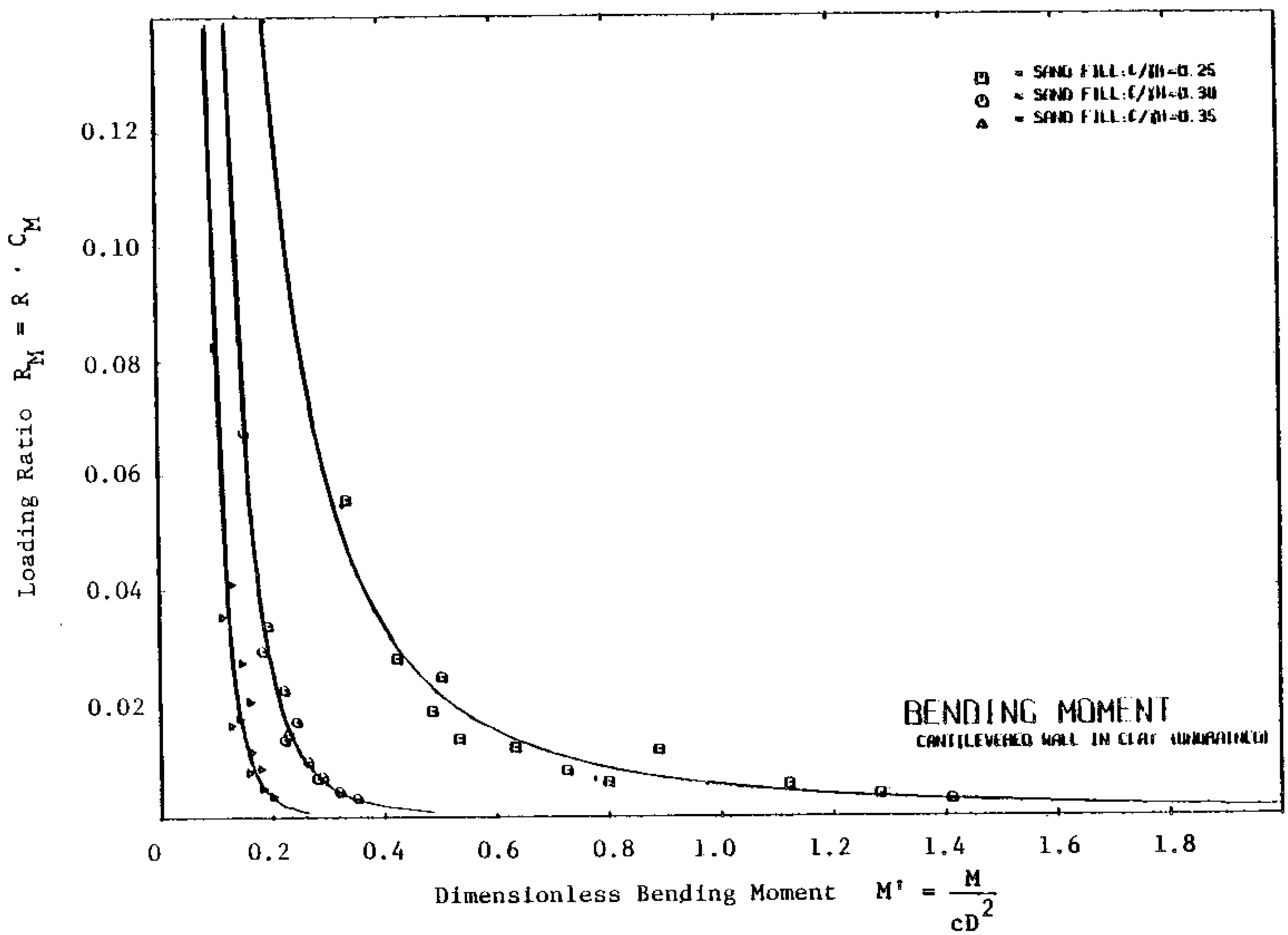


Figure 3-14. M' vs. R_M: clay (undrained)

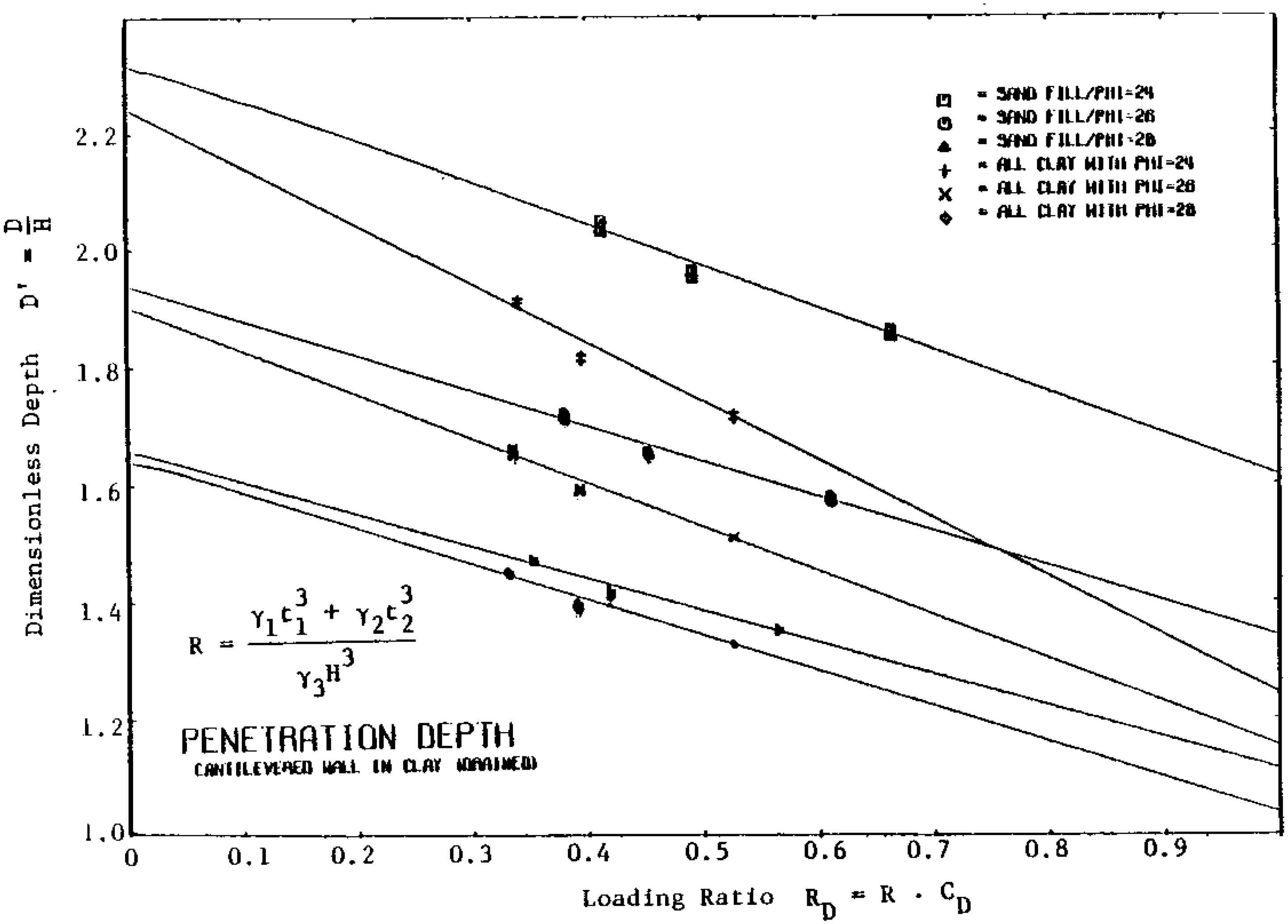


Figure 3-15. D' vs. R_D: clay (drained)

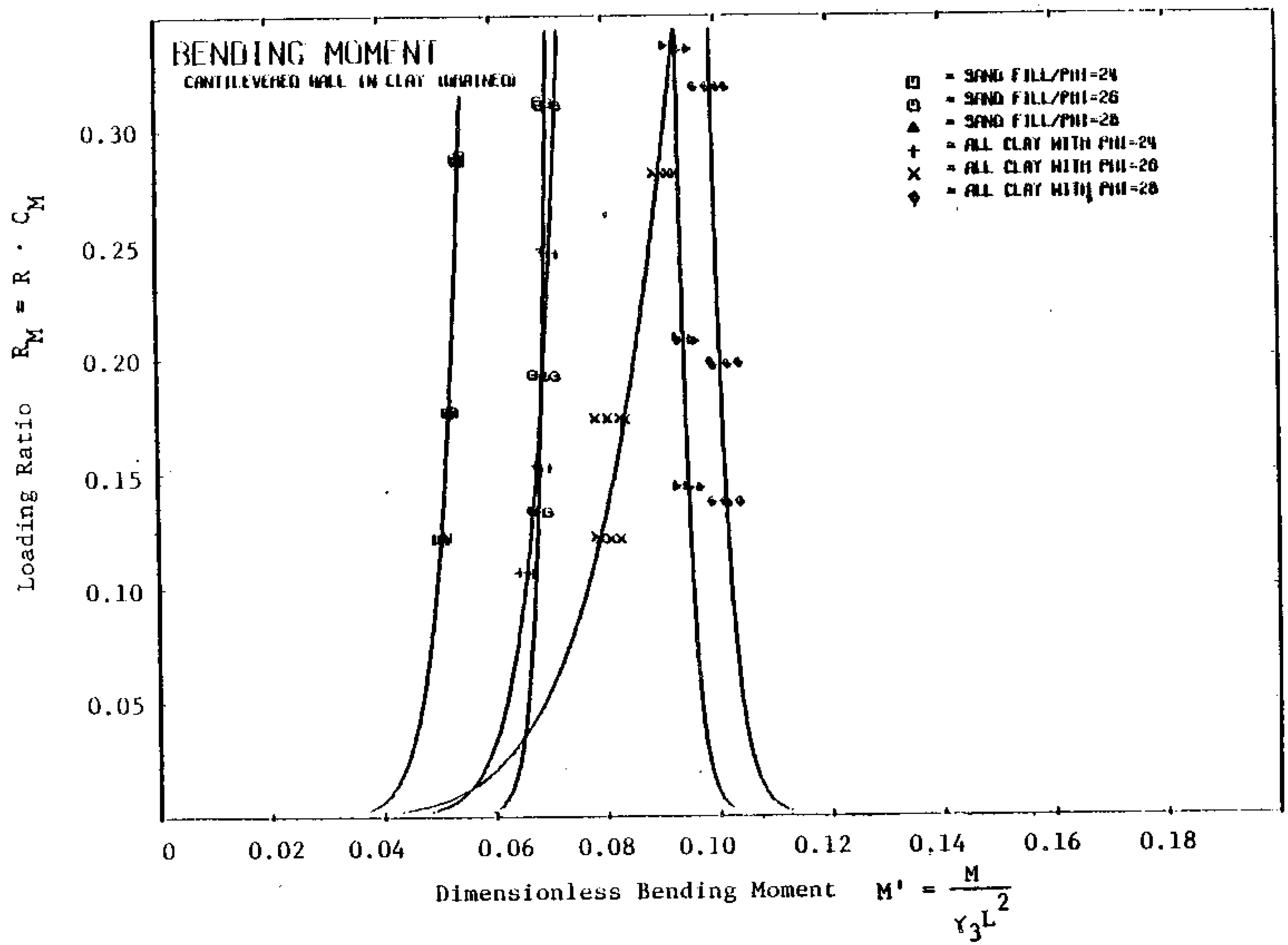


Figure 3-16. M' vs. R_M: clay (drained)

CHAPTER 4

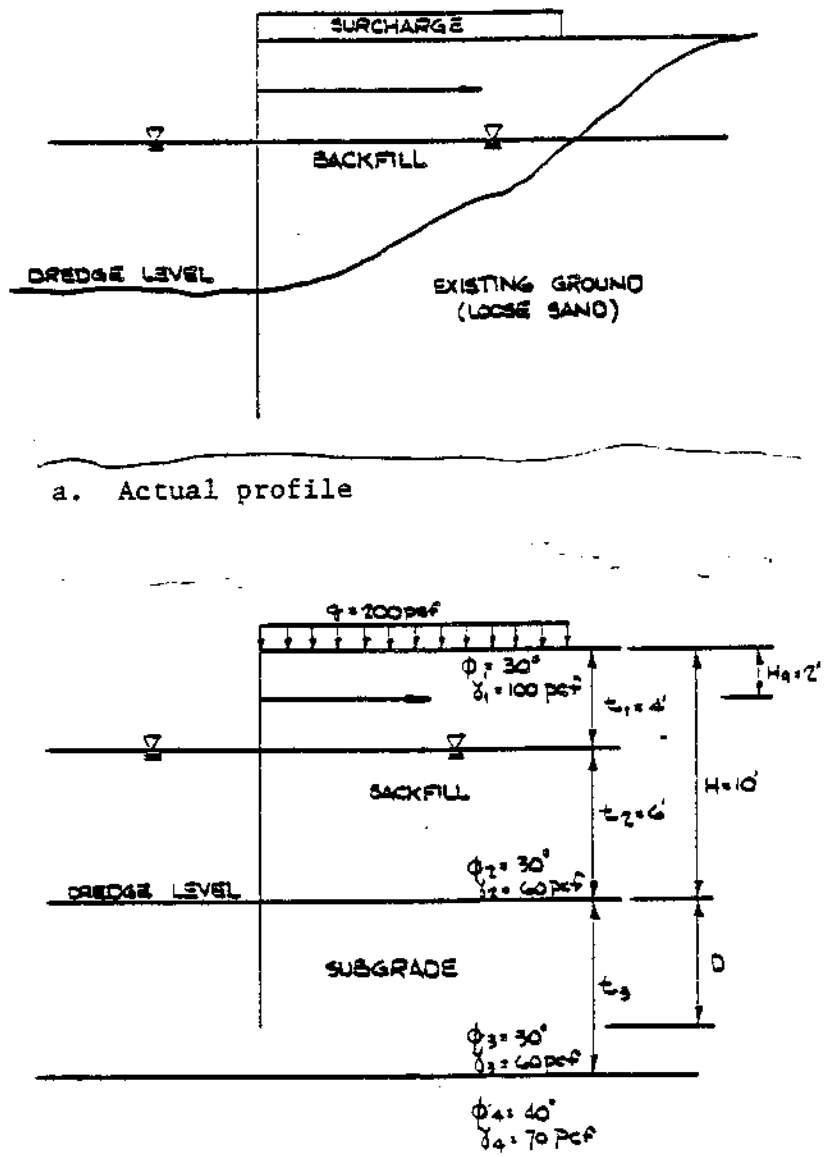
DESIGN PROCEDURES

The following pages outline the steps to be followed for the Free Earth Support, Rowe reduction, and simplified methods. Each of these is described in general terms. Specific examples illustrating the application of these methods in bulkhead design are contained in the Appendices.

4.1. Defining the Problem

Prior to any computations, the designer must take the information produced from the soils investigation and render it into a useful format. A sketch of the bulkhead geometry superimposed on the anticipated final soil profile is extremely helpful. For simplicity, soil layer interfaces should be horizontal planes. For example: the existing ground surface slopes downward as in Figure 4-1a. For design purposes, it is more convenient to assume a profile as in Figure 4-1b. A level slope is assumed to exist on the dredge side of the bulkhead.

It should be noted that the water table is identified as a soil layer interface. Although it is essentially the same soil below the water table as above, the moist unit weight is used above and the submerged unit weight below. Soil properties should be labeled for each layer.



a. Actual profile

b. Simplified profile

Figure 4-1. Defining the problem

The stress distribution, resultant forces, and centroids should be diagrammed as shown in Figure 4-2. Values should be tabulated in terms of rectangular and triangular stress distributions, resultant forces, centroids, moments about the tie-rod and moments about the point of application of the passive pressure resultant (2/3 D).

Penetration depth, tie-rod load and maximum bending moment computations are facilitated and may commence.

4.2. Anchored Walls in Sand

4.2.1. Free Earth Support Computations

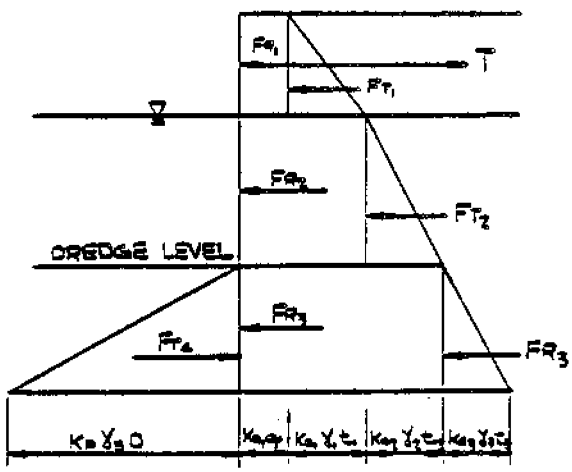
The Free Earth Support method uses statics to find the depth of penetration required for equilibrium, that is, the sum of moments taken about the tie-rod is zero. Using unfactored soil parameters would result in a factor of safety of unity, thus indicating imminent failure. Therefore, factored soil parameters are used to provide an adequate factor of safety against failure. For cohesionless soils,

$$\phi_f = \tan^{-1}\left(\frac{1}{SF}\tan\phi\right) \qquad (3\text{-}1)$$

in which ϕ_f = factored soil parameter, ϕ = unfactored soil parameter, and SF = a safety factor (commonly a minimum of 1.5). The factored active and passive stress coefficients are then computed in accordance with Equations 2-2 and 2-3. Figure 4-2 shows FES stress distributions and formulation to produce resultant forces, centroids, moment arms, and moments for the triangular and rectangular stress components.

Summing the moments about the tie-rod gives an equation:

$$aD^3 + bD^2 + cD + d = 0 \qquad (4\text{-}1)$$



	RECTANGULAR	TRIANGULAR
FORCES	$F_{R_1} = K_{a_1} q t_1$	$F_{T_1} = \frac{1}{2} K_{a_1} \gamma_1 t_1^2$
	$F_{R_2} = K_{a_2} (q + \gamma_1 t_1) t_2$	$F_{T_2} = \frac{1}{2} K_{a_2} \gamma_2 t_2^2$
	$F_{R_3} = K_{a_3} (q + \gamma_1 t_1 + \gamma_2 t_2) D$	$F_{T_3} = \frac{1}{2} K_{a_3} \gamma_3 D^2$
		$F_{T_4} = \frac{1}{2} K_p \gamma_3 D^2$
CENTROIDS	$C_{R_1} = \frac{1}{2} t_1$	$C_{T_1} = \frac{2}{3} t_1$
	$C_{R_2} = \frac{1}{2} t_2$	$C_{T_2} = \frac{2}{3} t_2$
	$C_{R_3} = \frac{1}{2} D$	$C_{T_3} = \frac{2}{3} D$
MOMENTS (TIE ROD)	$M_{R_1} = F_{R_1} (C_{R_1} - H_A)$	$M_{T_1} = F_{t_1} (C_{t_1} - H_A)$
	$M_{R_2} = F_{R_2} (C_{R_2} + t_1 - H_A)$	$M_{T_2} = F_{t_2} (C_{t_2} + t_1 - H_A)$
	$M_{R_3} = F_{R_3} (C_{R_3} + H - H_A)$	$M_{T_3} = F_{t_3} (C_{t_3} + H - H_A)$
		$M_{T_4} = -F_{t_4} (C_{t_3} + H - H_A)$
MOMENTS ($\frac{2}{3}$ D)	$M_{R_1} = F_{R_1} (\frac{2}{3} D + H - C_{R_1})$	$M_{T_1} = F_{t_1} (\frac{2}{3} D + H - C_{t_1})$
	$M_{R_2} = F_{R_2} (\frac{2}{3} D + t_2 - C_{R_2})$	$M_{T_2} = F_{t_2} (\frac{2}{3} D + t_2 - C_{t_2})$
	$M_{R_3} = F_{R_3} (\frac{1}{6} D)$	$M_{T_3} = 0$

Figure 4-2. Stress distribution and resultants

in which: $a = \frac{1}{3}(K'_{a3} - K'_p)\gamma_3$, $b = \frac{1}{2}(K'_{a3} - K'_p)\gamma_3(H-H_A) + \frac{1}{2}K'_{a2}(q + \gamma_1 t_1 + \gamma_2 t_2)$, $c = K'_{a2}(q + \gamma_1 t_1 + \gamma_2 t_2)(H-H_A)$, and $d = F_{R1}(\frac{1}{2}t_1 - H_A) + F_{R2}(\frac{1}{2}t_2 + t_1 - H_A) + F_{T1}(\frac{2}{3}t_1 - H_A) + F_{T2}(\frac{2}{3}t_2 + t_1 - H_A)$. K'_a and K'_p signify that ϕ_f was used.

A value for D is assumed and a trial-and-error process ensues until a satisfactory value for D is found, i.e., the sum of the moments is close to zero.

Including toe shear in the calculation tends to decrease the minimum penetration somewhat. Toe shear, T_s, is computed from the algebraic sum of the active and passive forces, the weight of pile and the effect of the soil-structure interface strength, such that:

$$T_s = (F_{T1} + F_{T2} + F_{T3} + F_{R1} + F_{R2} + F_{R3} - F_{T4})\tan^2(\delta_f) + W_p H_D \tan(\delta_f) \tag{4-2}$$

in which: W_p = weight per square foot of pile.
The toe shear is then added to the passive stress resultant (F_{T4}) and the iterations begin again. A reduced depth will result.

Once the penetration depth is established, the tie-rod load, P_{FES} (force per unit length of wall), is computed by summing moments about the point of application of the passive stress resultant, such that:

$$P_{FES}\,L = M_{R1} + M_{R2} + M_{R3} + M_{T1} + M_{T2}, \text{ and} \tag{4-3a}$$

$$L = (\frac{2}{3}D + H - H_A) \tag{4-3b}$$

This computation entails use of the unfactored soil parameters.

The maximum bending moment is then found by finding the point of zero shear, x, and summing moments about that point. If x is distance below the water table where shear is zero,

$$x = \frac{-b + \sqrt{b^2 - 4ad}}{2a} \qquad (4\text{-}4)$$

in which: $a = \frac{1}{2} K_{a2} \gamma_2$, $b = K_{a2}\gamma_1 t_1$, and $d = F_{T1} + F_{R1} - P$. The maximum moment (ft-lbs per unit length of wall) is found from:

$$M_{MAX} = P_{FES}\ (t_1 + x - H_A) - F_{T1}(\frac{1}{3}\ t_1 + x) - F_{R1}\ (\frac{1}{2}\ t_1 + x)$$

$$- \frac{1}{6}\ K_{a2}\gamma_2\ x^3 - \frac{1}{3}\ K_{a2}\gamma_1 t_1 x^2. \qquad (4\text{-}5)$$

Again, unfactored soil parameters are used.

4.2.2. Rowe Reduction

Since the actual tie-rod loads and bending moments differ from those calculated by the Free Earth Support method (Rowe, 1952), the Rowe reduction method is applied. To proceed with this method, the following parameters must be computed:

$$\alpha = \frac{H}{H_D} \qquad (4\text{-}6)$$

$$\beta = \frac{H_A}{H_D} \qquad (4\text{-}7)$$

$$\tau_{MAX} = \frac{12\ M_{MAX}}{H_D^3} \qquad (4\text{-}8)$$

Establishing the tie-rod load is simple when using Figure 2-17b: enter the tie-rod chart at the appropriate value and read off the factor, f_c, for the appropriate value. For unyielding anchorages, the factor r_t is also applied. The resulting tie-rod load

$$P = f_c \, P_{FES} \tag{4-9}$$

or, where appropriate

$$P = f_c \, r_t \, P_{FES} \tag{4-10}$$

Bending moment reductions are much more complex to figure. A pair of curves must be developed, one representing the loading and soil properties, the other representing flexibility characteristics of the pile. The operating curve is generated by values of

$$\tau_{op} = \tau_{MAX} \, r_d \tag{4-11}$$

Values of r_d are taken from the moment reduction chart in Figure 2-17a for values of log ρ.

The structural curve is generated by values of

$$\tau_S = \frac{\psi}{(H_D{}^2)^{2/3}} \tag{4-12}$$

in which ψ = the flexibility characteristic and

$$\psi = \frac{f_b}{(EI)^{2/3}} \tag{4-13}$$

where f_b = the allowable bending stress, S = the section modulus per unit length of wall, E = the elastic modulus, and I = the moment of inertia per unit length of wall. For rectangular sections, such as timber sheet piles,

$$\psi = \frac{2\ f_b}{E^{2/3}} \tag{4-14}$$

For a first approximation using Mariner steel sheet piling, ψ can be taken as 0.400 and, for A328 steel, ψ can be taken as 0.260. The intersection of operating and structural curves gives the design value τ, and the bending moment is found by

$$M = \tau H_D^3. \tag{4-15}$$

The section modulus required is

$$S = \frac{M}{f_b}\ . \tag{4-16}$$

This design section modulus is the minimum section required. The section modulus of the actual section used is then introduced into the computation of the structural curve values. In this case the actual flexibility characteristic of the section, ψ, is used. The design section resulting will most likely be the same as that calculated using the first approximation.

An example of the Free Earth Support method with Rowe reduction is given in the Appendices.

4.3. Cantilevered Walls in Sand

The procedures are similar to those for anchored walls in sand. The difference for depth calculations is that moments are taken about the toe of the wall because there is no tie-rod. Moment reductions proceed in the same manner, except that reduction factors are taken from Figure 2-20.

An example of the design of a cantilevered wall is contained in the Appendices.

4.4. Walls in Clay

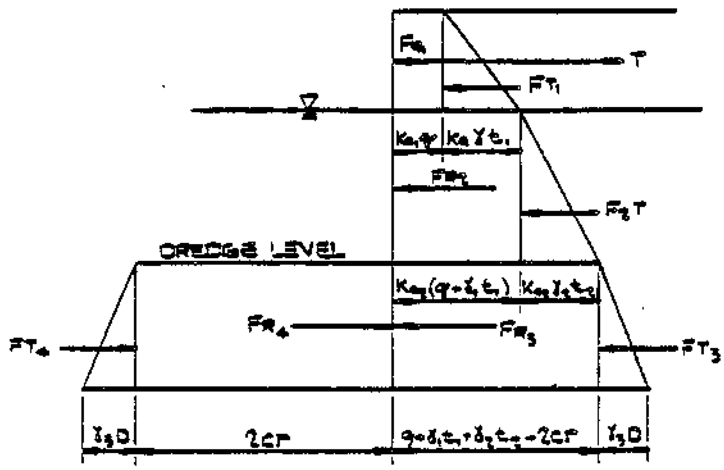
The short term behavior of anchored walls in clay is governed by the strength of the subgrade. The stability number, S_t, is the prime indicator of the ability of a wall to stand, where

$$S_t = \frac{2\ cr}{q + \gamma_1 t_1 + \gamma_2 t_1} \qquad (4\text{-}17)$$

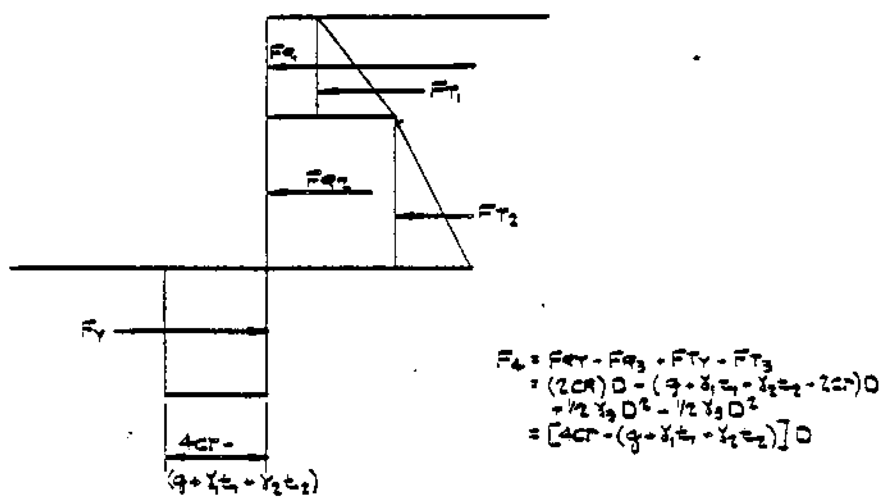
in which: c = the cohesion of the clay and r can be taken as 1.25.

From the geometry of the problem (Figure 4-3a), equilibrium cannot be achieved when the overburden is greater than 4 cr for any depth of penetration, or when S_t is less than or equal to 0.25. The first step in designing walls in clay is, therefore, to compute the stability number. Design should be abandoned for values of S_t less than or equal to 0.33.

If the stability number is of sufficient magnitude, depth of penetration is computed in the same manner as for walls in sand, except that the soil parameters are unfactored above the dredge level. The cohesion parameter is, however, factored. The ensuing computation is



a. Stress distribution



b. Net effect

Figure 4-3. Stress distribution for walls in clay simplified because of the resulting rectangular stress distribution below the dredge level (Figure 4-3b). The summation of moments about the tie-rod becomes

$$a^2D^2 + bD + d = 0 \qquad (4\text{-}18a)$$

in which: $a = \frac{1}{2}(4\ cr - q - \gamma_1 t_1 - \gamma_2 t_2)$, $b = (4\ cr - q - \gamma_1 t_1 - \gamma_2 t_2)\ (H - H_A)$, and $d = \frac{1}{2} Ka_1 \cdot \gamma_1 t_1^2\ (\frac{2}{3} t_1 - H_A) + \frac{1}{2} Ka_2\ \gamma_2 t_2^2\ (\frac{2}{3} t_2 + t_1 - H_A) + Ka_2(\gamma_1 t_1 + q)\ (\frac{1}{2} t_2 + t_1 - H_A)$.

The solution for depth becomes a matter of solving the quadratic equation

$$D = \frac{-b + \sqrt{b^2 - 4ad}}{2a} \qquad (4\text{-}18b)$$

The computations for tie-rod loads, point of zero shear, and maximum bending moment proceeds as for walls in sand.

4.4.1. Rowe Reduction Method, Anchored Walls in Clay

The procedure for moment reduction for walls in clay differs from that of walls in sand in the development of the operating curve. As seen in Figure 2-20a, a reduction factor, r_d, is given for only three different wall flexibilities:

$\log \rho = -3.6$ (stiff walls),

$\log \rho = -2.6$ (working stress), and

$\log \rho = -2.0$ (first yield).

Each selection of r_d is based upon the stability number, S_t, and the relative wall height, α.

The structural curve is developed in the same manner as for walls in sand. Tie-rod loads are also computed similarly, with the exception that factors are given in Figure 2-20b.

An example of the design of anchored walls in clay for the undrained (short term) case is contained in the Appendices.

4.4.2. Cantilevered Walls in Clay (Undrained)

As no investigation has been performed on cantilevered walls in clay subgrades, no reductions are allowed for bending moment. Penetration and bending moment calculations proceed by the Free Earth Support method. It can be anticipated that the resulting design will be conservative.

4.4.3. Undrained (Short Term) Condition vs. Drained (Long Term) Condition

Calculations should be made for both drained and undrained conditions. It is conceivable that soft clay subgrades could result in the short term case controlling while stiff clay subgrades would most likely result in the long term case controlling. The stability number may provide some hint, i.e., stability numbers greater than 0.5 indicate that the long term case will probably control.

4.5. Procedure for the Simplified Method

The essence of the simplified method is to utilize non-dimensional loading to find non-dimensional design parameters. The desired design parameter is then computed by multiplying the non-dimensional parameter by a factor.

The basic loading ratio, R, is merely the ratio of loading conditions above the dredge line to those below. For cohesionless conditions (walls in sand, walls in clay, drained)

$$R = \frac{\gamma_1 t_1^3 + \gamma_2 t_2^3}{\gamma_3 H^3} \tag{3-4}$$

and for clay (undrained)

$$R = \frac{\gamma_1 t_1^3 + \gamma_2 t_2^3}{(5c - \gamma_1 t_1 - \gamma_2 t_2) H^2} \tag{3-15}$$

in which γ_i = unit weight of the i^{th} layer, t_i = thickness of the i^{th} layer, H = free standing wall height, and c = cohesion of the subgrade.

A modifying coefficient, C, is used in conjunction with the loading factor for the particular design parameter sought, that is

$$R_D = R \cdot C_D \tag{3-17a}$$

$$R_P = R \cdot C_P, \text{ and} \tag{3-17b}$$

$$R_M = R \cdot C_M \tag{3-17c}$$

in which D = depth of penetration, P = tie-rod load and M = bending moment. A recap of the constituents of the modifying coefficients is shown in Table 3-4.

The non-dimensional design parameters are dimensionless penetration depth, D', dimensionless tie-rod load, P', and dimensionless moment, M', and are summarized in Table 4-1. L is the distance between tie-rod and point of passive stress application, or

Table 4-1. Normalizing parameters

Normalizing Parameter	Sand	Clay
Dimensionless Depth: D'	$\frac{D}{H}$	$\frac{D}{H}$
Dimensionless Tie-Rod Load: P'	$\frac{P}{\gamma_1 L^2}$	$\frac{P}{cD}$
Dimensionless Moment: M'	$\frac{M}{\gamma_2 L^3}$	$\frac{M}{cD^2}$

$$L = \frac{2}{3} D + H - H_A, \text{ and} \tag{4-14}$$

$$L = \frac{2}{3} D + H \tag{4-20}$$

for anchored and cantilevered walls in sand, respectively.

The non-dimensional design parameters are found by entering the appropriate curve (Figures 4-4 through 4-8) at the computed loading factor and reading off the result. An alternative is to use the equation of the curve, inserting the independent variable, the loading factor, and computing the resulting non-dimensional parameter.

Each case is comprised of different site conditions, i.e., different relative densities or cohesions for the fills and subgrades. If the design condition does not coincide with the conditions of the graph (Tables 3-1 and 3-2, Equations 3-2 and 3-3) interpolation, extrapolation, or assuming the most conservative condition are choices left to the designer. For instance, if the site has a subgrade whose angle of internal friction is 32 degrees, and loose fill will be placed, the designer may wish to interpolate between the "loose fill/loose subgrade" and "loose fill/medium subgrade" conditions. Or he may opt for the conservative approach and use "loose fill/loose subgrade."

The sequence for using the simplified method is to first compute the depth of penetration, D, then tie-rod load per unit length of wall, P, and finally, the bending moment, M. The design curves are entered using the appropriate loading factors, R. The non-dimensional design parameters are read from the curve and are multiplifed by the normalizing factors to give the design values sought.

An alternative to using the curves is to use the formulation provided. The operations can be performed easily with a hand calculator.

The design curves are contained in Figures 4-4 through 4-18 at the end of this chapter.

4.5.1. Walls in Sand

Each curve on a design chart refers to a particular condition. For walls in sand, the descriptions signify

loose: in which $\phi = 30°$, $\gamma_{moist} = 100$ pcf, $\gamma_{sat} = 120$ pcf;

medium: in which $\phi = 35°$, $\gamma_{moist} = 105$ pcf, $\gamma_{sat} = 125$ pcf; and

dense: in which $\phi = 40°$, $\gamma_{moist} = 110$ pcf, $\gamma_{sat} = 130$ pcf.

The first term of the description refers to the condition of the fill, and the second refers to the subgrade. Each curve is labelled such that

L/L = loose fill over loose subgrade,

L/M = loose fill over medium subgrade,

M/M = medium fill over medium subgrade,

M/D = medium fill over dense subgrade, and

D/D = dense fill over dense subgrade.

Variations in unit weight cause no significant problems in computations as these merely change the value of the loading factor, R. Deviations from the specified angle of internal friction on the other hand must be dealt with by interpolating or by assuming a conservative value. When actually performing the computations, the submerged unit weight should be used.

An example of the design for an anchored wall in sand appears in the Appendices.

4.5.2. Walls in Clay (Undrained)

Design curves for walls in clay (undrained) are identified by the condition describing the ratio of overburden stress to cohesion, that is, $c/\gamma H$, in which γ = the unit weight of the fill, taken as 100 pcf (15.7 kN/m^3), c = the subgrade cohesion, and H = the free standing wall height.

Granular soil of loose sand is assumed for the fill as cohesion in the fill renders an unconservative stress distribution in the undrained condition. The Rankine active stress distribution,

$$\sigma_H = \gamma h - 2c \tag{4-21}$$

results in no loading against the wall, even for modest amounts of cohesion. The drained condition would control in such situations.

To identify the site in terms of the proper design curve, the moist unit weight of the fill, free standing wall height, and cohesion are combined as above. It is likely that interpolation will be required. High values of cohesion generally result in low values of penetration depth, thus a small range of values is presented in the charts.

An example of the design of an anchored wall in clay (undrained) appears in the Appendices.

4.5.3. Walls in Clay (Drained)

The design curves for walls in clay (drained) are identified by the fill component and subgrade strength. The fill component may consist of loose granular fill or it may consist of the same material as the subgrade. The minimum value of subgrade strength is an angle of internal friction of 24 degrees. Lower values may be extrapolated from the curve data, but caution should be used since accuracy decreases as the range of extrapolation increases. Interpolation between curves should prove to be less of a problem.

An example of the design of an anchored wall in clay (drained) appears in the Appendices.

4.6. Conclusions

The use of the simplified curves enables the designer to compute the desired design parameters quickly. Because the Free Earth Support and Rowe methods involve many steps, there is greater potential for error than in using the design curves. In spite of the apparent simplicity, care must be taken to insure that graphs are read correctly and extrapolations do not extend beyond a reasonable range. Unusually high or low results should indicate that an error may have occurred.

4.7. Summary

The design procedure for the Free Earth Support method, Rowe reduction method, and the new simplified method were outlined. The complexity involved in the Free Earth Support and Rowe reduction methods renders those methods tedious and has high potential for error. The simplified method, if properly used, reduces the potential for error and is simple compared to the other methods. The examples found in the Appendices demonstrate the application of Free Earth Support, Rowe reduction, and new methods.

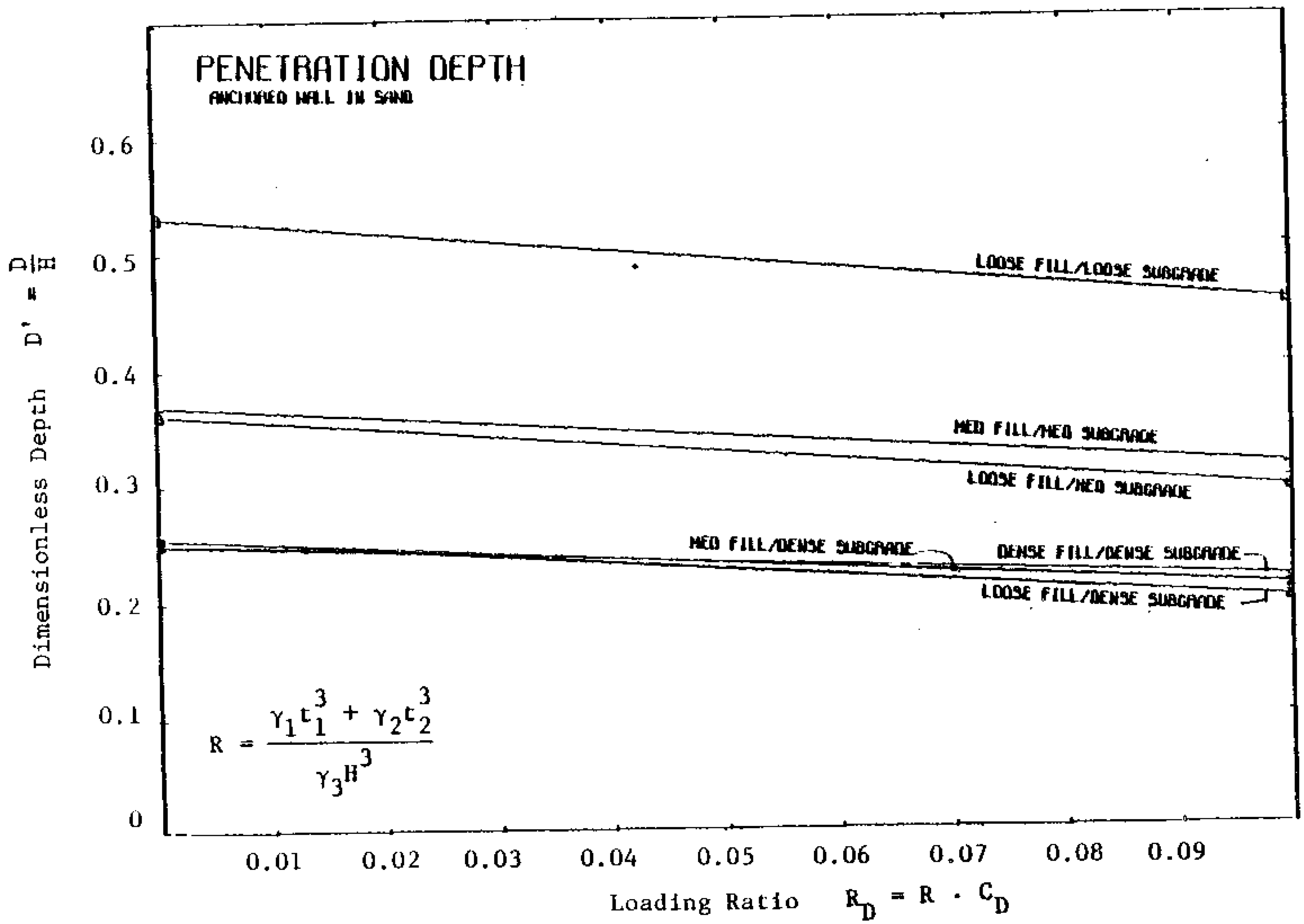


Figure 4-4. D' vs. R_D: sand

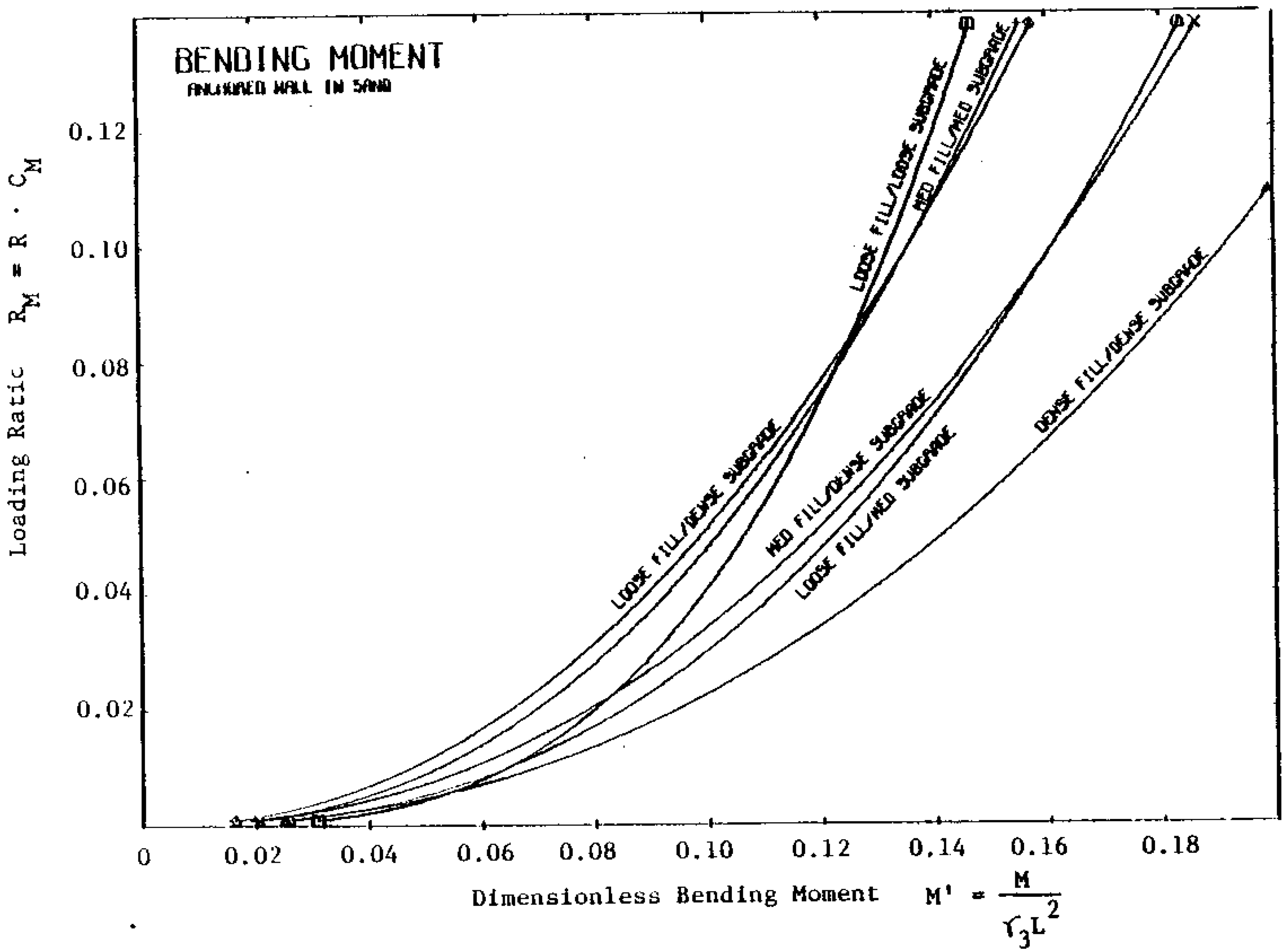


Figure 4-5. P' vs. R_p: sand

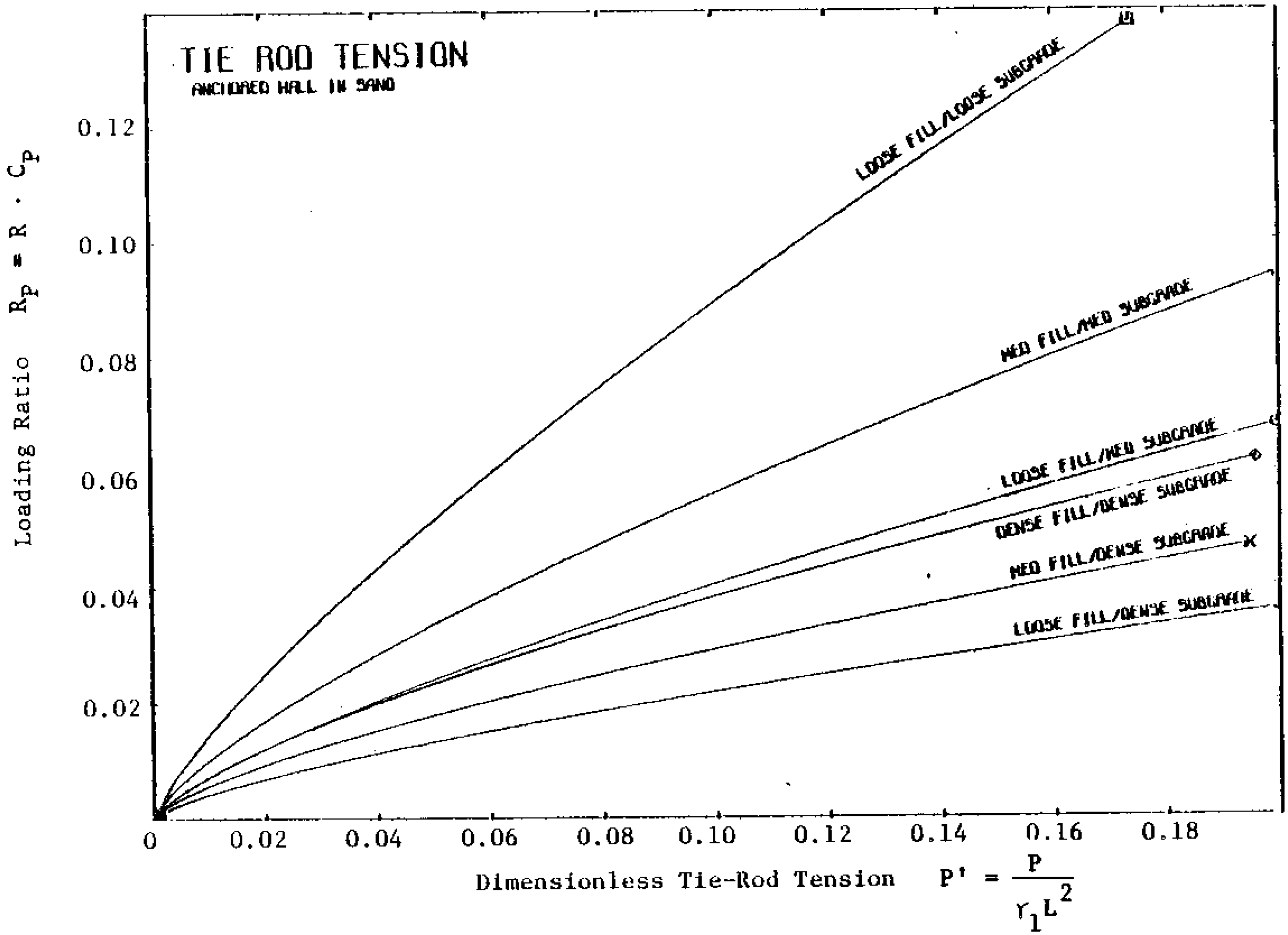


Figure 4-6. M' vs. R_M: sand

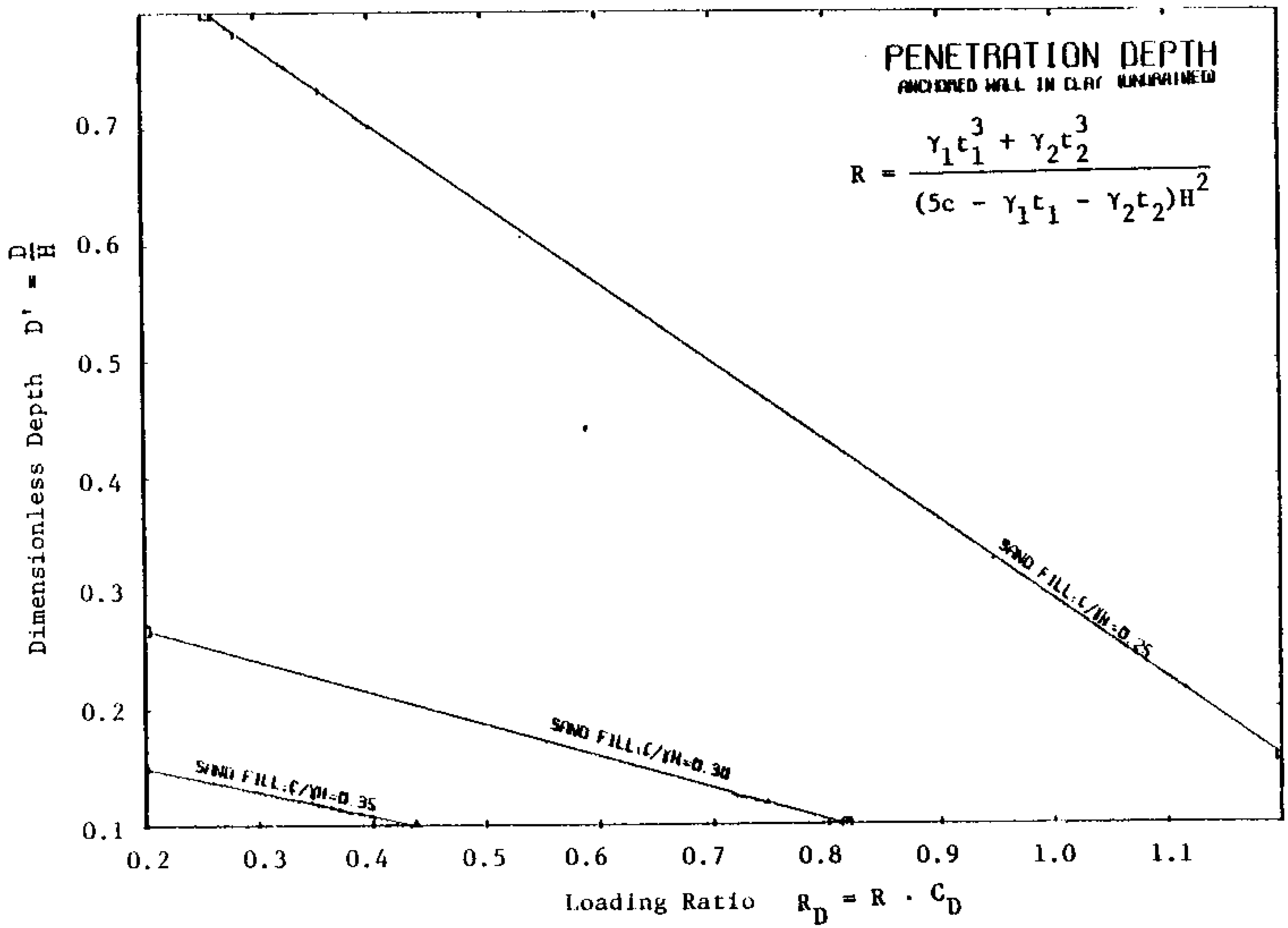


Figure 4-7. D' vs. R_D: clay (undrained)

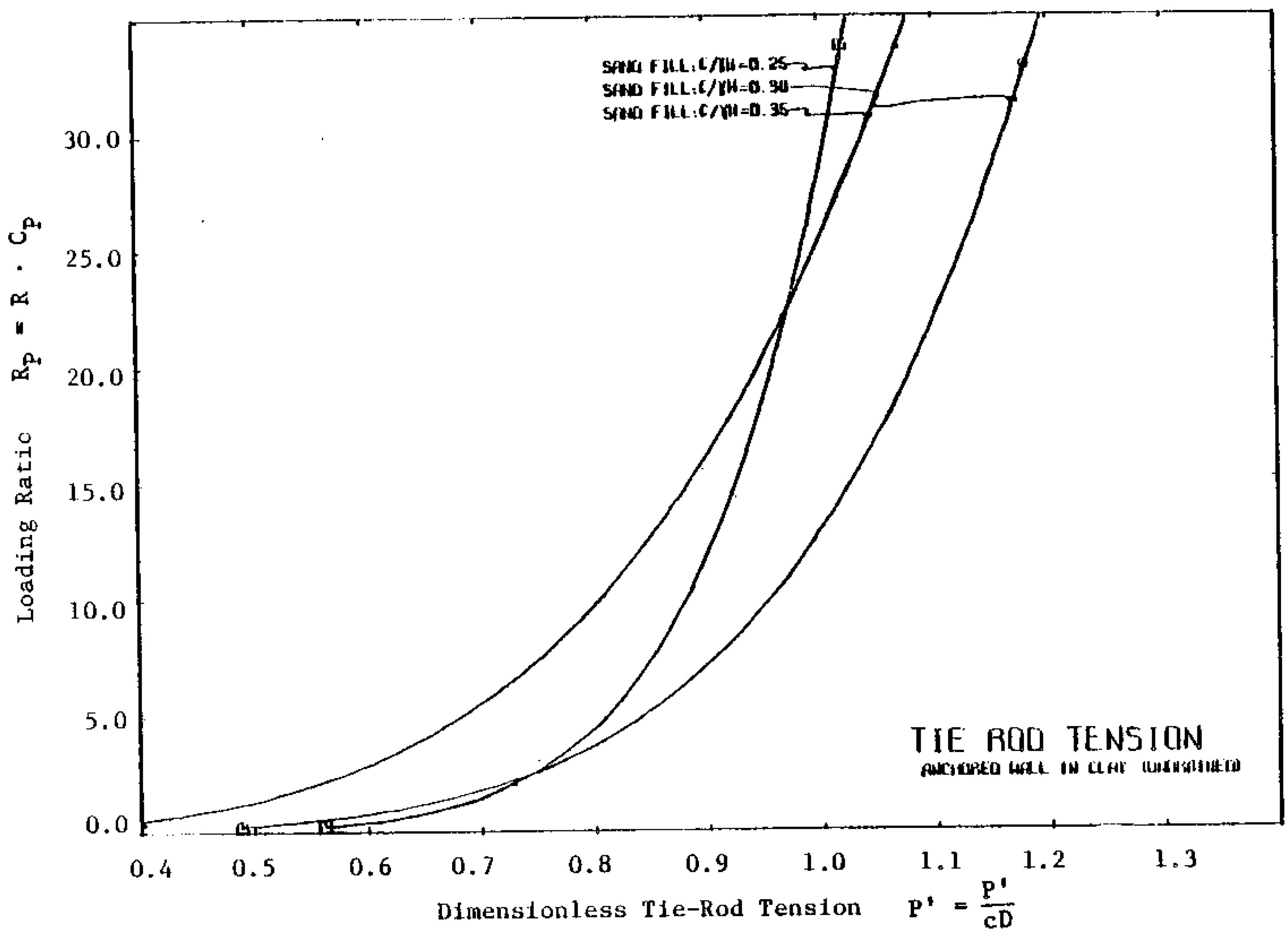


Figure 4-8. P' vs. R_p: clay (undrained)

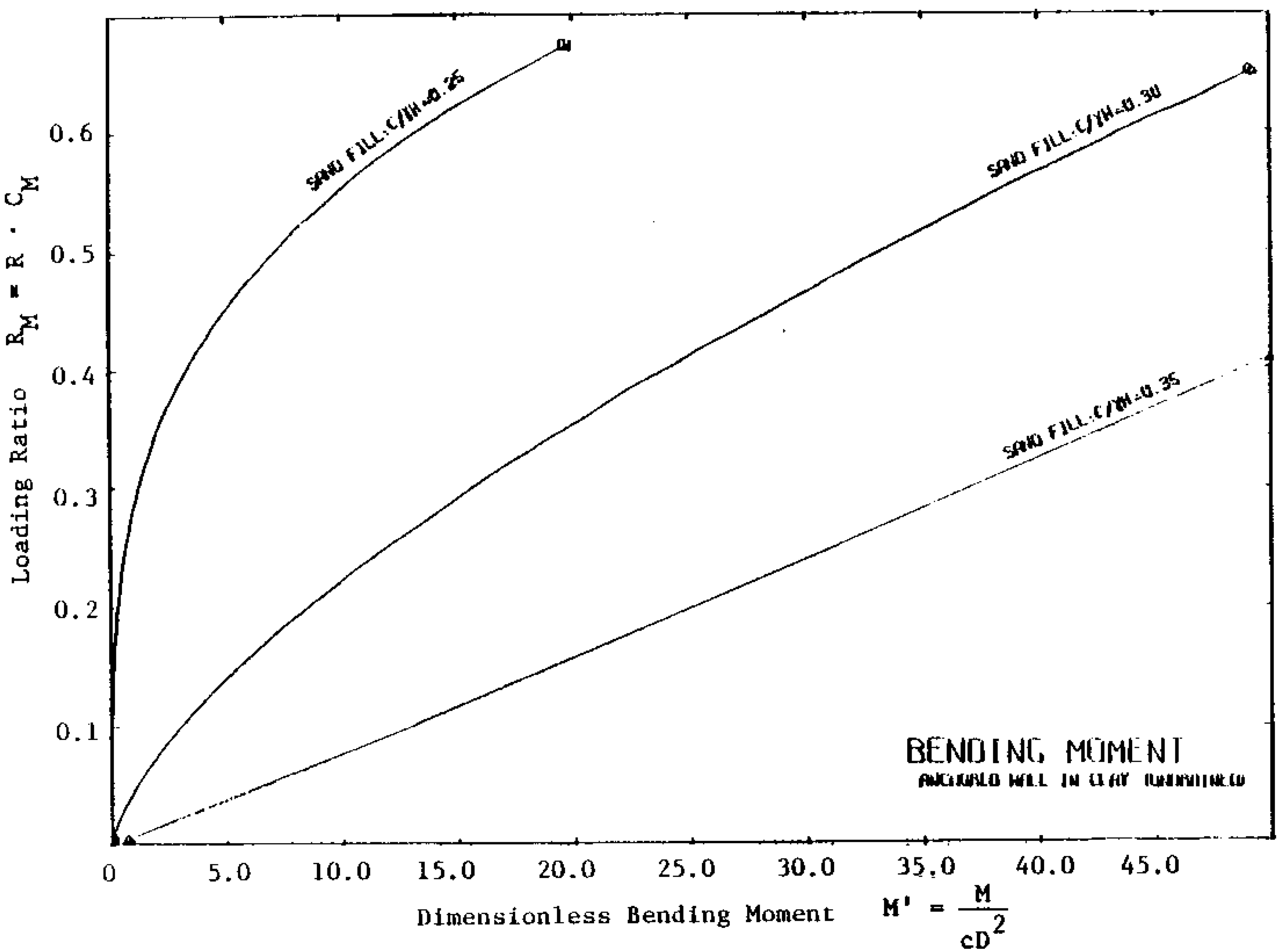


Figure 4-9. M' vs. R_M: clay (undrained)

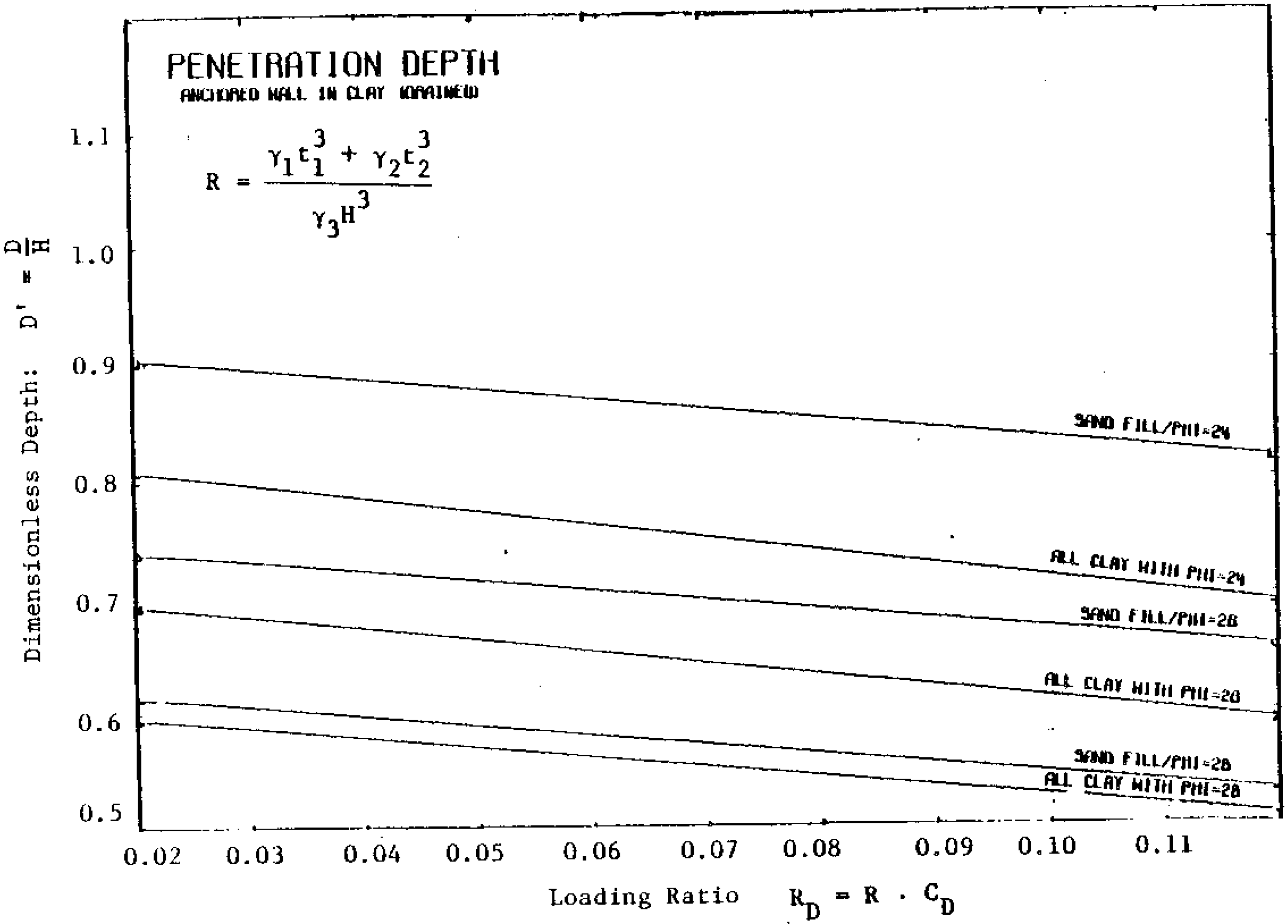


Figure 4-10. D' vs. R_D: clay (drained)

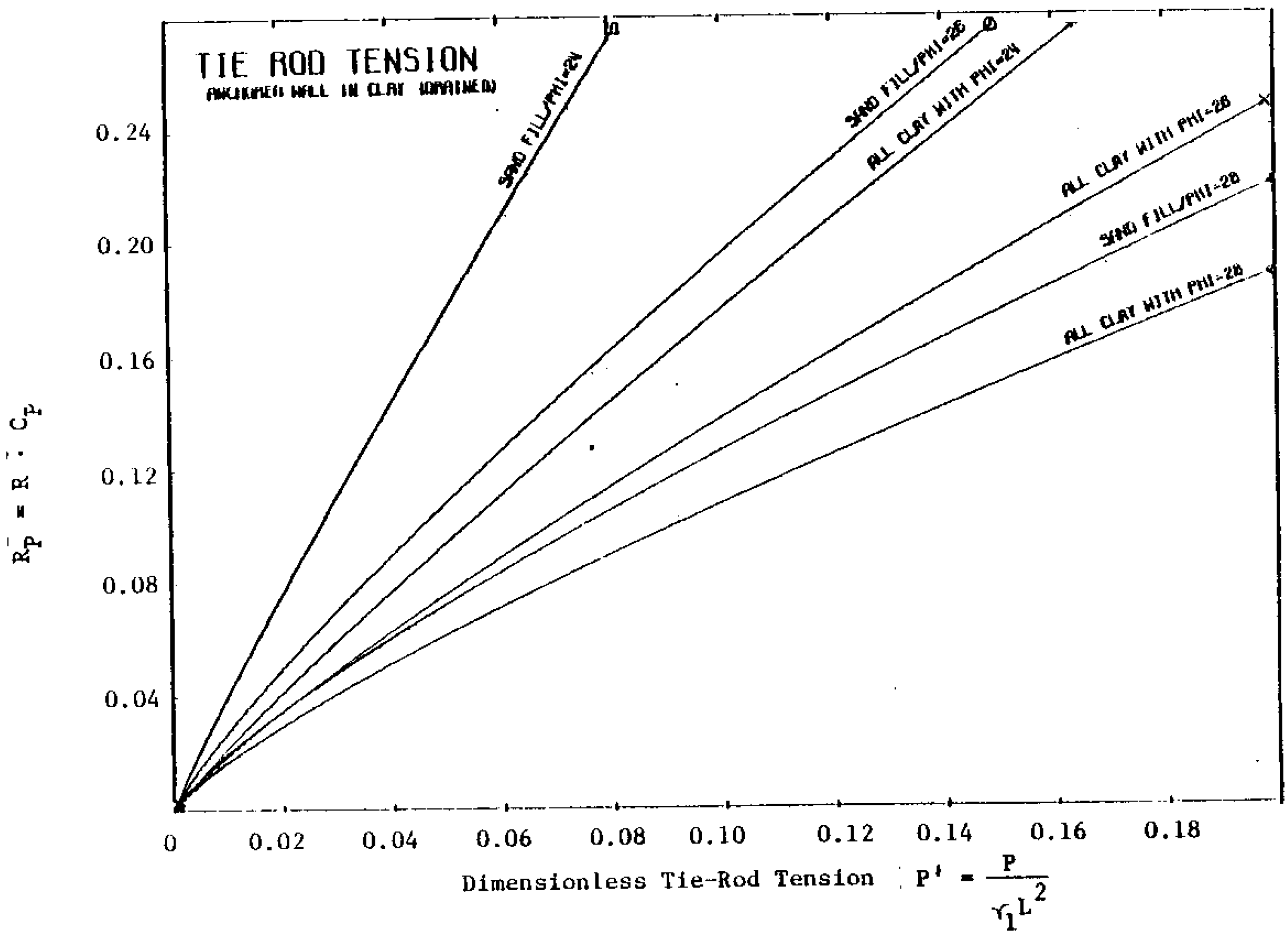


Figure 4-11. P' vs. R_p: clay (drained)

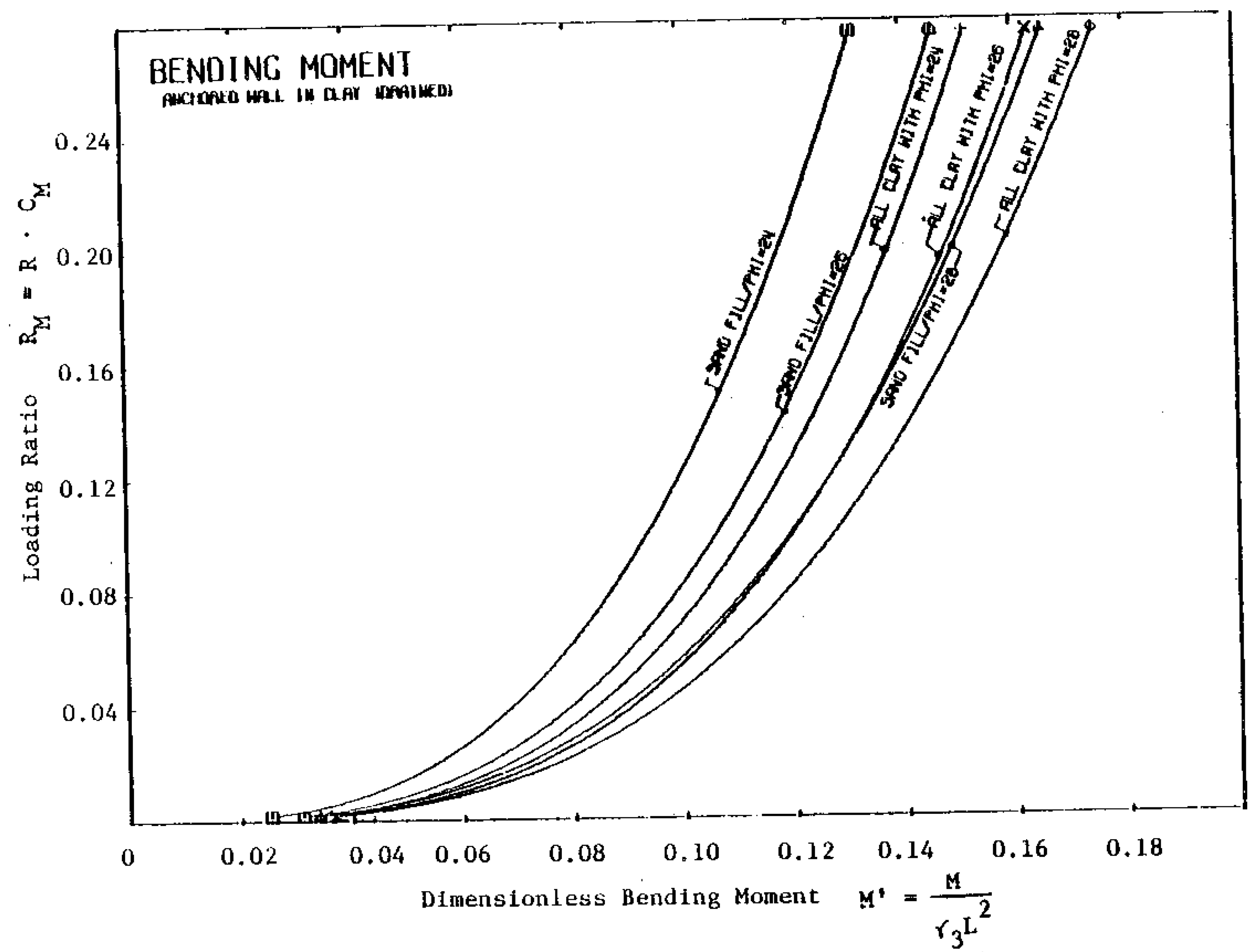


Figure 4-12. M' vs R_M: clay (drained)

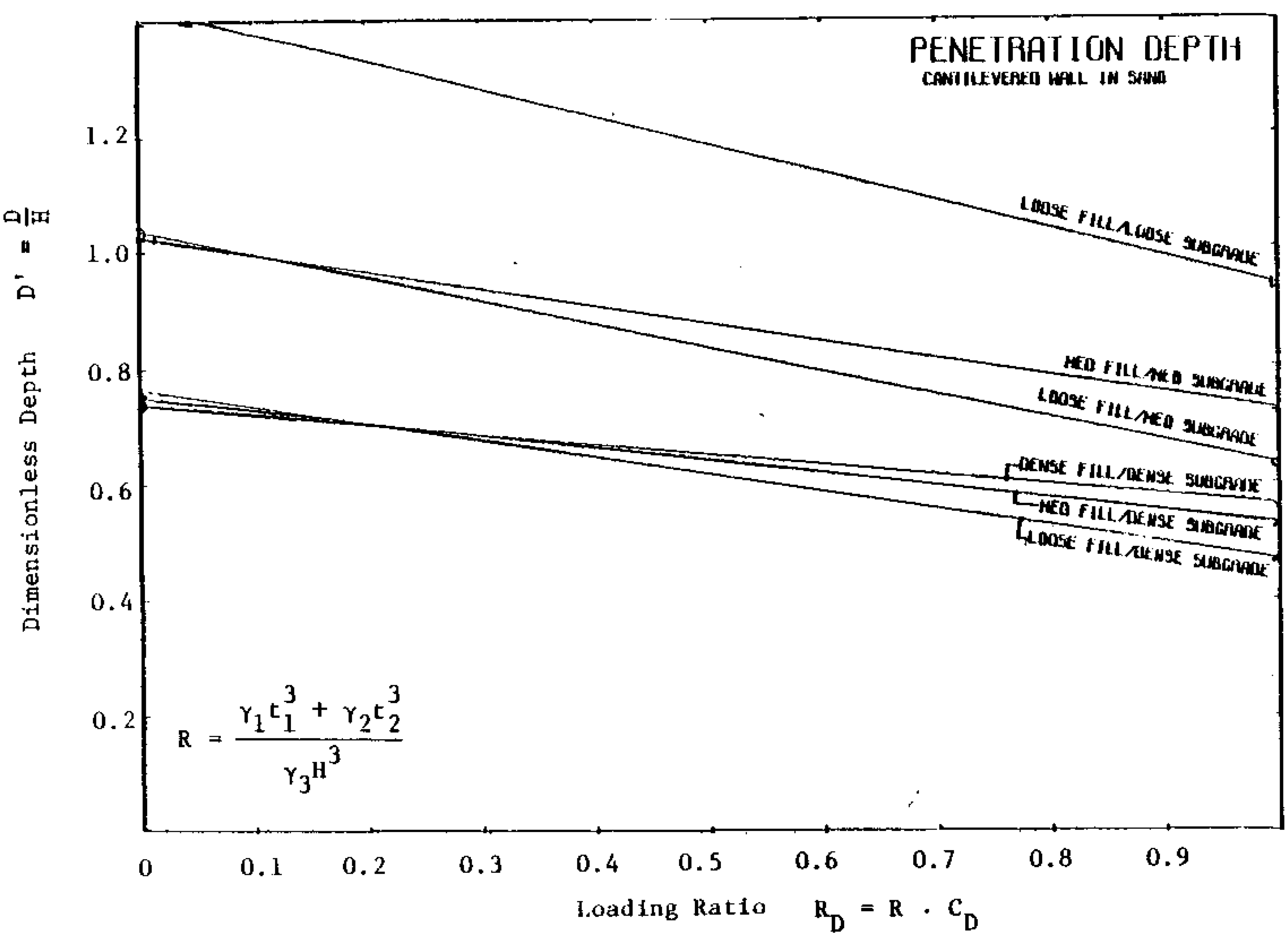


Figure 4-13. D' vs R_D: sand

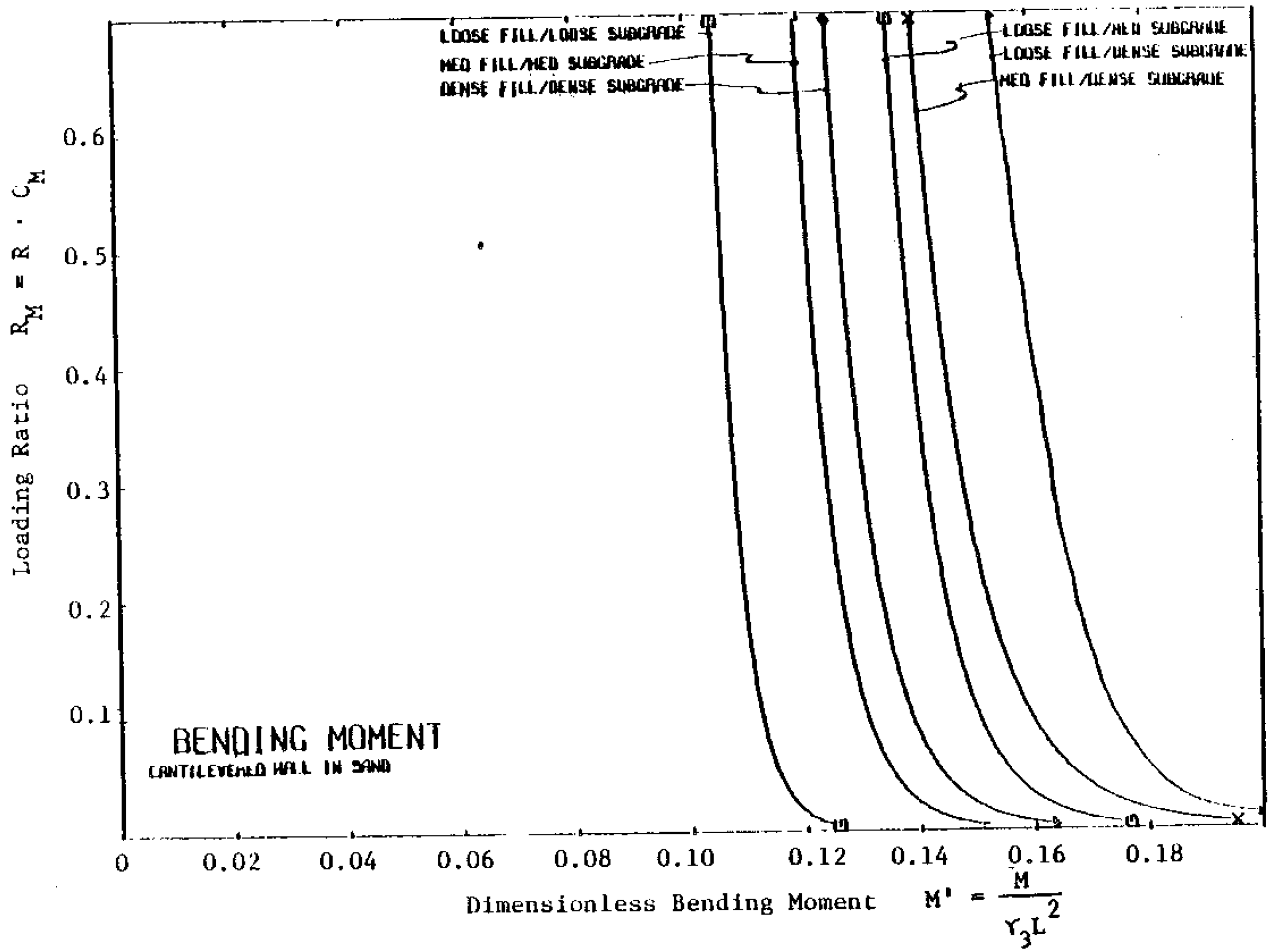


Figure 4-14. M'vs. R_M: sand

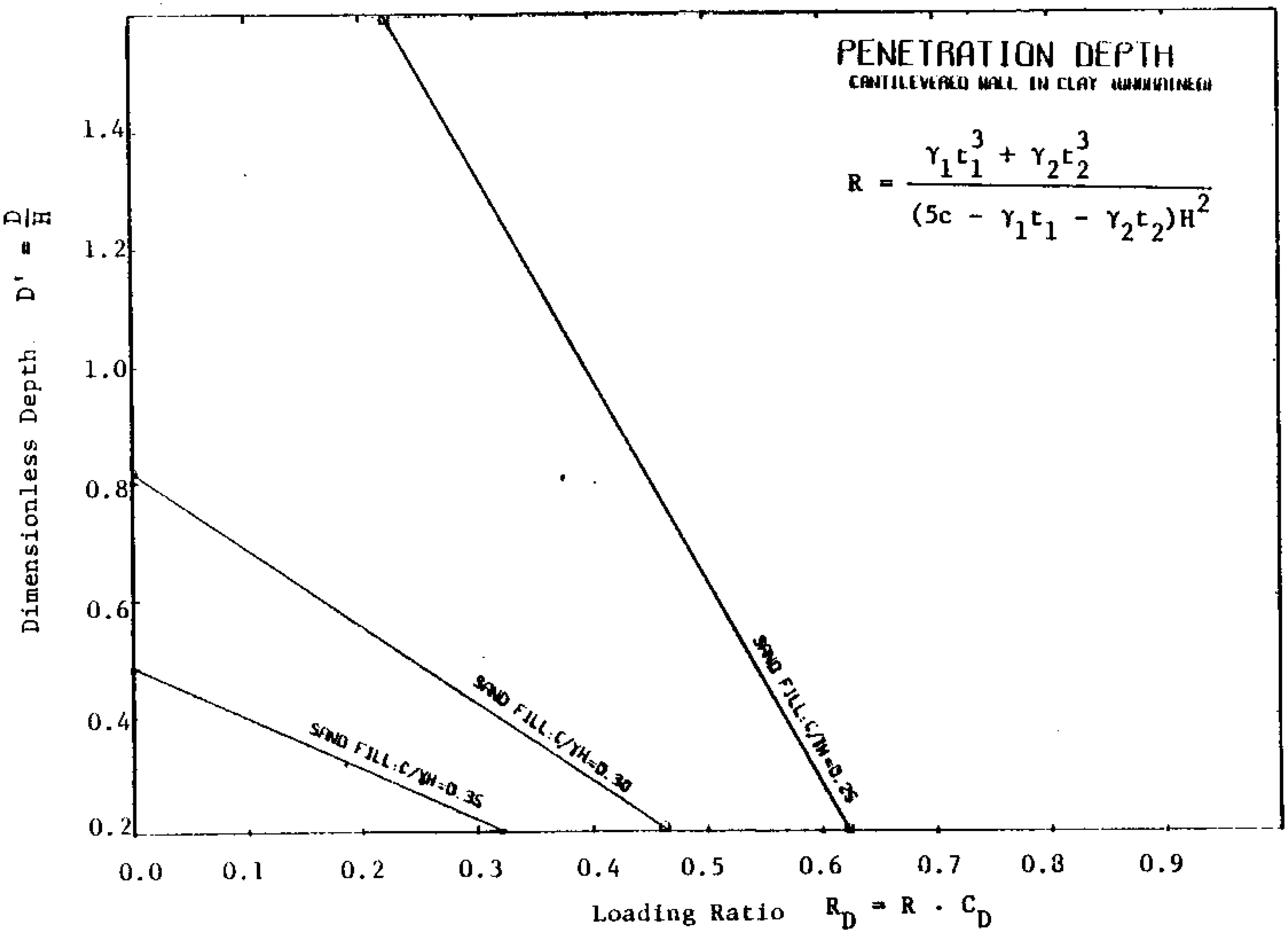


Figure 4-15. D' vs. R_D: clay (undrained)

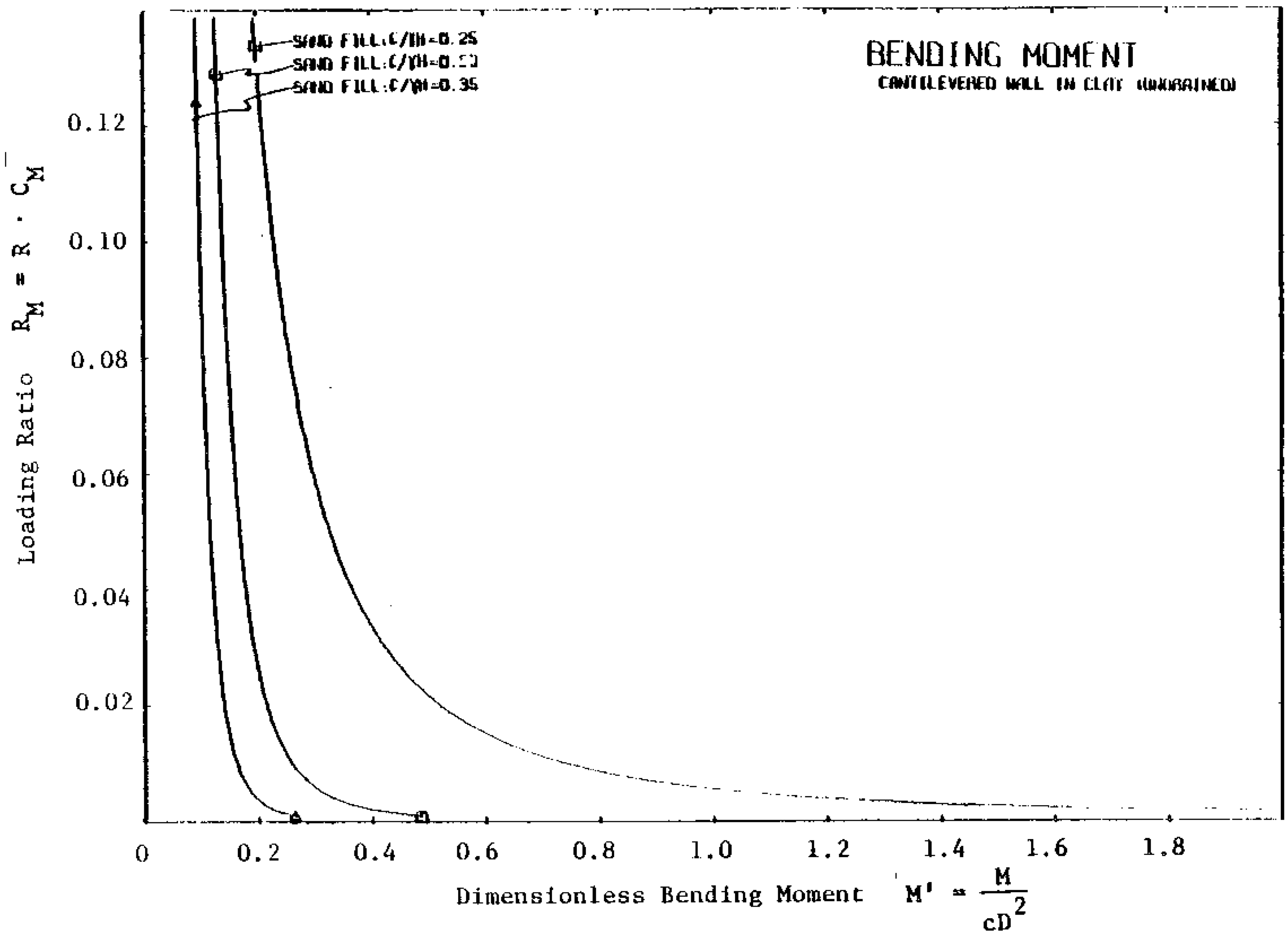


Figure 4-16. M' vs R_M: clay (undrained)

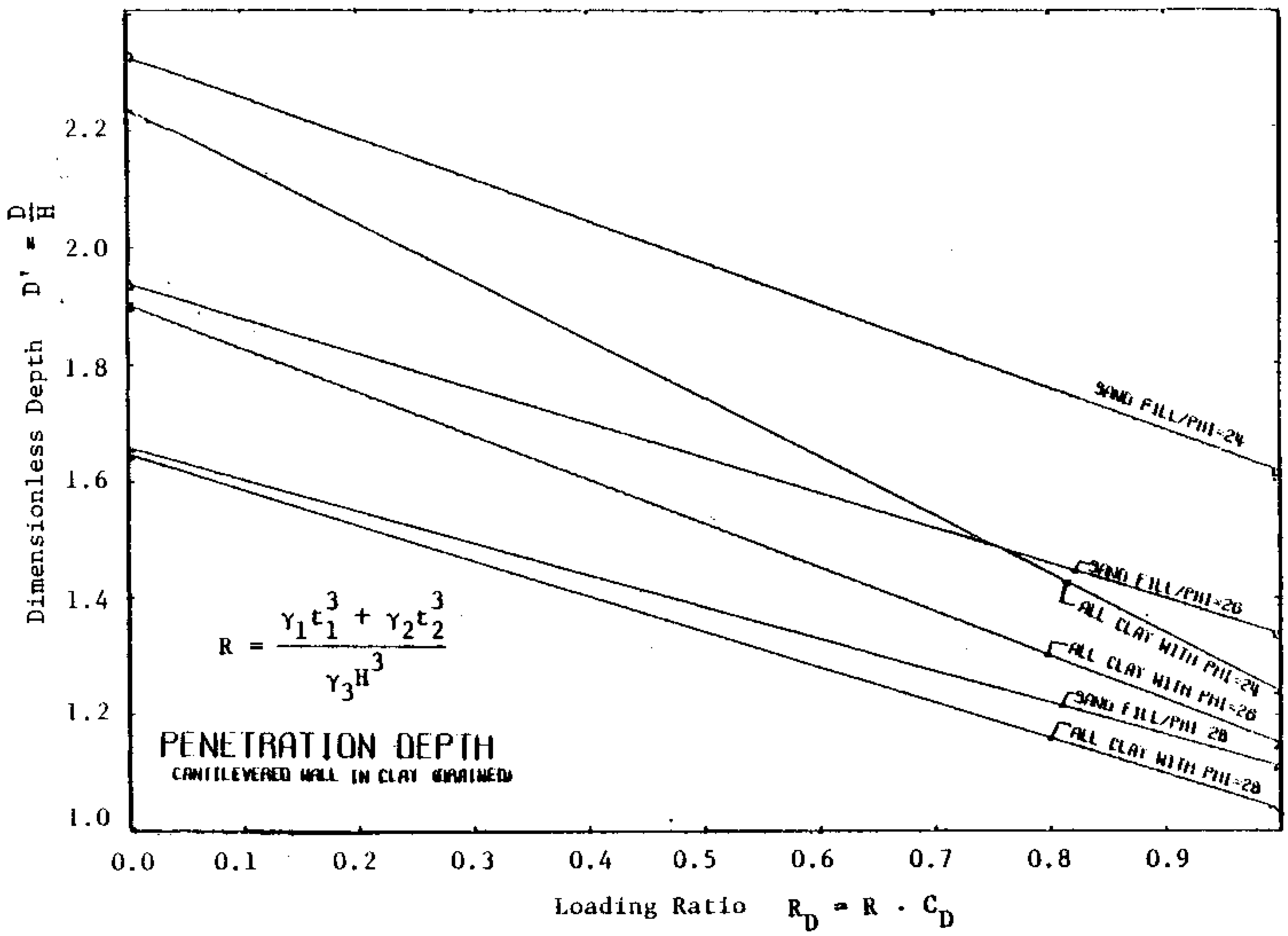


Figure 4-17. D' vs R_D: clay (drained)

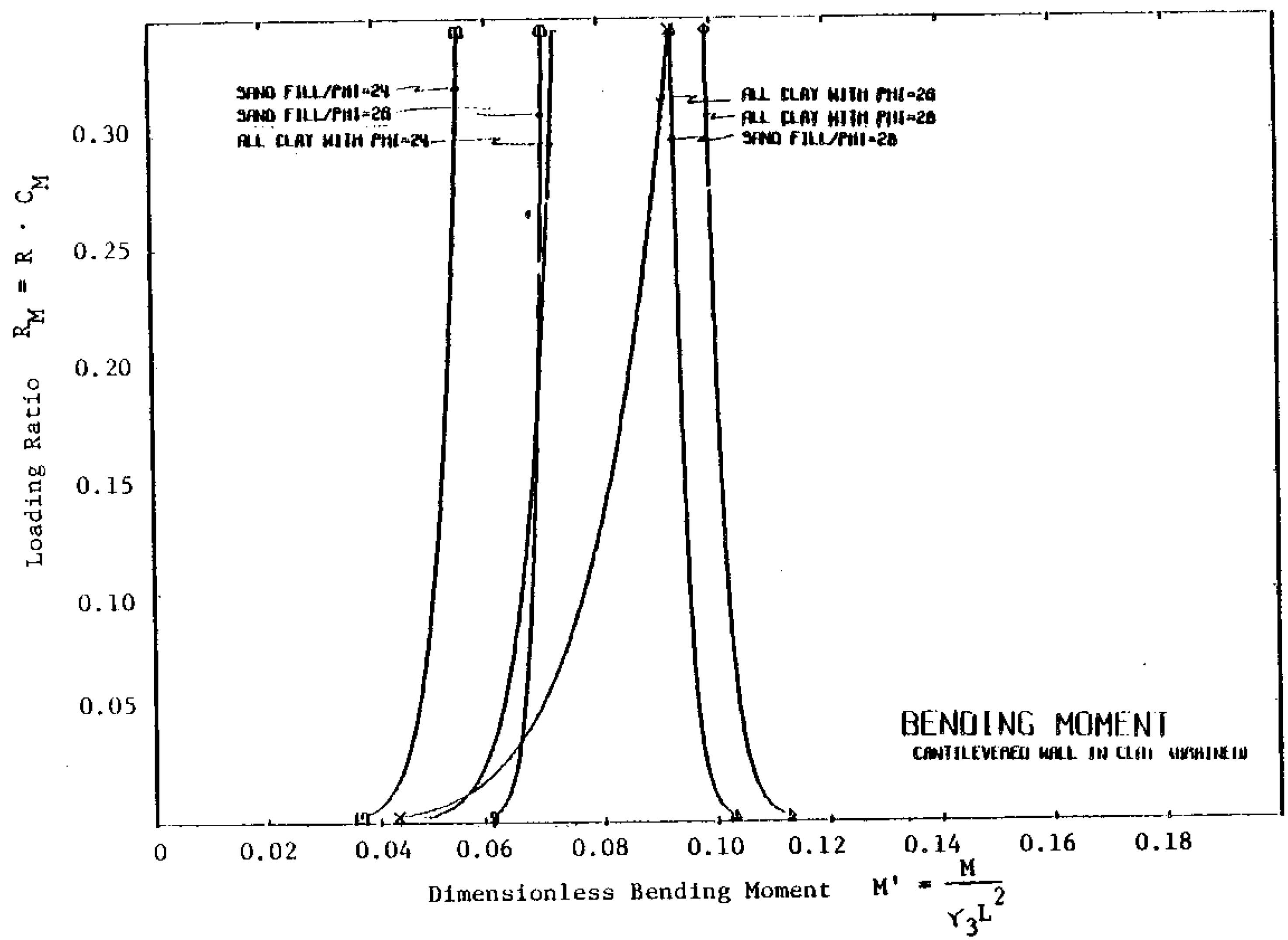


Figure 4-18. M' vs. R_M: clay (drained)

CHAPTER 5

DESIGN OF THE BULKHEAD SYSTEM

Bulkhead design requires more than determining penetration depth, bending moment, and tie-rod load. External loads must be considered and the structural components must be designed keeping in mind the cost effectiveness of various construction materials. External loads include surcharges imposed upon the backfill, hydrostatic imbalance in the backfill, ice-thrust, mooring loads, and impact loads. The structural components, i.e., sheet piles, tie-rods, wales, splices, and anchorages, must be dimensioned and detailed. The cost effectiveness of the entire system requires consideration of the strength, longevity, availability, and fastening methods of the component materials.

5.1. External Loading

External loads must be accounted for when designing an earth retaining system as these loads will increase the required penetration depth, maximum bending moment, and tie-rod load. The external loads that the designer must contend with are uniformly distributed loads, point loads, line loads, hydrostatic imbalance, ice thrust, mooring pull, and impact loads. Other environmental loads are discussed by Hubbell and Kulhawy (1979).

5.1.1. Uniformly Distributed Loads

Uniformly distributed loads are easily dealt with. The horizontal stress, p_h, resulting from a surcharge, q (force/unit area), is given by

$$p_h = K_a q \tag{5-1}$$

in which K_a = the active stress coefficient. The resulting stress distribution is rectangular (Figures 2-6 and 4-1). The resultant forces are then incorporated into the equilibrium calculations for penetration depth and tie-rod loads.

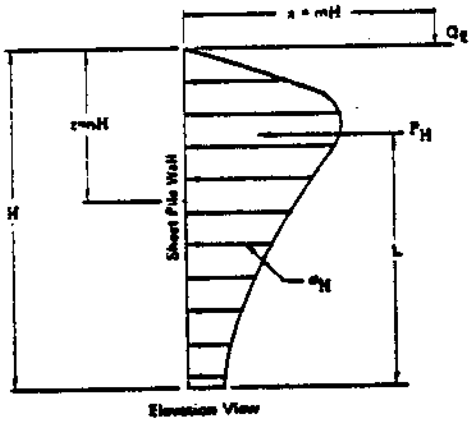
When the design charts are used, the surcharge can be converted into an equivalent height of soil, h_{eq}, given by

$$h_{eq} = \frac{q}{\gamma_1} \tag{5-2a}$$

in which γ_1 = the unit weight of soil comprising the backfill. The equivalent height of soil is merely added to the free standing wall height, H, and the resulting dimension is used throughout the computations. An example is given in the Appendices.

5.1.2. Point and Line Loads

The effects of point and line loads are treated in a semi-empirical manner (Terzaghi, 1954). Elastic theory, as expressed in the Boussinesq equation, was modified by experiment and the results given as in Figure 5-1. Knowing the intensity of the surcharge load, the designer uses the formula shown to compute the resultant horizontal force, P_H. The point of application is then found by choosing the appropriate dimension L for the corresponding value of m in Figure 5-1b and the computations may proceed.



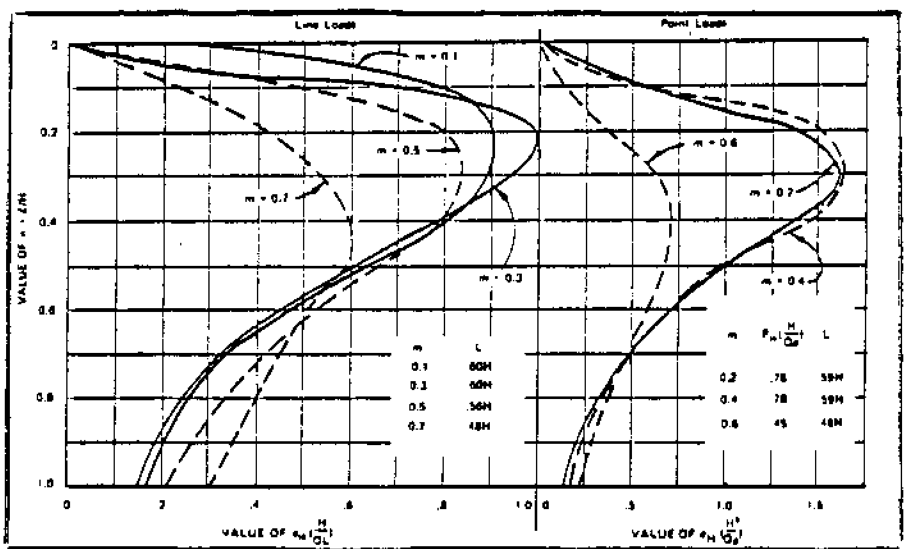
$$\sigma_H = 0.20\frac{Q_\ell}{H}\cdot\frac{n}{(0.16+n^2)^2} \quad (\text{for } m \leq 0.4)$$

$$P_H = 0.55\,Q_\ell, \text{ resultant force}$$

$$\sigma_H = 1.28\frac{Q_\ell}{H}\cdot\frac{m^2 n}{(m^2+n^2)^2} \quad (\text{for } m > 0.4)$$

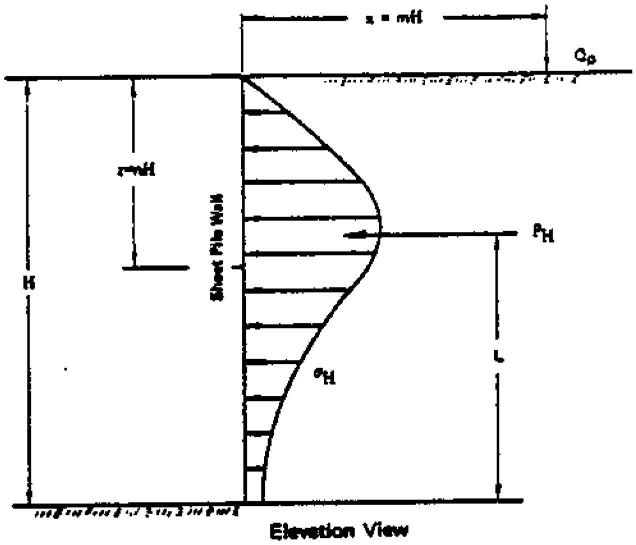
$$P_H = \frac{0.64\,Q_\ell}{(m^2+1)} \text{ resultant force}$$

a. Horizontal stress due to line load (Teng, 1962, p. 89)



b. Horizontal stress due to point and line load (Naval Facilities Engineering Command, p. 7-10-10)

Figure 5-1. Surcharge loads



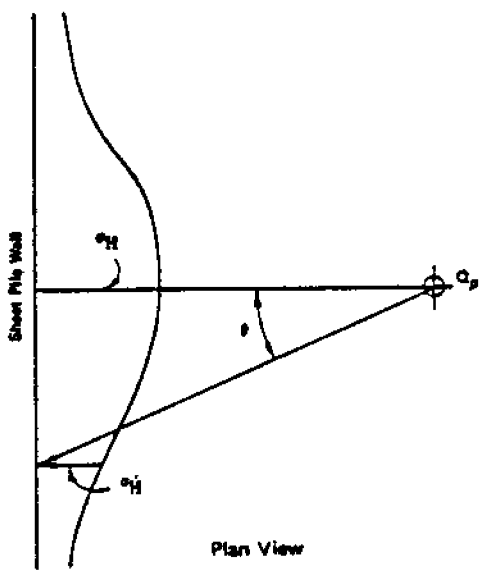
$$\sigma_H = 0.28\,\frac{Q_p}{H^2}\cdot\frac{n^2}{(0.16+n^2)^3}\quad(\text{for } m \le 0.4)$$

$$P_H = 0.78\,\frac{Q_p}{H}\quad(\text{see Fig. 11})$$

$$\sigma_H = 1.77\,\frac{Q_p}{H^2}\cdot\frac{m^2n^2}{(m^2+n^2)^3}\quad(\text{for } m > 0.4)$$

$$P_H = 0.45\,\frac{Q_p}{H}\quad(\text{see Fig. 11})$$

c. Horizontal stress due to point load (Teng, 1962, p. 91)



$\sigma'_H = \sigma_H \cos^2(1.1\,\theta)$

d. Horizontal stress due to point load (Teng, 1962, p. 91)

Figure 5-1. Continued

When the design charts are used, an equivalent height of soil is employed in a manner similar to the uniformly distributed case. For point and line loads,

$$h_{eq} = \frac{P_H}{\gamma_1 \ (H-L)} \tag{5-2b}$$

in which: H = the free standing wall height, and L = the distance from the dredge level to the point of application of P_H. The free standing wall height is then adjusted by increasing the dimension by h_{eq}. Design examples are given in the Appendices.

5.1.3. Hydrostatic and Seepage Effects

Fills containing significant amounts of soils of low permeability, such as clay, silt or fine sand, may cause a hydrostatic imbalance. Rapid tidal changes or substantial precipitation will cause saturation of the fill above the water level and, because of the low permeability of the fill, a hydrostatic imbalance results. The proper analysis of this condition calls for the use of a flow net (Figure 5-2a). If the soil is relatively homogeneous, an approximation of the pressure distribution as illustrated in Figure 5-2a (Terzaghi, 1954) may be used. As indicated by the flow net, the passage of water under the toe of the bulkhead has an upward gradient on the dredge side of the wall. The net result of this upward flow of water is a reduction of the effective unit weight of the soil, $\Delta\gamma$. The relationship between the hydrostatic imbalance H_u and reduced unit weight are shown in Figure 5-3 and described by the relationship

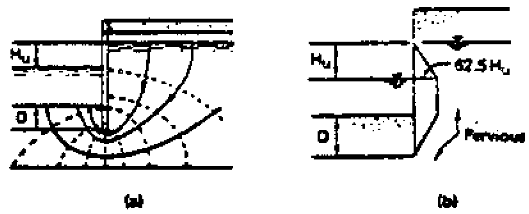


Figure 5-2. Hydrostatic and seepage stresses (Terzaghi, 1954, p. 1243)

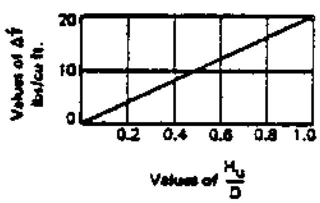


Figure 5-3. Reduction of effective unit weight (Terzaghi, 1954, p. 1243)

$$\Delta\gamma \text{ (pcf)} = 20 \frac{H_u}{D} . \tag{5-3}$$

The reduced unit weight of the soil is then used for passive stress computations.

5.1.4. Ice Thrust

Ice thrust is a phenomenon which occurs when there is ground water or capillary water above the frost line. Horizontal thrust is the result of volume expansion of ice upon temperature change. Horizontal loads due to ice thrust are often too large to be designed for and should, therefore, be eliminated by employing free draining soils for fill material (Teng, 1962).

In addition to reducing large lateral loads due to cohesive material in the backfill, sand dikes or sand blankets (Figure 5-4) can be incorporated to eliminate the potential for ice thrust and hydrostatic imbalance. A backfill consisting of clean, coarse-grained soil is highly permeable and precludes any significant capillary action in the intergranular voids.

5.1.5. Mooring and Ship Impact

Loads associated with mooring pull can be assumed to be equal to the capacity of the winch used on the boat (Teng, 1962).

Ship impact loads are usually too high to design for. As an alternative, a fendering system should be installed to minimize the amount of impact.

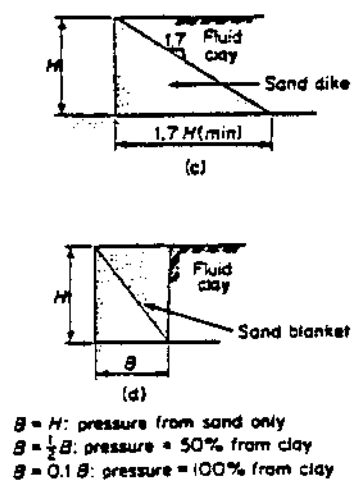


Figure 5-4. Reduction of horizontal stress in clay fills (Teng, 1962, p. 373)

5.1.6. Load Factors

Load factors are employed to provide an adequate safety margin in cases where the extent of variations in the actual loading are unknown. Such a situation occurs when tie-rods are employed.

Tie-rod loads may be higher than the values calculated for a number of reasons. Settlement of the fill, or soft soil in the subgrade, causes the tie-rods to sag. This additional elongation is accompanied by an increase in stress.

Such overstressing could be eliminated by installing the tie-rod within a PVC pipe. As the soil beneath the pipe settles, the pipe moves, but not the tie-rod (Teng, 1962).

Tie-rods may also become overstressed because of improper construction methods, i.e., placing the backfill unevenly, compacting the backfill, surcharging the backfill without first calculating the effect, or overtightening the tie-rod.

Since tie-rods are susceptible to overstressing, the loads on tie-rods should be increased by 1.2 in cases where the designer is reasonably assured of little overstressing, and by 1.4 in cases where the designer is uncertain.

Load factors need not be applied to penetration, sheet pile anchorage, wale, or splice calculations. The safety factor used in penetration calculations (Equation 3-1) accounts for any variation in direct soil stresses acting upon the wall. Although the unfactored soil parameters are used to compute bending moment in sheet piles, the values are still conservative. Additionally, allowable loads in materials are substantially lower than failure loads.

Although load factors are not applied to penetration depths, an increase in penetration must be applied to prevent failure from overdredging and scour. In such cases, the designer arbitrarily increases the penetration depth based upon local codes or the amount of scour and overdredging that the designer considers likely to occur.

5.2. Cost Effectiveness

The optimum design is that which is the most economical and performs the desired function for a specified lifetime, i.e., it is the most cost effective system. To attain this, the designer must consider the wall types, anchorage types, materials, and fastening methods available.

The discussion regarding materials is limited to steel and timber, as these comprise the majority of bulkheads. Reinforced concrete has been used for bulkheads. However, its use is often too costly for smaller walls and the complexity of the design procedure places its treatment beyond the scope of this work. Other structural materials, such as aluminum, are also available.

High strength bolts for steel walls, common bolts and nails for wood walls, and turnbuckles for tie-rods are the fasteners which will be discussed.

5.2.1. Wall Types

5.2.1.1. Anchored Wall vs. Cantilevered Wall

It may be advantageous to employ a cantilevered wall system when the standing wall height is small or when some aspect of the site precludes the installation of an anchorage. For example, the cost of utilizing an anchorage, with the required wales, tie-rods and connectors, may be higher than the cost of the increased depth of penetration required for a cantilevered wall; or, a utility line may be located which prevents employment of an anchorage. Use of the simplified design method facilitates the economic comparison between an anchored wall and a cantilevered wall in such cases.

5.2.1.2. Navy Bulkheads

A frequent sight along waterfronts is a structure commonly referred to as a navy bulkhead. It is characterized by wooden sheet pile members employed in conjunction with eight in (203 mm) diameter wooden timber piles (Figure 5-5). This structure gives the appearance of increased resistance to lateral loads when compared to smooth-faced bulkheads. The addition of the eight in (203 mm) piles does provide added strength, but the flexibility of the system is decreased and the interaction between the soil and structure is affected.

A qualitative analogy can be inferred from the discussion in Chapter 2 regarding a soldier pile and lagging system (Tsui, 1974). The soldier pile is very stiff as compared to the lagging and this is roughly analogous to the stiffness of an 8 in (203 mm) pile relative to the stiffness of the sheet piles. As shown by the finite element analysis of the discontinuous walls (Figure 2-21), the displacement of the lagging was two times that of the soldier piles for softer soils, and 1.5 times for stiffer soils. When deflections of an equivalent, continuous planar wall were computed, it was found that the displacement for the lagging was 1.6 times greater for the softer soils and 1.3

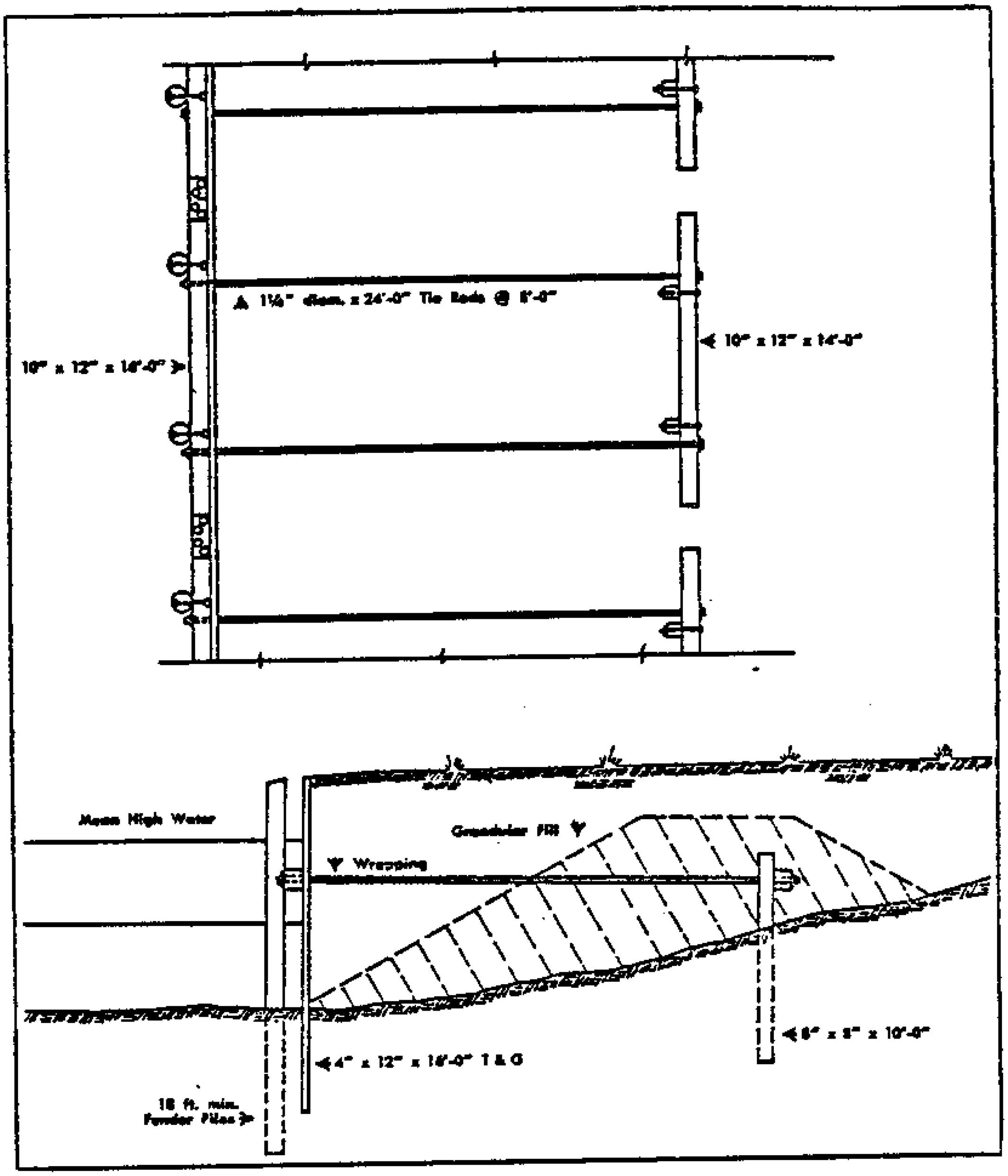


Figure 5-5. Navy bulkhead (AWPI, 1970, p. 5)

times greater for stiffer soils. One can therefore suggest that similar behavior occurs for the navy bulkheads. In other words, deflections, and therefore bending moments and bending stresses, are substantially greater at the midpoint between two piles than at the piles themselves.

As previously mentioned, the flexibility criteria for bulkhead design is determined by the flexibility number, ρ:

$$\rho = \frac{H_D^4}{EI} \qquad (2\text{-}15)$$

in which: H_D = total sheet pile length, E = elastic modulus of the members, and I = moment of inertia per unit length of wall (Rowe, 1952).

A brief investigation of varying member sizes leads to the essence of pile flexibility with respect to navy bulkheads. With total sheet pile length and the elastic modulus held constant, the governing factor determining wall flexibility is the moment of inertia. For rectangular members,

$$I = \frac{1}{12} bt^3 \qquad (5\text{-}4)$$

in which: b = member width, and t = thickness. With the addition of an 8 in (203 mm) pile, the moment of inertia is greatly increased and can be determined utilizing the parallel axis theorem:

$$I = I_1 + A_1 d_1^2 + I_2 + A_2 d_2^2 \qquad (5\text{-}5a)$$

in which: I_1 and I_2 = moment of inertia of sections 1 and 2, A_1 and A_2 = cross sectional areas of sections 1 and 2, and d_1 and d_2 = distance from the neutral axis to the centroids of sections 1 and 2. For the navy bulkhead (Figure 5-6):

$$I_1 = \frac{\pi}{4} r^4 = 201 \text{ in}^4 \ (8.37 \times 10^7 \text{ mm}^4) \quad (5\text{-}5b)$$

$$I_2 = \frac{1}{12} \ell \, t_s^3, \quad (5\text{-}5c)$$

$$A_1 = \pi r^2 = 50.3 \text{ in}^2 \ (3.24 \times 10^4 \text{ mm}^2), \quad (5\text{-}5d)$$

$$A_2 = \ell \, t_s, \quad (5\text{-}5e)$$

$$d_1 = c - \frac{1}{2} t_s, \text{ and} \quad (5\text{-}5f)$$

$$d_2 = t_s + t_w + 4 - c \text{ (in)} = t_s + t_w + 10.2 - c \text{ (mm)}, \quad (5\text{-}5g)$$

in which ℓ = length of wall under consideration, c = distance to the neutral axis, t_s = thickness of sheet pile, and t_w = thickness of the wale. Moments of inertia and planar, equivalent moments of inertia were computed and are given in Table 5-1 for varying combinations of sheet pile thickness, wale thickness and lengths of wall. It is obvious that the presence of the 8 in (203 mm) pile adds considerably to the stiffness of the system, even when a planar equivalent is computed with a distance of 7 ft (213 m) between 8 in (203 mm) piles.

The effect of the increased stiffness, or decreased flexibility, on bulkhead design can be appreciated when selecting sheet pile thickness. The values of critical pile flexibility, ρ_c, defined as the minimum flexibility to permit moment reductions based upon Free Earth Support computations are

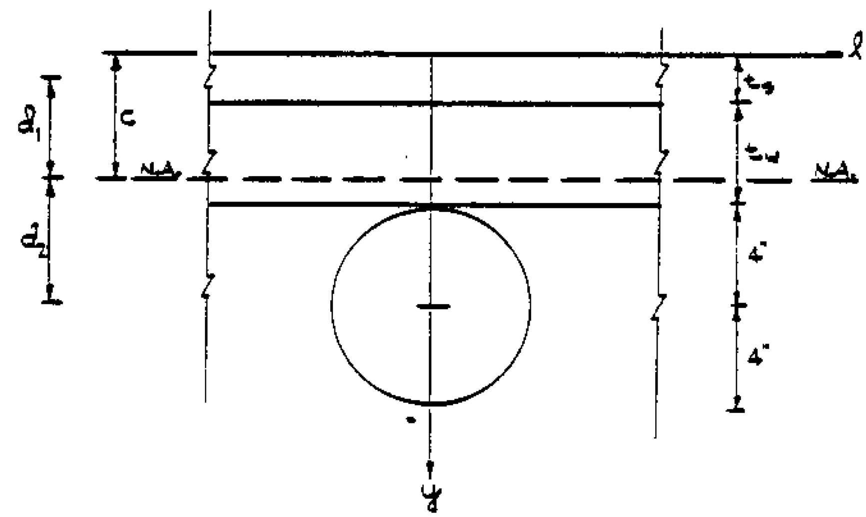


Figure 5-6. Dimensions of navy bulkhead

Table 5-1. Effect of 8 inch piles on flexibility

ℓ (inches)	t_w (inches)	t_s (inches)	c (inches)	I_4 (in^4)	I (in^4/ft)
12	8	2	9.79	2954	2954
		3	9.36	4051	4050
		4	9.16	5077	5080
	10	2	11.15	3864	3860
		3	10.53	5270	5270
		4	10.18	6550	6550
	12	2	12.51	4903	4900
		3	11.70	6652	6650
24	8	2	7.65	4367	2180
		3	7.05	5650	2830
		4	6.81	6795	3400
	10	2	8.67	5742	2870
		3	7.87	7367	3680
		4	7.50	8775	4390
	12	2	9.70	7310	3660
		3	8.69	8040	4020
		4	8.19	11,020	5510
48	8	2	5.46	5809	1450
		3	4.99	7100	1180
		4	4.90	8265	2070
	10	2	6.15	7656	1910
		3	5.51	9260	2320
		4	5.32	10,655	2660
	12	2	6.80	9235	2310
		3	6.03	9320	2330
		4	5.73	13,360	3340
84	8	2	3.99	6796	970
		3	.374	8028	1150
		4	3.82	9219	1320
	10	2	4.45	8962	1280
		3	4.08	10,460	1490
		4	4.08	11,840	1690
	12	2	4.92	11,440	1630
		3	4.41	12,060	1720
		4	4.34	15,560	2220
No piles	—	2	1.0	—	8
	—	3	1.5	—	27
	—	4	2.0	—	64

Note: 1 in = 25.4 mm
1 ft = 0.305 m

$1\ in^4 = 4.16 \times 10^{-7}\ m^4$

$\frac{1\ in^4}{ft} = 1.37 \times 10^{-6}\ \frac{m^4}{m}$

$\log \rho = -4.00$ for dense sand, and

$\log \rho = -3.50$ for loose sand (Rowe, 1952).

These values correspond to pile lengths:

$H_D = 19.2$ ft (5.85 m) for dense sand, and

$H_D = 25.6$ ft (7.80 m) for loose sand, for

$$I = 970 \frac{in^4}{ft} \left(1.33 \frac{m^4}{m}\right),$$

the moment of inertia per unit length of an equivalent, planar wall, with

$t_s = 2$ in (50.8 mm) and

$t_w = 8$ in (203 mm).

It can therefore be concluded that moment reduction should not be allowed for navy bulkheads of moderate height. It should also be noted that the planar equivalent should not be used for selecting sheet pile thickness because bending stresses can be considerably higher at the midpoint between 8 in (203 mm) piles than stresses computed for the planar equivalent.

Although the analogy between the soldier pile and lagging wall and the navy bulkhead is incomplete, it does suggest that a conservative approach be used in designing navy bulkheads. The consequence of this conservatism results in thicker sheet pile members and, therefore, higher costs. The convenience of a built-in fendering system may not be warranted because of this increased expense. However, large impact loads caused by large ships or breaking waves may necessitate the added cost of navy bulkheads.

5.2.2. Anchorage Type and Location

The anchorage may be deadmen, braced piles, sheet piles, or the footings of large structures (Figure 5-7). The passive stress developed in front of the anchorage determines the capacity of deadmen and sheet piles. Foundation footings derive their capacity to resist horizontal movement from the passive stress developed and from the friction developed along the bottom of the footing. Determination of pile capacity is beyond the scope of this work. Methods for computing pile capacity are given by Cheung and Kulhawy (1981).

The anchorage must be located so that it is not within the active failure wedge of the wall, which is defined by line segment $\overline{ab}$ in Figure 5-8. Since the anchorage develops passive stresses, the passive wedge of the anchorage must not intersect the active wedge of the wall. Line segment $\overline{be}$ represents the closest proximity of the wedges. The safe zone for anchorage location is outlined by segments $\overline{ed}$ and $\overline{dc}$,

Figure 5-8 represents the anchorage location for a sheet pile length, H_D, of 17.5 ft (5.33 m) and angle of internal friction, ϕ, of 32 degrees, the geometry and soil parameter for example #1. Point "a" marks the pile toe, and point "e" marks the intersection of line segment $\overline{ae}$, inclined at an angle equal to ϕ from the horizontal, with the surface of the fill.

The capacity of a continuous deadman or sheet pile anchorage (force per unit length of anchorage), is given by

$$P_{ULT} = P_p - P_a \tag{5-6}$$

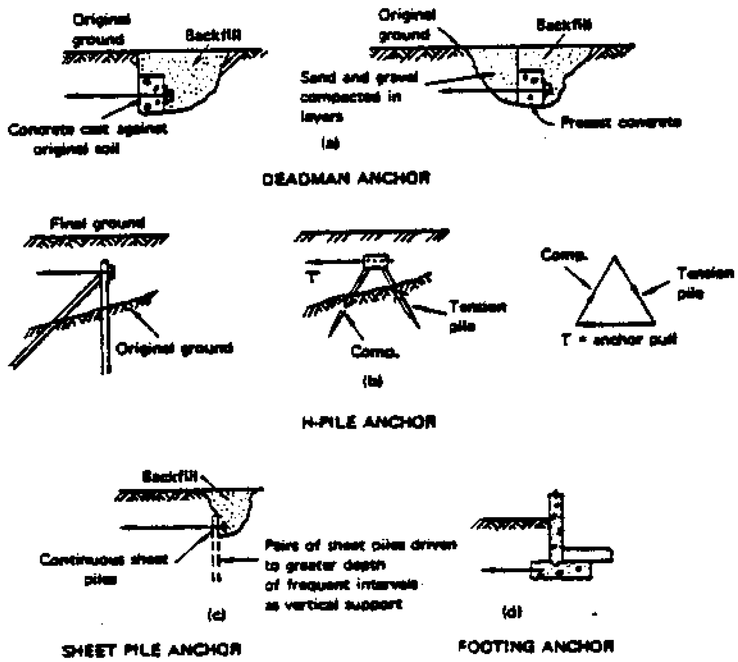


Figure 5-7. Types of anchorage (Teng, 1962, p. 374)

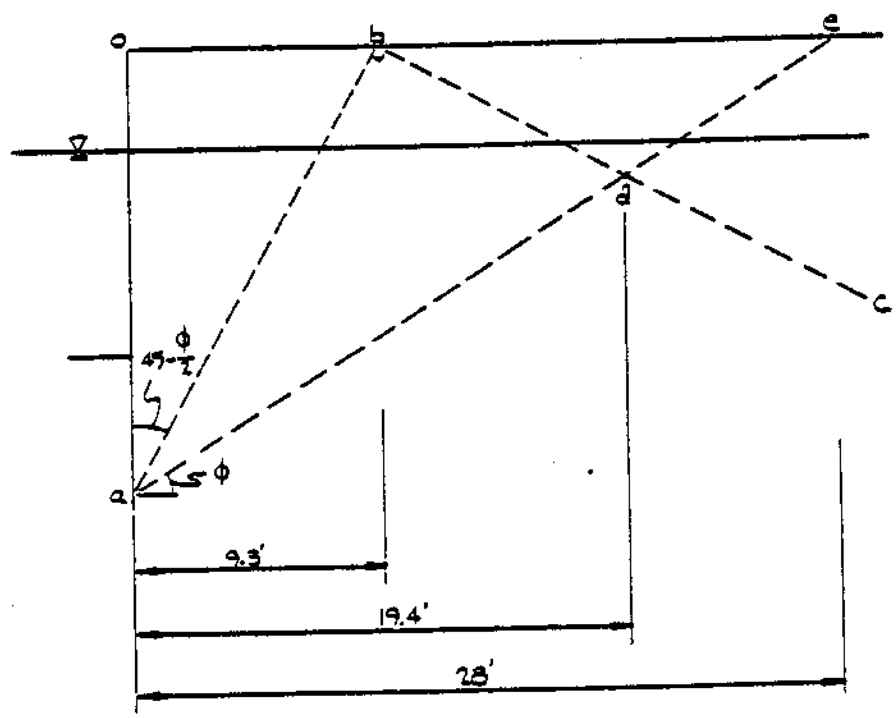


Figure 5-8. Location of the anchorage

in which P_p = passive stress resultant and P_a = active stress resultant (Figure 5-9a).

Short deadmen located near the ground surface provide added capacity because of end friction (Figure 5-9b). The capacity of short deadmen is given by (Teng, 1962)

$$T_{ULT} = L(P_p - P_a) + \frac{1}{3} K_o \gamma (\sqrt{K_p} + \sqrt{K_a}) h_L^3 \tan \phi \qquad (5\text{-}7)$$

in which L = the deadmen length, K_o = the at rest soil stress coefficient and may be taken as 0.40 (Teng, 1962), and h_L = the height of the deadman. For cohesive soils, the relationship is

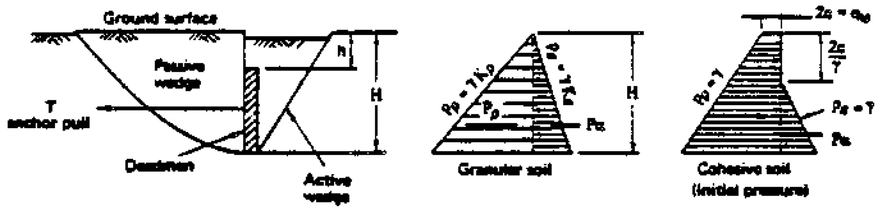
$$T_{ULT} = L(P_p - P_a) + 2c\, h_L^2 \qquad (5\text{-}8)$$

in which c = the soil cohesion.

5.2.3. Material Strength

Material strength affects the cost of components in two ways, i.e., higher strength materials are generally more expensive, and the strength of the material is a determinant of the component dimensions. Since the unit cost of materials is subject to wide fluctuation, the discussion of material strength will be confined to its influence on component dimensions.

Most of the structural components are flexural members, i.e., they must resist bending stresses. The dimensioning of the member is in terms of the section modulus, S, and is determined by the bending moment, M, and allowable bending stress of the material, f_b, such that:



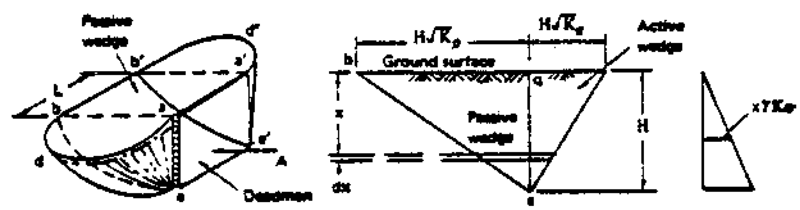


Figure 5-9. Capacity of deadmen (Teng, 1962, p. 376)

$$S = \frac{M}{f_b} \tag{5-9}$$

Since most of the timber components are rectangular, the dimensions may be selected using the relationship

$$S = \frac{1}{6}\, b\, h^2, \text{ or} \tag{5-10a}$$

$$S = \frac{1}{6}\, b^2\, h \tag{5-10b}$$

depending upon the direction of the bending. Equation 5-10a is used for bending about the major axis, and Equation 5-10b is used with respect to the minor axis, as shown in Figure 5-10.

The section moduli for structural steel members can be found in Table 5-2 for sheet pile sections and Table 5-3 for channel sections.

Member dimensions are determined from section moduli which are, in turn, directly proportional to the bending moment, M, and inversely proportional to the allowable bending stress, f_b. Hence, the cost of the member is related to its strength in terms of its allowable stress.

Table 5-4a contains a partial list of allowable stresses for southern pine, the wood type most commonly used in New York. A more exhaustive list may be found in Timber Design and Construction Manual by the Timber Engineering Co. Columns 3 through 7 of Table 5-4a indicate the allowable: Bending stress (f), tensile stress (t), shear stress (H), compressive stress perpendicular to the grain ($c\perp$) and parallel to the grain (c), and the elastic modulus (E). The shear stress is given by

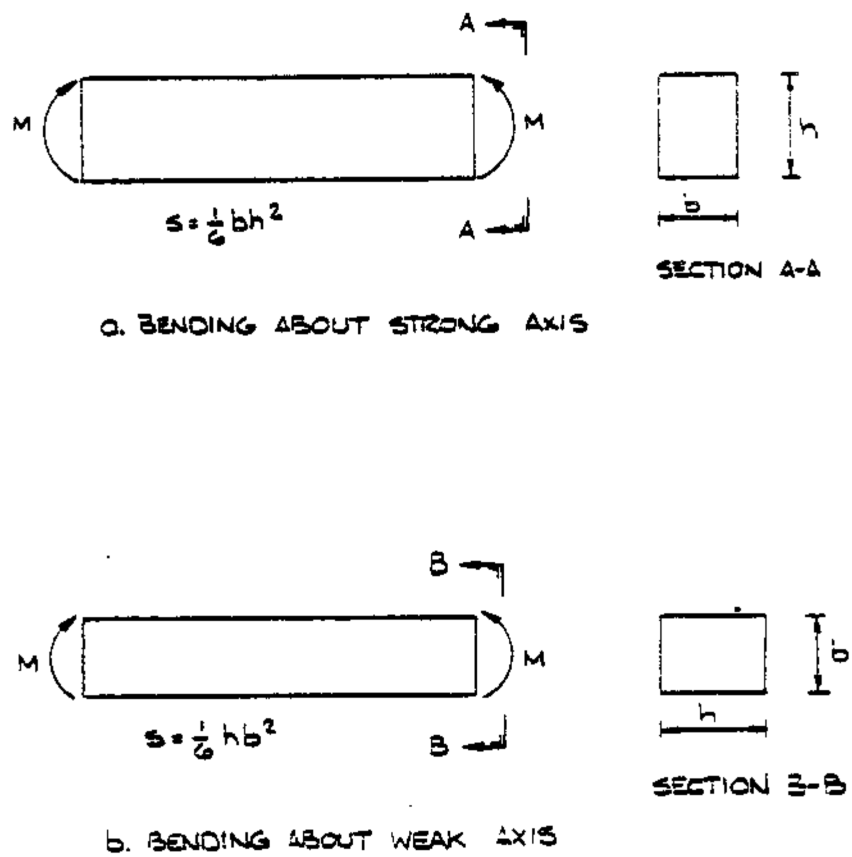


Figure 5-10. Section modulus of rectangular members

Table 5-2. Engineering properties of steel sheet piling (United States Steel, 1979, facing p. 1)

USS Steel Sheet Piling

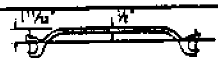
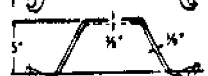
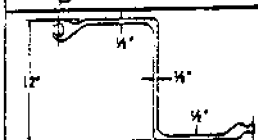
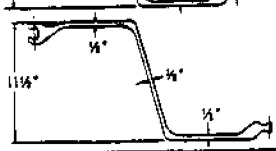
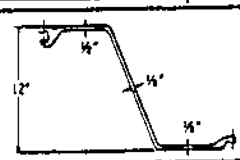
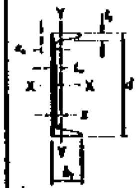
Profile	Designation		District Rolled	Driving Distance per Pile	Weight: Per Foot	Weight: Per Square Foot of Wall	Web Thickness	Section Modulus: Per Pile	Section Modulus: Per Foot of Wall	Area: Per Pile	Moment of Inertia: Per Pile	Moment of Inertia: Per Foot of Wall
				In.	Lbs.	Lbs.	In.	In.³	In.³	In.²	In.⁴	In.⁴
	Interlock with Each Other	PSX 32	H.	16½	44.0	32.0	29/64	3.3	2.4	12.94	5.1	3.7
		PS 32*	H.S.	15	40.0	32.0	½	2.4	1.9	11.77	3.6	2.9
		PS 28	H.S.	15	35.0	28.0	⅜	2.4	1.9	10.30	3.5	2.8
	Interlock with Each Other	PSA 28*	H.	16	37.3	28.0	½	3.3	2.5	10.98	6.0	4.5
		PSA 23	H.S.	16	30.7	23.0	⅜	3.2	2.4	8.99	5.5	4.1
		PDA 27	H.S.	16	36.0	27.0	⅜	14.3	10.7	10.59	53.0	39.8
		PMA 22	H.S.	19⅝	36.0	22.0	⅜	8.8	5.4	10.59	22.4	13.7
	Interlock with Each Other and with PSA23 or PSA28	PZ38	H.	18	57.0	38.0	⅜	70.2	46.8	16.77	421.2	280.8
		PZ32	H.	21	56.0	32.0	⅜	67.0	38.3	16.47	385.7	220.4
	Interlocks with Itself and with PSA23 or PSA28	PZ27	H.	18	40.5	27.0	⅜	45.3	30.2	11.91	276.3	184.2

Table 5-3. Channels (AISC, 1980, pp. 1-36, 1-37)

CHANNELS AMERICAN STANDARD
Properties for designing



Designation	Area A	Depth d	Flange Width b_f	Flange Average thickness t_f	Web thickness t_w	$\frac{d}{A_f}$	Axis X-X I	Axis X-X S	Axis X-X r	Nominal Weight per Ft.	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	$\bar{x}$	Shear Center Location E_o	Torsional Constant J	Warping Constant C_w
	In.^{2}	In.	In.	In.	In.		In.^{4}	In.^{3}	In.		In.^{4}	In.^{3}	In.	In.	In.	In.^{4}	In.^{6}
C 15×50	14.7	15.00	3.716	0.650	0.716	6.21	404	53.8	5.24	50	11.0	3.78	0.867	0.799	0.941	2.66	492
×40	11.8	15.00	3.520	0.650	0.520	6.56	349	46.5	5.44	40	9.23	3.36	0.886	0.778	1.03	1.46	410
×33.9	9.96	15.00	3.400	0.650	0.400	6.79	315	42.0	5.62	33.9	8.13	3.11	0.904	0.787	1.10	1.01	358
C 12×30	8.82	12.00	3.170	0.501	0.510	7.55	162	27.0	4.29	30	5.14	2.06	0.763	0.674	0.871	0.865	151
×25	7.35	12.00	3.047	0.501	0.387	7.85	144	24.1	4.43	25	4.47	1.88	0.780	0.674	0.940	0.541	131
×20.7	6.09	12.00	2.942	0.501	0.282	8.13	129	21.5	4.61	20.7	3.88	1.73	0.799	0.698	1.01	0.371	112
C 10×30	8.82	10.00	3.033	0.436	0.673	7.55	103	20.7	3.42	30	3.94	1.65	0.669	0.649	0.705	1.22	79.5
×25	7.35	10.00	2.886	0.436	0.526	7.94	91.2	18.2	3.52	25	3.36	1.48	0.676	0.617	0.757	0.690	68.4
×20	5.88	10.00	2.739	0.436	0.379	8.36	78.9	15.8	3.66	20	2.81	1.32	0.691	0.606	0.826	0.370	57.8
×15.3	4.49	10.00	2.600	0.436	0.240	8.81	67.4	13.5	3.87	15.3	2.28	1.16	0.713	0.634	0.916	0.211	45.5
C 9×20	5.88	9.00	2.648	0.413	0.448	8.22	60.9	13.5	3.22	20	2.42	1.17	0.642	0.583	0.739	0.427	39.5
×15	4.41	9.00	2.485	0.413	0.285	8.76	51.0	11.3	3.40	15	1.93	1.01	0.661	0.586	0.824	0.209	31.0
×13.4	3.94	9.00	2.433	0.413	0.233	8.95	47.9	10.6	3.48	13.4	1.76	0.962	0.668	0.601	0.859	0.169	28.2
C 8×18.75	5.51	8.00	2.527	0.390	0.487	8.12	44.0	11.0	2.82	18.75	1.98	1.01	0.599	0.565	0.674	0.436	25.1
×13.75	4.04	8.00	2.343	0.390	0.303	8.75	36.1	9.03	2.99	13.75	1.53	0.853	0.615	0.553	0.756	0.187	19.3
×11.5	3.38	8.00	2.260	0.390	0.220	9.08	32.6	8.14	3.11	11.5	1.32	0.781	0.625	0.571	0.807	0.131	16.5
C 7×14.75	4.33	7.00	2.299	0.366	0.419	8.31	27.2	7.78	2.51	14.75	1.38	0.779	0.564	0.532	0.651	0.268	13.1
×12.25	3.60	7.00	2.194	0.366	0.314	8.71	24.2	6.93	2.60	12.25	1.17	0.702	0.571	0.525	0.695	0.161	11.2
× 9.8	2.87	7.00	2.090	0.366	0.210	9.14	21.3	6.08	2.72	9.8	0.968	0.625	0.581	0.541	0.752	0.100	9.16
C 6×13	3.83	6.00	2.157	0.343	0.437	8.10	17.4	5.80	2.13	13	1.05	0.642	0.525	0.514	0.599	0.241	7.21
×10.5	3.09	6.00	2.034	0.343	0.314	8.59	15.2	5.06	2.22	10.5	0.865	0.564	0.529	0.500	0.643	0.131	5.94
× 8.2	2.40	6.00	1.920	0.343	0.200	9.10	13.1	4.38	2.34	8.2	0.692	0.492	0.537	0.512	0.699	0.075	4.73
C 5×9	2.64	5.00	1.885	0.320	0.325	8.29	8.90	3.56	1.83	9	0.632	0.449	0.489	0.478	0.590	0.109	2.93
× 6.7	1.97	5.00	1.750	0.320	0.190	8.93	7.49	3.00	1.95	6.7	0.478	0.378	0.493	0.484	0.647	0.055	2.22
C 4×7.25	2.13	4.00	1.721	0.296	0.321	7.84	4.59	2.29	1.47	7.25	0.432	0.343	0.450	0.459	0.546	0.082	1.24
× 5.4	1.59	4.00	1.584	0.296	0.184	8.52	3.85	1.93	1.56	5.4	0.319	0.283	0.449	0.458	0.594	0.040	0.923
C 3×6	1.76	3.00	1.596	0.273	0.356	6.87	2.07	1.38	1.08	6	0.305	0.268	0.416	0.455	0.500	0.073	0.463
× 5	1.47	3.00	1.498	0.273	0.258	7.32	1.85	1.24	1.12	5	0.247	0.233	0.410	0.438	0.521	0.043	0.380
× 4.1	1.21	3.00	1.410	0.273	0.170	7.78	1.66	1.10	1.17	4.1	0.197	0.202	0.404	0.437	0.546	0.027	0.307

Table 5-4a. Allowable stresses for southern pine (Timber Engineering Co., 1956, p. 483)

1 Species and commercial grade		2 Graded by	Allowable unit stresses, psi 3 f	4 H	5 $c_{\perp}$	6 c	7 E
Pine, southern							
DS 86 KD	2 in. thick only	SPIB	3,000	165	455	2,250	1,760,000
DS 72 KD	"		2,500	150	455	1,950	1,760,000
DS 65 KD	"		2,250	135	455	1,800	1,760,000
DS 58 KD	"		2,050	120	455	1,650	1,760,000
No. 1 Dense KD	"		2,050	135	455	1,750	1,760,000
No. 1 KD	"		1,750	135	390	1,500	1,760,000
No. 2 Dense KD	"		1,750	120	455	1,300	1,760,000
No. 2 KD	"		1,500	120	390	1,100	1,760,000
DS 86	2 in. thick only		2,900	150	455	2,200	1,760,000
DS 72	"		2,350	135	455	1,800	1,760,000
DS 65	"		2,050	120	455	1,600	1,760,000
DS 58	"		1,750	105	455	1,450	1,760,000
No. 1 Dense	"		1,750	120	455	1,550	1,760,000
No. 1	"		1,500	120	390	1,350	1,760,000
No. 2 Dense	"		1,400	105	455	1,050	1,760,000
No. 2	"		1,200	105	390	900	1,760,000
DS 86	3 in. & 4 in.	SPIB	2,900	150	455	2,200	1,760,000
DS 72	" thick		2,350	135	455	1,800	1,760,000
DS 65	"		2,050	120	455	1,600	1,760,000
DS 58	"		1,750	105	455	1,450	1,760,000
No. 1 Dense SR	"		1,750	120	455	1,750	1,760,000
No. 1 SR	"		1,500	120	390	1,500	1,760,000
No. 2 Dense SR	"		1,400	105	455	1,050	1,760,000
No. 2 SR	"		1,200	105	390	900	1,760,000
DS 86	5 or more in.		[14,2] 2,400	150	455	1,800	1,760,000
DS 72	" thick		[14,2] 2,000	135	455	1,550	1,760,000
DS 65	"		[14,1] 1,800	120	455	1,400	1,760,000
DS 58	"		[14,1] 1,600	105	455	1,300	1,760,000
No. 1 Dense SR	"		[14,1] 1,600	120	455	1,500	1,760,000
No. 1 SR	"		[14,1] 1,400	120	390	1,300	1,760,000
No. 2 Dense SR	"		[14,1] 1,400	105	455	1,050	1,760,000
No. 2 SR	"		[14,1] 1,200	105	390	900	1,760,000
IND 86 KD	1 in., 1¼ in.	SPIB	2,600	165	390	1,950	1,760,000
IND 72 KD	and 1½ in. thick		2,200	150	390	1,650	1,760,000
IND 65 KD	"		2,000	135	390	1,550	1,760,000
IND 58 KD	"		1,750	120	390	1,400	1,760,000
IND 50 KD	"		1,500	120	390	1,100	1,760,000
IND 86	"		2,500	150	390	1,900	1,760,000
IND 72	"		2,000	135	390	1,550	1,760,000
IND 65	"		1,750	120	390	1,350	1,760,000
IND 58	"		1,500	105	390	1,250	1,760,000
IND 50	"		1,200	105	390	900	1,760,000

$$H = \frac{3V}{2\ bh} \tag{5-11}$$

in which V = the total shear force.

Table 5-4b contains dimensions and properties for lumber.

The allowable bending stress in steel members is a function of its minimum yield point, f_y. For steel sheet piles, ASTM A328, A572, and A690 (United States Steel, 1975),

$$f_b = 0.65\ f_y \tag{5-12a}$$

For A36 steel, which is commonly used for channels, tie-rods, and plates, (AISC, 1973),

$$f_b = 0.60\ f_y, \tag{5-12b}$$

the allowable tensile stress, f_t, is evaluated the same as for bending stress, i.e.,

$$f_t = 0.60\ f_y, \tag{5-12c}$$

and the allowable shear stress may be taken as (AISC, 1973)

$$f_v = 0.40\ f_y. \tag{5-12d}$$

Table 5-5 reflects the minimum yield point for various ASTM steel specifications.

5.2.4. Fasteners

Timber components may be fastened by nails or common bolts. High strength bolts (ASTM A325) are used for steel.

Table 5-4b. Dimensional properties of lumber (Timber Engineering Co., 1956, pp. 362-363)

Nominal size in. $b \times h$	American Standard dressed size (S4S) in. $b \times h$	Area of section sq in. $A = bh$	Moment of inertia $I_{x-x} = bh^3/12$	Moment of inertia $I_{y-y} = b^3h/12$	Section modulus $S_{x-x} = bh^2/6$	Section modulus $S_{y-y} = b^2h/6$
1 × 4	$\frac{25}{32} \times 3\frac{5}{8}$	2.83	3.10	0.14	1.71	0.37
1 × 6	$\frac{25}{32} \times 5\frac{5}{8}$	4.39	11.59	0.22	4.12	0.57
1 × 8	$\frac{25}{32} \times 7\frac{1}{2}$	5.86	27.47	0.30	7.32	0.76
1 × 10	$\frac{25}{32} \times 9\frac{1}{2}$	7.42	55.82	0.38	11.75	0.97
1 × 12	$\frac{25}{32} \times 11\frac{1}{2}$	8.98	99.02	0.46	17.22	1.17
2 × 2	$1\frac{5}{8} \times 1\frac{5}{8}$	2.64	0.58	0.58	0.72	0.72
2 × 4	$1\frac{5}{8} \times 3\frac{5}{8}$	5.89	6.45	1.30	3.56	1.60
2 × 6	$1\frac{5}{8} \times 5\frac{5}{8}$	9.14	24.10	2.01	8.57	2.48
2 × 8	$1\frac{5}{8} \times 7\frac{1}{2}$	12.19	57.13	2.68	15.23	3.30
2 × 10	$1\frac{5}{8} \times 9\frac{1}{2}$	15.44	116.10	3.40	24.44	4.18
2 × 12	$1\frac{5}{8} \times 11\frac{1}{2}$	18.69	205.95	4.11	35.82	5.06
2 × 14	$1\frac{5}{8} \times 13\frac{1}{2}$	21.94	333.18	4.83	49.36	5.94
3 × 4	$2\frac{5}{8} \times 3\frac{5}{8}$	9.52	10.42	5.46	5.75	4.16
3 × 6	$2\frac{5}{8} \times 5\frac{5}{8}$	14.77	38.93	8.48	13.84	6.46
3 × 8	$2\frac{5}{8} \times 7\frac{1}{2}$	19.69	92.29	11.30	24.61	8.61
3 × 10	$2\frac{5}{8} \times 9\frac{1}{2}$	24.94	187.55	14.32	39.48	10.91
3 × 12	$2\frac{5}{8} \times 11\frac{1}{2}$	30.19	332.69	17.33	57.86	13.21
3 × 14	$2\frac{5}{8} \times 13\frac{1}{2}$	35.44	538.21	20.35	79.73	15.50
3 × 16	$2\frac{5}{8} \times 15\frac{1}{2}$	40.69	814.60	23.36	105.11	17.80
4 × 4	$3\frac{5}{8} \times 3\frac{5}{8}$	13.14	14.39	14.39	7.94	7.94
4 × 6	$3\frac{5}{8} \times 5\frac{5}{8}$	20.39	53.76	22.33	19.12	12.32
4 × 8	$3\frac{5}{8} \times 7\frac{1}{2}$	27.19	127.44	29.77	33.98	16.43
4 × 10	$3\frac{5}{8} \times 9\frac{1}{2}$	34.44	259.00	37.71	54.53	20.81
4 × 12	$3\frac{5}{8} \times 11\frac{1}{2}$	41.69	459.43	45.65	79.90	25.19
4 × 14	$3\frac{5}{8} \times 13\frac{1}{2}$	48.94	743.24	53.59	110.11	29.57
4 × 16	$3\frac{5}{8} \times 15\frac{1}{2}$	56.19	1,124.92	61.53	145.15	33.95
6 × 6	$5\frac{1}{2} \times 5\frac{1}{2}$	30.25	76.26	76.24	27.73	27.73
6 × 8	$5\frac{1}{2} \times 7\frac{1}{2}$	41.25	193.36	103.98	51.56	37.81
6 × 10	$5\frac{1}{2} \times 9\frac{1}{2}$	52.25	392.96	131.71	82.73	47.90
6 × 12	$5\frac{1}{2} \times 11\frac{1}{2}$	63.25	697.07	159.44	121.23	57.98
6 × 14	$5\frac{1}{2} \times 13\frac{1}{2}$	74.25	1,127.67	187.17	167.06	68.06
6 × 16	$5\frac{1}{2} \times 15\frac{1}{2}$	85.25	1,706.78	214.90	220.23	78.15
6 × 18	$5\frac{1}{2} \times 17\frac{1}{2}$	96.25	2,456.38	242.63	280.73	88.23
8 × 8	$7\frac{1}{2} \times 7\frac{1}{2}$	56.25	263.67	263.67	70.31	70.31
8 × 10	$7\frac{1}{2} \times 9\frac{1}{2}$	71.25	535.86	333.98	112.81	89.06
8 × 12	$7\frac{1}{2} \times 11\frac{1}{2}$	86.25	950.55	404.30	165.31	107.81
8 × 14	$7\frac{1}{2} \times 13\frac{1}{2}$	101.25	1,537.73	474.61	227.81	126.56
8 × 16	$7\frac{1}{2} \times 15\frac{1}{2}$	116.25	2,327.42	544.92	300.31	145.31
8 × 18	$7\frac{1}{2} \times 17\frac{1}{2}$	131.25	3,349.61	615.23	382.81	164.06
8 × 20	$7\frac{1}{2} \times 19\frac{1}{2}$	146.25	4,634.30	685.55	475.31	182.81
10 × 10	$9\frac{1}{2} \times 9\frac{1}{2}$	90.25	678.76	678.76	142.90	142.90
10 × 12	$9\frac{1}{2} \times 11\frac{1}{2}$	109.25	1,204.03	821.56	209.40	172.98

Table 5-4b. Continued

Nominal size in. $b \times h$	American Standard dressed size (S4S) in. $b \times h$	Area of section sq in. $A = bh$	Moment of inertia $I_{x-x} = bh^3/12$	$I_{y-y} = b^3h/12$	Section modulus $S_{x-x} = bh^2/6$	$S_{y-y} = b^2h/6$
10 × 14	9½ × 13½	128.25	1,947.80	964.55	288.56	203.06
10 × 16	9½ × 15½	147.25	2,948.07	1,107.44	380.40	233.15
10 × 18	9½ × 17½	166.25	4,242.84	1,250.34	484.90	263.23
10 × 20	9½ × 19½	185.25	5,870.11	1,393.23	602.06	293.31
10 × 24	9½ × 23½	223.25	10,274.15	1,679.03	874.40	353.48
12 × 12	11½ × 11½	132.25	1,457.51	1,457.51	253.48	253.48
12 × 14	11½ × 13½	155.25	2,357.86	1,710.98	349.31	297.56
12 × 16	11½ × 15½	178.25	3,568.71	1,964.46	460.48	341.65
12 × 18	11½ × 17½	201.25	5,136.07	2,217.94	586.98	385.73
12 × 20	11½ × 19½	224.25	7,105.92	2,471.42	728.81	429.81
12 × 22	11½ × 21½	247.25	9,524.28	2,724.90	885.98	473.90
12 × 24	11½ × 23½	270.25	12,437.13	2,978.38	1,058.48	517.98
14 × 14	13½ × 13½	182.25	2,767.92	2,767.92	410.06	410.06
14 × 16	13½ × 15½	209.25	4,189.36	3,177.98	540.56	470.81
14 × 18	13½ × 17½	236.25	6,029.30	3,588.05	689.06	531.56
14 × 20	13½ × 19½	263.25	8,341.73	3,998.11	855.56	592.31
14 × 24	13½ × 23½	317.25	14,600.11	4,818.23	1,242.56	713.81

Table 5-5. Minimum yield point

Steel Brand or Grade	f_y
A328	38,500 psi (265 MN/m^2)
A592 Gr 50	50,000 psi (344 MN/m^2)
A640	50,000 psi (344 MN/m^2)
A36	36,000 psi (248 MN/m^2)

The capacity of a nail as a fastener is determined by its resistance to withdrawal, W_r, which is in turn a function of the effective length of embedment, ℓ_e, allowable load in withdrawal per inch of embedment, p, and specific gravity, G. The effective length of a nail fastening a sheet pile to a wale is the length of embedment in the wale, i.e. the nail length minus the thickness of the sheet pile.

To find the allowable load in withdrawal of a particular nail size, the specific gravity, G_s, of the wood is first found by using Table 5-6, then entering Table 5-7 for the desired nail size and specific gravity. The resistance to withdrawal is given by

$$W_r = p\ell_e \tag{5-13}$$

The allowable lateral loads on nails should be checked. Nails fastening southern pine and douglas fir are allowed a maximum shear of

$$V = 1650\ D^{3/2} \tag{5-14}$$

in which V = the allowable shear in pounds and D is the nail diameter in inches (Timber Engineering Co., 1956).

Common bolts may be used in wood splices and their allowable loads may be found in Table 5-8. Allowable loads are for bolts in double shear, i.e., bolts used in 3 member joints, as in splice plates for wales (Figure 5-11a). The controlling factors in Figures 5-10 and 5-11 are the bolt diameter, d, the length of bolt in the main member, b, and the relative size of the splice members and the main member. The

Table 5-6. Specific gravity of wood members (Timber Engineering Co., 1956, p. 553)

Species of wood	Specific Gravity (G)	Species of wood	Specific Gravity (G)
Alder, red	0.43	Hackberry	0.56
Ash, black	.53	Hemlock, eastern	.43
Ash, Oregon	.58	Hemlock, west coast	.44
Ash, white	.64	Hickory	.80
Aspen	.41	Larch, western	.59
Basswood	.40	Locust, black	.71
Beech	.67	Locust, honey	.67
Birch, paper	.60	Madrone	.69
Birch, sweet	.71	Magnolia, cucumber	.52
Birch, yellow	.68	Magnolia, evergreen	.53
Buckeye, yellow	.35	Maple, big leaf	.51
Butternut	.40	Maple, hard (black)	.62
Cedar, Alaska	.46	Maple, hard (sugar)	.68
Cedar, eastern red	.49	Maple, soft (red)	.55
Cedar, incense	.37	Maple, soft (silver)	.51
Cedar, northern white	.32	Oak, commercial red	.66
Cedar, Port Orford	.44	Oak, commercial white	.71
Cedar, southern white	.35	Pecan	.69
Cedar, western red	.34	Pine, lodgepole	.43
Cherry, black	.53	Pine, northern white	.37
Chestnut	.45	Pine, Norway	.47
Cottonwood, black	.37	Pine, ponderosa	.42
Cottonwood, eastern	.43	Pine, southern	.59
Cypress, southern	.48	Pine, southern longleaf	.64
Douglas fir, Coast Region	.51	Pine, sugar	.38
Douglas fir	.51	Pine, western white	.42
Douglas fir, Coast Region, dense	.54	Poplar, yellow	.43
Douglas fir, dense	.54	Redwood	.42
Elm, American	.55	Spruce, Engelmann	.36
Elm, rock	.66	Spruce, red	.41
Elm, slippery	.57	Spruce, sitka	.42
Fir, balsam	.41	Spruce, white	.45
Fir, commercial white	.42	Sugarberry	.54
Gum, black	.55	Sycamore	.54
Gum, red	.53	Tamarack	.58
Gum, tupelo	.52	Walnut, black	.56
		Willow, black	.42

Table 5-7. Allowable load in withdrawal (Timber Engineering Co., 1956, pp. 558-559)

When specific gravity (G) (see Appendix G) of wood is	SIZE OF NAIL										
	d=	6	8	10	12	16	20	30	40	50	60
	l=	2	2½	3	3¼	3½	4	4½	5	5½	6
	D=	0.113	0.131	0.148	0.148	0.162	0.192	0.207	0.225	0.244	0.263
0.31	p=	8	10	11	11	12	14	15	16	18	19
.32		9	10	12	12	13	15	16	18	20	21
.33		10	11	13	13	14	17	18	20	21	23
.34		10	12	14	14	15	18	19	21	23	24
.35		11	13	15	15	16	19	21	23	25	26
.36		12	14	16	16	17	21	22	24	26	28
.37		13	15	17	17	19	22	24	26	28	30
.38		14	16	18	18	20	24	25	28	30	32
.39		15	17	19	19	21	25	27	30	32	34
.40		16	18	21	21	23	27	29	31	34	37
.41		17	20	22	22	24	29	31	33	36	39
.42		18	21	23	23	25	30	33	35	38	41
.43		19	22	25	25	27	32	35	38	41	44
.44		20	23	26	26	29	34	37	40	43	46
.45		21	25	28	28	30	36	39	42	46	49
.46		22	26	29	29	32	38	41	45	48	52
.47		24	27	31	31	34	40	43	47	51	55
.48		25	29	33	33	36	42	46	50	54	58
.49		26	30	34	34	38	45	48	52	57	61
.50		28	32	36	36	40	47	51	55	60	64
.51		29	34	38	38	42	49	53	58	63	68
.52		31	35	40	40	44	52	56	61	66	71
.53		32	37	42	42	46	54	59	64	69	74
.54		33	39	44	44	48	57	61	67	72	78
.55		35	41	46	46	50	60	64	70	76	82
.56		37	43	48	48	53	62	67	73	79	85
.57		38	44	50	50	55	65	70	76	82	89
.58		40	46	52	52	57	68	73	79	86	93
.59		42	48	55	55	60	71	76	83	90	97
.60		44	50	57	57	62	74	80	87	94	101

When specific gravity (G) (see Appendix G) of wood is	SIZE OF SPIKE										
	d=	10	12	16	20	30	40	50	60	5/16"	3/8"
	l=	3	3¼	3½	4	4½	5	5½	6	7	8-12
	D=	0.192	0.192	0.207	0.225	0.244	0.263	0.283	0.283	0.312	0.375
0.31	p=	14	14	15	16	18	19	21	21	23	27
.32		15	15	16	18	20	21	23	23	25	30
.33		17	17	18	20	21	23	25	25	27	33
.34		18	18	19	21	23	24	26	26	29	35
.35		19	19	21	23	25	26	28	28	31	38
.36		21	21	22	24	26	28	30	30	34	40
.37		22	22	24	26	28	30	33	33	36	42
.38		24	24	25	28	30	32	35	35	38	46
.39		25	25	27	30	32	34	37	37	41	49
.40		27	27	29	31	34	37	39	39	43	52
.41		29	29	31	33	36	39	42	42	46	56
.42		30	30	33	35	38	41	45	45	49	59
.43		32	32	35	38	41	44	48	48	52	63
.44		34	34	37	40	43	47	50	50	55	66
.45		36	36	39	42	46	49	53	53	59	70
.46		38	38	41	45	48	52	56	56	62	74
.47		40	40	43	47	51	55	59	59	65	78
.48		42	42	46	50	54	58	62	62	69	83
.49		45	45	48	52	57	61	66	66	72	87
.50		47	47	51	55	60	64	69	69	76	92
.51		49	49	53	58	63	68	73	73	80	96
.52		52	52	56	61	66	71	77	77	84	101
.53		54	54	59	64	69	74	80	80	88	106
.54		57	57	61	67	72	78	84	84	92	111
.55		60	60	64	70	76	82	88	88	97	116
.56		62	62	67	73	79	85	92	92	101	122
.57		65	65	70	76	82	89	96	96	105	127
.58		68	68	73	79	86	93	100	100	110	132
.59		71	71	76	83	90	97	104	104	115	138
.60		74	74	80	87	94	101	109	109	120	144

Table 5-7. Continued

When specific gravity (G) (see Appendix G) of wood is	SIZE OF NAIL											SIZE OF SPIKE										
	d=	6	8	10	12	16	20	30	40	50	60	d=	10	12	16	20	30	40	50	60	5/16"	3/8"
	l=	2	2½	3	3¼	3½	4	4½	5	5½	6	l=	3	3¼	3½	4	4½	5	5½	6	7	8-12
	D=	0.113	0.131	0.148	0.148	0.162	0.192	0.207	0.225	0.244	0.263	D=	0.192	0.192	0.207	0.225	0.244	0.263	0.283	0.283	0.312	0.375
.61		46	53	59	59	65	77	83	90	98	108		77	77	83	90	98	108	114	114	125	150
.62		47	55	62	62	68	80	86	94	102	110		80	80	86	94	102	110	116	118	130	156
.63		49	57	65	65	71	84	90	98	106	115		84	84	90	98	106	115	123	123	136	163
.64		51	59	67	67	73	87	93	102	110	119		87	87	93	102	110	119	128	128	141	168
.65		53	62	70	70	76	91	97	106	115	124		91	91	97	106	115	124	134	134	147	177
.66		55	64	72	72	79	94	101	110	119	128		94	94	101	110	119	128	138	138	152	183
.67		57	66	75	75	82	97	105	114	124	133		97	97	105	114	124	133	143	143	158	190
.68		60	69	78	78	85	101	109	119	128	138		101	101	109	119	128	138	149	149	164	198
.69		62	71	81	81	88	105	113	123	133	143		105	105	113	123	133	143	154	154	170	204
.70		64	74	84	84	92	109	117	128	138	149		109	109	117	128	138	149	160	160	177	213
.75		76	88	99	99	108	129	139	151	164	177		129	129	139	151	164	177	190	190	210	252
.80		89	103	117	117	128	152	163	178	193	208		152	152	163	178	193	208	223	223	248	296

Table 5-8. Allowable load per bolt in double shear (Timber Engineering Co., 1956, pp. 520-521)

l	d	l/d	A = ld	OAK, RED AND WHITE P	OAK, RED AND WHITE Q	PINE, NORTHERN (Eastern White), PONDEROSA, SUGAR, AND IDAHO WHITE P	PINE, NORTHERN (Eastern White), PONDEROSA, SUGAR, AND IDAHO WHITE Q	PINE, LODGEPOLE P	PINE, LODGEPOLE Q	PINE, NORWAY P	PINE, NORWAY Q	PINE, SOUTHERN P	PINE, SOUTHERN Q	POPLAR, YELLOW P	POPLAR, YELLOW Q	REDWOOD P	REDWOOD Q
Inches	Inches		Sq. in.														
1⅝	½	3.3	.8125	940	750	710	380	680	330	750	330	1,010	480	680	330	940	380
	⅝	2.6	1.0156	1,200	850	890	420	850	370	930	370	1,290	540	850	370	1,200	420
	¾	2.2	1.2188	1,450	950	1,070	470	1,010	420	1,120	420	1,550	600	1,010	420	1,450	470
	⅞	1.9	1.4219	1,690	1,040	1,250	520	1,190	460	1,310	460	1,810	670	1,190	460	1,690	520
	1	1.6	1.625	1,930	1,140	1,430	570	1,360	500	1,500	500	2,070	730	1,360	500	1,930	570
2	½	4.0	1.00	1,100	920	890	460	840	410	900	410	1,180	590	840	410	1,100	460
	⅝	3.2	1.25	1,460	1,050	1,100	520	1,050	460	1,160	460	1,560	670	1,050	460	1,460	520
	¾	2.7	1.50	1,780	1,160	1,320	580	1,260	510	1,390	510	1,910	740	1,260	510	1,780	580
	⅞	2.3	1.75	2,080	1,280	1,540	640	1,470	560	1,620	560	2,230	820	1,470	560	2,080	640
	1	2.0	2.00	2,380	1,400	1,760	700	1,670	610	1,850	610	2,550	890	1,670	610	2,380	700
2⅝	½	5.3	1.3125	1,180	1,210	980	610	1,010	530	1,030	530	1,280	780	1,010	530	1,180	610
	⅝	4.2	1.6406	1,760	1,370	1,390	690	1,360	600	1,460	600	1,890	880	1,360	600	1,760	690
	¾	3.5	1.9688	2,250	1,530	1,720	760	1,650	670	1,810	670	2,430	980	1,650	670	2,250	760
	⅞	3.0	2.2969	2,700	1,680	2,020	840	1,920	740	2,120	740	2,900	1,080	1,920	740	2,700	840
	1	2.6	2.625	3,110	1,830	2,310	920	2,190	810	2,430	810	3,340	1,170	2,190	810	3,110	920
3	½	6.0	1.50	1,200	1,330	1,000	690	1,070	610	1,050	610	1,200	890	1,070	610	1,200	690
	⅝	4.8	1.875	1,850	1,570	1,490	780	1,510	690	1,560	690	1,980	1,000	1,510	690	1,850	780
	¾	4.0	2.25	2,470	1,740	1,930	870	1,870	770	2,030	770	2,660	1,120	1,870	770	2,470	870
	⅞	3.4	2.625	3,020	1,920	2,300	960	2,190	840	2,420	840	3,250	1,230	2,190	840	3,020	960
	1	3.0	3.00	3,530	2,100	2,640	1,050	2,510	920	2,770	920	3,790	1,340	2,510	920	3,530	1,050
3⅝	½	7.3	1.8125	1,200	1,390	1,000	830	1,070	730	1,050	730	1,290	1,020	1,070	730	1,200	830
	⅝	5.8	2.2656	1,860	1,850	1,560	950	1,660	830	1,640	830	2,010	1,210	1,660	830	1,860	950
	¾	4.8	2.7188	2,680	2,110	2,170	1,050	2,190	930	2,280	930	2,860	1,350	2,190	930	2,680	1,050
	⅞	4.1	3.1719	3,450	2,320	2,700	1,160	2,630	1,020	2,840	1,020	3,680	1,490	2,630	1,020	3,450	1,160
	1	3.6	3.625	4,120	2,530	3,160	1,270	3,030	1,110	3,320	1,110	4,430	1,620	3,030	1,110	4,120	1,270
4	½	8.0	2.00	1,200	1,390	1,000	890	1,070	780	1,050	780	1,290	1,040	1,070	780	1,200	890
	⅝	6.4	2.50	1,870	1,950	1,600	1,050	1,670	920	1,640	920	2,010	1,330	1,670	920	1,870	1,050
	¾	5.3	3.00	2,690	2,310	2,230	1,160	2,330	1,020	2,340	1,020	2,890	1,490	2,330	1,020	2,690	1,160
	⅞	4.6	3.50	3,570	2,560	2,800	1,280	2,850	1,130	3,000	1,130	3,830	1,640	2,850	1,130	3,570	1,280
	1	4.0	4.00	4,400	2,790	3,430	1,400	3,330	1,230	3,600	1,230	4,720	1,790	3,330	1,230	4,400	1,400
4½	½	9.0	2.25	1,200	1,340	1,000	900	1,070	790	1,050	790	1,290	1,020	1,070	790	1,200	900
	⅝	7.2	2.8125	1,870	1,980	1,580	1,170	1,670	1,030	1,640	1,030	2,010	1,440	1,670	1,030	1,870	1,170
	¾	6.0	3.375	2,690	2,520	2,260	1,310	2,420	1,150	2,360	1,150	2,890	1,680	2,420	1,150	2,690	1,310
	⅞	5.1	3.9375	3,670	2,880	3,010	1,440	3,110	1,270	3,160	1,270	3,920	1,840	3,110	1,270	3,670	1,440
	1	4.5	4.50	4,660	3,140	3,710	1,570	3,670	1,380	3,900	1,380	4,980	2,010	3,670	1,380	4,660	1,570
	1⅛	4.0	5.0625	5,560	3,420	4,340	1,710	4,210	1,510	4,560	1,510	5,980	2,190	4,210	1,510	5,560	1,710
5	½	10.0	2.50	1,200	1,280	1,000	800	1,070	770	1,050	770	1,290	990	1,070	770	1,200	800
	⅝	8.0	3.125	1,870	1,960	1,600	1,260	1,670	1,100	1,640	1,100	2,010	1,470	1,670	1,100	1,870	1,260
	¾	6.7	3.75	2,690	2,620	2,250	1,450	2,420	1,280	2,360	1,280	2,890	1,840	2,420	1,280	2,690	1,450
	⅞	5.7	4.375	3,670	3,140	3,000	1,600	3,200	1,410	3,210	1,410	3,940	2,060	3,260	1,410	3,670	1,600
	1	5.0	5.00	4,750	3,490	3,890	1,750	3,990	1,540	4,080	1,540	5,100	2,240	3,990	1,540	4,750	1,750
	1⅛	4.4	5.625	5,910	3,810	4,680	1,900	4,610	1,670	4,910	1,670	6,280	2,440	4,610	1,670	5,910	1,900

Table 5-8. Continued

l	d		A = l×d	OAK, RED AND WHITE P	OAK, RED AND WHITE Q	PINE, NORTHERN (Eastern White), PONDEROSA, SUGAR, AND IDAHO WHITE P	PINE, NORTHERN (Eastern White), PONDEROSA, SUGAR, AND IDAHO WHITE Q	PINE, LODGEPOLE P	PINE, LODGEPOLE Q	PINE, NORWAY P	PINE, NORWAY Q	PINE, SOUTHERN P	PINE, SOUTHERN Q	POPLAR, YELLOW P	POPLAR, YELLOW Q	REDWOOD P	REDWOOD Q
Inches	*Inches*		*Sq. in.*														
5½	5/8	8.8	3.4375	1,870	1,910	1,660	1,270	1,670	1,120	1,640	1,120	2,010	1,460	1,670	1,120	1,870	1,270
	3/4	7.3	4.125	2,690	2,660	2,250	1,590	2,420	1,400	2,380	1,400	2,890	1,940	2,420	1,400	2,690	1,590
	7/8	6.3	4.8125	3,670	3,310	3,060	1,760	3,310	1,550	3,230	1,550	3,940	2,250	3,310	1,550	3,670	1,760
	1	5.5	5.50	4,770	2,800	3,970	1,920	4,180	1,690	4,170	1,690	5,120	2,400	4,180	1,690	4,770	1,920
	1⅛	4.9	6.1875	6,000	4,190	4,870	2,090	4,960	1,840	5,110	1,840	6,440	2,080	4,960	1,840	6,000	2,090
6	5/8	9.6	3.75	1,870	1,840	1,570	1,250	1,670	1,100	1,650	1,100	2,010	1,420	1,670	1,100	1,870	1,250
	3/4	8.0	4.50	2,690	2,620	2,250	1,680	2,420	1,480	2,380	1,480	2,890	1,970	2,420	1,480	2,690	1,680
	7/8	6.9	5.25	3,670	3,380	3,000	1,920	3,310	1,690	3,230	1,690	3,940	2,410	3,310	1,690	3,670	1,920
	1	6.0	6.00	4,790	4,010	4,000	2,100	4,290	1,840	4,210	1,840	5,140	2,680	4,290	1,840	4,790	2,100
	1⅛	5.3	6.75	6,050	4,540	5,020	2,280	5,240	2,010	5,270	2,010	6,500	2,920	5,240	2,010	6,050	2,280
6½	5/8	10.4	4.0625	1,870	1,800	1,570	1,230	1,670	1,090	1,650	1,090	2,010	1,390	1,670	1,090	1,870	1,230
	3/4	8.7	4.875	2,690	2,560	2,250	1,700	2,420	1,490	2,380	1,490	2,890	1,940	2,420	1,490	2,690	1,700
	7/8	7.4	5.6875	3,670	3,410	3,060	2,070	3,310	1,820	3,230	1,820	3,940	2,510	3,310	1,820	3,670	2,070
	1	6.5	6.50	4,790	4,180	4,000	2,270	4,290	2,000	4,210	2,000	5,140	2,880	4,290	2,000	4,790	2,270
	1⅛	5.8	7.3125	6,050	4,820	5,040	2,470	5,370	2,180	5,290	2,180	6,500	3,170	5,370	2,180	6,050	2,470
7	5/8	11.2	4.375	1,870	1,720	1,570	1,210	1,670	1,070	1,650	1,070	2,010	1,350	1,670	1,070	1,870	1,210
	3/4	9.3	5.25	2,690	2,610	2,250	1,690	2,420	1,490	2,380	1,490	2,890	1,910	2,420	1,490	2,690	1,690
	7/8	8.0	6.125	3,670	3,360	3,060	2,150	3,310	1,890	3,230	1,890	3,940	2,530	3,310	1,890	3,670	2,150
	1	7.0	7.00	4,790	4,250	4,000	2,440	4,290	2,150	4,210	2,150	5,140	3,040	4,290	2,150	4,790	2,440
	1⅛	6.2	7.875	6,050	5,050	5,000	2,600	5,450	2,340	5,340	2,340	6,500	3,410	5,450	2,340	6,050	2,600
7½	5/8	12.0	4.6875	1,870	1,670	1,570	1,200	1,670	1,050	1,650	1,050	2,010	1,300	1,670	1,050	1,870	1,200
	3/4	10.0	5.625	2,690	2,420	2,250	1,660	2,420	1,460	2,380	1,460	2,890	1,880	2,420	1,400	2,690	1,660
	7/8	8.6	6.5625	3,670	3,300	3,060	2,180	3,310	1,920	3,230	1,920	3,940	2,500	3,310	1,920	3,670	2,180
	1	7.5	7.50	4,790	4,230	4,000	2,590	4,290	2,280	4,210	2,280	5,140	3,130	4,290	2,280	4,790	2,590
	1⅛	6.7	8.4375	6,050	5,140	5,000	2,850	5,450	2,510	5,340	2,510	6,500	3,610	5,450	2,510	6,050	2,850
8	5/8	12.8	5.00	1,870	1,600	1,570	1,180	1,670	1,040	1,650	1,040	2,010	1,260	1,670	1,040	1,870	1,180
	3/4	10.7	6.00	2,690	2,360	2,250	1,630	2,420	1,430	2,380	1,430	2,890	1,840	2,420	1,430	2,690	1,630
	7/8	9.1	7.00	3,670	3,260	3,060	2,180	3,310	1,920	3,230	1,920	3,940	2,470	3,310	1,920	3,670	2,180
	1	8.0	8.00	4,790	4,190	4,000	2,680	4,290	2,300	4,210	2,300	5,140	3,150	4,290	2,300	4,790	2,680
	1⅛	7.1	9.00	6,050	5,210	5,000	3,040	5,450	2,680	5,340	2,680	6,500	3,750	5,450	2,680	6,050	3,040
	1¼	6.4	10.00	7,480	6,000	6,260	3,270	6,780	2,880	6,570	2,880	8,040	4,170	6,780	2,880	7,480	3,270
9½	3/4	12.7	7.125	2,690	2,150	2,250	1,570	2,420	1,380	2,380	1,380	2,890	1,800	2,420	1,380	2,690	1,570
	7/8	10.9	8.3125	3,670	2,980	3,060	2,080	3,310	1,830	3,230	1,830	3,940	2,350	3,310	1,830	3,670	2,080
	1	9.5	9.50	4,790	3,900	4,000	2,680	4,290	2,300	4,210	2,300	5,140	3,050	4,290	2,300	4,790	2,680
	1⅛	8.4	10.6875	6,050	5,120	5,090	3,350	5,450	2,950	5,340	2,950	6,500	3,830	5,450	2,950	6,050	3,350
	1¼	7.6	11.875	7,480	6,190	6,260	3,820	6,780	3,360	6,570	3,360	8,040	4,590	6,780	3,360	7,480	3,820
10	7/8	11.4	8.75	3,670	2,940	3,060	2,080	3,310	1,830	3,230	1,830	3,940	2,290	3,310	1,830	3,670	2,080
	1	10.0	10.00	4,790	3,870	4,000	2,660	4,290	2,340	4,210	2,340	5,140	3,000	4,290	2,340	4,790	2,660
	1⅛	8.9	11.25	6,050	4,990	5,090	3,330	5,450	2,930	5,340	2,930	6,500	3,790	5,450	2,930	6,050	3,330
	1¼	8.0	12.50	7,480	6,140	6,260	3,930	6,780	3,460	6,570	3,400	8,040	4,610	6,780	3,400	7,480	3,930
11½	1	11.5	11.50	4,790	3,640	4,000	2,580	4,290	2,270	4,210	2,270	5,140	2,850	4,290	2,270	4,790	2,580
	1⅛	10.2	12.9375	6,050	4,710	5,090	3,260	5,450	2,860	5,340	2,860	6,500	3,600	5,450	2,860	6,050	3,200
	1¼	9.2	14.375	7,480	5,890	6,260	3,950	6,780	3,480	6,570	3,480	8,040	4,490	6,780	3,480	7,480	3,950
12	1	12.0	12.00	4,790	3,560	4,000	2,560	4,290	2,250	4,210	2,250	5,140	2,790	4,290	2,250	4,790	2,560
	1⅛	10.7	13.50	6,050	4,640	5,090	3,200	5,450	2,810	5,340	2,810	6,500	3,600	5,450	2,810	6,050	3,200
	1¼	9.6	15.00	7,480	5,770	6,260	3,920	6,780	3,450	6,570	3,450	8,040	4,450	6,780	3,450	7,480	3,920

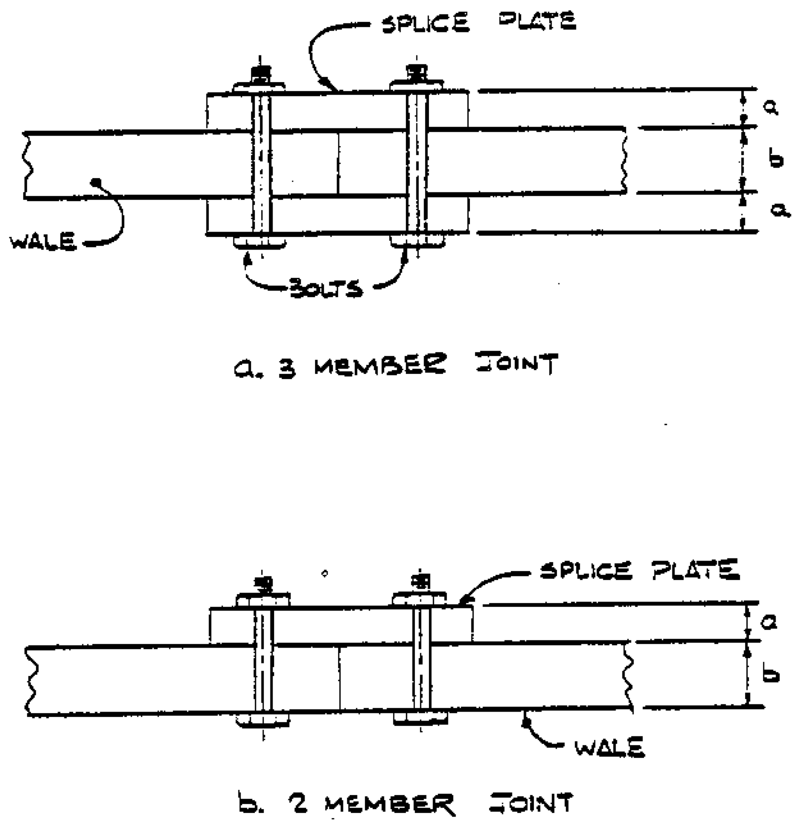


Figure 5-11. Common bolts as fasteners

tabulated loads are for members where side members are at least 1/2 the dimension of the main member. Where side members are thinner than 1/2 the main member, then, for the purpose of determining the allowable load, b = 2a. For example, if a wale is 6 in (152 mm), then the b dimension used for Table 5-8 would be 3 in (76.2 mm).

The values in Table 5-8 are represented by P for loads parallel to the grain and by Q for loads perpendicular to the grain. For the purpose of wale splices, the allowable load in shear per bolt, V, can be taken as Q in Table 5-8.

For 2 member joints (Figure 5-11b) of equal thickness, the allowable load is 1/2 the tabulated value for a main member whose thickness is twice that of the actual member. For example, for a 2 in (50.8 mm) member in southern pine, enter Table 5-8 at 4 in (101.6 mm) for the appropriate bolt diameter. The allowable load, Q, for a 1 in (25.4 mm) bolt is 4720 pounds (21.0 kN) and the allowable load in shear per bolt, V, is 2360 pounds (10.5 kN).

For 2 member joints of unequal thickness, the procedure outlined in the previous paragraph is applied with respect to the thinner member.

Where steel plates are used as splice members, the allowable load is increased by 25 percent.

The criteria for allowable loads in common bolts are summarized in Table 5-9.

The allowable loads on high strength bolts (ASTM A325) are 40,000 psi (276 MN/m^2) in tension, f_t, and 15,000 psi (103 MN/m^2) in shear, f_v (AISC, 1973).

Table 5-9. Summary of allowable loads in common bolts used for splice plates

Joint Type	Relative Dimensions	Enter Table 5-8, Column b at	Allowable Load, V
3 Member	$a \geq \frac{1}{2}b$	$b = b$	P
	$a < \frac{1}{2}b$	$b = 2a$	P
2 Member	$a = b$	$b = 2a$	$\frac{1}{2}P$
	$a < b$	$b = 2a$	$\frac{1}{2}P$
	$a > b$	$b = 2b$	$\frac{1}{2}P$
Steel Side Plate	n/a	$b = b$	1.25P

The cost of fasteners is dependent upon their required size and number which are, in turn, determined by their capacity and loads. Another determinant to be considered is the location of the wale, i.e., whether it is located on the fill side or the dredge side of the wall. Locating the wale on the fill side presents a smooth face for the user, whereas locating the wale on the dredge side presents a protrusion which may interfere with mooring. However, with the wale located inside the fill, more fasteners are required as the fill tends to push the sheet piles away from the wale, exerting a prying force (Figure 5-12a). On the other hand, a wale outside the fill bears against the sheet piles, thereby eliminating the consideration of prying forces. The number of nails required per wood pile section, n, is

$$n = \frac{Pw}{W_r} \tag{5-15}$$

in which P = the tie-rod pull (force per unit length of wall), w = the width of the pile section, and W_r = the resistance to withdrawal per nail. The number of high strength bolts per steel sheet pile, n, is

$$n = \frac{4Pw}{\pi d^2 f_t}$$

in which d = the bolt diameter and f_t = the allowable tensile stress per bolt, taken as 40,000 psi (275 MN/m^2) for A325 bolts. The allowable shear stress, f_v, in A325 bolts is 15,000 psi (103 MN/m^2).

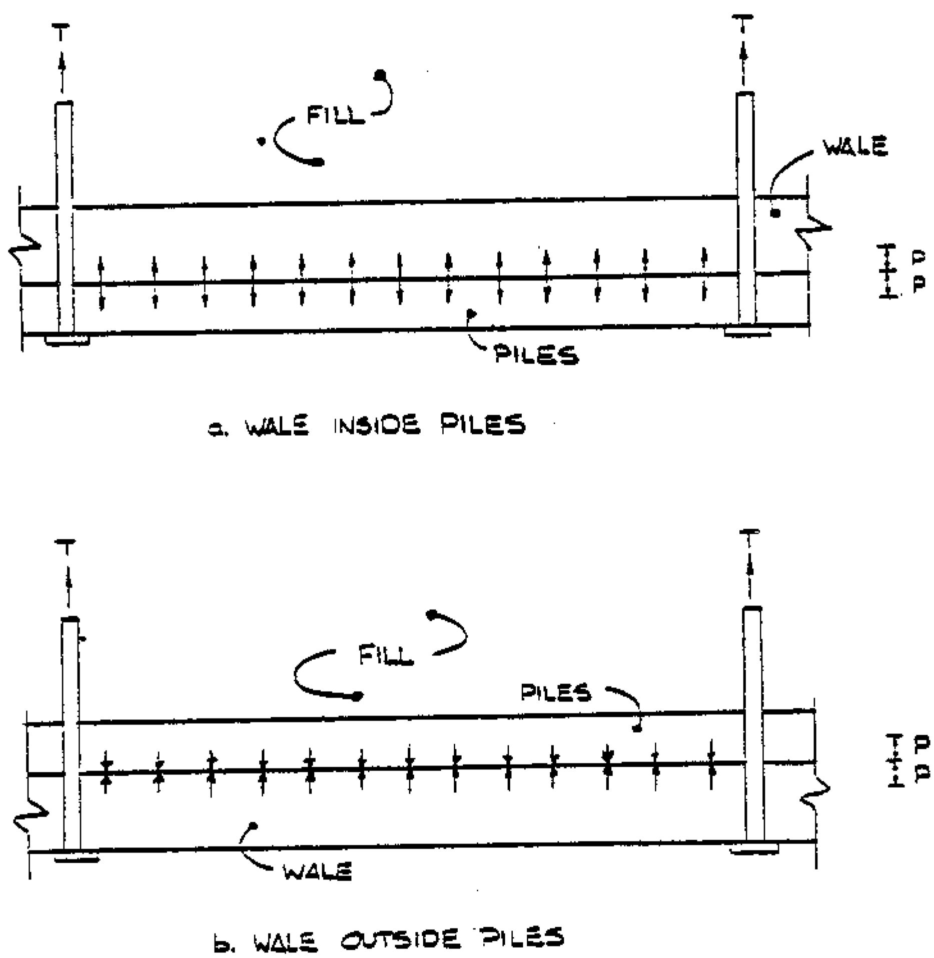


Figure 5-12. Transfer of loads from piles to wales

Holes are 1/32 in (0.79 mm) larger than the bolt diameter for wood and 1/6 in (1.6 mm) for steel.

5.2.5. Bulkhead Lifetime

The life expectancy of a bulkhead depends upon the components of the system, i.e., if one component fails, the system is no longer viable. Obviously, the lifetimes of components vary from material to material, and the material with the shortest lifetime will control the bulkhead lifetime. The designer must, therefore, insure that the material of each component is the optimum.

The structure must be protected from harmful agents that exist in the environment. Timber must be protected from rot and other biological agents by an appropriate treatment as recommended by the American Wood Preservation Institute (AWPI) and the American Wood Preservative Association (AWPA).

Timber sheet piles usually consist of heartwood instead of sapwood. This may cause the purchaser some consternation as standards established for preservative penetration are for sapwood, not heartwood. Since heartwood is more resistant to preservative penetration, it follows that the preservative penetration of many sheet piles will be less than optimum.

Steel sheet pile and tie-rod life can be prolonged by applying special coatings. Corrosion and decay rates should be determined for a particular environment so that the life of the structure can be estimated. A detailed discussion of materials and the hazards present in certain environments is contained in "Coastal Structure Materials" (Hubbell and Kulhawy, 1979).

Tie-rods, turnbuckles, bolts, nuts, washers, and nails receive protection from corrosion by galvanizing. Electro-deposited zinc coatings, in accordance with ASTM B633, or hot-dip coatings, in accordance with ASTM A513, may be specified to increase the life of steel components.

When the cost is favorable, hardware may be comprised of wrought iron.

If no coating or treatment is specified for the hardware, the required dimensions will be reduced by corrosion. If the amount of deterioration is known, the dimensions of the hardware should be increased by this amount to preclude failure. Recommended increases in hardware dimensions are shown in Table 5-10 (Johnson, 1965).

Bulkheads sited in erosion zones should incorporate returns on the flanks of the bulkhead (see Chapter 6, Figure 6-1). These are sections constructed perpendicular to the wall which prevent the washout of backfill around the flanks.

5.2.6. Compliance with Industry Standards

The designer may enhance the quality assurance of the product by making certain that suppliers comply with industry standards, such as ASTM and AWPA specifications. This may be accomplished by inspecting timber products for grademarks (Figure 5-13) and by requesting certificates of compliance from the supplier. Such requests are reasonable and the documents certify that the provisions of the specifications are met.

Table 5-10. Recommended increase in dimensions of hardware (summarized from Johnson, 1965)

Dimension	Exterior Exposure (Except Marine)	Marine Exposure: In and Below Splash Zone	Marine Exposure: Above Splash Zone
Bolt Diameter	1/8 in (3.18 mm)	1/2 in (12.7 mm)	1/4 in (6.35 mm)
Plate Thickness	1/8 in (3.18 mm)	1/4 in (6.35 mm)	1/8 in (3.18 mm)

Note: Washers for marine exposure (in and below splash zone) should be ogee. For other exposures, 1/4 in (6.35 mm) plate types are unsuitable, ogee optional.

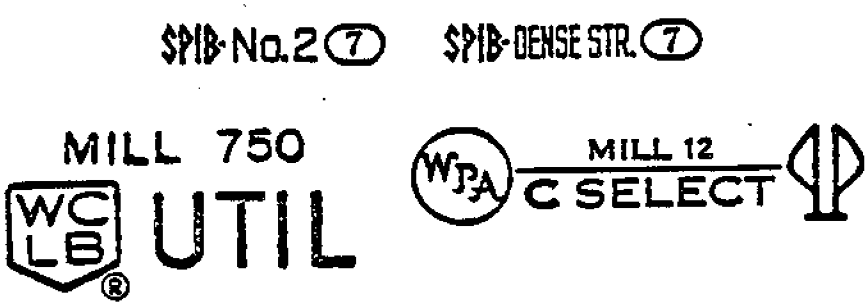


Figure 5-13. Typical grademarks (Timber Engineering Co., 1956, p. 37)

5.3. Design of Components

5.3.1. Sheet Piles

When the maximum moment has been determined (Chapters 2 and 4), the required section modulus is found by employing Equation 5-9. Since the moment is computed in terms of moment per unit length of wall, the section modulus must also be in terms of unit length per wall. For steel sheet piles, Table 5-2 is consulted, as is demonstrated in design examples found in the Appendices.

For rectangular wood piles, the required thickness is found by employing Equation 5-10a, as is also demonstrated in design examples.

No load factors are required for sheet pile calculations. A material factor is already employed in the allowable bending stress, f_b, for steel and wood.

5.3.2. Tie-Rod Diameter

The computation of the tie-rod diameter is quite simple. Once the tie-rod pull, P (force per unit length of wall), is found, the tie-rod tension, T, is found by multiplying the tie-rod load times the spacing between ties (see Section 6.1.5. for further discussion on the tie-rod spacing). A load factor is then applied (Section 5.2.6.) and the diameter found by

$$d = \sqrt{\frac{4T\ LF}{\pi f_t}} \qquad (5\text{-}17)$$

in which LF = a load factor of 1.2 to 1.4 and f_t = the allowable tension of A36 steel (Equation 5-12c and Table 5-5). At this point the designer may decide to increase the diameter of the tie-rod for corrosion if no other precautions were taken (Section 5.2.5.).

Tables 5-11 and 5-12 contain data for tie-rods and turnbuckles, respectively.

An example of determining the tie-rod diameter is given in the Appendices.

5.3.3. Wale Design

The bending moment in wales is somewhere between that for a single span, simply supported, and that for three continuous spans, simply supported. The maximum bending moment can therefore be taken as (Teng, 1962)

$$M = \frac{1}{9} P\ell^2 \qquad (5\text{-}18)$$

in which P = the tie-rod force (per unit length of wall) and ℓ = the distance between tie-rods.

The section modulus is determined from Equation 5-9. Once this is found, Table 5-3 may be used to find the appropriate channel size or, if wood wales are used, Equation 5-10a or Table 5-4b is used to find the proper dimensions. Examples of steel and wood wale designs may be found in the Appendices.

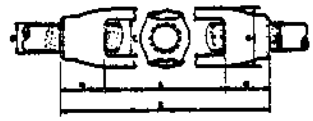
5.3.2.1. Fastening Wood Piles and Wales

Wales located on the fill side of the wall have a tendency to separate from the sheet piles. The prying force exerted on each sheet pile may be taken as the tie-rod load per unit length of wall, P, since

Table 5-11. Tie rods (AISC, 1967, p. 4-93)

ROUND BAR			UPSET END									
				LENGTH L		ADD'L LENGTH REQUIRED		ADD'L WEIGHT REQUIRED				% EXCESS ROOT AREA OVER CROSS BAR AREA
DIAM. D	CROSS AREA	WT. PER FT.	DIAM D	FOR TURN-B'KLE	FOR CLEVIS	FOR TURN-B'KLE	FOR CLEVIS	FOR TURN-B'KLE	FOR CLEVIS	THDS. PER INCH	THD ROOT AREA	
IN	IN²	LB	IN	IN	IN	IN.	IN	LB	LB		IN²	
¾	.442	1.50	1	4½	4	4¾	4	.53	.50	8	.551	24.7
⅞	.601	2.04	1⅛	4¾	4	4	3¼	.68	.55	7	.693	15.3
1	.785	2.67	1⅜	5	4	5½	4½	1.22	1.00	6	1.054	34.3
1⅛	.994	3.38	1½	5¼	4	5	4	1.41	1.13	6	1.294	30.2
1¼	1.227	4.17	1¾	5½	4	6¼	4¾	2.26	1.65	5	1.74	41.8
1⅜	1.485	5.05	1¾	5½	4	4½	3¼	1.89	1.37	5	1.74	17.2
1½	1.767	6.01	2	5¾	4½	5½	4½	2.75	2.25	4½	2.30	30.2
1⅝	2.074	7.05	2¼	6¼	5	7¼	5½	4.26	3.23	4½	3.02	45.6
1¾	2.405	8.18	2¼	6½	5	5½	4¼	3.75	2.90	4½	3.02	25.6
1⅞	2.761	9.39	2½	6¾	5½	6½	5¾	5.08	4.11	4	3.72	34.7
2	3.142	10.68	2½	6¾	5½	5	4	4.45	3.56	4	3.72	18.4
2⅛	3.547	12.06	2¾	7¼	5½	6¼	4¾	6.28	4.77	4	4.62	30.3
2¼	3.976	13.52	2¾	7¼	5½	4¾	3½	5.35	3.94	4	4.62	16.2
2⅜	4.430	15.06	3	7½	6	5¾	4½	7.22	5.65	4	5.62	26.9
2½	4.909	16.69	3¼	8¼	6½	7¼	5¾	10.08	8.00	4	6.72	36.9
2⅝	5.412	18.40	3¼	8¼	6½	5¾	4½	8.83	6.90	4	6.72	24.2
2¾	5.940	20.20	3½	8¾	7	6½	5½	10.94	9.26	4	7.92	33.3
2⅞	6.492	22.07	3½	8¾	7	5¼	4½	9.66	8.28	4	7.92	22.0
3	7.069	24.03	3¾	9	7	6½	5	13.02	10.01	4	9.21	30.3
3⅛	7.670	26.08	4	9	7½	7¼	6	15.76	13.04	4	10.61	38.3
3¼	8.296	28.21	4	9	7½	6	5	14.10	11.75	4	10.61	27.9
3⅜	8.946	30.42	4¼	9¾	—	7	—	17.75	—	4	12.10	35.3
3½	9.621	32.71	4¼	9¾	—	6	—	16.36	—	4	12.10	25.8
3⅝	10.321	35.09	4½	10	—	7	—	20.47	—	4	13.69	32.6
3¾	11.045	37.55	4¾	10½	—	8	—	25.03	—	4	15.38	39.2
3⅞	11.793	40.10	4¾	10½	—	7	—	23.39	—	4	15.38	30.4

Table 5-12. Turnbuckles (AISC, 1980, p. 4-143)



DIAM D	STANDARD TURNBUCKLES					WEIGHT OF TURNBUCKLES, POUNDS						TURNBUCKLE SAFE WORKING LOAD, KIPS*
	DIMENSIONS, INCHES					LENGTH, A, INCHES						
IN	A	N	C	E	G	6	9	12	18	24	36	
3/8	6	9/16	7 1/8	5/16	1 5/32	.41						1.2
1/2	6	3/4	7 1/2	15/32	1 5/16	.75	.80	1.00				2.2
5/8	6	29/32	7 13/16	9/16	1 1/2	1.00	1.38	1.50	2.43			3.5
3/4	6	1 1/16	8 1/8	11/16	1 25/32	1.45	1.63	2.13	3.06	4.25		5.2
7/8	6	1 7/32	8 7/16	1 3/32	1 7/8	1.85		2.83	4.20	5.43		7.2
1	6	1 3/8	8 3/4	1 9/32	2 1/32	2.60		3.20	4.40	6.85	10.0	9.3
1 1/8	6	1 9/16	9 1/8	1 13/32	2 5/16	2.72		4.70	6.10			11.6
1 1/4	6	1 3/4	9 1/2	1 9/16	2 17/32	3.58		4.70	7.13	11.30	13.1	15.2
1 3/8	6	1 15/16	9 7/8	1 11/16	2 3/4	4.50						17.4
1 1/2	6	2 1/8	10 1/4	1 27/32	3 1/32	5.50		8.00	9.13	16.00	19.4	21.0
1 5/8	6	2 1/4	10 1/2	1 31/32	3 9/32	7.50				*		24.5
1 3/4	6	2 1/2	11	2 1/8	3 9/16	9.50		15.25	16.00	19.50		28.3
1 7/8	6	2 3/4	11 1/2	2 1/4	4	11.50						37.2
2	6	2 3/4	11 1/2	2 1/4	4	11.50		15.25		27.50		37.2
2 1/4	6	3 1/8	12 1/4	2 11/16	4 1/2	18.00		35.25		43.50		48.0
2 1/2	6	3 3/4	13 1/2	3	5	23.25		33.60		42.38		60.0
2 3/4	6	4 1/8	14 1/4	3 1/4	5 1/2	31.50				54.00		75.0
3	6	4 1/2	15	3 3/8	6 1/8	39.50						96.7
3 1/4	6	5 1/4	16 1/2	3 7/8	6 3/4	60.50						122.2
3 1/2	6	5 1/4	16 1/2	3 7/8	6 3/4	60.50						122.2
3 3/4	6	6	18	4 1/2	8 1/2	95.00						167.8
4	6	6	18	4 1/2	8 1/2	95.00						167.8
4 1/4	9	6 3/4	22 1/2	5 1/4	9 3/4		152.0					233.8
4 1/2	9	6 3/4	22 1/2	5 1/4	9 3/4		152.0					233.8
4 3/4	9	6 3/4	22 1/2	5 1/4	9 3/4		152.0					233.8
5	9	7 1/2	24	6	10		200.0					294.7

each sheet pile is approximately one foot wide. The number of nails, n, required per pile may be found by selecting a nail size, determining its allowable load in withdrawal, W_r, from Tables 5-6 and 5-7 and Equation 5-13, then using the simple relationship

$$n = \frac{P}{W_r} \qquad (5\text{-}19)$$

An example may be found in the Appendices.

Wales located on the dredge side of the bulkhead require nails for construction only. Using two nails per sheet pile should be sufficient. The nails should, however, be long enough to have adequate embedment in the wale so as to be capable of transmitting shear, i.e., 2/5 of their length (Timber Engineering Co., 1956) or

$$\ell_e = 2/5 \;\; \ell. \qquad (5\text{-}20a)$$

Since the effective length, ℓ_e, is the length, ℓ, minus the pile thickness, t, the nail length should be

$$\ell = 5/3 \; t. \qquad (5\text{-}20b)$$

An example may be found in the Appendices.

5.3.3.2. Splices in Wood Wales

Advantages are gained by locating the splices of outside wales at the tie-rods (Figure 5-14). The bending moments here cause compression of the outside edge of the wale and tension at the inside edge.

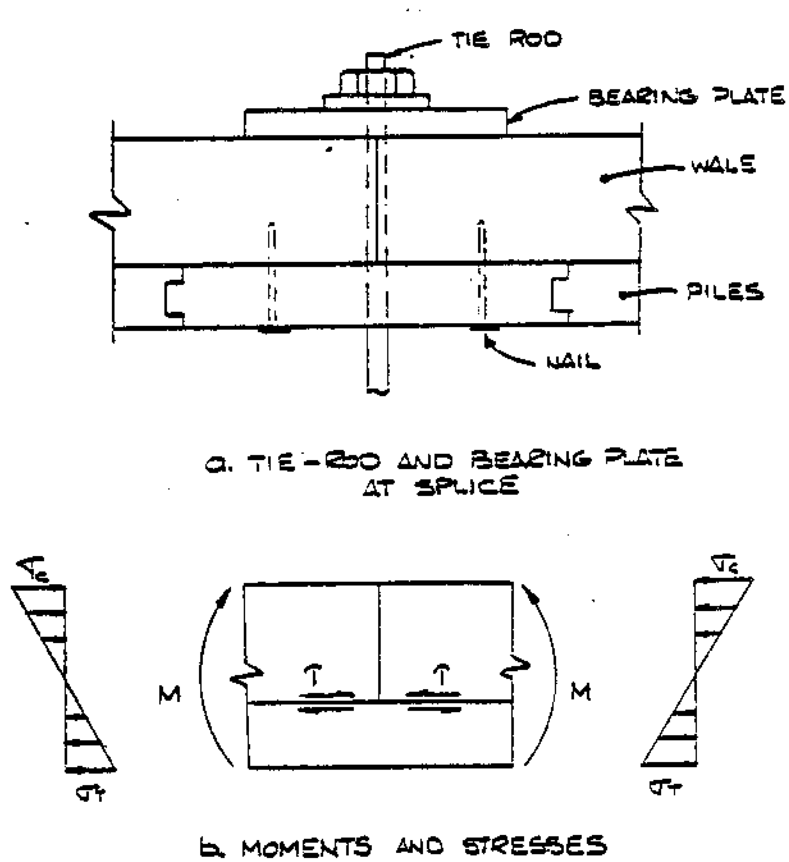


Figure 5-14. Locating the splice at tie-rods

The tension is resisted by the sheet pile attached to the wale at this location (Figure 5-14b).

A splice requiring a 2- or 3-member joint (Figure 5-11) may be eliminated. In addition to cost savings, elimination of the splice removes the potential for ponding that would occur between the horizontal members of the splices. Ponding hastens the decay of the wood.

An advantage is also gained as the tie-rod hole in the wale occurs in an area which is penetrated with preservative throughout the entire length of the hole.

The bearing plate is designed in a manner similar to the design for bearing of a steel beam on a masonry wall. The plate area is determined from the allowable bearing pressure, f_p, taken as $c \perp$ from Table 5-4. The area, A, is found from

$$A = \frac{T}{f_p} \tag{5-21}$$

The thickness of the plate is given by (AISC, 1973)

$$t = \frac{3\ F_p\ N^2}{f_b} \tag{5-22}$$

in which: F_p = the actual bearing pressure, N = 1/2 the short dimension of the plate minus the hole radius , and f_b = the allowable bending stress of the steel. An example of the bearing plate design for an outside wale may be found in the Appendices.

Inside wale splices must be 2- or 3-member joints (Figure 5-11). The average shear force, V, that the bolts must resist may be found from

$$V = \frac{1}{2} T - \frac{1}{4} PL_b \tag{5-23}$$

in which L_b = the distance between extreme bolts. Equation 5-23 is valid for splices centered over the tie-rod. The splice should also be designed to resist the maximum moment.

Bolts in the splice have minimum requirements for end distance, edge distance, bolt spacing, and distance between rows of bolts. A summary of these requirements appears in Table 5-13. These are for loads acting perpendicular to the grain (Timber Engineering Co., 1956).

The procedure for designing a splice is to select L_b, compute V, select a bolt size in accordance with Section 5.2.4, determine the arrangement of bolts, and determine the final length of the splice member. Examples of 2- and 3-member splice designs may be found in the Appendices.

5.3.3.3. Fasteners and Splices for Steel Wales

Figure 5-15 displays typical details for inside and outside wales used with steel sheet piles. Inside wales are fastened using high strength bolts in conjunction with a fixing plate. The number of bolts is determined by Equation 5-16 and the fixing plate may be dimensioned by approximating it as a simply supported beam with a point load.

The minimum distance from the center of the bolt hole to the edge of the member may be taken as 1.5 times the bolt diameter for rolled or

Table 5-13. Distance requirements for bolted connections (Timber Engineering Co., 1956).

Distance	Number of Bolt Diameters, n_d
End	$1\frac{1}{2}$
Edge	4
Bolt Spacing	4
Rows of Bolts	$2\frac{1}{2}$ (for $l/d \leq 2$)
	5 for $l/d \leq 6$)
	$(5/8)(l/d) + 1\frac{1}{4}$ (for $2 < l/d < 6$)

l/d = bolt length/bolt diameter

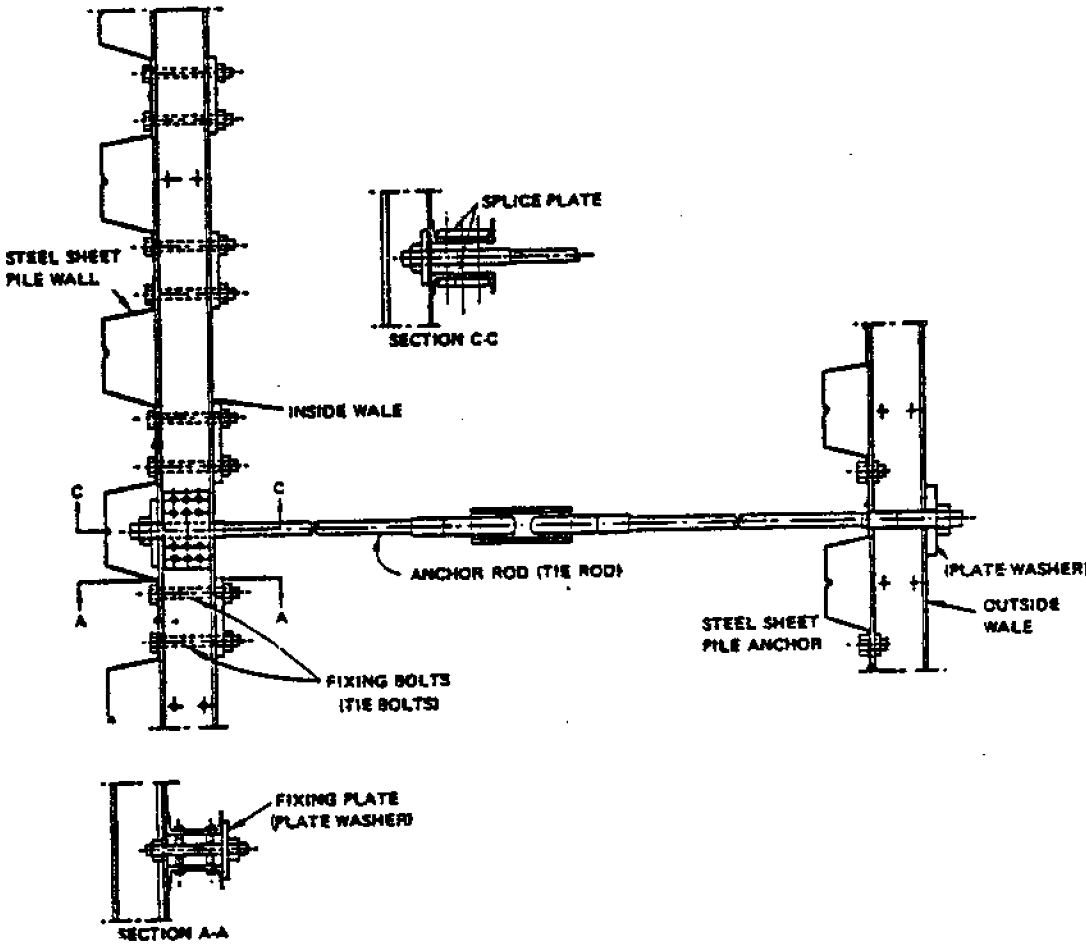


Figure 5-15. Typical wale and anchor details (U.S. Steel, 1975, p. 39)

gas cut edges. Minimum spacing is three bolt diameters (AISC, 1973). An example of the design of an inside wale may be found in the Appendices.

An outside wale may be fastened by merely employing a plate of sufficient dimensions between the wale and the tie-rod nut. A plate washer will suffice if the separators allow the channels to be close enough to each other.

Splices in wales should be able to transfer the maximum moment in the wales (Equation 5-18). The splice plate may be dimensioned using Equations 5-9, 5-10a, and 5-23. Design of splice plates for steel channel wales may be found in the Appendices.

5.4. Anchorage

Once the anchorage is adequately located with respect to the geómetry and soil strength of the site, the type of anchorage must be chosen and dimensioned.

5.4.1. Continuous Deadman

The capacity of a continuous deadman stems from the net resultant of the soil stresses acting, as shown in Figure 5-16. When considering these stresses, the distance to the high water mark should be considered as this represents the lowest capacity of the deadman. The stress coefficients K_p' and K_a' used are factored, thus requiring no additional load factors for the design. An example is given in the Appendices.

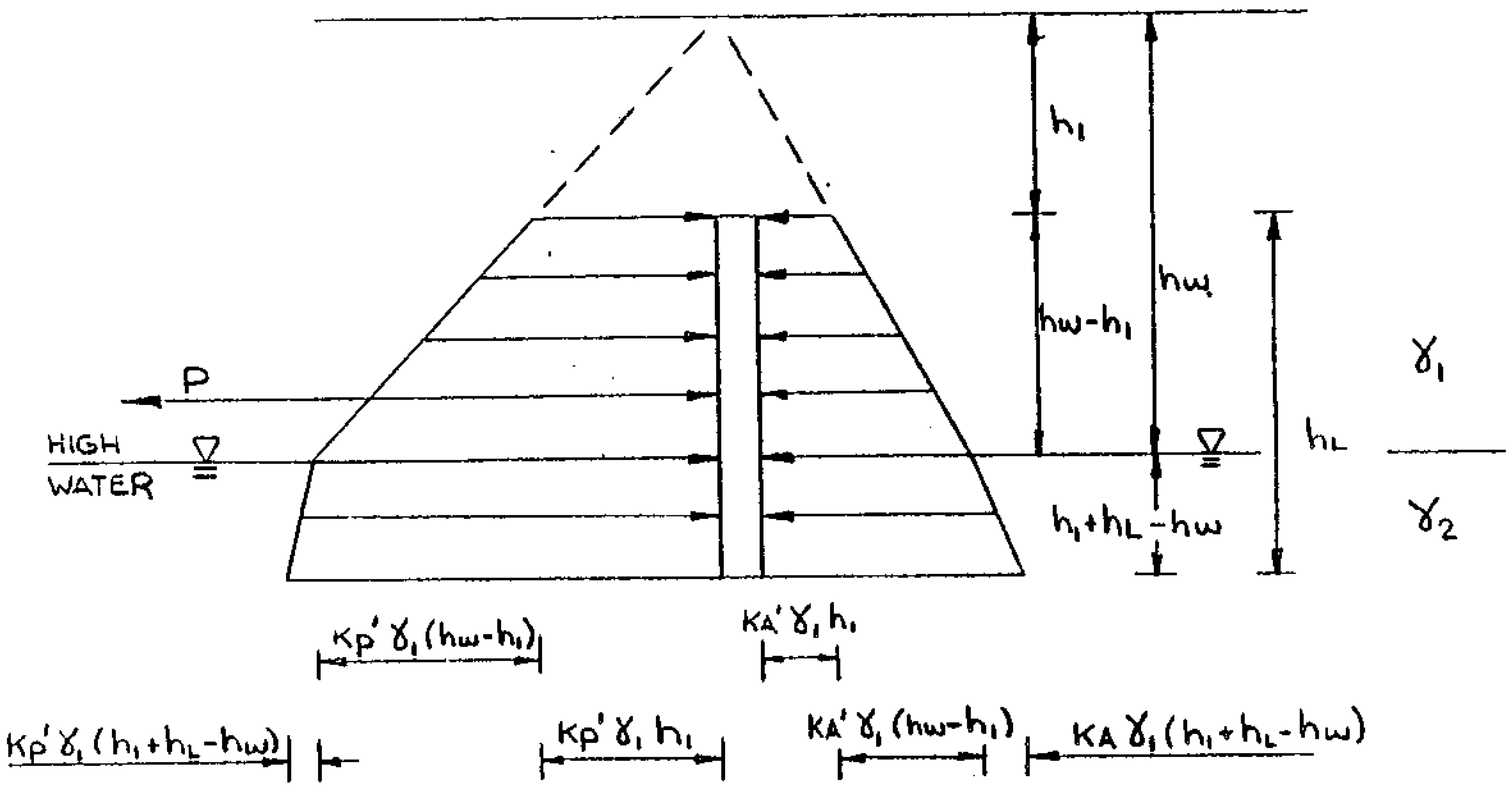


Figure 5-16. Soil stresses acting on the anchorage

5.4.2. Short Deadman Near the Surface

The calculation for short deadmen near the surface can be facilitated using the information obtained from the design of a continuous deadman. The net capacity per length of anchorage $(P_p - P_a)$ is already computed in terms of h_L, deadman height. The remaining values are merely substituted into Equation 5.7. The Appendices contain an example of the design of a short deadman.

5.5. Summary

Bulkhead design requires the integrated consideration of loading, cost-effectiveness, and the design of the basic bulkhead components. A detailed examination of these considerations has been presented in this chapter.

The bulkhead may have to withstand loads other than those stemming from the retained soil. These include surcharges placed on the backfill, hydrostatic imbalance, ice thrust, mooring loads and ship impact. The loads imposed on some components should be increased by load factors, depending upon the inherent uncertainties.

Cost-effectiveness is dependent upon such interrelated factors as type and configuration of the wall, material strength of the components, ability to withstand harmful agents of the environment, and fastening methods.

Each structural component must be dimensioned and the type, number, and spacing of fasteners must be determined. As each item is being selected, the designer must keep in mind alternative materials and schemes, costs, and the desired function of the component.

CHAPTER 6

CONSTRUCTION CONSIDERATIONS

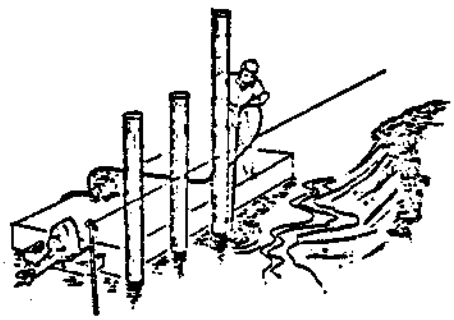
The construction of bulkheads is less complicated than the design proess. Figures 6-1a through 6-1f are a pictorial sequence of a typical navy bulkhead construction operation. In spite of the apparent simplicity, there are factors which must be considered to comply with design criteria and result in optimum performance. This section includes a discussion of these factors.

6.1. <u>General Construction Procedure</u>

6.1.1. <u>Pile Installation</u>

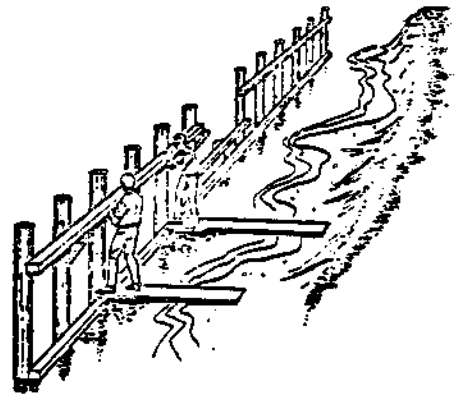
Prior to installing the sheet piles, the bulkhead alignment is determined and guides are placed, such as wales placed on temporary stakes. This is not necessary for navy bulkheads because the fender piles and wales provide the proper horizontal alignment. Vertical alignment may include a slight batter in the direction of the fill side of wall. This is standard practice in areas subject to freezing and tide changes. The overall effect is to diminish pile uplift by ice on a rising tide. A temporary wale may be placed below the upper wale to facilitate construction. This lower wale is not necessary for the permanent structure.

Sheet piles are generally installed by driving, jetting, or a combination of both. Driving is more desirable from a soil mechanics



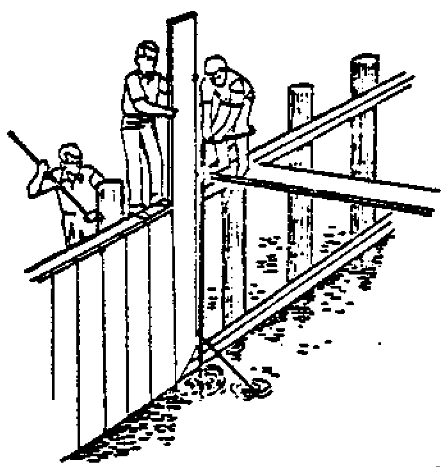
After it is determined where the bulkhead should be, piles are driven into the bottom with the aid of a water jet.

a



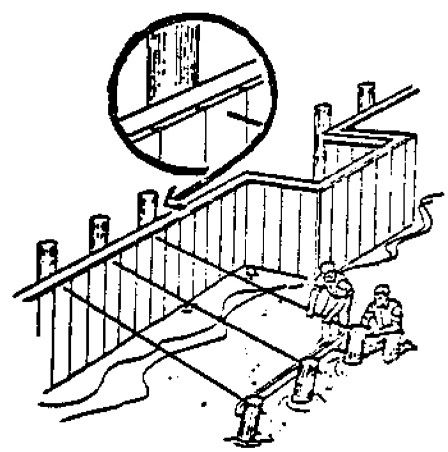
WALERS (top and bottom) are attached to hold the piles in line.

b



SHEATHING is lowered into position with the aid of a water jet and nailed to the WALERS.

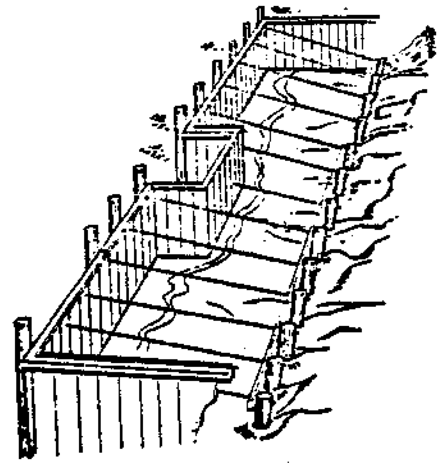
c



Galvanized TIE RODS are connected to DEADMEN PILES anchoring the system back into the land.

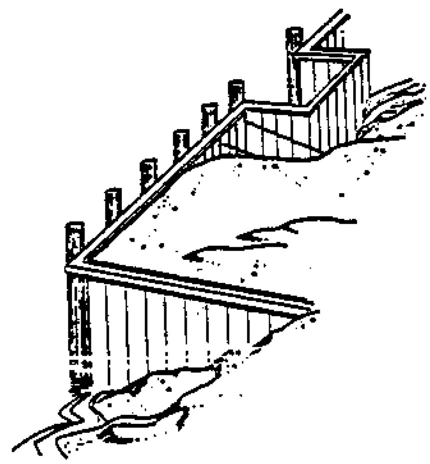
d

Figure 6-1. Typical construction sequence (BBS Creosote Lumber Co. Inc., undated)



An INTERLACE between the DEADMEN PILES completes the basic strength of the bulkhead. If necessary, a RETURN into the land prevents scouring behind the bulkhead.

e



BACKFILLING to the desired height conceals the TIE RODS and provides for a level property to the waterfront.

f

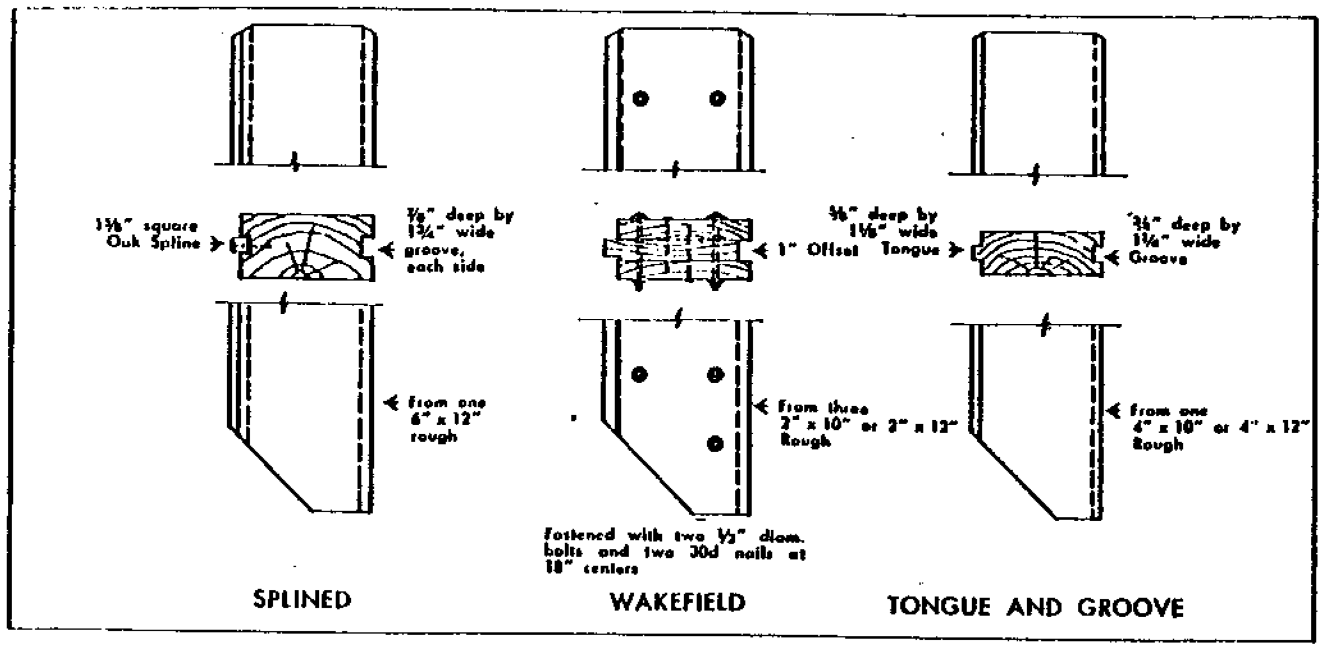
Figure 6-1. Continued

standpoint as the downward force of the pile tip tends to locally compact the soil, thus increasing its strength. Jetting is more commonly practiced where timber sheet piling is installed. This procedure entails pumping water through a pipe under approximately 100 psi (689 N/m^2) pressure and advancing the pipe into the subgrade closely followed by the pile. Jetting is not effective in gravel, silt, or clay and tends to loosen the soil locally, thus decreasing the soil strength. Because jetting facilitates installation and driving enhances soil strength, a combination of these creates the optimum operation where the pile is jetted to within a few feet of the required depth and the remainder is driven.

As piles interlock using tongue-and-groove or ball-and-socket fittings (Figure 6-2), it is recommended that the direction of construction leads with the tongue, or ball. This will eliminate the danger of soil clogging the groove, or socket, and subsequent improper interlock and leaning.

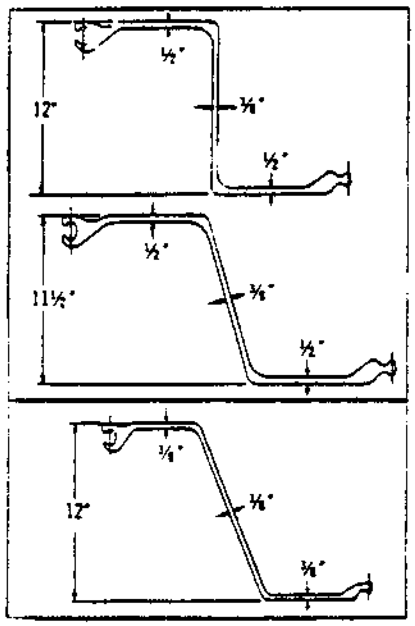
Driving in pairs or in panels (Figure 6-3) eliminates some of the interlock friction occurring between piles. This also facilitates driving as rigidity is increased and leaning is reduced.

Other causes of leaning may include defective guides, pile deformation, improper driving and improper jetting. Remedies include pulling the heads of piles during installation (Figure 6-4a), use of guide piles in conjunction with driving in panels (Figure 6-4b), applying the driving force at an angle (Figure 6-4c), use of piles with chamfers at the foot (Figure 6-4d), and use of specialty fabricated wedge-shaped piles (Figure 6-4e) (Teng, 1962).



a. Typical timber piles

Figure 6-2. Typical piles (AWPI, p. 3)



b. Typical ball and socket (U.S. Steel, 1975, f p. 1)

Figure 6-2. Continued

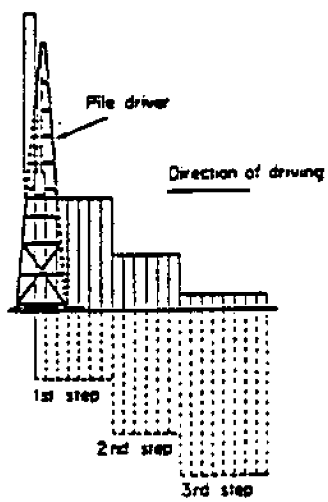


Figure 6-3. Driving sheet piles in panels (Teng, 1962, p. 378)

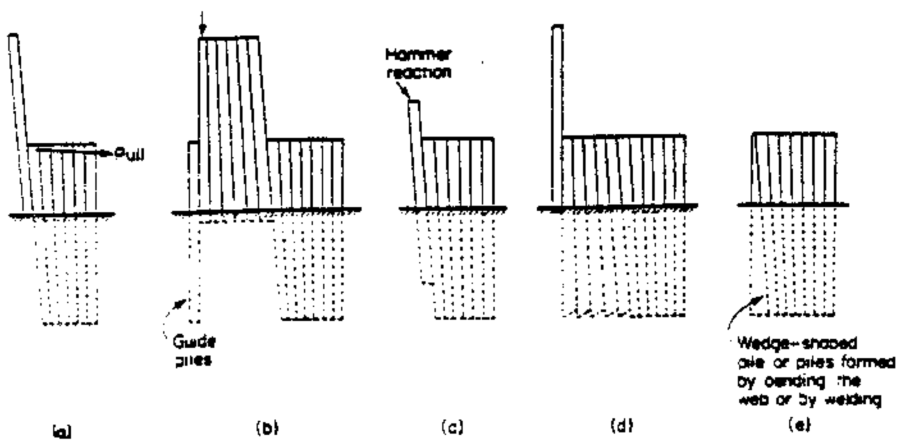


Figure 6-4. Remedial actions (Teng, 1962, p. 379)

6.1.2. Wales

After the piles are installed, wales are connected by bolting channels to each steel sheet pile section or by nailing timber wales to timber sheet piles (Section 5.4.3.).

Splices are made in wales where required. Locating the splices of wooden wales at the tie-rod eliminates the need for splice plates and reduces the potential for ponding, thereby accruing some economic advantages.

Typical details of wales for steel walls are shown in Figure 6-5.

6.1.3. Anchorage

The anchorage should be installed in parent material a safe distance from the wall (Section 5.3.2.). If the parent material is undesirable, it should be removed and the backfill in front of the anchorage should be compacted.

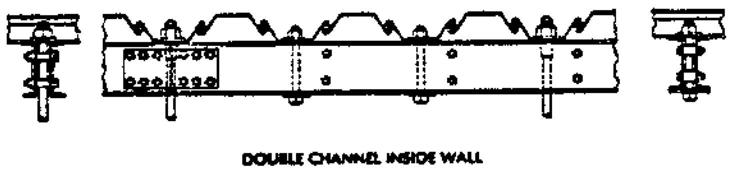
Alternative anchoring schemes are shown in Figure 6-6 and alternative anchorage schemes are shown in Figure 6-7.

6.1.4. Tie-Rods

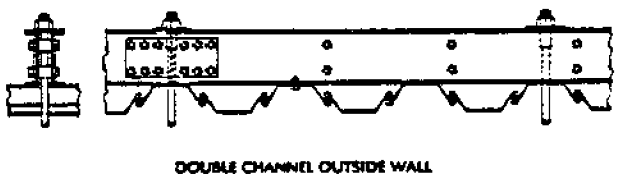
Holes are drilled through fender piles (if used), wales, sheet piles and anchorages. One tie-rod segment is passed through the wall, another segment through the anchorage, and the two segments are joined using a turnbuckle. If settlement of the tie-rods is considered a problem, PVC pipe should surround the tie-rod (Section 6.2.6.).

If the tie-rod is not horizontal, the design load should be increased by a load factor

$$LF = \frac{1}{\cos \theta} \qquad (6\text{-}1)$$

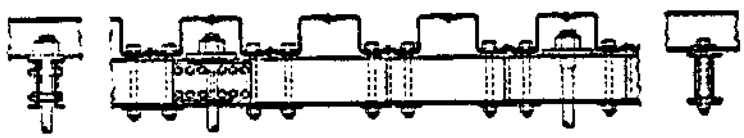


a



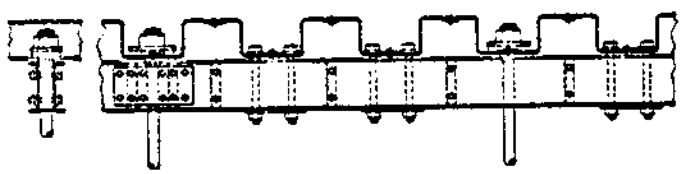
b

Figure 6-5. Standard wale details (U.S. Steel, 1976, pp. 71-73)



DOUBLE INSIDE CHANNEL WALES—WELDED INTERMEDIATE BEAM OR CHANNEL SEPARATORS

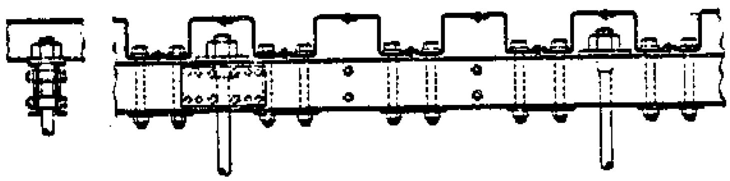
c



DOUBLE INSIDE CHANNEL WALES—BOLTED CHANNEL SEPARATORS

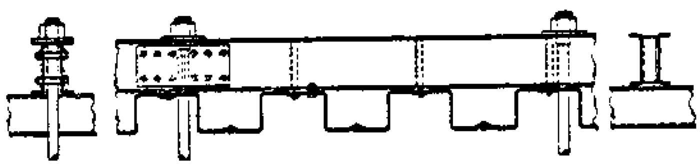
d

Table 6-5. Continued



DOUBLE INSIDE CHANNEL WALES—BOLTED PIPE SEPARATORS

e



DOUBLE OUTSIDE CHANNEL WALES—WELDED INTERMEDIATE BEAM OR CHANNEL SEPARATORS

f

Figure 6-5. Continued

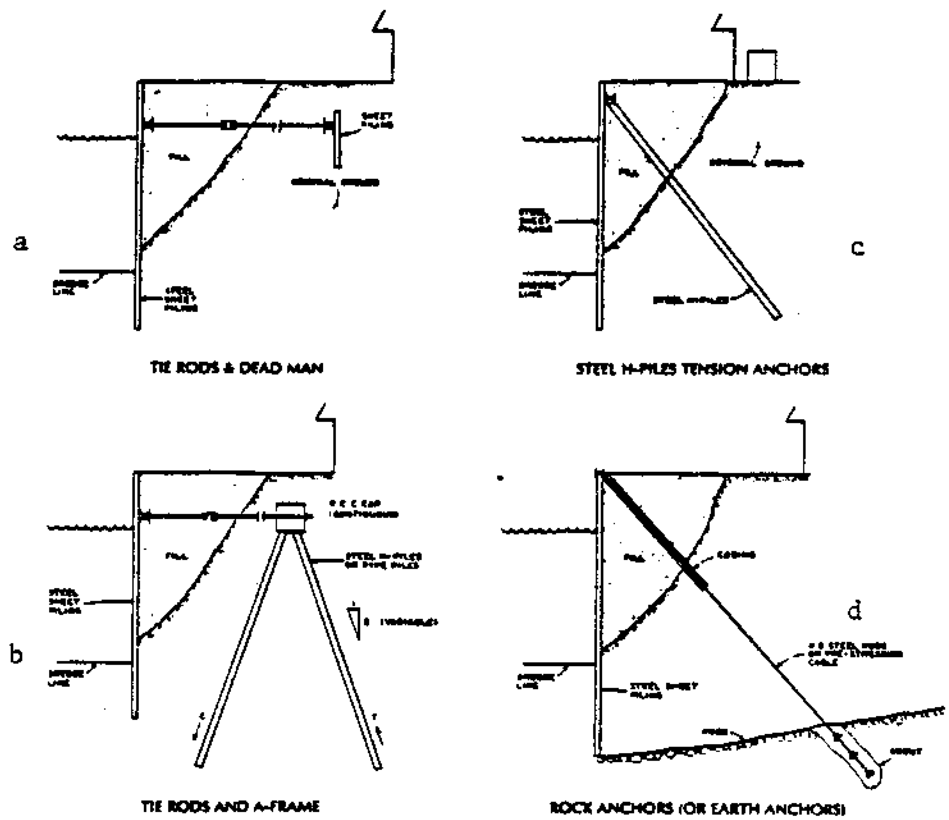


Figure 6-6. Alternative anchoring schemes (U.S. Steel, 1976, pp. 74-75)

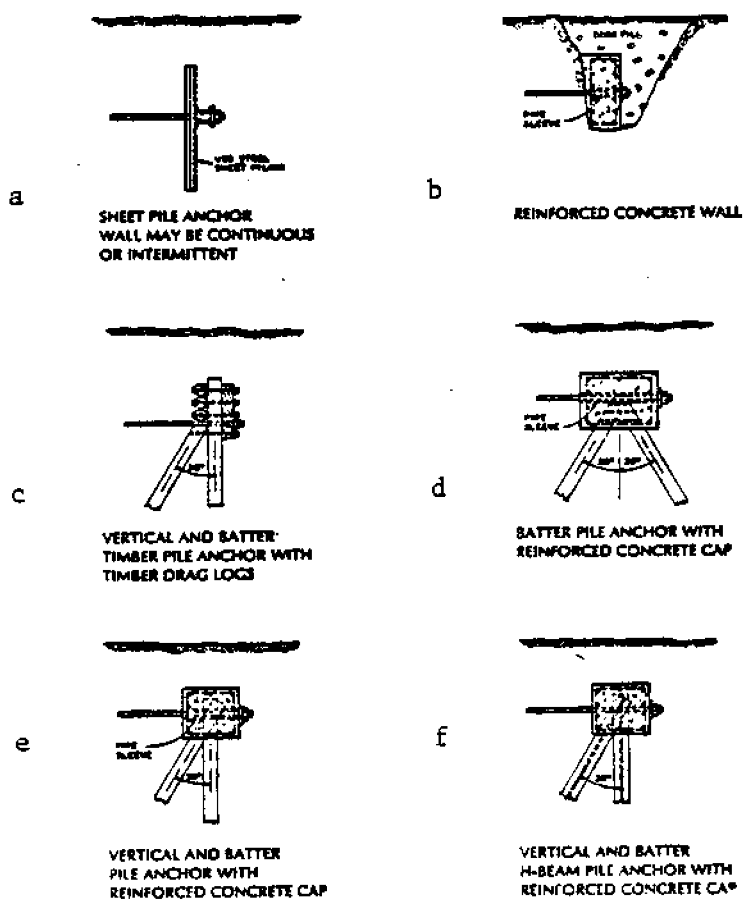


Figure 6-7. Alternative anchorages (U.S. Steel, 1976, p. 82)

in which θ = the angle between the tie-rod and the horizontal plane.

In corrosive environments the tie-rod should be protected by using galvanized steel and employing protective wraps, bituminous treatment or special painting.

Turnbuckles should be tightened until slack is removed from the tie-rods. Overtightening causes anchor yield and excess stresses in the tie-rod and sheet piling.

6.1.5. Tie-Rod Spacing

Tie-rods in wood bulkheads are frequently spaced at 7.5 ft (2.27 m) intervals. Construction details do not interfere with this spacing or any variation thereof. Steel bulkheads, on the other hand, limit the designer's flexibility in choosing the interval as pile sections differ in driving width (Table 5-2). For example, the section shown in Figure 6-5a is a PDA 27 with a width of 16 in (0.41 m) and tie-rods at every seventh section for an interval of 8 ft (2.44 m); Figure 6-5c shows a P238 pile with an 18 in (0.46 m) width and tie-rods at every seventh section for an interval of 9 ft (2.74 m).

The designer must be aware of these constraints because the tie-rod tension is a function of the spacing, as well as the computed pull per unit length of wall. An interval used for computations that is different from the interval permitted by the pile section configuration will result either in overdesigned, uneconomic tie-rods and wales, or a design prone to failure from overstressing.

6.1.6. Backfill and Dredging

Free-draining backfill material should be used. If the expense is too great to employ coarse material for the entire fill, a sand drain or sand blanket should be employed (Figure 5-4). If either of these is not feasible, then the additional load of saturated material must be considered, as well as the reduction of the effective depth of penetration because of hydrostatic imbalance (Section 5.2.3.).

The fill should be placed in equal lifts across the entire length of the bulkhead. Piling up the fill in one area results in local overstressing of pile members and tie-rods. The backfill should not be compacted as this increases the stresses beyond the designed values.

Dredging, if required, should be accomplished after backfilling is completed. The net result of this sequence is to provide additional reduction of the bending moment because of arching of soil between the tie-rod and dredge level.

6.1.7. Tightening of Nuts

For timber structures, the proper tightening tension is reached when washers begin to indent the adjacent timber. High strength bolts used for steel sheet piling are tightened in accordance with the Specification for Structural Joints using ASTM A325 or A490 bolts, Manual of Steel Construction (AISC, 1976).

6.2. Other Considerations

6.2.1. Construction Equipment

Bulkheads are often the first structures completed in new developments. This implies that construction activity will take place nearby. If this is anticipated, surcharges from heavy equipment must be accounted for in the design procedure or restrictions must be made as to the allowable proximity of the equipment. A horizontal distance equal to the wall height is recommended as the closest a piece of equipment may be allowed. If the tie-rod and anchorage are shallow, the equipment should not be allowed to pass over these.

6.2.2. Quality Assurance of Materials

To insure that materials are in compliance with design specifications, some measures need to be taken. The most fundamental step is an inspection of the material for obvious defects. If timber is the basic structural material, grademarks (Figure 5-13) should be found on the members which indicate the grade marking service and stress grade. A certificate is also available from the grading agency. Certificates of compliance may be requested from suppliers for assurance that the proper preservation process and amount was used. Certification may also be requested to insure compliance with the proper ASTM designations and any ordered special treatment such as bituminous coating.

6.2.3. Cutting and Notching

Treated timber members should not be cut to size. This practice subjects the cut ends to attack from the elements from which protection was desired. Preservation treatment should be specified as being applied to all surface areas of timber members.

A similar argument applies for notching or countersinking recesses for tie-rods to provide a flush face. In addition to limiting the effectiveness of preservatives, it reduces the net area of the section in terms of its effectiveness to carry a load. An alternative to this practice is to nail a coil of rope around the protruding tie-rod. This will offer the desired protection to the moored vehicles.

If any cutting is done, preservative should be post-applied at the site. This is not as effective as pressure treatment, but it is a vast improvement over leaving the cut unprotected.

6.2.4. Regulations Pertaining to Coastal Use

The use of coastal zones implies that some change in the environment will occur stemming from such use. Permission may be required prior to using coastal lands by the U.S. Army Corps of Engineers, Environmental Protective Agency, county or local governments. In New York State a Coastal Zone Management Program exists under the auspices of the Department of State, although regulatory functions are delegated to localities. At any rate, the structure's impact upon the environment must be assessed and the need to obtain permits must be ascertained. For details, see "Regulatory Processes in Coastal Structure Construction" (Ronan, 1979).

6.2.5. Construction Details

Typical construction details appear in Figures 6-8 through 6-12.

6.3. Summary

Although the construction of bulkheads is relatively straightforward some factors must be taken into account which may affect the desired performance of the system. Certain problems inherent in the installation of sheet piles can be overcome with some suggested techniques. Connection of wales and tie-rods and installation of the anchorage must be

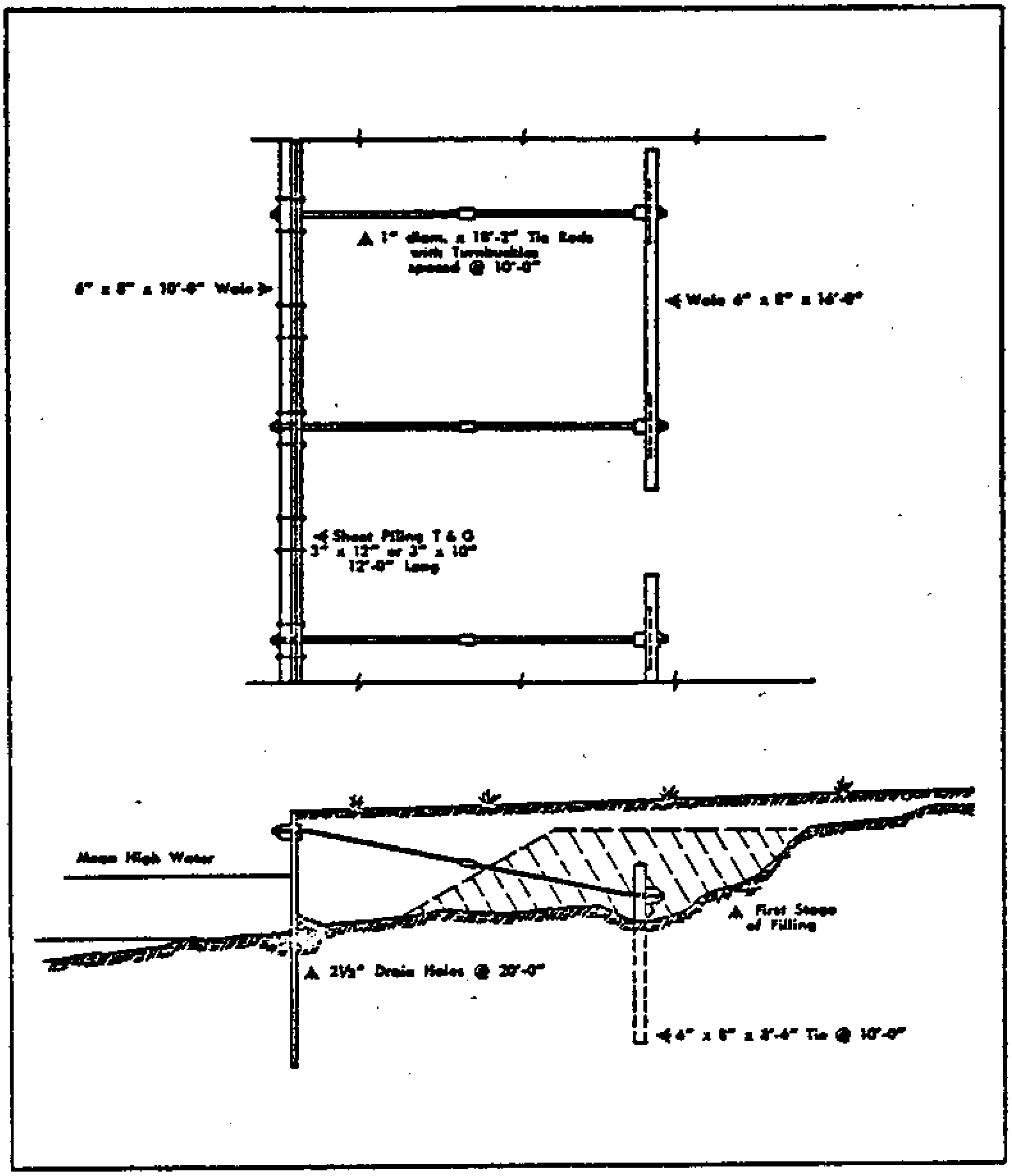


Figure 6-8. Typical bulkhead, wale outside (AWPI, p. 4)

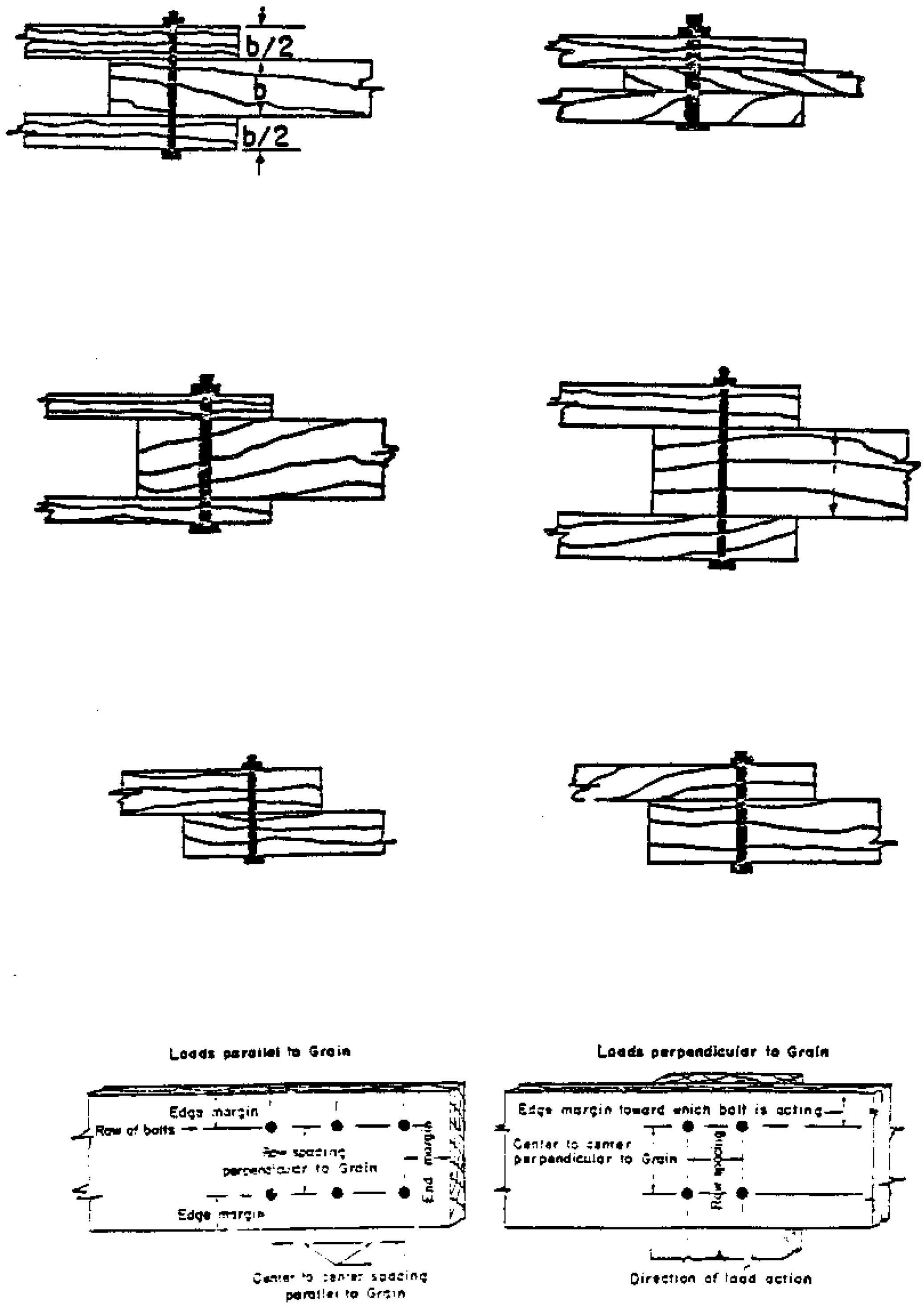


Figure 6-9. Typical bolting details, timber (Timber Engineering Co., 1956, pp. 511-513)

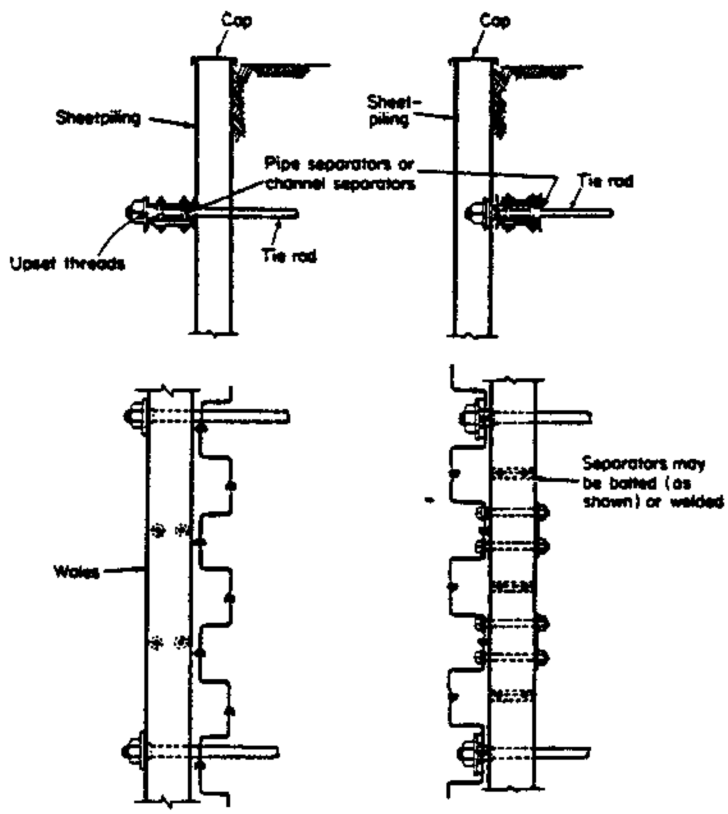


Figure 6-10. Common arrangement of wales and tie-rods (Teng, 1962, p. 372)

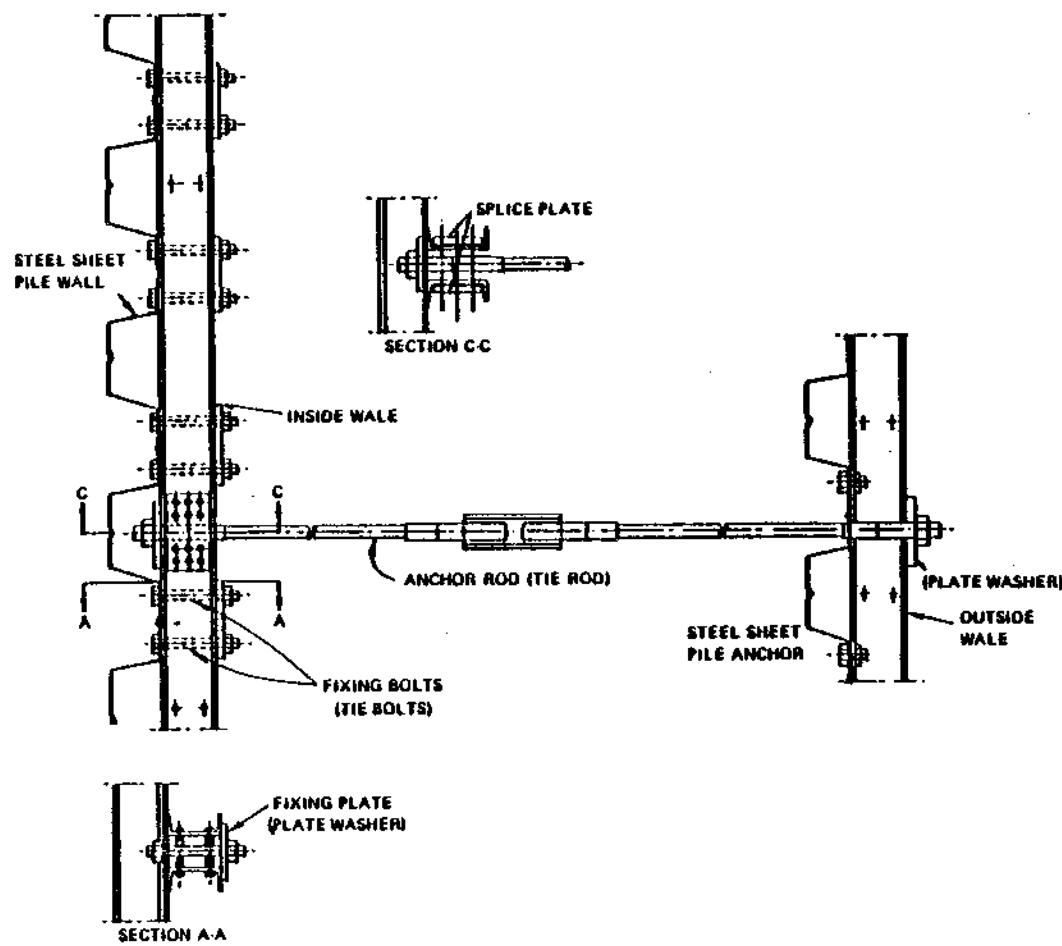


Figure 6-11. Typical wale and tie-rod details (U.S. Steel, 1975, p. 43)

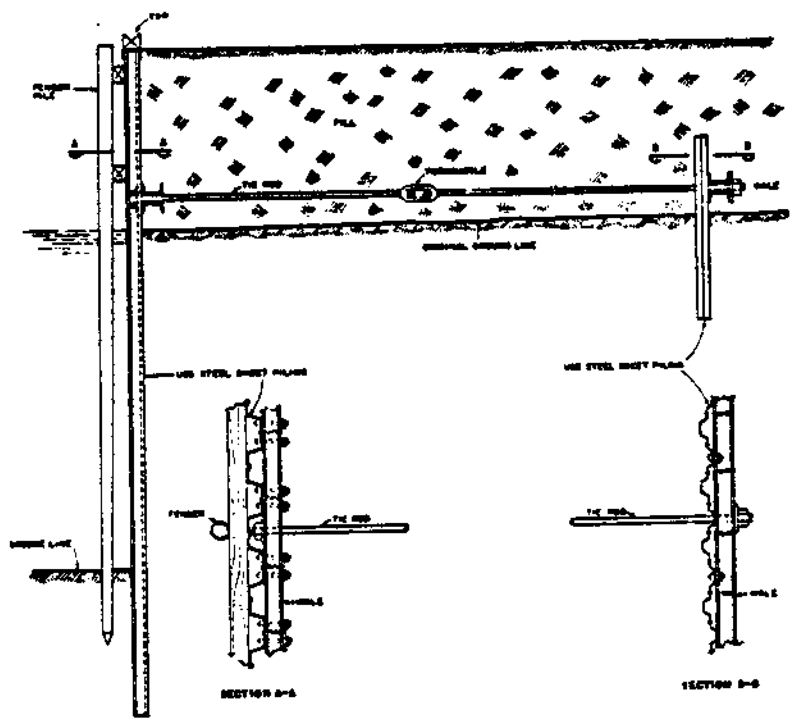


Figure 6-12. Steel bulkhead with timber fender piles (U.S. Steel, 1976, p. 74)

accomplished with respect to conditions imposed by the design. Benefits may accrue from the optimum sequencing of dredging and undesirable consequences may result in the improper placement of backfill. Surcharges imposed by construction equipment must be accounted for or damage to the system may occur. Measures should be taken to assure that the material purchased complies with the quality specified in the design. Field alterations performed on treated timber reduce the effectiveness of the preservative. Consideration of these factors during construction will enhance the longevity and proper functioning of the bulkhead.

CHAPTER 7

RELIABILITY AND FACTOR OF SAFETY

The chance of a system performing successfully is termed its reliability, R. The complement of reliability is the probability of failure, P_f, which is defined as

$$P_f = 1 - R \tag{7-1}$$

Every system has a finite probability of failure that depends upon: the system's ability to sustain loads, i.e., the capacity; the loads placed upon the system, i.e., the demand; and the variability of the capacity and demand.

Capacity-demand models involving penetration depth, tie-rod pull and bending moment for a particular hypothetical situation cannot be used to determine the probability of failure of all bulkhead systems. It can, however, suggest the order of magnitude of reliability to be expected, if realistic values and assumptions are chosen. A portion of this chapter is, therefore, dedicated to such a hypothetical situation where the reliability and factors of safety are explored.

The situation presented here is a bulkhead designed in accordance with Rowe's reduction method. Probabilistic methods are employed to determine the probability of failure of the design and some qualitative conclusions are drawn. Since the simplified design procedure suggested in this work is based on the Rowe method and some variability exists between the Rowe and simplified methods solutions, probabilistic methods are again utilized to investigate reliability.

7.1. Assumptions

Certain assumptions are inherent in the simplified design procedure and the argument presented in this chapter. A discussion of these assumptions should help to establish the validity of this work.

A very basic, yet critical, assumption is that the soil strength and unit weight are established by virtue of sufficient investigation. Some variability in these parameters can be expected and some variability will, consequently, occur in the loadings and the capacity to resist failure.

Variability in loadings caused by faulty construction procedure is not addressed.

As suggested in Chapters 2 and 3, the Free Earth Support and Rowe methods have been established as accurate means of describing bulkhead behavior. They have been corroborated by experiment and by comparison to theoretical and sophisticated analytical techniques. It can then be readily assumed that these methods can be modified to portray adequate capacity-demand models.

Some variability exists in the ultimate strengths of construction materials comprising bulkheads. It is suggested that the average factor of safety of stress graded timber is 2.5 and that 99 percent of all tests will demonstrate a minimum factor of safety of 1.25 (Timber Engineering Co., 1974). If a design value of 2,000 psi (13.8 MN/m^2) is assumed for the flexural strength of timber sheet piles composed of southern pine, the average ultimate strength can be assumed as 5,000 psi (34.4. MN/m^2) and 99 percent of the same material can be assumed to possess an ultimate strength of 2,500 psi (17.2 MN/m^2). Tie-rods made from grade A36 steel must possess a minimum yield strength of 36,000 psi (248 MN/m^2). The average yield strength of all A36 steel members is not known, but a conservative value may be assumed to be 40,000 psi (275 MN/m^2). It may also be assumed, conservatively speaking, that 99 percent of all A36 steel possesses at least the minimum required yield strength, 36,000 psi (248 MN/m^2).

Conservative assumptions are also made for selecting the appropriate mean value of soil parameters. The variabilities of these parameters reflect data taken from the technical literature. The random values chosen for soil and material parameters are assumed to be normally distributed and to represent infinite populations.

A hypothetical situation may be used to illustrate the factors of safety against penetration failure, tie-rod failure, and bending moment failure, and the associated probabilities of failure. With the factor of safety defined as the ratio of demand, D, to capacity, C, or

$$FS = \frac{C}{D} \tag{7-2}$$

then a factor of safety of unity or less signifies imminent failure, i.e., when the capacity is equal to the demand. The margin of safety, SM, is the difference of capacity and demand, or

$$SM = C - D \tag{7-3a}$$

Failure will occur when $SM \leq 0$.

The capacity and demand will vary depending upon many factors, such as material flaws, heterogeneity, etc., and are, therefore, termed variates. The value that occurs most frequently is termed the expected value, or mean, and a measure of the amount that values differ from the mean is termed the standard deviation.

If C and D are normal variates, then $\bar{C}$ and $\bar{D}$ are the means and S_C and S_D are the standard deviations. The mean safety margin may be defined as

$$\overline{SM} = \bar{C} - \bar{D}, \text{ and} \tag{7-3b}$$

the standard deviation of the safety margin may be defined as

$$S_{SM} = \sqrt{S_C^2 + S_D^2} \tag{7-3c}$$

A standardized value, z, is determined by

$$z = \frac{\overline{SM}}{S_{SM}} \tag{7-4}$$

From this value can be determined the probability that $\overline{SM} \leq 0$, or the probability of failure. Such a determination is made from probability density functions which may be found in statistical tables.

Capacity and demand for the three modes of failure previously mentioned will be analyzed statistically to find the mean and standard deviation of the safety margin. The standard score will then be determined and converted to the probability of failure.

7.2. Anchored Walls in Sand

7.2.1. Hypothetical Situation

A design will be illustrated for a bulkhead whose geometry is given in Figure 4-1, with the dimensions

H = 10' (3.05 m)

H_W = 6' (1.83 m)

H_A = 2' (0.61 m)

t_1 = 4' (1.22 m), and

t_2 = 6' (1.83 m)

The material comprising the fill and subgrade is loose sand. The mean values of the design parameters assigned to layer t_1 and t_2 are assumed as:

γ_1 = 100 pcf (15.8 kN/m^3)

ϕ_1 = 30 degrees .

γ_2 = 120 - 62.4

= 57.6 pcf (9.09 kN/m^3), and

ϕ_2 = 30 degrees

The design proceeded by calculating the depth of penetration by the Free Earth Support method and the tie-rod pull and bending moments by the Rowe reduction method. A factored angle of internal friction was used for computing the required depth of penetration, such that

$$\phi_f = \tan^{-1} \left(\frac{1}{SF} \tan \phi\right) \qquad (3\text{-}1)$$

in which SF = an appropriate safety factor, taken as 1.5, ϕ = angle of internal friction, unfactored, and ϕ_f = angle of internal friction, factored. The tie-rod diameter is then calculated based on an allowable tensile strength, f = 22,000 psi (151 MN/m^2). Finally, the sheet pile member thickness is selected based upon an allowable flexural stress of f = 2,000 psi (13.8 N/m^2). The resulting minimum parameters required are a penetration depth, D = 4.8 ft (1.46 m), tie-rod diameter, d = 0.68 in (17.2 mm), and sheet pile thickness, t = 1.81 in (46.0 mm).

Penetration depth stems from the demand found by summing moments about the tie-rod. The demand moment is from active stress applied against the wall. This motivating phenomena is computed as

$$\begin{aligned} M = {} & \frac{1}{2} K_{a1}\gamma_1 t_1^2 \left(\frac{2}{3} t_1 - H_A\right) \\ & + \frac{1}{2} K_{a2}\gamma_2 t_2^2 \left(\frac{2}{3} t_2 + t_1 - H_A\right) \\ & + \frac{1}{2} K_{a3}\gamma_3 D^2 \left(\frac{2}{3} D + H - H_A\right) \\ & + K_{a2}\gamma_1 t_1 t_2 \left(\frac{1}{2} t_2 + t_1 - H_A\right) \\ & + K_{a3} (\gamma_1 t_1 + \gamma_2 t_2) D \left(\frac{1}{2} D + H - H_A\right). \end{aligned} \tag{7-6a}$$

For the geometry of this situation and for $\gamma_2 = \gamma_3$, and $K_{a1} = K_{a2} = K_{a3} = K_a$,

$$M = K_a \left[(318)\ \gamma_1 + (517)\ \gamma_2\right] \tag{7-6b}$$

The capacity to resist this demand is provided by the moment about the tie-rod produced by the application of passive stress such that

$$R = \frac{1}{2} K_p \gamma_3 D^2 (\frac{2}{3} D + H - H_A), \text{ or} \tag{7-7a}$$

$$R = K_p \gamma_3 (121). \tag{7-7b}$$

The variability of a parameter, x, can be demonstrated in terms of its coefficient of variation

$$V = \frac{\bar{x}}{S_x} \cdot 100\% \tag{7-8}$$

in which $\bar{x}$ = the mean value of the parameter, and S_x = the standard deviation.

A correlation was found between variance of horizontal stress coefficients and the angle of internal friction (Singh, 1972), such that

$$V_{KA} = 1.15\ V\phi \text{ , and} \tag{7-9a}$$

$$V_{KP} = 1.10\ V\phi. \tag{7-9b}$$

For example, for an angle of internal friction, ϕ = 30 degrees, V_{KA} = 16.1 percent and V_{KP} = 15.4 percent.

The standard deviations associated with stress coefficients K_A = 0.279 and K_p = 5.74 are S_{KA} = 0.0449 and S_{KP} = 0.884 respectively.

Other pertinent parameters with variability are void ratio, e (Schultze, 1972), and specific gravity of the soil solids, G_S (Schultze, 1972; Padilla and Vanmarcke, 1974). Appropriate values assigned to these parameters are a mean void ratio of 0.663 with a standard deviation 0.088, and a mean specific gravity of 2.65 with a standard deviation of 0.01.

The relationship existing between the unit weight, void ratio, and specific gravity for saturated soil is

$$\gamma = \frac{(G_s + e)}{(1 + e)} \gamma_w \tag{7-10}$$

in which γ_w = unit weight of water.

A mechanism relating the variability of n independent parameters x_n to the dependent parameter y is (Hahn and Shapiro, 1967)

$$S_y^2 = \sum_{i=1}^{n} \left(\frac{\partial y}{\partial x_i}\right)^2 (S_{xi})^2 \tag{7-11}$$

Therefore, for the relationship between unit weight, void ratio and specific gravity

$$\frac{\partial \gamma}{\partial e} = \frac{(1 - G_s)}{(1 + e)^2} \gamma_w = -37.2,$$

$$\frac{\partial \gamma}{\partial G_s} = \frac{\gamma_w}{1+e} = 37.5,$$

$$S_\gamma^2 = \left(\frac{\partial \gamma}{\partial e}\right)^2 (S_e)^2 + \left(\frac{\partial \gamma}{\partial G_s}\right)^2 (S_{GS})^2, \text{ and}$$

$$S_\gamma = 3.30 \text{ lb/ft}^3 \ (0.521 \text{ kN/m}^3).$$

Using Equations 7-7 through 7-11 and the selected values, the means and standard deviations can be computed for the motivating moments, M, the resisting moments, R, and the probability of failure. The results are shown in Table 7-1.

Table 7-1. Probability of failure and factor of safety

Parameter	Penetration D (ft-lb)	Penetration C (ft-lb)	Tie-Rod Pull D (lb)	Tie-Rod Pull C (lb)	Bending Stress D (psi)	Bending Stress C (psi)
Mean	17,200	40,000	7,100	14,500	1,900	5,000
Standard Deviation	2,830	6,600	1,162	560	311	970
Standard Score	3.17		5.74		3.04	
Probability of Failure	8.00×10^{-4}		5.10^{-9}		1.20×10^{-3}	
Factor of Safety	2.33		2.04		2.63	

Note: 1 ft-lb = 1.356 N-m
1 lb = 0.00444 kN
1 psi = 0.00689 MN/m^2

Similar approaches can be taken with the tie-rod and bending stress demands. Tie-rod load is given by

$$T = K_a \, [(200) \, \gamma_1 + (9417) \, \gamma_2]. \tag{7-12}$$

Hence

$$S_T^2 = \left(\frac{\partial T}{\partial \gamma_1}\right)^2 (S_{\gamma_1})^2 + \left(\frac{\partial T}{\partial \gamma_2}\right)^2 (S_{\gamma_2})^2$$

$$+ \left(\frac{\partial T}{\partial K_a}\right)^2 (S_{K_a})^2, \text{ and}$$

$$S_T = 1162 \text{ lb } (5.16 \text{ kN}).$$

The maximum bending moment for this situation is given by

$$M_{MAX} = 5.88 \, P - K_a \, [(71.8) \, \gamma_1 + (12.6) \, \gamma_2)]$$

$$- K_a \, [(71.8) \, \gamma_1 + (9.74) \, \gamma_2] \tag{7-13b}$$

$$= K_a \, [(67.2) \, \gamma_1 + (50.8) \, \gamma_2). \tag{7-13c}$$

For a reduction factor in bending of 0.30 and section modulus of 6.55 in^3/ft in this case, the maximum bending stress is

$$\sigma = (0.304) \, (12) \, M_{MAX}/(6.55) \tag{7-14a}$$

$$= K_a \, [(47.5) \, \gamma_1 + (35.9) \, \gamma_2] \tag{7-14b}$$

The standard deviation for bending stress is given by

$$S_\sigma^2 = \left(\frac{\partial \sigma}{\partial \gamma_1}\right)^2 (S_{\gamma_1})^2 + \left(\frac{\partial \sigma}{\partial \gamma_2}\right)^2 (S_{\gamma_2})^2$$

$$+ \left(\frac{\partial \sigma}{\partial K_a}\right)^2 (S_{K_a})^2, \text{ and}$$

$S_\sigma = 311$ psi (2.14 MN/m^2).

As previously established, the mean flexural strength of wood sheet piles can be taken as 5,000 psi (34.4 MN/m^2) and mean yield strength of A36 steel can be taken as 40,000 psi (275 MN/m^2) so that

$$T_{ULT} = \frac{\pi}{4} d^2 f_y \tag{7-15}$$

$$= \frac{\pi}{4} (0.68)^2 (40{,}000)$$

$$= 14{,}500 \text{ lb } (64.4 \text{ kN}).$$

The standard deviations of the capacities can be found by back-calculation. Assumed cumulative probabilities of 99 percent associated with a minimum yield strength of 36,000 psi (248 MN/m^2) for A36 steel and a minimum flexural strength of 2,000 psi (13.8 MN/m^2) for timber sheet piles result in standard deviations of 560 lb (2.49 kN), for T_{ULT}, and 970 psi (6.68 MN/m^2) for σ.

The probability of failure in penetration depth, tie-rod pull and bending stress may now be computed using Equations 7-2 through 7-5. The results are given in Table 7-1.

7.2.2. Reliability of the Design Curves

The preceding hypothetical situation clearly demonstrates high reliability and comfortable factors of safety against failure for a 10 foot (3.05 m) wall in loose sand. One is able to surmise that similar results would occur in analyses of various geometries and soil conditions.

The same reliability might be expected from the design curves which comprise the basis for the simplified method as they were derived from the Rowe procedure. The design curves, however, do not coincide exactly with design solutions provided by the Rowe method, since the curves represent mean values of the solutions. The variabilities of the differences between the Rowe solutions and mean values of the design curves are demonstrated in Figures 3-4 through 3-15 and Table 3-5.

The variation of the design curves is expressed in terms of percent difference. This can be converted to the same units that express the variation in the hypothetical situation. Since the design curves are the result of a least squares method of best fit, the mean percent difference between the curve and the data points is very close to zero. The means of the design curves can thus be assumed to be equal to the means of the demand of the hypothetical situation, i.e., the mean percent difference between the curve and the demand of all hypothetical situations is zero. The standard deviations can be dimensionalized by multiplying the standard deviation, expressed as a percent, by the associated mean of the hypothetical situation. For example, a 10 percent standard deviation for tie-rod loads would convert to

$$S_T = (0.10)\ (7100)$$
$$= 710 \text{ lb } (3.16 \text{ kN}).$$

The reliability of the design curves, expressed in terms of the probability of failure, is shown in Table 7-2.

Table 7-2. Reliability of the design curves (anchored walls in sand)

Parameter	Penetration D (ft-lb)	Penetration C (ft-lb)	Tie-Rod Pull D (lb)	Tie-Rod Pull C (lb)	Bending Stress D (psi)	Bending Stress C (psi)
Mean	17,200	40,000	7,270	14,500	2,000	5,000
Standard Deviation	530	4,750	511	560	210	970
Standard Score	4.77		4.53		3.02	
Probability of Failure	$\sim 10^{-6}$		$\sim 10^{-22}$		1.30×10^{-3}	

Note: 1 ft-lb = 1.356 N-m
1 lb = 0.00444 kN
1 psi = 0.00689 MN/m^2

7.2.3. Reliability of the Simplified Method

The simplified method may be considered as a system consisting of 2 components: the Rowe reduction method and the design curves. The reliability of a system whose components operate in series may be expressed as

$$R_s = \prod_{i=1}^{n} R_i \tag{7-16}$$

in which R_i = the reliability of the i^{th} component and n = the number of components in the system. In terms of probability of failure, the relationship is

$$P_f = \prod_{i=1}^{n} 1 - (1 - P_i) \tag{7-17}$$

in which P_i = the probability of failure of the i^{th} component (Harr, 1977). The reliability of the simplified method may thus be assessed from the combinatorial probability of failure of its components as shown in Table 7-3.

7.3. Anchored Walls in Clay

7.3.1. Hypothetical Situation (Undrained)

The conditions assumed for anchored walls in sand remains the same with the exception of a cohesive subgrade where c = 250 psi (1.72 MN/m^2), an anchored wall in clay may be designed in accordance with the Rowe reduction method. The design depth of penetration, tie-rod pull, tie-rod diameter, bending stress and pile thickness are

Table 7-3. Reliability of the simplified method (anchored walls in sand

Parameter	Probability of Failure
Penetration	8.00×10^{-4}
Tie-Rod Pull	$<10^{-10}$
Bending Stress	2.50×10^{-3}

$D = 5.54$ feet (1.69 m)

$P = 6{,}530$ pounds (28.2 kN)

$d = 0.615$ inches (15.6 mm)

$\sigma = 1{,}990$ psi (13.7 Pa) and

$t = 1.92$ inches (48.8 mm)

The analysis proceeds as before with additions of another variant, the cohesion parameter, whose coefficient of variation may be taken as $V_c = 18.6$ percent (Lumb, 1972), which gives a standard distribution of $S_c = 46.5$. The resulting capacities, demands, standard scores and probabilities of failure are shown in Table 7-4.

The most striking aspect of the results is the relatively large probability of failure in penetration as compared to what is virtually a very substantial factor of safety. This disparity stems from the large variance of the cohesion parameter.

Coefficients of variation for the cohesion range as high as 50 percent (Harr, 1977). Incorporating this value into the foregoing analysis results in a probability of failure in penetration of $P_f = 0.25$.

7.3.2. Hypothetical Situation: Penetration Computed for Drained Condition

If the long-term case (drained condition) is considered, the design results in a depth of penetration $D = 9.2$ ft (2.8 m), factor of safety $FS = 2.2$ and probability of failure $P_f = 0.003$. This is based on the assumption that the variance of the parameters is the same as the variance for cohesionless soils. If this depth of penetration is

Table 7-4. Probability of failure, anchored walls in clay (undrained)

Parameter	Penetration D (ft-lb)	Penetration C (ft-lb)	Tie-Rod Load D (lb)	Tie-Rod Load C (lb)	Bending Stress D (psi)	Bending Stress C (psi)
Mean	5,230	30,500	6,530	11,500	1,990	5,000
Standard Deviation	920	13,800	1,070	450	320	970
Standard Score	1.83		4.45		2.95	
Probability of Failure	3.40×10^{-2}		10^{-6}		1.60×10^{-3}	
Factor of Safety	5.83		1.76		2.51	

Note: 1 ft-lb = 1.356 N-m
1 lb = 0.00444 kN
1 psi = 0.00689 MN/m^2

used to compute the probability of failure for the short term case, the probability of failure would be almost zero for a coefficient of variation of 18.6 percent on cohesion, and approximately 10^{-6} for a coefficient of variation of 50 percent.

7.4. Summary and Conclusions

The investigation of a hypothetical situation provided a conceptualization of the reliability of anchored bulkheads. By incorporating variations in the pertinent soil and material parameters found in the technical literature, a means was established whereby the probability of failure in penetration, tie-rod pull, and bending stress could be estimated.

A capacity-demand model was formulated for each of the three potential modes of failure for walls in a sand subgrade, in a clay subgrade under undrained conditions, and in a clay subgrade under drained conditions. Penetration failure was seen to be the most probable mode of failure while tie-rod failure was virtually improbable under the assumptions declared. The probability of flexural failure of timber members was less than penetration failure, but not nearly as low as tie-rod failure.

Recalling that the safety margin, variance in capacity and demand, and the probability of failure are related by

$$\overline{SM} = \bar{C} - \bar{D} \ , \tag{7-3a}$$

$$S_{SM} = \sqrt{S_c^2 - S_D^2} \ , \text{ and} \tag{7-3b}$$

$$P_f = (\frac{\overline{SM}}{S_{SM}}) \ , \tag{7-17}$$

the reasons for the general trend appear clear: a high safety margin results in a low probability of failure, while a high variance in either capacity or demand has the opposite effect.

Since the specified engineering properties of steel can be relatively easy to attain with low variance, steel products will show a rather high capacity. Added reliance stems from the fact that, to achieve the minimum yield for each lot manufactured, the metallurgical design process is conservative and an average yield results which is substantially higher than the required minimum. Rigid quality control insures that a very low percentage of the final product has a yield less than the specified minimum.

Since timber cannot be processed and refined to the extent that iron ore can, the final product exhibits more variability in its engineering properties. Designs using timber show high reliability which is derived from the quality assurance provided by stress grading.

Both demand and capacity of the penetration model are functions of the soil parameters and penetration depth. Since high variance in soil parameters pertains to both capacity and demand, a high safety margin is required to achieve an acceptable reliability. Obviously, increasing the safety margin may be accomplished by decreasing the demand or increasing the capacity. The only choices available to obtain either end are to replace the in-situ material with a more suitable one, or to increase the depth of penetration. Additional excavation and backfilling is costly, thus increasing the penetration depth is more attractive. Unfortunately, large increases in depth are necessary to offset high variability, low soil strength, or both.

Harr states that, "For most problems in geotechnical engineering, $P_f \leq 10^{-3}$" (Harr, 1977). It is not unreasonable therefore, to consider this order of magnitude as a desired standard and to declare as acceptable any probability of failure that is less than 0.01.

The numerical results of the analysis of the hypothetical situation demonstrate the acceptable reliability except for one case. The reliability of tie-rods and flexural member (sheet piles) are acceptable in all cases. Penetration depth, however, is unreliable for clays in the undrained condition, even for the moderate coefficient of variation of 18.6 percent. This realization is important as the apparent factor of safety against failure of 5.83 is very substantial and falsely suggests an adequate design. However, when the wall is redesigned for the drained condition, an acceptable reliability results for both long and short term.

The design curves possess small variability and show high reliability as a result. When considered as a component of a design system which incorporates the Free Earth Support method with Rowe reduction, the design curves lead to reliable designs providing, of course, that there is not excessive variability exhibited by the soil parameters.

The technical literature suggests that the undrained strength of cohesive soils demonstrates high variability. Deterministic designs based upon undrained strength produce an inherent risk of failure. Designs based upon drained strength, however, show good reliability; hence the drained condition can be considered to control the design process.

The reliability of a particular design can be estimated provided that the site was adequately investigated. One important aspect regarding the adequacy of the investigation is the number of data points used to determine the mean soil parameters. Since the investigation entails sampling from a population whose standard distribution is unknown, the desired probability of failure (confidence interval) may be investigated by utilizing a cumulative probability function described by a student distribution (Harr, 1977), where the standard score is given by

$$t = \frac{\overline{SM}}{S_{SM}} \tag{7-18}$$

A table is consulted to ascertain the probability of failure for a particular number of data points.

The t scores for a desired probability of failure less than 0.01 are shown in Table 7-5. It is readily observed that as the number of data points decreases, the t score increases. This indicates that for the desired reliability a greater safety margin, lower variance in soil parameters, or both, is required for fewer data points. The only option left to the designer confronted with scant data is to increase the safety margin. This is very likely to be less cost-effective than an increased scope in site investigation.

It may be concluded that the Free Earth Support, Rowe, and simplified methods are inherently reliable for walls in sand subgrades. To extend this high reliability to walls in cohesive subgrades, an adequate site investigation is required whose scope will be determined by the variability of the data.

Table 7-5. t Score required for a probability of failure less than 0.01

No. Data Points	t Score
3	31.821
4	6.965
5	4.451
6	3.747
7	3.365
8	3.143
9	2.998
10	2.896
11	2.821
12	2.764
13	2.718
14	2.681
15	2.650
16	2.624
17	2.602
18	2.583
19	2.567
20	2.552
21	2.539
22	2.528
23	2.518
24	2.508
25	2.500
26	2.492
27	2.485
28	2.479
29	2.473

CHAPTER 8

SUMMARY AND CONCLUSIONS

Bulkheads must be designed to resist failure from bending and from lack of sufficient penetration below the dredge level. The forces causing failure stem from horizontal stresses exerted upon the wall from the soil on the backfill side. Resistance to bending failure is derived from the properties of the wall, and outward movement of the toe of the wall is resisted by the soil on the dredge side. Required penetration depth may be reduced by employing a tie-rod and anchorage on the fill side, adequately dimensioned and located.

Bulkhead behavior is governed by the complicated interaction of many variables, requiring equally complex procedures to determine the design loads. Overly simplified methods tend to over- or under-design the system. A simplified procedure is needed which addresses the pertinent variables, and this is described herein.

Various approaches have been used to determine the horizontal stress distribution and the resultant forces and moments. Of the seven approaches reviewed in Chapter 2, the Free Earth Support method with Rowe reductions was found to be the most extensively examined and covered the widest range of conditions. In spite of its technical merit, the FES/Rowe procedure is complex. A simplified method was therefore derived from the more complicated one.

A computer program was devised which calculated penetration depth, moment and tie-rod load in accordance with the FES/Rowe method for a wide variety of soil conditions and site geometries. Chapter 3 explains the methodology by which the pertinent parameters were combined and correlated to generate simplified design curves.

A detailed explanation of the FES/Rowe and simplified methods is given in Chapter 4. The expediency of the simplified method is made apparent in that explanation and is substantiated by the procedural flow tables and design examples that appear in the Appendices.

Although the determination of penetration depth and loadings is of prime importance in bulkhead design, there are other items that require careful consideration to complete the design. Chapter 5 provides a discussion of other pertinent factors, i.e., overall system cost-effectiveness, external loads, component dimensioning and detailing. Procedural flow tables and examples are provided in the Appendices for the design of components.

Proper construction practices are also required for a properly functioning system. A general construction procedure is discussed in Chapter 6, as well as some other practical considerations concerning construction methods.

A qualitative description of bulkhead reliability was developed by inference in Chapter 7. A capacity-demand model of a typical bulkhead was examined with respect to penetration depth, moment, and tie-rod load. Both sand and clay subgrades were considered. Soil and material strength parameters and variability were selected from the technical literature and incorporated into the model. The models showed that, because of the high variability of clay strength parameters, walls in clay were less reliable than walls in sand. However, a design based upon the long-term strength of clay results in a reliable design, even when the short-term parameters are considered.

By examining the capacity-demand model using probabilistic methods, several concepts were reinforced, i.e., once an adequate penetration depth is found, the probability of system failure is low; the risk of penetration failure in a clay subgrade is high when considering short-term strength, but is reduced when the long-term strength is used for design; and as the number of data points used to determine the strength parameters of the soil increases, the probability of system failure decreases.

APPENDIX A

COMPUTER PROGRAM USER'S GUIDE

Title

Bulkhead Design for Anchored or Cantilevered Walls in Sand or Clay Subgrades.

Purpose

The purpose of this computer program is to determine the depth of penetration of bulkhead sheet-piles, determine the tie-rod load per unit length of wall, compute the maximum bending moment, and select the appropriate USS steel sheet pile and timber sheet pile. The design method is Free Earth Support as modified by Rowe.

Input

Cards 1 through 30 comprise moment and tie-rod reduction factors and USS steel sheet pile design data. These data cards are provided with the program.

Control Cards: 2 each. Must be right-justified.

Card 1

1-2	NP	- Number of designs to be run.

Card 2

1-2	KC	- Type of wall to be designed. KC = 0: Anchored wall only. KC = 1: Cantilevered wall only. KC = 2: Both types will be designed.
3-4	N	- Number of soil layers in the site. N must be 2 or greater.

Soil Parameter Cards: 1 card for each soil layer. English units. Not right or left-justified, but a decimal is required.

1-10	PHI - Angle of internal friction.
11-20	GAMMA - Total unit weight (lb/ft^3).
21-30	C - Cohesion ($\#/ft^2$). Must be zero if $\phi \neq 0$.

Site Geometry Cards: 2 cards

Card 1

1-10	BOMEGA	- Angle of backfill slope.
11-20	DOMEGA	- Angle of dredge slope.

Card 2

1-10	H	- Free standing wall height (ft).
11-20	HW	- Height of water above dredge level (DL). This is the low water level.
21-30	HHW	- Height of tie-rod above DL.
31-40	T1	- Distance from top of wall to 2nd soil layer.
41-50	T2	- Distance from top of wall to 3rd soil layer.
51-60	T3	- Distance from top of wall to 4th soil layer.

Surcharge Cards: 1 card

Card 1

1-10	QS	- Uniformly distributed load (lb/ft^2).
11-20	QL	- Line load (lb/ft).
21-30	QP	- Point load (lb).
31-40	X	- Horizontal distance from wall to load (for QL and QP only).

Explanation

Most sites can be approximated using 3 layers: the first layer consisting of moist (not saturated) soil between the top of the wall and the water level; the second layer extending to the DL; and the third layer extending beyond. Input of T3 = 50 ft is a good value since any distance beyond the depth of penetration will be neglected.

The field width for each soil layer is 10 spaces. Each additional soil layer may be input utilizing this width, e.g., T4 would be input using columns 61-70.

Values of zero must be input on soil parameter, site geometry and surcharge cards with a decimal point.

The use of cohesion parameters above the DL will result in unconservative designs. An explanation is contained in Chapter 3. Long term strength parameters should be used instead.

APPENDIX B

SOURCE PROGRAM

```
FILE: WALL     FORTRAN  A                CORNELL VM/SP SUBSET CMS LEVEL 104

C       FLEXIBLE RETAINING WALLS
        DIMENSION PHI(10),GAMMA(10),C(10),T(10),FAC(30,30),BET(6,12)
        REAL KA1,KP1
        REAL KP,KA,MOM
        DOUBLE PRECISION RAD,PI,PP
        DIMENSION KA1(10),KP1(10),C1(10),C2(10)
        DIMENSION KP(10),KA(10)
        DIMENSION Z(10),GAMMAE(10)
        DIMENSION DELTA(10),FT(10),FS(10),CT(10),CS(10),ASM(10)
        DIMENSION SZ(10)
        READ(5,38)((FAC(I,J),J=1,21),I=1,6)
        READ(5,36)((FAC(I,J),J=1,13),I=7,15 )
        READ(5,37)((FAC(I,J),J=1,9),I=16,23)
        READ(5,38)((BET(I,J),J=1,6),I=1,3)
        READ(5,35)((BET(I,J),J=1,6),I=4,6)
        READ(5,37)(SZ(I),I=1,8)
        DO 382 I=1,23
        KI=21
        IF(I.GE.7) KI=13
        IF(I.GE.16) KI=9
        DO 383 J=1,KI
        FAC(I,J)=FAC(I,J)/1000.
        IF(KI.EQ.13)FAC(I,J)=FAC(I,J)*10.
        IF(KI.EQ.9) FAC(I,J)=FAC(I,J)*10.
        IF(I.LT.16) GO TO 383
  383 CONTINUE
  382 CONTINUE
        DO 391 I=1,6
        DO 392 J=1,6
        BET(I,J)=BET(I,J)/100.
  392 CONTINUE
  391 CONTINUE
        DO 371 I=1,8
        SZ(I)=SZ(I)/10.
  371 CONTINUE
        PP=1.
        PI=DATAN(PP)
        PI=4.*PI
        RAD=180./PI
C
C       FAC= REDUCTION FACTORS (MOMENT)
C       BET=REDUCTION FACTORS (TIE ROD)
C       SZ=SECTION MODULII FOR STEEL SHEET PILES
C
  101 CONTINUE
        WRITE(11,1000)
        READ(5,1) NP
        DO 1111 II=1,NP
        READ(5,1)KC,N
        WRITE(6,1)KC,N
C
C       PHI=FRICTION ANGLE OF SOIL LAYER
C       GAMMA=UNIT WEIGHT OF SOIL LAYER
C       GAMMAE=EFFECTIVE UNIT  WT.(CALCULATED W/R TO WATER LEVEL)
C       BOMEGA=SLOPE OF GROUND SURFACE AT TOP OF WALL
```

```
FILE: WALL     FORTRAN  A                     CORNELL VM/SP SUBSET CMS LEVEL 104

C     DOMEGA=SLOPE OF GROUND SURFACE AT DREDGE LEVEL                          WAL00560
C                                                                             WAL00570
      READ (5,2)(PHI(I),GAMMA(I),C(I),I=1,N)                                  WAL00580
      READ(5,2) BOMEGA,DOMEGA                                                 WAL00590
C                                                                             WAL00600
C     H=FREE STANDING WALL HEIGHT                                             WAL00610
C     HHW=HIGH WATER LEVEL =TIE ROD LEVEL  (FROM DREDGE LEVEL)                WAL00620
C     HW=HEIGHT OF WATER (LOW WATER LEVEL)                                    WAL00630
C     HA=TIE ROD LEVEL (DEPTH FROM TOP OF WALL)                               WAL00640
C     D=DEPTH OF PENETRATION                                                  WAL00650
C     HD=H+D=TOTAL PILE LENGTH                                                WAL00660
C     T=DISTANCE FROM TOP OF WALL TO BOTTOM OF SOIL LAYER                     WAL00670
C     QS=EVENLY DISTRIBUTED SURCHARGE LOAD                                    WAL00680
C   QL=LINE LOAD                                                              WAL00690
C   QP=POINT LOAD                                                             WAL00700
C   X=HORIZONTAL DISTANCE FROM WALL TO LOAD                                   WAL00710
C                                                                             WAL00720
C                                                                             WAL00730
      PW=0.                                                                   WAL00740
      READ(5,2) H,HW,HHW,(T(I),I=1,N)                                         WAL00750
      READ(5,2) QS,QL,QP,X                                                    WAL00760
      WRITE(11,25)                                                            WAL00770
      WRITE(11,25)                                                            WAL00780
      DO 21 I=1,N                                                             WAL00790
      WRITE(11,22)I,T(I),GAMMA(I),PHI(I),C(I)                                 WAL00800
      PHI(I)=PHI(I)/RAD                                                       WAL00810
   21 CONTINUE                                                                WAL00820
      WRITE(11,100)                                                           WAL00830
      WRITE(11,24) H,HW,HHW,QS,QL,QP,BOMEGA,DOMEGA                            WAL00840
      BOMEGA=BOMEGA/RAD                                                       WAL00850
      DOMEGA=DOMEGA/RAD                                                       WAL00860
      WRITE(11,100)                                                           WAL00870
      WRITE(11,110)                                                           WAL00880
C                                                                             WAL00890
C     KC=0 : COMPUTES ANCHORED WALLS ONLY                                     WAL00900
C     KC=1 : COMPUTES CANTILEVERED WALLS ONLY                                 WAL00910
C     KC=2 : COMPUTES ANCHORED WALLS, THEN COMPUTES CANTILEVERED WALLS        WAL00920
C     KK=0 : FACTORED SOIL PARAMETERS                                         WAL00930
C     KL=0 : ANCHORED BULKHEAD                                                WAL00940
C     KL=1 : CANTILEVERED BULKHEAD                                            WAL00950
C     KM=0 : DEPTH OF PENETRATION ITERATION NOT COMMPLETED                    WAL00960
C     KO=0 : ALLOWS CALCULATION OF DEPTHS,ETC                                 WAL00970
C     KR=0 : NO COHESION IN ANY SOIL LAYERS                                   WAL00980
C                                                                             WAL00990
      KK=0                                                                    WAL01000
      KL=0                                                                    WAL01010
      IF(KC.EQ.1) KL=1                                                        WAL01020
      KR=0                                                                    WAL01030
      KM=0                                                                    WAL01040
   72 CONTINUE                                                                
      CALL FACTOR(KC,F,C,PHI,RAD,KR,KK,BOMEGA,DELTA,KP,KA,N,KL,KP1,KA1,DWAL01050
     COMEGA,C1,C2)                                                            WAL01060
C                                                                             WAL01070
C     HO=DISTANCE TO WATER LEVEL                                              WAL01080
C                                                                             WAL01090
      HO=H-HW                                                                 WAL01100
```

FILE: WALL   FORTRAN A   CORNELL VM/SP SUBSET CMS LEVEL 104

```
      SUM=0.
      D=H
      HD=H+D
      CALL DEPTH(T,Z,N,N3,M,K1,K2,K22,H,HD,HO)
      CALL PARAM(N,KR,Z,ST,T,KP,KA,DELTA,GAMMA,GAMMAE,C,N3,K1,H,K2,K22,
     CKP1,KA1,PHI,M,C1,C2,KK)
      IF(KR.EQ.0) GO TO 59
      CALL DEPTH(T,Z,N,N3,M,K1,K2,K22,H,HD,HO)
      CALL PARAM(N,KR,Z,ST,T,KP,KA,DELTA,GAMMA,GAMMAE,C,N3,K1,H,K2,K22,
     CKP1,KA1,PHI,M,C1,C2,KK)
      FOS=ST/.25
      WRITE(11,57) FOS,ST
      IF(FOS.LE.1.) WRITE(11,56)
      IF(FOS.LE.1.) WRITE(11,1000)
      IF(FOS.LE.1.) GO TO 1111
   59 CONTINUE
      HA=H-HHW
      KM1=0
  721 CONTINUE
      CALL FORCES(HA,KA,KP,C,GAMMAE,Z,FT,FS,CT,CS,KR,K1,K2,K22,M,KL,
     C KM,DELTA,MOM,DS,DL,DP,S,X,PH,EL,H,KK)
      IF(KR.EQ.1) GO TO 723
      IF(Z(M).EQ.H) KM1=1
      IF(KM1.EQ.1) GO TO 722
      IF(MOM.GT.0.) Z(M)=Z(M)-1.0
      IF(MOM.GT.0.) GO TO 721
      KM1=1
  722 CONTINUE
      IF(MOM.LT.0.)Z(M)=Z(M)+.10
      IF(MOM.LT.0.) GO TO 721
  723 HO=Z(M)
      D=HO-H
C     ID=D
C     D=ID+1
      HD=H+D
C
C     KM=1 : DEPTH OF PENETRATION ITERATION COMPLETED
C
      KM=1
      KK=1
      CALL FORCES(HA,KA,KP,C,GAMMAE,Z,FT,FS,CT,CS,KR,K1,K2,K22,M,KL,
     C KM,DELTA,MOM,DS,DL,DP,S,X,PH,EL,H,KK)
      CALL FACTOR(KC,R,C,PHI,RAD,KR,KK,BOMEGA,DELTA,KP,KA,N,KL,KP1,KA1,O
     COMEGA,C1,C2)
      CALL DEPTH(T,Z,N,N3,M,K1,K2,K22,H,HD,HO)
      CALL PARAM(N,KR,Z,ST,T,KP,KA,DELTA,GAMMA,GAMMAE,C,N3,K1,H,K2,K22,
     CKP1,KA1,PHI,M,C1,C2,KK)
      IF(KR.EQ.0) GO TO 59
      CALL DEPTH(T,Z,N,N3,M,K1,K2,K22,H,HD,HO)
      CALL PARAM(N,KR,Z,ST,T,KP,KA,DELTA,GAMMA,GAMMAE,C,N3,K1,H,K2,K22,
     CKP1,KA1,PHI,M,C1,C2,KK)
   59 CONTINUE
      CALL FORCES(HA,KA,KP,C,GAMMAE,Z,FT,FS,CT,CS,KR,K1,K2,K22,M,KL,
     C KM,DELTA,MOM,DS,DL,DP,S,X,PH,EL,H,KK)
      WRITE(11,1000)
```

FILE: WALL   FORTRAN A   CORNELL VM/SP SUBSET CMS LEVEL 104

```
   75 CONTINUE
      ALPHA=H/HO
      BETA=HA/HO
      WRITE(11,69) D
   69 FORMAT(T55,'DEPTH OF PENETRATION = ',F10.2,' FT')
      WRITE(11,1010)
      IF(KL.EQ.1) HA=0.
      IF(KL.EQ.1) PULL=0.
      IF(KL.EQ.1) V=0.
      IF(KL.EQ.1) GO TO 77
      CALL TIE(APM,HO,Z,CT,CS,FT,FS,V,M,HA,PULL,PH,EL,H,KK)
      PULL=V
   77 CONTINUE
      CALL DESMOM(Z,CT,CS,FT,FS,M,MOM,HA,V,C,PH,EL,H,KK)
      MOM=ABS(MOM)
      CALL ROWE(MOM,PHI,FAC,BET,SZ,PULL,ALPHA,BETA,EI,H,ST,KL,KR,RAO,Z1,
     CZ2,Z3,HO)
      IF(KC.EQ.2) GO TO 1111
      IF(KC.EQ.1) GO TO 1111
      IF(KL.EQ.1) GO TO 1111
      KL=1
      KK=0
      KM=0
      D=H
      WRITE(11,1010)
      WRITE(11,111)
      GO TO 72
 1111 CONTINUE
      STOP
    1 FORMAT(2I2)
    2 FORMAT(2F10.2)
   22 FORMAT(T35,I2,T38,4F15.3)
   24 FORMAT(T55,'WALL HT        = ',T71,F5.2,' FT ',/,T55,'LOW WTR        =
     C',T71,F5.2,' FT'
     C,/,T55,'TIE ROD        = ',T71,F5.2,' FT',
     C/,T55,'SURCHARGE      = ',T71,F10.0,' PSF (DISTRIBUTED LOAD)',
     C/,T55,'SURCHARGE      = ',T71,F10.0,' PLF (LINE LOAD)',
     C/,T55,'SURCHARGE      = ',T71,F10.0,' POUNDS (POINT LOAD)',
     C/,T55,'FILL SLOPE     = ',T71,F5.2,'DEGREES',
     C/T55,'DREDGE SLOPE   = ',T71,F5.2,' DEGREES')
   25 FORMAT(T25, 'SOIL LAYER DEPTHS ARE FROM GROUND SURFACE... WALL HT
     C & WTR LEVELS ARE FROM DREDGE LINE',
     C/,T10,'__________________________________________________________
     C_____________________________________________________________')
   26 FORMAT(/,T31,'SOIL LAYER',T48,'DEPTH',T61,'UNIT WEIGHT',T77,'PHI'
     C,T90,'COHESION',
     C/,T48,'(FT)',T62,'(PCF)',T75,'(DEG)',T92,'(PSF)')
   35 FORMAT( 6F3.0)
   36 FORMAT(10F3.0)
   37 FORMAT(9F3.0)
   38 FORMAT(21F3.0)
   39 FORMAT(18F3.0)
   57 FORMAT(     T43,'SAFETY FACTOR =',F5.2,'      AGAINST STAB = ',F5.2
     C)
   58 FORMAT(T43,'THIS WALL CANNOT STAND')
```

FILE: WALL   FORTRAN A   CORNELL VM/SP SUBSET CMS LEVEL 104

```
  110 FORMAT( T37,' ANCHORED BULKHEAD')
  111 FORMAT( T53,' CANTILEVERED BULKHEAD')
 1000 FORMAT(T11,'--------------------------------------------------------
     C---------------------------------------------------------------')
      END
      SUBROUTINE FACTOR (KC,R,C,PHI,RAD,KR,KK,BOMEGA,DELTA,KP,KA,N,KL,
     C KP1,KA1,DOMEGA,C1,C2)
C
C     SUBROUTINE TO FACTOR SOIL  PARAMETERS FOR PENETRATION
C     CALCULATIONS, UNFACTOR FOR MOMENT CALCULATIONS
C
      REAL KA1,KP1
      REAL KP,KA
      DIMENSION KA1(10),KP1(10),C1(10),C2(10),PH1(10),PH2(10)
      DIMENSION C(10),PHI(10),DELTA(10),KP(10),KA(10)
      DOUBLE PRECISION RAD,PP,PI
      DO 34 I=1,N
      IF(C(I).GT.0.) KP=1
      IF(KK.EQ.0) C2(I)=C(I)
      IF(KK.EQ.0) C1(I)=C(I)/1.5
      IF(KK.EQ.0) C(I)=C1(I)
      IF(KK.EQ.1) C(I)=C2(I)
   34 CONTINUE
      DO 3 I=1,N
      IF(KR.EQ.1.AND.KK.EQ.1) GO TO 3
      IF(KK.EQ.1) GO TO 31
      IF(KC.EQ.2.AND.KL.EQ.1) GO TO 31
      IF(KK.EQ.0) PH2(I)=PHI(I)
      PH1(I)=TAN(PHI(I))/1.5
      PH1(I)=ATAN(PH1(I))
   31 CONTINUE
      PHI(I)=PH2(I)
      IF(KK.EQ.0) PHI(I)=PH1(I)
      IF(KR.EQ.1) PHI(I)=PH2(I)
      DELTA(I)=2.*PHI(I)/3.
      IF(PHI(I).EQ.0.) GO TO 33
      A=COS(PHI(I))
      A=A**2.
      DELT=DELTA(I)
      A1=(SIN(PHI(I)+DELT ))*(SIN(PHI(I)-BOMEGA))
      A2=(COS(DELT ))*(COS(BOMEGA))
      AA2=(COS(DELT ))*(COS(DOMEGA))
      A3=(SIN(PHI(I)+DELT ))*(SIN (PHI(I)+BOMEGA))
      AA3=(SIN(PHI(I)+DELT ))*(SIN (PHI(I)+DOMEGA))
      A4=SQRT(A1/A2)
      A5=(1.+A4)**2
      KA(I)=A/A5
      A4=-SQRT(AA3/AA2)
      A5=(1.+A4)**2
      KP(I)=A/A5
   33 CONTINUE
      IF(PHI(I).EQ.0.) KP(I)=1.
      IF(PHI(I).EQ.0.) KA(I)=1.
      KP1(I)=KP(I)
      KA1(I)=KA(I)
```

FILE: WALL FORTRAN A CORNELL VM/SP SUBSET CMS LEVEL 104

```
    7 CONTINUE
      WRITE(11,1000)
      IF(KK.EQ.0) WRITE(11,310)
      IF(KK.EQ.1) WRITE(11,320)
      IF(KR.EQ.1.AND.KK.EQ.0) WRITE(11,330)
  310 FORMAT(T34,'FACTORED SOIL PARAMETERS')
  320 FORMAT(T34,'UNFACTORED SOIL PARAMETERS')
  330 FORMAT ( T34,'(FACTORED COHESION PARAMETR ONLY)')
 1000 FORMAT(T17,'______________________________________________________')
C
      RETURN
      END
      SUBROUTINE DEPTH(T,Z,N,N3,M,K1,K2,K22,H,HO,HD)
C
C     SUBROUTINE TO INTEGRATE WATER, DREDGE AND PILE TIP LEVELS WITH
C     SOIL INTERFACE LEVELS
C
      DIMENSION Z(10),T(10)
      N1=N+1
      N2=N+2
      N3=N+3
      T(N1)=HO
      T(N2)=H
      T(N3)=HD
      DO 4 J=1,N3
    4 Z(J)=T(J)
      DO 41 J=1,N3
      AMIN=Z(J)
      K=J
      DO 40 I=J,N3
      IF(Z(I).GE.AMIN) GO TO 40
      K=I
      AMIN=Z(K)
   40 CONTINUE
      IF(K.EQ.J) GO TO 41
      TEMP=Z(K)
      Z(K)=Z(J)
      Z(J)=TEMP
   41 CONTINUE
C
C     ELIMINATE ANY DUPLICATIONS IN IDNET. LAYER INTERFACES
C     IDENT. INTIGER VARIABLES ASSOC'D W/ INTFC. @ WATER LVL.,D.L.,
C     AND PILE TIP (HO,H,HD)
C
      K=1
      DO 42 I=1,N3
      J=I-1
      IF(Z(I).EQ.0.) GO TO 42
      IF(I.EQ.1) GO TO 43
      IF(Z(I).EQ.Z(J)) GO TO 42
      IF(Z(I).GT.H.AND.Z(I).NE.HD) GO TO 42
   43 Z(K)=Z(I)
      IF(Z(I).EQ.HO) K1=K
      IF(Z(K).EQ.H) K2=K
      IF(Z(K).EQ.HD) M=K
```

```
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      IF(Z(K).EQ.HD) GO TO 44
      K=K+1
   42 CONTINUE
   44 CONTINUE
      RETURN
      END
      SUBROUTINE PARAM(N,KP,Z,ST,T,KP,KA,DELTA,GAMMA,GAMMAE,C,N3,K1,M,K2
     C,K22,KP1,KA1,PHI,P,C1,C2,KK)
      REAL KA1,KP1
      REAL KP,KA
      DIMENSION T(10),Z(10),KP(10),KA(10),DELTA(10),GAMMA(10),GAMMAE(10)
     C ,C(10),C1(10),C2(10)
      DIMENSION KA1(10),KP1(10),PHI(10)
      J=1
      ST=0.
      S=0.
      DO 5 I=1,M
   53 CONTINUE
      IF(J.GT.N) GO TO 5
      IF(Z(I).LE.T(J)) GO TO 52
      J=J+1
      GO TO 53
   52 CONTINUE
      IF(Z(I).NE.H) GO TO 54
      K2=I
      K22=K2+1
   54 CONTINUE
      KP(I)=KP(J)
      KA(I)=KA(J)
      DELTA(I)=DELTA(J)
      KP(I)=KP1(J)
      KA(I)=KA1(J)
      GAMMAE(I)=GAMMA(J)
      IF(KK.EQ.0)C(I)=C1(J)
      IF(KK.EQ.1)C(I)=C2(J)
      PHI(I)=PHI(J)
      IF(I.GT.K1) GAMMAE(I)=GAMMAE(I)-62.4
      IF(KR.EQ.0) GO TO 5
      IF(Z(I).GT.H) GO TO 51
      X=Z(I)-S
      ST=ST+GAMMAE(I)*X
      IF(I.EQ.1) ST1=ST
   51 CONTINUE
    5 S=Z(I)
      IF(KR.EQ.1)ST=C(M)*1.25/ST
      RETURN
      END
      SUBROUTINE FORCES(HA,KA,KP,C,GAMMAE,Z,FT,FS,CT,CS,KR,K1,K2,K22,
     C M,KL,KM,DELTA,MOM,QS,QL,DP,D,X,PH,EL,H,KK)
C
C     SUBROUTINE TO CALCULATE PRESSURES, FORCES, MOMENTS, CENTROIDS
C     AND MOMENT ARMS FOR EACH SOIL LAYER
C
      REAL KP,KA,MOM
      DIMENSION KA(10),KP(10),ARM(10)
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```
      DIMENSION FT(10),FZ(10),CT(10),CS(10),DELTA(10)
      DIMENSION Z(10),T(10),GAMMA(10),GAMMAE(10),C(10)
      HD=Z(M)
      MOM=0.
      Q=0.
      IF(QL.LT.1.) GO TO 65
      EM=X/H
      IF(EM.LE.0.3) EL=(0.6)*H
      IF(EM.GT.0.3.AND.EM.LE.0.5)  EL=(0.56)*H
      IF(EM.GT.0.5) EL=(0.48)*H
      PH=-(0.64)*QL/(EM**2+1.)
      IF(EM.LE.0.4) PH=-(0.55)*QL
      PMOM=PH*(H-HA-EL)
      MOM=MOM+PMOM
      IF(KK.EQ.1) GO TO 65
      IF(KM.EQ.1) WRITE(11,651) PH,EL,PMOM
  651 FORMAT(T55,'RESULTANT OF LINE LOAD = ',F10.2,'PLF',/
     C,T55,'ACTING ',F5.2,'FT FROM DREDGE LEVEL',/
     C,T55,'CONTRIBUTING MOMENT = ',F10.2,'  FT-LB')
      Q=Q+PH*TAN(DELTA(2))
   65 CONTINUE
      IF(QP.LT.1.) GO TO 66
      EM=X/H
      IF(EM.LE.0.4) EL=(0.59)*H
      IF(EM.GT.0.4) EL=(0.45)*H
      PH=-(0.45)*QP/H
      IF(EM.LE.0.4) PH=-(0.78)*QP/H
      PMOM=PH*(H-HA-EL)
      MOM=MOM+PMOM
      IF(KK.EQ.1) GO TO 66
      IF(KM.EQ.1) WRITE(11,661) PH,EKM,PMOM
  661 FORMAT(T55,'RESULTANT OF POINT LOAD = ',F10.2,'POUNDS',/
     C,T55,'ACTING ',F5.2,'FT FROM DREDGE LEVEL',/
     C,T55,'CONTRIBUTING MOMENT = ',F10.2,'  FT-LB')
      Q=Q+PH*TAN(DELTA(2))
   66 CONTINUE
      S=0.
      R=1.25
      APT=0.
      APS=QS
      ST=0.
      IF(KM.EQ.1) WRITE(11,63)
      DO 5 I=1,M
      IF(I.LE.K2) KP(I)=0.
      IF(I.LE.K22) PPS=0.
      CR=C(I)*R
      APT=GAMMAE(I)*(Z(I)-S)
      A=APS+APT
      CR2=CR*2.
      IF(KM.EQ.0.AND.KR.EQ.1.AND.I.EQ.M) A=2.*CR2-APT
      IF(KM.EQ.0.AND.KR.EQ.1.AND.I.EQ.M) B=A*(Z(K2)-HA)
      IF(KM.EQ.0.AND.KR.EQ.1.AND.I.EQ.M) A=A/2.
      IF(KM.EQ.0.AND.KR.EQ.1.AND.I.EQ.M) D=(-B+SQRT(B**2-4.*A*MOM))/(2.*
     CA)
      IF(KM.EQ.0.AND.KR.EQ.1.AND.I.EQ.M) Z(M)=Z(K2)+D
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      FT(I)=.5*(KP(I)-KA(I))*(APT)*(Z(I)-S)
      FS(I)=(KP(I)*(PPS+CR2)-KA(I)*(APS-CR2))*(Z(I)-S)
      IF(CR2.GE.A) FT(I)=0.
      IF(I.NE.M.AND.CR2.GE.A) FS(I)=0.
      IF(I.NE.M.AND.CR2.GT.APS) FS(I)=0.
      CT(I)=(Z(I)-S)*2./3.
      CS(I)=CT(I)*3./4.
      ARM(I)=0-HA
      IF(KL.EQ.1) ARM(I)=Z(M)-Z(I)
      CTT=CT(I)+ARM(I)
      IF(KL.EQ.1) CTT=CT(I)/2.+ARM(I)
      CSS=CS(I)+ARM(I)
      FSM=CSS*FS(I)
      FTM=CTT*FT(I)
      MOM=MOM+FTM+FSM
      Q=Q+(FT(I)+FS(I))*TAN(DELTA(I))
      IF(KM.EQ.1)
     CWRITE(11,64)Z(I),KP(I),KA(I),GAMMAE(I),CR2,APS,APT,FS(I),FT(I),
     C CS(I),CT(I),CSS,CTT,FSM,FTM
      APS=APS+APT
      PPS=PPS+APT
      S=Z(I)
    6 CONTINUE
      Q=ABS(Q)
      Q=(Q+22.*HO)*TAN(DELTA(M))
      MOM=MOM+Q*CTT
   63 FORMAT(T3,'Z',T8,'KP',T13,'KA',T16,'GAMMA',T23,'2CP',
     CT31,'OVERBURDEN STRESS',
     C T60,'FORCES',T80,'CENTROIDS',T99,'MOMENT ARMS',T118,'MOMENTS',/,
     CT31,'REC',T42,'TRI',T52,'REC',T62,'TRI',T72,'REC
     C',T82,'TRI',T92,'REC',T102,'TRI',T112,'REC',T122,'TRI')
   64 FORMAT(1X,F4.1,2F5.2,2F5.0,F9.0,10F10.2)
      RETURN
      END
      SUBROUTINE TIE(ARM,HO,Z,CT,CS,FT,FS,Q,M,HA,PULL,PH,EL,H,KK)
C
C  TIE ROD PULL ... SUM MOMENTS ABOUT PT. OF PASS. FORCE APPLICATION
C
      DIMENSION FT(10),FS(10),CT(10),CS(10),ARM(10),Z(10)
      REAL MOM
      MOM=0.
      HO=Z(M)
      DO 8 I=1,M
      ARM(I)=HO-Z(I)-CT(M)/2.
      IF(FT(M).EQ..0) ARM(I)=HO-Z(I)-CS(M)
      CTT=CT(I)/2.+ARM(I)
      CSS=CS(I)+ARM(I)
      FTM=FT(I)*CTT
      FSM=FS(I)*CSS
      MOM=MOM+FTM+FSM
    8 CONTINUE
      IF(PH.EQ.0.) GO TO 65
      PMOM=PH*(EL+CT(M))
      IF(FT(M).EQ.0.) PMOM=PH*(EL+CS(M))
      MOM=MOM+PMOM
```

```
   65 CONTINUE
      IF(PH.EQ.0.) GO TO 66
      PMOM=PH*(EL+CT(M))
      IF(FT(M).EQ.0.) PMOM=PH*(EL+CS(M))
      MOM=MOM+PMOM
   66 CONTINUE
      PULL=MOM/(HA+CT(M)/2.-HD)
      IF(FT(M).EQ.0.) PULL=MOM/(HA+CS(M)-HD)
      SA=HA
      V=PULL
      WRITE(11,75) PULL
      WRITE(11,1000)
   75 FORMAT(T55,'TIE ROD PULL = ',F9.0,' LB/FT')
 1000 FORMAT(T10,'____________________________________________________
     *')
    C____________________________________________________________
      RETURN
      END
      SUBROUTINE DESMOM(Z,CT,CS,FT,FS,M,MOM,HA,V,C,PH,EL,H,KK)
C
C     SUBROUTINE TO FIND POINT OF ZERO SHEAR AND COMPUT MAXIMUM
C     MOMENT AT THAT POINT
C
      DIMENSION Z(10),CT(10),CS(10),FS(10),FT(10),APM(10),C(10)
      REAL KP,KA,MOM
           MOM=0.
      G=V
      S=0.
      DO 81 I=1,M
      V=V+FT(I)+FS(I)
      IF(PH.EQ.0.) GO TO 87
      XEL=H-EL
      IF(XEL.LT.0.OR.XEL.GT.Z(I)) GO TO 87
      V=V+PH
      MOM=MOM+PH*(XEL-HA)
   87 CONTINUE
      IF(S.LT.HA) GO TO 81
      IF(G.EQ.0..AND.I.EQ.1) GO TO 81
      V=V-FT(I)-FS(I)
      Y=Z(I)-S
      IF(Y.EQ.0.) V=V+FT(I)+FS(I)
      IF(Y.EQ.0.) GO TO 81
      F1=FT(I)/Y**2
      F2=FS(I)/Y
      IF(F2.EQ.0..AND.V.EQ.0.) V=V+FT(I)+FS(I)
      IF(F2.EQ.0..AND.V.EQ.0.) GO TO 81
      IF(F1.EQ.0..AND.F2.EQ.0.) GO TO 81
      F3=F2**2-4.*F1*V
      IF(F3.LT.0.) V=V+FT(I)+FS(I)
      IF(F3.LT.0.) GO TO 81
      IF(F1.EQ.0.) B=V/F2
      IF(F1.EQ.0.) GO TO 85
      B=(-F2+SQRT(F3))/(2.*F1)
      A=(-F2-SQRT(F3))/(2.*F1)
      IF(A.GT.B) B=A
   86 CONTINUE
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      X=I+B
      IF(B.GT.Z(I).OR.B.LT.0.) V=V+FT(I)+FS(I)
      IF(B.GT.Z(I).OR.B.LT.0.) GO TO 81
      GO TO 83
   81 S=Z(I)
   83 CONTINUE
      I1=I-1
      IF(FT(M).EQ.0..AND.FS(M).EQ.0.) X=MA
      IF(FT(M).EQ.0..AND.FS(M).EQ.0.) B=0.
      M1=M-1
      MOM=MOM+G*(X-MA)+(F1/3.)*B**3+(F2/2.)*B**2
      IF(I.EQ.1) GO TO 68
      DO 67 I=1,I1
      ARM(I)=X-Z(I)
      CTT=CT(I)/2.+ARM(I)
      CSS=CS(I)+ARM(I)
      FSM=CSS*FS(I)
      FTM=CTT*FT(I)
      MOM=MOM+FTM+FSM
   67 CONTINUE
   68 CONTINUE
   54 CONTINUE
      WRITE(11,85) X ,MOM
      WRITE(11,1000)
   85 FORMAT(T25,'ZERO SHR @ X = ',F4.0,'FT BELOW GROUND SURFACE...
     CMAXIMUM MOMENT = ',F10.2,'FT-LBS/FT')
 1000 FORMAT(T10,'                                                   ')
     C
      RETURN
      END
      SUBROUTINE FORC(MOM,PHI,FAC,BET,SZ,PULL,ALPHA,BETA,EI,M,ST,KL,KR,R
     CAD,Z1,Z2,Z3,HO)
C
C     SUBROUTINE TO GENERATE OPERATING AND STRUCTURAL CURVES
C     AND CALCULATE DESIGN MOMENTS, SECTIONS FOR STEEL AND WOOD PILES
C     AND TIE ROD LOADS
C
      DOUBLE PRECISION SLOPE, YINT,A(50),B(50),RAD
      DIMENSION FAC(30,30),BET(6,12),TO(30),TS(30),RP(30)
      DIMENSION PHI(10),SZ(10)
      REAL MOM,MO1,MO2,MO3,MO
C
C     ALLOWABLE STRESS FOR TIE ROD = 25000 PSI
C
      SATIE=25000.
      TMAX=MOM*12./HO**3
      P1=30./RAD
      P2=40./RAD
      IF(PHI(M).GT.P2) PHI(M)=P2
      IF(PHI(M).LT.P1) PHI(M)=P1
      IF(KR.EQ.0.AND.KL.EQ.0) CALL SAND(FAC,BET,ALPHA,BETA,TMAX,PULL,PHI
     C,M,TO,P1,P2,FC)
      IF(KR.EQ.1.AND.KL.EQ.0) CALL CLAY(FAC,BET,ALPHA,BETA,TMAX,PULL,ST,
     CTO,SLOPE,YINT,YI,SL)
      IF(KR.EQ.0.AND.KL.EQ.1) CALL CANT(FAC,ALPHA,TMAX,TO,PHI,P,P1,
```

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```
     C*2)
      IF(KL.EQ.1) PULL=0.
      IF(KL.EQ.1) FC=0.
      DO 9 IM=1,3
      IM=0
      IF(IN.EQ.1) SA=25000.
      IF(IM.EQ.2) SA=33500.
      IF(IN.EQ.3) SA=2000.
      IF(IN.EQ.1) E=3.*10.**7
      IF(IM.EQ.2) E=3.*10.**7
      IF(IN.EQ.3) E=1.5*10.**6
   99 CONTINUE
      IM=IM+1
      IF(IN.EQ.1) WRITE(11,2)
      IF(IN.EQ.2) WRITE(11,3)
      IF(IN.EQ.3) WRITE(11,4)
      IF(IM.EQ.1) WRITE(11,5)
      IF(IM.EQ.2) WRITE(11,6)
      IF(KR.EQ.1.AND.KL.EQ.1) GO TO 18
      IF(KL.EQ.1) KX=11
      IF(KL.EQ.3) KX=26
      DO 72 K=1,KX
      RHO=K
      IF(KR.EQ.1.AND.IN.EQ.1.AND.IM.EQ.1) TO(K)=YINT+RHO**SLOPE
      IF(KX.EQ.11) RHO=(-.25*RHO)+.75
      IF(KX.EQ.26) RHO=(-RHO*.1)+0.9
      RP(K)=RHO
      RHO=10.**(RHO)
      HR=(HO*RHO**2)**(1./3.)
      PSI=0.39
      IF(IN.EQ.2) PSI=PSI*(50000.)/(38500.)
      IF(IN.EQ.3) PSI=2.*SA/(E**(2./3.))
      IF(IM.EQ.2)  PSI=SA*SZ(I)/(E*EI)**(2./3.)
      TO(K)=PSI/HR
   72 CONTINUE
      IF(IN.NE.1.OR.IM.NE.1) GO TO 7
      DO 71 II=1,KX
      IF(KX.EQ.26)J=22-II
      IF(KX.EQ.26)K=J+5
      IF(KX.EQ.11)J=9-II
      IF(KX.EQ.11)K=J+3
      L=K+1
      IF(K.LE.0) GO TO 7
      IF(KX.EQ.26.AND.K.GT.5) TO(K)=TO(J)
      IF(KX.EQ.11.AND.K.GT.3) TO(K)=TO(J)
      IF(KX.EQ.26.AND.K.LE.5) TO(K)=TO(L)
      IF(KX.EQ.11.AND.K.LE.3) TO(K)=TO(L)
   71 CONTINUE
    7 CONTINUE
      IF(TO(1).LT.TS(1)) TAU=TMAX
      IF(TO(1).LT.TS(1)) FC=1.0
      IF(TO(1).LT.TS(1)) GO TO 18
      DO 73 I=1,KX
      KI=KX+1
      IF(I.EQ.KX) TO(KI)=TO(KX)
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```
      IF(I.EQ.KX) TS(KI)=TS(KX)
      I1=I+1
      X2=I1
      X1=I
      Y1=TO(I)
      Y2=TO(I1)
      Y3=TS(I)
      Y4=TS(I1)
      IF(KX.EQ.26.AND.TO(KX).GT.TS(KX)) TAU=TO(26)
      IF(KX.EQ.26.AND.TO(KX).GT.TS(KX)) XP=26
      IF(KX.EQ.26.AND.TO(KX).GT.TS(KX)) GO TO 74
      CALL POI(X1,X2,Y1,Y2,Y3,Y4,XP,YP)
      IF(XP.LT.X1.OR.XP.GT.X2) GO TO 73
      TAU=YP
      GO TO 74
   73 CONTINUE
   74 CONTINUE
      IF(KP.EQ.1) PC=YI*XF**SL
   18 CONTINUE
      IF(KR.EQ.1.AND.KL.EQ.1) TAU=TMAX
      IF(KR.EQ.1.AND.KL.EQ.1) GO TO 19
      WRITE(11,1)( RP(I),TO(I),TS(I),I=1,KX)
      WRITE(11,11) TAU
      WRITE(11,1200)
   19 MOM=TAU*MO**3
      IF(IN.EQ.1) MO1=MOM
      IF(IN.EQ.2) MO2=MOM
      IF(IN.EQ.3) MO3=MOM
      IF(KL.EQ.1) PULL=0.
C
C     ALLOWABLE STRESS = YIELD POINT STRESS X 0.65
C
      IF(IN.EQ.1) Z=MOM/(TA*0.65)
      IF(IN.EQ.2) Z=MOM/(SA*0.65)
      IF(IN.EQ.3) Z=MOM/(SA)
      IF(IN.EQ.1) Z1=Z
      IF(IN.EQ.2) Z2=Z
      IF(IN.EQ.3) Z3= Z
      IF(IN.EQ.3) GO TO 80
      DO 8 I=1,10
      I1=I+1
      IF(Z.GT.SZ(1)) WRITE(11,900)
      IF(Z.GT.SZ(1))  GO TO 800
      IF(I.EQ.8) GO TO 80
      IF(Z.GT.SZ(I1).AND.Z.LT.SZ(I)) GO TO 80
    8 CONTINUE
   80 CONTINUE
      IF(IN.EQ.3) I=9
      IF(IN.EQ.3) SZ(I)=SQRT(Z/2.)
      IF(IN.EQ.3) IM=2
      IF(I.EQ.1) EI=230.8
      IF(I.EQ.2) EI=220.4
      IF(I.EQ.3) EI=194.2
      IF(I.EQ.4) EI=39.8
      IF(I.EQ.5) EI=13.7
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                                                                      WAL07160
      IF(I.EQ.6) EI=4.1                                               WAL07170
      IF(I.EQ.7) EI=3.7                                               WAL07180
      IF(I.EQ.8) EI= 2.8                                              WAL07190
      IF(I.EQ.9) EI=TZ(I)                                             WAL07200
      IF(IN.EQ.1) EI1=EI                                              WAL07210
      IF(IN.EQ.2) EI2=EI                                              WAL07220
      IF(IN.EQ.3) EI3=EI                                              WAL07230
      IF(IN.EQ.1) K1=I                                                WAL07240
      IF(IN.EQ.2) K2=I                                                WAL07250
      IF(IN.EQ.3) K3=I                                                WAL07260
      IF(IN.EQ.1) GO TO 99                                            WAL07270
    9 CONTINUE                                                        WAL07280
      MOM=MOM/12.                                                     WAL07290
      PULL=PULL*FC                                                    WAL07300
      WRITE(11,10[illegible]) Z1,Z2,Z3                                WAL07310
      WRITE(11,1000)                                                  WAL07320
      DO 101 IN=1,3                                                   WAL07330
      IF(IN.EQ.1) WRITE(11,[illegible])                               WAL07340
      IF(IN.EQ.1) MO=MO1                                              WAL07350
      IF(IN.EQ.2) MO=MO2                                              WAL07360
      IF(IN.EQ.3) MO=MO3                                              WAL07370
      IF(IN.EQ.1) EI=EI1                                              WAL07380
      IF(IN.EQ.2) EI=EI2                                              WAL07390
      IF(IN.EQ.3) EI=EI3                                              WAL07400
      IF(IN.EQ.1) I=K1                                                WAL07410
      IF(IN.EQ.2) I=K2                                                WAL07420
      IF(IN.EQ.3) I=K3                                                WAL07430
      IF(IN.EQ.1.AND.I.EQ.1) WRITE(11,81)  SZ(I),EI,MO,PULL           WAL07440
      IF(IN.EQ.1.AND.I.EQ.2) WRITE(11,82)  SZ(I),EI,MO,PULL           WAL07450
      IF(IN.EQ.1.AND.I.EQ.3) WRITE(11,83)  SZ(I),EI,MO,PULL           WAL07460
      IF(IN.EQ.1.AND.I.EQ.4) WRITE(11,84)  SZ(I),EI,MO,PULL           WAL07470
      IF(IN.EQ.1.AND.I.EQ.5) WRITE(11,85)  SZ(I),EI,MO,PULL           WAL07480
      IF(IN.EQ.1.AND.I.EQ.6) WRITE(11,86)  SZ(I),EI,MC,PULL           WAL07490
      IF(IN.EQ.1.AND.I.EQ.7) WRITE(11,87)  SZ(I),EI,MO,PULL           WAL07500
      IF(IN.EQ.1.AND.I.EQ.8) WRITE(11,88)  SZ(I),EI,MO,PULL           WAL07510
      IF(IN.EQ.2.AND.I.EQ.1) WRITE(11,91)  SZ(I),EI,MC,PULL           WAL07520
      IF(IN.EQ.2.AND.I.EQ.2) WRITE(11,92)  SZ(I),EI,MO,PULL           WAL07530
      IF(IN.EQ.2.AND.I.EQ.7) WRITE(11,97)  SZ(I),EI,MO,PULL           WAL07540
      IF(IN.EQ.2.AND.I.EQ.3) WRITE(11,93)  SZ(I),EI,MO,PULL           WAL07550
      IF(IN.EQ.2.AND.I.EQ.4) WRITE(11,94)  SZ(I),EI,MO,PULL           WAL07560
      IF(IN.EQ.2.AND.I.EQ.5) WRITE(11,95)  SZ(I),EI,MC,FULL           WAL07570
      IF(IN.EQ.2.AND.I.EQ.6) WRITE(11,96)  SZ(I),EI,MC,FULL           WAL07580
      IF(IN.EQ.2.AND.I.EQ.8) WRITE(11,98)  SZ(I),EI,MO,PULL           WAL07590
      IF(IN.EQ.3) WRITE(11,99)             SZ(I),EI,MG,PULL           WAL07600
  101 CONTINUE                                                        WAL07610
      WRITE(11,1200)                                                  WAL07620
  300 CONTINUE                                                        WAL07630
      RETURN                                                          WAL07640
    1 FORMAT(T42,'LOG RHO = ',F10.3,'   TO = ',F10.3,'  TS = ',F10.3) WAL07650
    2 FORMAT(T45,'A328 STEEL OPERATING AND STRUCTURAL CURVES')        WAL07660
    3 FORMAT(T45,'A572/69- STEEL OPERATING AND STRUCTURAL CURVES')    WAL07670
    4 FORMAT(T45,'WOOD OPERATING AND STRUCTURAL CURVES')              WAL07680
    5 FORMAT(T60,'TYPICAL SECTION')                                   WAL07690
    6 FORMAT(T60,'SPECIFIC SECTION')                                  WAL07700
   11 FORMAT(T60,'TAU =  ',F10.3)
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```
 81 FORMAT(T28,'PZ 38 ',T38,'A328'       ,T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
 82 FORMAT(T29,'PZ 32 ',T38,'A328'       ,T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
 83 FORMAT(T28,'PZ 27 ',T38,'A328'       ,T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
 84 FORMAT(T28,'POA 27',T38,'A328'       ,T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
 85 FORMAT(T28,'PMA 22',T38,'A328'       ,T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
 86 FORMAT(T28,'PSA 23',T38,'A328'       ,T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
 87 FORMAT(T28,'PSX 32',T38,'A328'       ,T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
 88 FORMAT(T28,'PS 28 ',T38,'A328'       ,T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
 89 FORMAT(T28,'WOOD PILE THICKNESS (IN)',  T55,F5.2,T65,F5.1,T75,F9
   C.0,
   CT90,F6.3)
 90 FORMAT(T28,'SECTION',T38,'MATERIAL',T55,'SEC MOD',T65,'M/INERT',
   CT76,'DES/MOM',T93,' TIE-ROD PULL',/,T55,'(IN-3)',T65,'(IN-4)',
   CT75,' (IN-LB/FT)',T90,'(LB/FT)')
 91 FORMAT(T28,'PZ 38 ',T38,'A 572/690',T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
 92 FORMAT(T29,'PZ 32 ',T38,'A 572/690',T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
 93 FORMAT(T28,'PZ 27 ',T38,'A 572/690',T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
 94 FORMAT(T28,'POA 27',T38,'A 572/690',T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
 95 FORMAT(T28,'PMA 22',T38,'A 572/690',T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
 96 FORMAT(T28,'PSA 23',T38,'A 572/690',T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
 97 FORMAT(T28,'PSX 32',T38,'A 572/690',T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
 98 FORMAT(T28,'PS 28 ',T38,'A 572/690',T55,F5.1,T65,F5.1,T75,F9.0,
   CT90,F6.0)
100 FORMAT(T41,'CALCULATED SECTION MODULI : ',T55,
   CF10.3,' IN-3 (A328)',/,T65,
   CF10.3,' IN-3 (A572/690)',/,T65,
   CF10.3,' IN-3 (WOOD)')
900 FORMAT(T50,'NO FLEXIBLE WALL IS SUITABLE UNDER THESE CONDITONS')
1000 FORMAT(T10,'______________________________________________________')
C______________________________________________________________________
      END
      SUBROUTINE FIT(A,B,SLOPE,YINT,K)
C______________________________________________________________________
C     PROGRAM TO FIT DATA POINTS TO A POWER CURVE
C______________________________________________________________________
      DOUBLE PRECISION X,Y,SLOPE,YINT,A(50),B(50),C(50),D(50)
      M=K
      SUMX=0.
      SUMY=0.
      SUM2X=0.
```

```
FILE: WALL      FORTRAN  A             CORNELL VM/SP SUBSET CMS LEVEL 104

      SUM2Y=0.
      SUMXY=0.
C
C     PERFORM REGRESSION ANALYSIS ON NAT. LOG. OF DATA POINTS
C     USE GAUSSIAN ELIMINATION
C
      DO 2 I=1,M
      X=DLOG(A(I))
      Y=DLOG(B(I))
      SUMX=SUMX+X
      SUMY=SUMY+Y
      SUMXY=SUMXY+X*Y
      SUM2X=SUM2X+X**2
      SUM2Y=SUM2Y+Y**2
      XBAR=SUMX/M
      YBAR=SUMY/M
    2 CONTINUE
      VARX=SUM2X/M-XBAR**2
      VARY=SUM2Y/M-YBAR**2
      SIGMAX=SQRT(VARX*M/(M-1))
      SIGMAY=SQRT(VARY*M/(M-1))
      SLOPE=(SUMX*SUMY/M-SUMXY)/(SUMX**2/M-SUM2X)
      CORR= SLOPE*SIGMAX/SIGMAY
      YINT=YBAR-SLOPE*XBAR
      WRITE(11,3) CORR
    3 FORMAT(T53,'CORRELATION = ',F10.3)
      YINT=DEXP(YINT)
C
C     SLOPE OF LOG-LOG CURVE = POWER OF DESIRED FUNCTION
C     THE Y-INT. OF THE LOG-LOG CURVE RAISED AS A POWER OF NAT. EXP.
C     GIVES THE COEFF. OF DESIRED FUNCTION
C
      DO 4 I=1,M
      C(I)=    (YINT)*A(I)**(SLOPE)
      D(I)= (I)-B(I)
    4 CONTINUE
      WRITE(11,5)
      WRITE(11,6)(A(I),B(I),C(I),D(I),I=1,M)
      WRITE(11,1000)
    5 FORMAT(T40,'DATA POINTS',T61,'ORD. OF POWER FUNC.',T85,'DIFFERENCE
     C',/,T41,'X',T51,'Y',T66,'Y (FITTED)',T85,'DELTA Y')
    6 FORMAT(T34 ,F10.4,T44,F10.4,T62,F10.4,T80,F10.4)
 1000 FORMAT(T10,'                                                        ')
    C
      RETURN
      END
      SUBROUTINE POI(X1,X2,Y1,Y2,Y3,Y4,XP,YP)
C
C
C     SUBROUTINE TO FIND POINT OF INTERSECTION OF TWO LINES
C
      S1=(Y2-Y1)/(X2-X1)
      B1=-S1*X1+Y1
      S2=(Y4-Y3)/(X2-X1)
      B2=-S2*X1+Y3
```

FILE: WALL FORTRAN A CORNELL VM/SP SUBSET CMS LEVEL 104

```
      XP=(B2-B1)/(S1-S2)
      YP=S2*XP+B2
      RETURN
      END
      SUBROUTINE SAND (FAC,BET,ALPHA,BETA,TMAX,PULL,PHI,M,TO,P1,P2,FC)
C
C     SUBROUTINE TO INTERPOLATE DATA FROM  REDUCTION CURVES FOR SAND
C
      DIMENSION FAC(30,30),BET(6,12),TO(30)
      DIMENSION PHI(10)
      SATIE=25000.
      IF(ALPHA.GT..7) GO TO 2
      IF(ALPHA.GT..6) GO TO 3
      IF(ALPHA.GT..5) GO TO 3
    2 F=(ALPHA-.7)/.1
      J=1
      J1=2
      L=4
      L1=5
      GO TO 7
    3 F=(ALPHA-.6)/.1
      J=2
      J1=3
      L=5
      L1=6
      GO TO 7
    7 CONTINUE
      DO 71 K=1,21
      C=F*(FAC(J,K)-FAC(J1,K))+FAC(J1,K)
      D=F*(FAC(L,K)-FAC(L1,K))+FAC(L1,K)
      R=PHI(M)-P1
      R=R/(P2-P1)
      R=R*(D-C)+C
      TO(K)=TMAX*R
C
C     TIE ROD REDUCTIONS (WALLS IN SAND)
C
      JK=BETA*10.+1.
      B=JK
      B=B-1.
      B=B/10.
      IF(JK.NE.K) GO TO 71
      IF(K.GT.6) GO TO 71
      V=F*(BET(J,K)-BET(J1,K))+BET(J1,K)
      K1=K+1
      IF(K.EQ.6) K1=6
      W=F*(BET(J,K1)-BET(J1,K1))+BET(J1,K1)
      FC=(BETA-B)*(W-V)+V
   71 CONTINUE
      RETURN
      END
      SUBROUTINE CLAY (FAC,BET,ALPHA,BETA,TMAX,PULL,ST,TO,SLOPE,YINT,YI,
     CCL)
C
C     SUBROUTINE TO INTERPOLATE DATA FROM REDUCTION CURVES FOR CLAY
```

```
C
      DOUBLE PRECISION SLOPE, YINT,A(50),B(50)
      DIMENSION FAC(30,30),BET(6,12),TO(50)
      K=0
      DO 5 I=4,6
      DO 6 J=1,6
      J1=J-1
      IF(J.EQ.1) J1=1
      S=J1
      S=S+0.4
      S1=J
      S1=S1+0.4
      IF(ST.GT.2..AND.J.EQ.6) GO TO 1
      IF(J.EQ.1) S=.25
      IF(ST.LT.S.OR.ST.GT.S1) GO TO 6
    1 K=K+1
      IF(K.EQ.1) A(K)=17.
      IF(K.EQ.2) A(K)=12.
      IF(K.EQ.3) A(K)=6.
      P=ST-S
      Q=S1-S
      B(K)=P/Q  *(BET(I,J)-BET(I,J1))+BET(I,J)
    6 CONTINUE
    5 CONTINUE
      WRITE(11,300)
      CALL FIT(A,B,SLOPE,YINT,K)
      YI=YINT
      SL=SLOPE
      IF(ALPHA.GT..7) GO TO 2
      IF(ALPHA.GT..6) GO TO 3
    2 F=(ALPHA-.7)/.1
      J=1
      J1=2
      L=7
      L1=8
      GO TO 4
    3 F=(ALPHA-.6)/.1
      J=2
      J1=3
      L=8
      L1=9
    4 CONTINUE
      DO 10 I=1,10
C
C     MANIPULATIONS TO ACCOUNT FOR PRONOUNCED CURVATURE OF RODS
C
      I1=I+1
      S=I-1
      S1=I
      S=S+0.25
      S1=S1+0.25
      IF(I.EQ.1) S=0.25
      IF(I1.EQ.2) S1=0.375
      IF(I.EQ.2) S=0.375
      IF(ST.GT.2.5.AND.I.EQ.6) GO TO 12
```

```
FILE: WALL     FORTRAN  A                 CORNELL VM/SP SUBSET CMS LEVEL 104

      IF(ST.GT.31) GO TO 10
   12 K=0
      DO 13 J=7,15
      IF(L.NE.J) GO TO 13
      P=ST-S
      Q=S1-S
      C=P/Q    *(FAC(L,I1)-FAC(L  ,I))+FAC(L  ,I)
      D=P/Q    *(FAC(L1,I1)-FAC(L1,I))+FAC(L1,I)
      R=F*(C-D)+D
      L=L+3
      L1=L1+3
      K=K+1
      TO(K)=R*TMAX
      IF(K.EQ.1) A(K)=17.
      IF(K.EQ.2) A(K)=12.
      IF(K.EQ.3) A(K)=6.
      B(K)=TO(K)
   13 CONTINUE
      GO TO 11
   10 CONTINUE
   11 CONTINUE
      WRITE(11,200)
      CALL FIT(A,B,SLOPE,YINT,K)
  200 FORMAT(T50,'CURVE-FITTING FOR PILE MEMBERS',/)
  300 FORMAT(T50,'CURVE-FITTING FOR TIE-ROD',/)
      RETURN
      END
      SUBROUTINE CANT(FAC,ALPHA,TMAX,TO,PHI,M,P1,P2)
C
C     SUBROUTINE TO INTERPOLATE DATA FROM REDUCTION CURVES FOR
C     CANTILEVERED WALLS
C
      DIMENSION FAC(30,30),BET(6,12),TO(30)
      DIMENSION PHI(10)
      IF(ALPHA.GT..6) GO TO 4
      IF(ALPHA.GT..5) GO TO 5
      IF(ALPHA.GT..4) GO TO 6
    4 F=(ALPHA-.6)/.1
      J=16
      J1=17
      L=20
      L1=21
      GO TO 7
    5 F=(ALPHA-.5)/.1
      J=17
      J1=18
      L=21
      L1=22
      GO TO 7
    6 F=(ALPHA-.4)/.1
      J=18
      J1=19
      L=22
      L1=23
    7 CONTINUE
```

```
FILE: WALL     FORTRAN  A                 CORNELL VM/SP SUBSET CMS LEVEL 104

      DO 71 K=1,3                                                       WAL10460
      C=F*(FAC(J,K)-FAC(J1,K))+FAC(J1,K)                                WAL10470
      D=F*(FAC(L,K)-FAC(L1,K))+FAC(L1,K)                                WAL10480
      R=PHI(M)-P1                                                       WAL10490
      R=R/(P2-P1)                                                       WAL10500
      R=R*(C-D)+D                                                       WAL10510
      TO(K)=TMAX-R                                                      WAL10520
   71 CONTINUE                                                          WAL10530
      RETURN                                                            WAL10540
      END                                                               WAL10550
```

APPENDIX C

SAMPLE OUTPUT

Site Geometry and Soil Parameters

The geometric and soil parameters are listed in the output to provide a check. This output should be checked first when debugging.

Factored Soil Parameters

Factored soil parameters are used to compute the following in each soil layer:

Depth of soil layer interface (from top of wall)
Active and passive stress coefficients
Effective unit weight
Triangular stress distribution (overburden and horizontal)
Rectangular stress distribution (overburden and horizontal)
Resultant force for each stress distribution
Centroid for each stress distribution
Moment arm for each stress distribution
Resultant moment for each stress distribution

Depth of Penetration

The required penetration depth is printed out. If the subgrade cohesion renders an unstable wall, a message reading "THIS WALL CANNOT STAND" will appear and the program will terminate. The stability number of factor of safety against failure in penetration are listed for cohesive subgrades.

Unfactored Soil Parameters

A listing appears of the same parameters output for "Factored Soil Parameters," the difference being that this listing is computed for tie-rods loads and bending moments using unfactored soil parameters.

Tie-Rod Load

The tie-rod load is listed in lb/ft of wall.

Maximum Moment

The maximum bending moment, as computed by the Free Earth Support method is displayed. The location of the maximum moment is also shown (point of zero shear).

Operating and Structural Curves

Ordered pairs of τ and log ρ are shown for A328 steel sections, A570/A690 steel sections, and wood piles. Ordered pairs are first given for typical sections, then the actual design section. Curve-fitting data is given for clay subgrades where there are only three values of pile flexibility given in the Rowe reduction curves. The value of representing the point of intersection of the operating and structural curves is shown.

Design Section Modulus

The results of the Rowe reduction procedure are listed in in^3/ft of wall for A328 steel, A570/A690 steel and timber.

Design Section

The final USS section is listed for A328 steel, A570/A690 steel, as well as the required actual thickness for a timber pile. The tie-rod load is also output.

SOIL LAYER DEPTHS ARE FROM GROUND SURFACE... WALL HT & WTR LEVELS ARE FROM DREDGE LINE

SOIL LAYER	DEPTH (FT)	UNIT WEIGHT (PCF)	PHI (DEG)	COHESION (PSF)
1	4.000	100.000	30.000	0.000
2	12.000	122.400	32.000	0.000
3	100.000	122.400	32.000	0.000

```
WALL HT        =   12.00 FT
LOW WTR        =    0.00 FT
TIE ROD        =   10.00 FT
SURCHARGE      =           200. PSF (DISTRIBUTED LOAD)
SURCHARGE      =             0. PLF (LINE LOAD)
SURCHARGE      =             0. POUNDS (POINT LOAD)
FILL SLOPE     =   0.00DEGREES
DREDGE SLOPE   =   0.00 DEGREES
```

ANCHORED BULKHEAD

FACTORED SOIL PARAMETERS

Z	KP	KA	GAMMA	ZCR	OVERBURDEN STRESS REC	OVERBURDEN STRESS TRI	FORCES REC	FORCES TRI	CENTROIDS REC	CENTROIDS TRI	REC	MOMENT ARMS TRI	REC	MOMENTS TRI
4.0	0.00	0.41	100.	0.	200.	400.	-326.	-326.	2.	3.	-0.	1.	0.	-218.
12.0	0.03	0.38	60.	0.	680.	480.	-1834.	-734.	4.	5.	6.	7.	-11803.	-5379.
18.1	3.32	0.38	60.	0.	1089.	366.	-2517.	3281.	5.	4.	13.	14.	-32846.	46150.

UNFACTORED SOIL PARAMETERS

Z	KP	KA	GAMMA	ZCR	OVERBURDEN STRESS REC	OVERBURDEN STRESS TRI	FORCES REC	FORCES TRI	CENTROIDS REC	CENTROIDS TRI	REC	MOMENT ARMS TRI	REC	MOMENTS TRI
4.0	0.00	0.28	100.	0.	200.	400.	-224.	-224.	2.	3.	-0.	1.	0.	-149.
12.0	0.00	0.26	60.	0.	600.	480.	-1230.	-492.	4.	5.	6.	7.	-7378.	-3607.
13.1	6.63	0.26	60.	0.	1088.	366.	-1668.	7339.	5.	4.	13.	14.	-22824.	183237.

DEPTH OF PENETRATION = 6.10 FT

TIE ROD PULL = 1499. LB/FT

ZERO SHR @ X = 9.FT BELOW GROUND SURFACE... MAXIMUM MOMENT = 5289.FT-LBS/FT

A328 STEEL OPERATING AND STRUCTURAL CURVES
TYPICAL SECTION

```
LOG RHO =   -1.000   TO =   2.834   TS =   0.680
LOG RHO =   -1.100   TO =   2.834   TS =   0.804
LOG RHO =   -1.200   TO =   2.834   TS =   0.937
LOG RHO =   -1.300   TO =   2.834   TS =   1.093
LOG RHO =   -1.400   TO =   2.834   TS =   1.279
LOG RHO =   -1.500   TO =   2.834   TS =   1.485
LOG RHO =   -1.600   TO =   2.859   TS =   1.732
LOG RHO =   -1.700   TO =   2.883   TS =   2.019
LOG RHO =   -1.800   TO =   2.914   TS =   2.354
LOG RHO =   -1.900   TO =   2.953   TS =   2.745
LOG RHO =   -2.000   TO =   3.084   TS =   3.200
LOG RHO =   -2.100   TO =   3.165   TS =   3.731
```

LOG RHO	=	-2.200	TO	=	3.354	TS	=	4.350
LOG RHO	=	-2.300	TO	=	3.571	TS	=	5.072
LOG RHO	=	-2.400	TO	=	3.781	TS	=	5.913
LOG RHO	=	-2.500	TO	=	4.061	TS	=	6.895
LOG RHO	=	-2.600	TO	=	4.341	TS	=	8.038
LOG RHO	=	-2.700	TO	=	4.672	TS	=	9.372
LOG RHO	=	-2.800	TO	=	5.057	TS	=	10.927
LOG RHO	=	-2.900	TO	=	5.509	TS	=	12.740
LOG RHO	=	-3.000	TO	=	5.993	TS	=	14.854
LOG RHO	=	-3.100	TO	=	6.499	TS	=	17.318
LOG RHO	=	-3.200	TO	=	7.174	TS	=	20.192
LOG RHO	=	-3.300	TO	=	7.834	TS	=	23.542
LOG RHO	=	-3.400	TO	=	8.536	TS	=	27.448
LOG RHO	=	-3.500	TO	=	9.209	TS	=	32.002

TAU = 3.837

A329 STEEL OPERATING AND STRUCTURAL CURVES
SPECIFIC SECTION

LOG RHO	=	-1.000	TO	=	2.834	TS	=	0.438
LOG RHO	=	-1.100	TO	=	2.834	TS	=	0.510
LOG RHO	=	-1.200	TO	=	2.834	TS	=	0.595
LOG RHO	=	-1.300	TO	=	2.834	TS	=	0.694
LOG RHO	=	-1.400	TO	=	2.834	TS	=	0.809
LOG RHO	=	-1.500	TO	=	2.839	TS	=	0.943
LOG RHO	=	-1.600	TO	=	2.858	TS	=	1.100
LOG RHO	=	-1.700	TO	=	2.883	TS	=	1.282
LOG RHO	=	-1.800	TO	=	2.914	TS	=	1.495
LOG RHO	=	-1.900	TO	=	2.953	TS	=	1.743
LOG RHO	=	-2.000	TO	=	3.084	TS	=	2.032
LOG RHO	=	-2.100	TO	=	3.165	TS	=	2.369
LOG RHO	=	-2.200	TO	=	3.354	TS	=	2.762
LOG RHO	=	-2.300	TO	=	3.571	TS	=	3.221
LOG RHO	=	-2.400	TO	=	3.781	TS	=	3.755
LOG RHO	=	-2.500	TO	=	4.061	TS	=	4.378
LOG RHO	=	-2.600	TO	=	4.341	TS	=	5.104
LOG RHO	=	-2.700	TO	=	4.672	TS	=	5.951
LOG RHO	=	-2.800	TO	=	5.057	TS	=	6.939
LOG RHO	=	-2.900	TO	=	5.509	TS	=	8.090
LOG RHO	=	-3.000	TO	=	5.993	TS	=	9.432
LOG RHO	=	-3.100	TO	=	6.499	TS	=	10.997
LOG RHO	=	-3.200	TO	=	7.174	TS	=	12.822
LOG RHO	=	-3.300	TO	=	7.834	TS	=	14.949
LOG RHO	=	-3.400	TO	=	8.536	TS	=	17.429
LOG RHO	=	-3.500	TO	=	9.209	TS	=	20.321

TAU = 3.903

A572/690 STEEL OPERATING AND STRUCTURAL CURVES
TYPICAL SECTION

LOG RHO	=	-1.000	TO	=	2.834	TS	=	0.895
LOG RHO	=	-1.100	TO	=	2.834	TS	=	1.044
LOG RHO	=	-1.200	TO	=	2.834	TS	=	1.217
LOG RHO	=	-1.300	TO	=	2.834	TS	=	1.419
LOG RHO	=	-1.400	TO	=	2.834	TS	=	1.655
LOG RHO	=	-1.500	TO	=	2.834	TS	=	1.929
LOG RHO	=	-1.600	TO	=	2.838	TS	=	2.249

LOG RHO =	-1.700	TO =	2.883	TS =	2.622
LOG RHO =	-1.800	TO =	2.914	TS =	3.057
LOG RHO =	-1.900	TO =	2.953	TS =	3.565
LOG RHO =	-2.000	TO =	3.094	TS =	4.156
LOG RHO =	-2.100	TO =	3.165	TS =	4.846
LOG RHO =	-2.200	TO =	3.354	TS =	5.650
LOG RHO =	-2.300	TO =	3.571	TS =	6.567
LOG RHO =	-2.400	TO =	3.781	TS =	7.680
LOG RHO =	-2.500	TO =	4.061	TS =	8.954
LOG RHO =	-2.600	TO =	4.341	TS =	10.440
LOG RHO =	-2.700	TO =	4.672	TS =	12.172
LOG RHO =	-2.800	TO =	5.057	TS =	14.191
LOG RHO =	-2.900	TO =	5.509	TS =	16.546
LOG RHO =	-3.000	TO =	5.993	TS =	19.291
LOG RHO =	-3.100	TO =	6.489	TS =	22.491
LOG RHO =	-3.200	TO =	7.174	TS =	26.223
LOG RHO =	-3.300	TO =	7.834	TS =	30.574
LOG RHO =	-3.400	TO =	8.536	TS =	35.646
LOG RHO =	-3.500	TO =	9.209	TS =	41.561

TAU = 2.903

A572/690 STEEL OPERATING AND STRUCTURAL CURVES
SPECIFIC SECTION

LOG RHO =	-1.000	TO =	2.834	TS =	0.674
LOG RHO =	-1.100	TO =	2.834	TS =	0.786
LOG RHO =	-1.200	TO =	2.834	TS =	0.917
LOG RHO =	-1.300	TO =	2.834	TS =	1.069
LOG RHO =	-1.400	TO =	2.834	TS =	1.246
LOG RHO =	-1.500	TO =	2.834	TS =	1.453
LOG RHO =	-1.600	TO =	2.858	TS =	1.694
LOG RHO =	-1.700	TO =	2.883	TS =	1.975
LOG RHO =	-1.800	TO =	2.914	TS =	2.302
LOG RHO =	-1.900	TO =	2.953	TS =	2.684
LOG RHO =	-2.000	TO =	3.094	TS =	3.129
LOG RHO =	-2.100	TO =	3.165	TS =	3.649
LOG RHO =	-2.200	TO =	3.354	TS =	4.254
LOG RHO =	-2.300	TO =	3.571	TS =	4.960
LOG RHO =	-2.400	TO =	3.781	TS =	5.783
LOG RHO =	-2.500	TO =	4.061	TS =	6.742
LOG RHO =	-2.600	TO =	4.341	TS =	7.861
LOG RHO =	-2.700	TO =	4.672	TS =	9.165
LOG RHO =	-2.800	TO =	5.057	TS =	10.686
LOG RHO =	-2.900	TO =	5.593	TS =	12.459
LOG RHO =	-3.000	TO =	5.993	TS =	14.526
LOG RHO =	-3.100	TO =	6.489	TS =	16.936
LOG RHO =	-3.200	TO =	7.174	TS =	19.746
LOG RHO =	-3.300	TO =	7.834	TS =	23.022
LOG RHO =	-3.400	TO =	8.536	TS =	26.841
LOG RHO =	-3.500	TO =	9.209	TS =	31.295

TAU = 3.065

WOOD OPERATING AND STRUCTURAL CURVES
TYPICAL SECTION

LOG RHO =	-1.000	TO =	2.834	TS =	0.540
LOG RHO =	-1.100	TO =	2.834	TS =	0.629

LOG RHO =	-1.200	TO =	2.834	TS =	0.734
LOG RHO =	-1.300	TO =	2.834	TS =	0.855
LOG RHO =	-1.400	TO =	2.834	TS =	0.997
LOG RHO =	-1.500	TO =	2.834	TS =	1.163
LOG RHO =	-1.600	TO =	2.859	TS =	1.356
LOG RHO =	-1.700	TO =	2.883	TS =	1.580
LOG RHO =	-1.800	TO =	2.914	TS =	1.843
LOG RHO =	-1.900	TO =	2.953	TS =	2.148
LOG RHO =	-2.000	TO =	3.044	TS =	2.505
LOG RHO =	-2.100	TO =	3.165	TS =	2.920
LOG RHO =	-2.200	TO =	3.354	TS =	3.405
LOG RHO =	-2.300	TO =	3.571	TS =	3.970
LOG RHO =	-2.400	TO =	3.781	TS =	4.628
LOG RHO =	-2.500	TO =	4.061	TS =	5.396
LOG RHO =	-2.600	TO =	4.341	TS =	6.292
LOG RHO =	-2.700	TO =	4.672	TS =	7.336
LOG RHO =	-2.800	TO =	5.057	TS =	8.553
LOG RHO =	-2.900	TO =	5.509	TS =	9.972
LOG RHO =	-3.000	TO =	5.993	TS =	11.626
LOG RHO =	-3.100	TO =	6.489	TS =	13.555
LOG RHO =	-3.200	TO =	7.174	TS =	15.804
LOG RHO =	-3.300	TO =	7.834	TS =	18.426
LOG RHO =	-3.400	TO =	8.536	TS =	21.484
LOG RHO =	-3.500	TO =	9.209	TS =	25.048
		TAU =	3.321		

CALCULATED SECTION MODULI : 1.398 IN-3 (A328)
0.726 IN-3 (A572/690)
9.446 IN-3 (WOOD)

SECTION	MATERIAL	SEC MOD (IN-3)	M/INERT (IN-4)	DEF/MOM (IN-LB/FT)	TIE-ROD PULL (LB/FT)
PS 28	A328	1.9	2.4	22549.	1419.
PS 29	A 572/690	1.9	2.4	18177.	1419.
WOOD PILE THICKNESS (IN)		2.22	2.2	19691.	1419.

CANTILEVERED BULKHEAD

FACTORED SOIL PARAMETERS

Z	KP	KA	GAMMA	2CR	OVERBURDEN STRESS REC	OVERBURDEN STRESS TRI	FORCES REC	FORCES TRI	CENTROIDS REC	CENTROIDS TRI	MOMENT ARMS REC	MOMENT ARMS TRI	MOMENTS REC	MOMENTS TRI
4.0	0.90	1.41	100.	0.	200.	400.	-326.	-326.	2.	3.	26.	25.	-8353.	-8135.
12.0	0.19	1.33	50.	0.	600.	400.	-1034.	-734.	4.	5.	20.	18.	-35942.	-13393.
27.6	3.12	0.39	60.	0.	1060.	336.	-6417.	21456.	8.	16.	8.	5.	-57284.	111310.

UNFACTORED SOIL PARAMETERS

Z	KP	KA	GAMMA	2CR	OVERBURDEN STRESS REC	OVERBURDEN STRESS TRI	FORCES REC	FORCES TRI	CENTROID REC	CENTROID TRI	MOMENT ARMS REC	MOMENT ARMS TRI	MOMENTS REC	MOMENTS TRI
4.0	0.30	1.26	100.	0.	200.	400.	-224.	-224.	2.	3.	26.	25.	-3722.	-5573.
12.0	0.30	1.26	60.	0.	600.	400.	-1230.	-492.	4.	3.	20.	18.	-24101.	-8765.
27.6	6.93	1.25	60.	0.	1080.	936.	-4316.	41997.	4.	10.	8.	5.	-33654.	249581.

DEPTH OF PENETRATION = 15.64 FT

ZERO SHR @ X = 16.FT BELOW GROUND SURFACE... MAXIMUM MOMENT = -17237.FT-LBS/FT

A328 STEEL OPERATING AND STRUCTURAL CURVES
TYPICAL SECTION

LOG RHO =	-1.000	TO =	3.918	TS =	0.599
LOG RHO =	-1.250	TO =	3.918	TS =	0.879
LOG RHO =	-1.500	TO =	3.919	TS =	1.291
LOG RHO =	-1.750	TO =	3.918	TS =	1.894
LOG RHO =	-2.000	TO =	3.997	TS =	2.786
LOG RHO =	-2.250	TO =	4.123	TS =	4.081
LOG RHO =	-2.500	TO =	4.306	TS =	5.990
LOG RHO =	-2.750	TO =	4.772	TS =	8.792
LOG RHO =	-3.000	TO =	5.174	TS =	12.905
LOG RHO =	-3.250	TO =	5.631	TS =	18.942
LOG RHO =	-3.500	TO =	5.983	TS =	27.803
		TAU =	4.127		

A328 STEEL OPERATING AND STRUCTURAL CURVES
SPECIFIC SECTION

LOG RHO =	-1.000	TO =	3.918	TS =	0.375
LOG RHO =	-1.250	TO =	3.918	TS =	0.551
LOG RHO =	-1.500	TO =	3.919	TS =	0.808
LOG RHO =	-1.750	TO =	3.919	TS =	1.186
LOG RHO =	-2.000	TO =	3.997	TS =	1.741
LOG RHO =	-2.250	TO =	4.123	TS =	2.555
LOG RHO =	-2.500	TO =	4.306	TS =	3.751
LOG RHO =	-2.750	TO =	4.772	TS =	5.506
LOG RHO =	-3.000	TO =	5.174	TS =	8.081
LOG RHO =	-3.250	TO =	5.631	TS =	11.861
LOG RHO =	-3.500	TO =	5.983	TS =	17.410
		TAU =	4.507		

A572/690 STEEL OPERATING AND STRUCTURAL CURVES
TYPICAL SECTION

LOG RHO =	-1.000	TO =	3.918	TS =	0.778
LOG RHO =	-1.250	TO =	3.919	TS =	1.142
LOG RHO =	-1.500	TO =	3.918	TS =	1.676
LOG RHO =	-1.750	TO =	3.918	TS =	2.460
LOG RHO =	-2.000	TO =	3.997	TS =	3.611
LOG RHO =	-2.250	TO =	4.123	TS =	5.300
LOG RHO =	-2.500	TO =	4.306	TS =	7.779
LOG RHO =	-2.750	TO =	4.772	TS =	11.418
LOG RHO =	-3.000	TO =	5.174	TS =	16.760
LOG RHO =	-3.250	TO =	5.631	TS =	24.600
LOG RHO =	-3.500	TO =	5.983	TS =	36.104
		TAU =	4.923		

A572/690 STEEL OPERATING AND STRUCTURAL CURVES
SPECIFIC SECTION

LOG RHO =	-1.000	TO =	3.918	TS =	0.571
LOG RHO =	-1.250	TO =	3.918	TS =	0.846
LOG RHO =	-1.500	TO =	3.919	TS =	1.244
LOG RHO =	-1.750	TO =	3.919	TS =	1.927
LOG RHO =	-2.000	TO =	3.997	TS =	2.681
LOG RHO =	-2.250	TO =	4.123	TS =	3.936

LOG RHO =	-2.500	TO =	4.306	TS =	5.776
LOG RHO =	-2.750	TO =	4.772	TS =	8.479
LOG RHO =	-3.000	TO =	5.174	TS =	12.445
LOG RHO =	-3.250	TO =	5.631	TS =	18.267
LOG RHO =	-3.500	TO =	5.983	TS =	26.812
		TAU =	4.143		

WOOD OPERATING AND STRUCTURAL CURVES
TYPICAL SECTION

LOG RHO =	-1.000	TO =	3.910	TS =	0.469
LOG RHO =	-1.250	TO =	3.910	TS =	0.688
LOG RHO =	-1.500	TO =	3.910	TS =	1.010
LOG RHO =	-1.750	TO =	3.910	TS =	1.403
LOG RHO =	-2.000	TO =	3.997	TS =	2.176
LOG RHO =	-2.250	TO =	4.123	TS =	3.194
LOG RHO =	-2.500	TO =	4.306	TS =	4.600
LOG RHO =	-2.750	TO =	4.772	TS =	6.882
LOG RHO =	-3.000	TO =	5.174	TS =	10.101
LOG RHO =	-3.250	TO =	5.631	TS =	14.826
LOG RHO =	-3.500	TO =	5.983	TS =	21.762
		TAU =	4.252		

CALCULATED SECTION MODULI : 5.031 IN-3 (A328)
3.901 IN-3 (A572/690)
44.761 IN-3 (WOOD)

SECTION	MATERIAL	SEC MOD (IN-3)	M/INERT (IN-4)	DES/MOM (IN-LB/FT)	TIE-ROD PULL (LB/FT)
PDA 27	A328	10.7	39.0	94746.	0.
PMA 22	A 572/690	5.4	13.7	87111.	0.
WOOD PILE THICKNESS (IN)		4.73	4.7	69401.	0.

APPENDIX D: FLOW TABLES FOR DESIGN

Table D-1. Preliminary actions

Step	Action	Reference Section
1	Establish soil profile	4.2
2	Determine bulkhead type (fill or dredge, anchored or cantilevered) and geometry, i.e., wall height, anchor level, dredge level, high and low water levels	
3	Determine soil parameters for each soil layer (ϕ, c, γ)	4.2
4	Compute soil stress coefficients using factored soil parameters (ϕ', c') for penetration depth and unfactored (ϕ, c) for tie-rod and moment calculations	2.3.1, Eq. 2-2, 2-3; 4.3
5	Compute stability number for walls in clay	4.5, Eq. 4-17
6	Produce a soil stress diagram to aid in calculations	4.3.1, 4.5

Table D-2. Free Earth Support calculations

Step	Action	Reference Section
1	Compute soil stresses, resultant forces, centroids sum moments about	
	a. Tie-rod (anchored walls in sand)	4.3.1
	b. Tie-rod (anchored walls in clay)	4.5.1
	c. Pile toe (cantilevered walls in sand)	4.4
	d. Pile toe (cnatilevered walls in clay)	4.5.3
2	Solve for penetration depth, D, using factored soil parameters for	
	a. Anchored walls in sand	4.3.1
	b. Anchored walls in clay	4.5.1
	c. Cantilevered walls in sand	4.4
	d. Cantilevered walls in clay	4.5.3
3	Compute tie-rod pull, P (force per unit length of wall) by summing moments about	
	a. 2/3D (anchored walls in sand)	4.3.1
	b. 1/2D (anchored walls in clay)	4.5.1
4	Find point of zero shear for:	
	a. Anchored walls	4.3.1, 4.5.1
	b. Cantilevered walls	4.4
5	Compute maximum bending moment at point of zero shear	4.3.1, 4.5.1 4.4

Table D-3. Rowe reduction calculations

Step	Action	Reference Section
1	Compute M_{max} from FES maximum moment	4.3.1, 4.4, 4.5.1
2	Develop an operating curve based upon M_{max} and moment reduction factors for	
	a. Anchored walls in sand	4.3.1
	b. Anchored walls in clay	4.5.1
	c. Cantilevered walls in sand	4.4
3	Develop structural curves based upon the average properties of the sheet pile material under consideration	4.3.1, 4.4, 4.5.1, 2.7.1.3, Fig. 2-17a, 2.7.4, Fig. 2-19a, 2.7.6, Fig. 2-20
4	Find τ from the intersection of the operating and structural curves	4.3.1, 4.4, 4.5.1, 2.7.1.3, Fig. 2-18
5	Determine the member size from τ	4.3.1, 4.4, 4.5.1
6	Recompute the structural curve based upon the properties of the selected section	4.3.1, 4.4, 4.5.1
7	Repeat steps 4 and 5 to insure that the selected section is adequate	4.3.1, 4.4, 4.5.1
8	Apply tie-rod factors	
	a. Walls in sand	4.3.1, 2.3.7.1, Fig. 2-17b
	b. Walls in clay	4.5.1, 2.7.4, Fig. 2-19b
	c. Non-yielding anchorages	2.7.1.3, Fig. 2-17c

Table D-4. Computations for simplified procedure

Step	Action	Reference Section
1	Compute loading ratio, R	4.6
2	Compute modifying coefficient for depth, C_D	4.6
3	Compute $R_D = R \times C_D$, find dimensionless depth, D' from design charts or equations	4.6
4	Compute $D = D' \times H$	4.6
5	Compute modifying coefficient for moment and tie-rod pull, $C_M = C_P$	4.6
6	Compute $R_M = R \times C_M$, find dimensionless bending moment, M', from charts or equation	4.6
7	Compute moment $M = M' \gamma_3 L^3$	4.6
8	Compute $R_P = R \times C_P$, find dimensionless tie-rod pull, P'	4.6
9	Compute pull, $P = P' \gamma L^2$	4.6

Table D-5. Component design computations

Step	Action	Reference Section
1	Design tie-rod	
	a. Compute tie-rod tension based on pull per unit length of wall times the tie-rod spacing	5.4.2
	b. Apply load factors	5.4.2., 5.2.6
	c. Compute required diameter	5.4.2
	d. Determine length based on anchorage location	5.4.2, 5.3.2
2	Wale design	
	a. Compute bending moment in wale	5.4.3
	b. Dimension the wale	5.4.3
3	Fastening wales to sheet piles	
	a. Inside wales, wood: select a nail size and determine the number of nails required per section to resist the prying force, P (tie-rod pull/unit length of wall)	5.4.3.1
	b. Outside wales, wood: use 2 nails/pile. Select nail size with adequate length to transmit shear	5.4.3.1
	c. Inside wales, steel	5.4.3.3
	1) Select a bolt size and determine the number of bolts required to resist the prying force, P (tie-rod pull/unit length of wall)	5.4.3.3
	2) Compute tensile force in each bolt	5.4.3.3
	3) Compute bending moment in fixing plate	5.4.3.3
	4) Dimension the fixing plate	5.4.3.3
	d. Outside wale, steel: use number of bolts required to facilitate construction	5.4.3.3

Table D-5. Continued

Step	Action	Reference Section
4	Splices for wales	
	a. Outside wales, wood: locate splices at the tie-rod. Design a bearing plate for the tie-rod nut	5.4.3.2
	b. Inside wales, wood:	5.4.3.2
	1) Select splice plate dimensions (2- or 3-member splice) to resist maximum moment in wale	
	2) Select bolt size and number to resist shear	5.4.3.2
	3) Determine edge distance, end distance, spacing between bolts, and spacing between rows	
	4) Select final length of splice plate	5.4.3.2
	c. Splices for channels	
	1) Select splice plate width and thickness to fit between the channel flanges and to resist maximum moment in the wale	5.4.3.2
	2) Select bolts to resist shear (double shear as bolts will attach 2 plates, one on each channel)	5.4.3.2
	3) Allow for edge distance and spacing	5.4.3.2
	4) Select a convenient length	5.4.3.2
5	Anchorage design	
	a. Determine loads on:	5.4.4
	1) Continuous anchorage	5.4.4.1
	2) Short deadman	5.4.4.2
	b. Check bearing stress of tie-rod nut and design a bearing plate, if required	5.4.3.2

APPENDIX E

DESIGN EXAMPLES

EXAMPLE #1: GIVEN THE FOLLOWING SITE GEOMETRY AND SOIL CONDITIONS, FIND THE PENETRATION DEPTH, BENDING MOMENT AND TIE-ROD PULL USING THE FREE EARTH SUPPORT METHOD WITH ROWE REDUCTION:

$H = 12'$	$t_1 = 4'$	$\gamma_1 = 100$ pcf	$\phi_1 = 30°$	(FIG. 4-1)
$H_W = 8'$	$t_2 = 8'$	$\gamma_2 = 122.4$ pcf	$\phi_2 = 32°$	
$H_A = 2'$		$\gamma_3 = 122.4$ pcf	$\phi_3 = 32°$	

1.) FIND FACTORED AND UNFACTORED SOIL PARAMETERS

$\phi_1 = 30°$ $\quad \phi_1 f = \tan^{-1}\left(\frac{1}{1.5}\tan 30°\right) = 21°$ (EQ 3-1)

$\phi_2 = 32°$ $\quad \phi_2 f = 22.6° = \phi_3 f$

$\delta_1 = 20°$ $\quad \delta_1 f = 14°$

$\delta_2 = 21.3°$ $\quad \delta_2 f = 15° = \delta_3 f$

$$K_a = \frac{\cos^2\phi}{\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin\phi}{\cos\delta}}\right]^2} \qquad \text{(EQ 2-2)}$$

$$K_p = \frac{\cos^2\phi}{\left[1-\sqrt{\frac{\sin(\phi+\delta)\sin\phi}{\cos\delta}}\right]^2} \qquad \text{(EQ 2-3)}$$

FACTORED: $K_a'{}_1 = 0.408$ $\quad K_p'{}_1 = 3.00$

$K_a'{}_2 = 0.382 = K_a'{}_3$ $\quad K_p'{}_2 = 3.32 = K_p'{}_3$

UNFACTORED: $K_{a1} = 0.279$ $\quad K_{p1} = 5.74$

$K_{a2} = 0.256 = K_{a3}$ $\quad K_{p2} = 6.33 = K_{p3}$

$\gamma_2 = 122.4 - 62.4$ (BOUYANT UNIT WEIGHT)

$= 60$ pcf $= \gamma_3$

2.) COMPUTE RESULTANT FORCES AND SUM MOMENTS ABOUT TIE ROD (FIG. 4-2, EQ. 4-3)

$$\tfrac{1}{2} Ka'_1 \gamma_1 t_1^2 \left(\tfrac{2}{3}t_1 - HA\right) + \tfrac{1}{2} Ka'_2 \gamma_2 t_2^2 \left(\tfrac{2}{3}t_2 + t_1 - HA\right) + Ka'_2 \gamma_1 t_1 t_2$$
$$\left(\tfrac{1}{2}t_2 + t_1 - HA\right) + Ka'_3 (\gamma_1 t_1 + \gamma_2 t_2)\, D \left(\tfrac{1}{2}D + H - HA\right) + \tfrac{1}{2}(Kp'_3 - Ka'_3)$$
$$\gamma_3 D^2 \left(\tfrac{2}{3}D + H - HA\right) = 0$$

$$218 - 5380 + 7330 + 3360D + (168 - 881)\, D^2 - 58.8\, D^3 = 0$$
$$12{,}930 + 3360D - 713D^2 - 58.8\, D^3 = 0$$
$$D = 5.6'$$

3.) COMPUTE TOE SHEAR BASED UPON $HD = H + D = 17.6'$

$$(FT_1 + FT_2 + FR_2 + FR_3 + FT_3 - FT_4)\tan\delta = \qquad \text{(EQ. 4-2)}$$
$$\left[\tfrac{1}{2} Ka'_1 \gamma_1 t_1^2 + \tfrac{1}{2} Ka_2 \gamma_2 t_2^2 + Ka'_2 \gamma_1 t_1 t_2 + Ka'_3 (\gamma_1 t_1 + \gamma_2 t_2) D\right.$$
$$\left. - \tfrac{1}{2}(Ka'_3 - Kp'_3)\gamma_3 D^2\right] \tan\delta_1 =$$
$$(326 + 733 + 1220 + 1880 - 2760)(\tan 14°) = 349\ \#$$

FOR WEIGHT OF PILE PER FOOT OF HEIGHT USE $W = 22\ \#/\text{ft.}$

$$TS = [(22)(17.6) + (349)](\tan 14°) = 184\ \#$$

4.) APPLY FORCE AT 2/3 D AND SUM MOMENTS ABOUT TIE-ROD:

$$TS\left(H + \tfrac{2}{3}D - HA\right) = (184)(13.7) = 2530$$
$$(12{,}930 - 2530) + 3360D - 713D^2 - 58.8D^3 = 0$$
$$10{,}400 + 3360D - 713D^2 - 58.8D^3 = 0$$
$$D = 5.3', \text{ USE } D = 5.5'$$

5.) FIND TIE-ROD USING UNFACTORED SOIL PARAMETER BY SUMMING MOMENTS OF RESULTANT FORCES ABOUT 2/3 D: (EQ. 4-3)

$$\tfrac{1}{2} Ka_1 \gamma_1 t_1^2 \left(\tfrac{1}{3}t_1 + t_2 + \tfrac{2}{3}D\right) + \tfrac{1}{2} Ka_2 \gamma_2 t_2^2 \left(\tfrac{1}{3}t_2 + \tfrac{2}{3}D\right) + Ka_2 \gamma_1 t_1 t_2 \left(\tfrac{1}{2}t_2 + \tfrac{2}{3}D\right)$$
$$+ Ka_3 (\gamma_1 t_1 + \gamma_2 t_2)\left(\tfrac{1}{6}D^2\right) - P\left(H + \tfrac{2}{3}D - HA\right) = 0$$
$$(2900 + 3110 + 6280 - 1140) = 13.7P$$
$$P_{FES} = 983\ \#/\text{FT.}$$

6.) FIND POINT OF ZERO SHEAR (EQ. 4-4)

$P = F_{T1} - \frac{1}{2} Ka_2 \gamma_2 x^2 - Ka_2 \gamma_1 t_1 x = 0$

$X = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$ WHERE $a = \frac{1}{2} Ka_2 \gamma_2 = 7.68$

$b = Ka_2 \gamma_2 t_1 = 61.4$, AND

$c = FT - T = \frac{1}{2} Ka_1 \gamma_1 t_1^2 - P$

$= -760$

$X = 6.72'$ BELOW THE WATER LINE (t_1)

7.) FIND MAXIMUM MOMENT (EQ. 4-5)

$M_{MAX} = P(t_1 + x - HA) - F_{T1}(\frac{1}{3}t_1 + x) - \frac{1}{6} Ka_2 \gamma_2 x^3 - \frac{1}{2} Ka_2 \gamma_2 t_1 x^2$

$= (983)(8.72) - (223)(8.05) - (777) - (2310)$

$= 3690$ ft. #/ft.

8.) COMPUTE TIE-ROD LOAD BASED UPON ROWE METHOD:

$\alpha = \frac{H}{HD} = \frac{12}{17.5} = 0.69$

$\beta = \frac{HA}{HD} = \frac{2}{17.5} = 0.11$

$f_c = 1.02$ (FIG. 2-17b)

$p = f_c \cdot P_{FES}$

$p = (1.02)(983) = 1000$ #/ft.

FOR SPACING OF TIES AT 7'-6" CENTERS

$T = P \cdot 7.5 = 7{,}500$ #

9.) COMPUTE BENDING MOMENT

a) $T_{MAX} = (12)\, M_{MAX}/HD^3 = (12)(3690)/(17.5)^3 = 8.26$ (EQ. 4-8)

USING FIG. 2-17b FOR VALUES OF r_d : INTERPOLATE 0.20 x DISTANCE BETWEEN LOOSE SAND AND DENSE SAND @ $\alpha = 0.7$. USE OF 20% FOR INTERPOLATION STEMS FROM CHOOSING $\phi = 30°$ FOR LOOSE SAND, $\phi = 40°$ FOR DENSE SAND, AND $\phi = 32°$ FOR THE SUBGRADE SO THAT:

$$\frac{32-30}{40-30} = 20\%$$

$T_{op} = T_{max} \times rd$ (EQ. 4-11)

$T_{STR.} = \frac{\psi}{(H\alpha p^2)^{1/3}}$ (EQ 4-12)

$\psi = \frac{2f}{E^{2/3}} = \frac{2(2000)}{(1.5\times10^6)^{2/3}} = 0.305$ (WOOD)

$= 0.260$ (APPROX. FOR A328 STEEL)

$= 0.400$ (APPROX. FOR A690 STEEL)

$\log \rho$	-3.00	-2.75	-2.50	-2.25	-2.00	-1.75
rd	0.57	0.48	0.39	0.29	0.27	0.27
T_{op}	4.54	3.97	3.22	2.81	2.40	2.23
$(H\alpha p^2)^{-\frac{1}{3}}$	38.5	26.2	17.9	12.2	8.30	3.85
T_{STR} (WOOD)	11.8	8.0	5.45	3.71	2.53	1.17
(A328)	10.0	6.81	4.65	3.17	2.16	1.60
(A690)	15.4	7.16	4.88	3.32	2.26	1.54

(SEE PLOT NEXT PAGE)

b) DESIGN SECTION

$M = T \cdot H\alpha^3$ (EQ. 4-15)

$S = \frac{M}{f_b}$ (EQ. 4-16)

MATERIAL	T	M (in-#)	f (psi)	S (in³/ft.)
WOOD	2.37	12,700	2,000	6.35
A328	2.48	13,300	25,000	0.532
A690	2.22	11,900	32,000	0.372

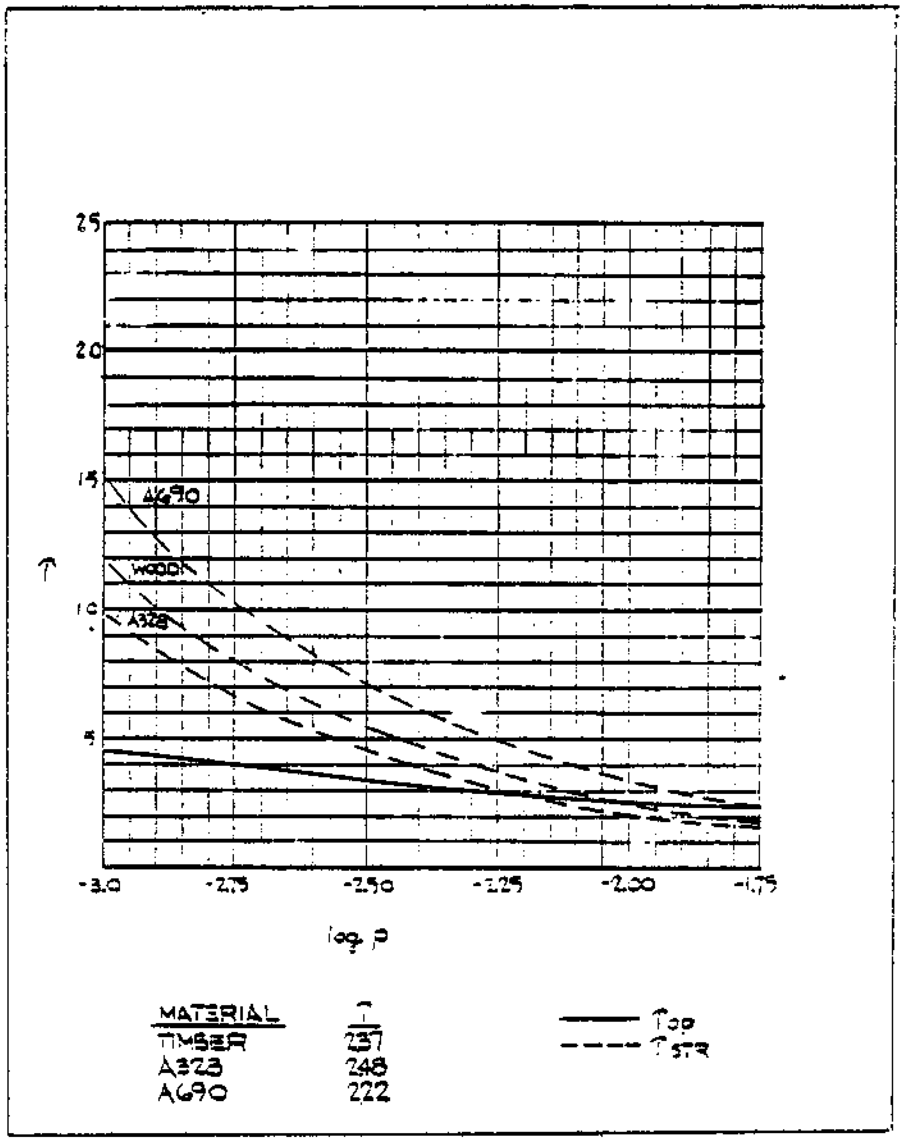
FOR WOOD SECTION:

$S = \frac{1}{6} bt^2$, for $b = 12"$, $t = \sqrt{\frac{S}{2}}$ (EQ. 5-10a)

$t = 1.78$, USE 3x12 (NOMINAL SIZE)

FOR A328 & A690 STEEL, THE SMALLEST SECTION, PS28 HAS

$S = 1.9$ in³/ft. > S req. (TABLE 5-2)


25
20
15
10
5
A690
WOOD
A328
-3.0
-2.75
-2.50
-2.25
-2.00
-1.75
log p
MATERIAL
TIMBER
A328
A690
237
248
222
T_{op}
T_{STR}

c.) RECOMPUTE FLEXIBILITY CHARACTERISTICS:

$$\psi = \frac{f}{E^{2/3}} \cdot \frac{S}{I^{2/3}} \qquad \text{(Eq. 4-13)}$$

MATERIAL	f (psi)	E (psi)	S (in³/ft.)	I (in⁴/ft.)	ψ
A328	25,000	30×10^6	1.9	2.8	0.248
A690	32,000	30×10^6	1.9	2.8	0.317

d.) RECOMPUTE τ_{STR} AND FIND INTERSECTION OF THE OPERATING AND NEW STRUCTURAL CURVES FOR A328 AND A690 STEEL:

$\log \rho$	-2.25	-2.00	-1.75	-1.50
$(H_0 \rho^3)^{-1/3}$	12.2	8.30	5.65	3.85
τ_{STR} (A328)	3.03	2.06	1.40	0.95
(A690)	4.65	4.65	3.16	2.15
τ_{op}	2.81	2.40	2.23	2.23

e.) RECOMPUTED VALUES:

MATERIAL	τ	M_{MAX}	$S_{req.}$
A328	2.66	14,300	0.572
A690	2.21	11,800	0.307

THE SECTIONS SELECTED ARE SATISFACTORY. A COST ANALYSIS WILL DETERMINE WHICH MATERIAL IS BEST: WOOD, A328 OR A690.

EXAMPLE 2: USING THE CONDITIONS OF EXAMPLE #1, ASCERTAIN THE DESIRABILITY OF A CANTILEVERED WALL.

1.) COMPUTE DEPTH OF PENETRATION: SUM MOMENTS ABOUT TOE

$$\frac{1}{2} K a_1' \gamma_1 t_1^2 \left(\frac{1}{3} t_1 + t_2 + D\right) + \frac{1}{2} K a_2' \gamma_2 t_2^2 \left(\frac{1}{3} t_2 + D\right)$$
$$+ K a_2' \gamma_1 t_1 t_2 \left(\frac{1}{2} t_2 + D\right) + \frac{1}{2} K a_3' (\gamma_1 t_1 + \gamma_2 t_2) D^2$$
$$+ \frac{1}{6} (K a_3' - K p') \gamma_3 D_3 = 0$$
$$321 (9.33 + D) + 733 (2.67 + D) + 1220 (4 + D) + 168 D^2 - 29.4 D^3 = 0$$
$$D = 13.4$$

2) NEGLECT TOE SHEAR: MOMENT ARM IS $\frac{1}{3}$D AND THE RESULTING MOMENT COMPUTED FROM TOE SHEAR IS VERY SMALL.

3) FIND MAXIMUM MOMENT:

a.) POINT OF ZERO SHEAR IS SOME DISTANCE X BELOW DREDGE LEVEL (USE UNFACTORED SOIL PARAMETERS)

$$FT_1 + FT_2 + FR_2 + Ka_3 (\gamma_1 t_1 + \gamma_2 t_2) X - \frac{1}{2} (Kp - Ka_3) \gamma_3 X^2 = 0$$
$$(224 + 492 + 819) + 225 X - 197 X^2$$
$$X = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}, \text{ WHERE } a = 197,\ b = -225,\ c = -1535$$
$$X = 3.42$$

b) $$M_{max} = FT_1 \left(\frac{1}{3} t_1 + t_2 + X\right) + FT_2 \left(\frac{1}{3} t_2 + X\right) + FR_2 \left(\frac{1}{2} t_2 + X\right)$$
$$+ \frac{1}{2} Ka_3 (\gamma_1 t_1 + \gamma_2 t_2) X^2 - \frac{1}{6} (Kp - Ka_3) \gamma_3 X^3$$
$$= (224)(12.9) + (492)(6.28) + (819)(7.61)$$
$$+ \frac{1}{2} (225)(3.41)^2 - \frac{1}{6} (394)(3.41)^3$$
$$= 10916 \text{ ft-\#}$$

4) COMPUTE BENDING MOMENT (WOOD ONLY)

a.) $T_{max.} = M_{max} \times 12 \div HD^3$
$= (10920)(12) / (13.4 + 12)^3$
$= 8.00$

b) $\alpha = \frac{H}{HD} = \frac{(12)}{(13.4+12)} = \frac{(12)}{(25.4)} = 0.472$

C.) GENERATE OPERATING AND STRUCTURAL CURVES. FROM FIG. 3.4, SELECT VALUES OF r_d FOR THE CORRESPONDING VALUES OF LOG ρ:

$$\tau_{op} = \tau_{max} \times r_d$$

$$\tau_{ST} = \frac{\psi}{(H_D\rho^2)^{\frac{1}{3}}}$$

FOR WOOD, $\psi = \frac{2F}{E^{2/3}} = \frac{(2)(2000)}{[(1.5)(10^6)]^{2/3}} = 0.309$

$\log \rho$	-3.0	-2.75	-2.50	-2.25	-2.00	-1.75	-1.50
r_d	0.60	0.53	0.49	0.48	0.45	0.44	0.44
τ_{op}	4.80	4.24	3.92	3.84	3.60	3.52	3.52
$(H_D\rho^2)^{-\frac{1}{3}}$	34.0	23.2	15.8	10.8	7.33	4.99	3.40
τ_{ST}	10.4	7.07	4.82	3.28	2.24	1.52	1.04

IT CAN BE SEEN FROM INSPECTION, THAT THE INTERSECTION OF THE GRAPHS FALLS BETWEEN $\log \rho = -2.50$ AND $\log \rho = -2.25$. APPROXIMATING THE STRUCTURAL AND OPERATING CURVE SEGMENTS AS STRAIGHT AND EMPLOYING SIMPLE COORDINATE GEOMETRY YIELDS:

$$\tau = 3.87 \ @ \ \log \rho = -2.35$$

$M = \tau \times H_D = (3.87)(25.4)^3 = 63420 \text{ in-\#}$

$S = \frac{M}{F} = \frac{63420}{2000} = 31.7 \text{ in}^3$

$S = \frac{1}{6} bt^2$, for $b = 12''$, $t = \sqrt{\frac{S}{2}}$

$t = 3.98$ in

5x12 (NOMINAL) SHEET-PILES ARE REQUIRED. THIS SIZE SECTION IS PROBABLY NOT AVAILABLE. A STEEL SECTION OR NAVY WALL WOULD BE APPROPRIATE.

FOR A328 STEEL:

$\psi \approx 0.260$

FOR WOOD:

$\psi = 0.305$

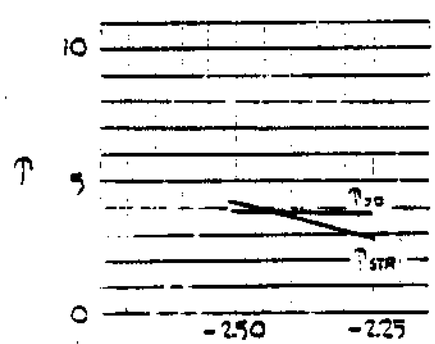
FORMING A RATIO OF A328/WOOD AND APPLYING IT AGAINST THE EXISTING VALUES OF τ_{ST} PRECLUDES GENERATING ANOTHER STRUCTURAL CURVE.

FOR $\log \rho = -2.25$, $\tau_{ST} = \left(\frac{0.260}{0.305}\right)(3.28) = 2.79$

FOR $\log \rho = -2.50$, $\tau_{ST} = \left(\frac{0.260}{0.305}\right)(4.82) = 4.10$

THIS SEGMENT OF THE CURVES IS IDENTIFIED BY:

$\log \rho$	-2.50	-2.25
τ_{OP}	3.92	3.84
τ_{ST}	4.10	2.79



$\tau = 3.91$

$M = (3.91)(25.4)^3 = 64,100$ in-lb.

$S = \frac{M}{f} = \frac{(64,100)}{25,000} = 2.56 \text{ in}^3$

USE PMA22, WHERE $S = 5.4\,in^3$ NO RECOMPUTATION IS NEEDED AS THE SECTION MODULUS IS SUBSTANTIALLY GREATER THAN THE MINIMUM REQUIRED.

5.) THE CANTILEVERED WALL IS MUCH LESS ECONOMICAL OWING TO THE GREAT INCREASES IN THE REQUIRED SECTION AND OVERALL PILE LENGTH.

EXAMPLE 3: USING CONDITIONS GIVEN IN EXAMPLE 1, FIND THE PENETRATION DEPTH, TIE-ROD LOAD AND BENDING MOMENT, USING THE DESIGN CHARTS:

1.) COMPUTE RD:

$$R = \frac{\gamma_1 t_1^3 + \gamma_2 t_2^3}{\gamma_s H^3} = \frac{(100)(4)^3 + (60)(8)^3}{(60)(12)^3}$$

$$= 0.358$$

$$C_D = \left(\frac{H_W}{H}\right)^2 \left(\frac{H_A}{H-H_A}\right) = \left(\frac{8}{12}\right)^2 \left(\frac{2}{12-2}\right)$$

$$= 0.0889$$

$$R_D = R \cdot C_D = (0.358)(0.0889) = 0.0318$$

2.) FIND D': SINCE THE SUBGRADE IS SOMEWHERE BETWEEN THE "LOOSE" AND "MEDIUM" CONDITIONS, INTERPOLATION WILL GIVE THE DESIRED VALUES.

ENTER FIG. 4-4 @ $R_D = 0.0318$ AND READ OFF D' FOR "L/L" AND "L/M." INTERPOLATION BY CONSIDERING THE DESIRED VALUE TO LIE 0.40 TIMES THE DISTANCE FROM "L/L" TO "L/M" GIVES THE PROPER VALUE.

CONDITION	ØS	D'
L/L	30	0.503
DESIRED	32	0.503 − x
L/M	35	0.336

Differences: 30 to 32 = 2; 30 to 35 = 5; 0.503 to 0.503 − x = x; 0.503 to 0.336 = −0.167

$$\frac{x}{-0.167} = \frac{2}{5}$$

$$x = 0.067$$

$$D' = 0.436 \rightarrow D = D' \cdot H = (0.436)(12) = 5.23'$$

3.) <u>COMPUTE R_M & R_P:</u>

$$R_M = R_p = R \cdot C_M = R \cdot C_p$$

$$C_p = C_M = \left(\frac{D}{H}\right)\left(\frac{H_A}{H_W}\right) = \left(\frac{5.23}{12}\right)\left(\frac{2}{8}\right) = 0.109$$

$$R_M = R \cdot C_M = (0.358)(0.109) = 0.039 = R_p$$

4.) <u>FIND M':</u> INTERPOLATE BY ENTERING "L/L" AND "L/M" @ $R_M = 0.039$

CONDITION	ϕ_s			P'		
L/L	30	2	5	0.099	x	0.012
DESIRED	32			0.099 + x		
L/M	35			0.111		

$$\frac{X}{0.012} = \frac{2}{5}$$

$$X = 0.0048$$

$$L = \frac{2}{3} D + H - H_A = \left(\frac{2}{3}\right)(5.23) + 12 - 2$$
$$= 13.5'$$

$$M' = M' \gamma_3 L^3 = (0.103)(60)(13.5)^3$$
$$= 15{,}200 \text{ in-\#} \text{ (A328 STEEL)}$$

5.) <u>FIND P':</u> ENTER "L/L" AND "L/M" @ $R_P = 0.041$

CONDITION	ϕ_s			P'		
L/L	30	2	5	0.352	x	0.0580
DESIRED	32			0.0352 + x		
L/M	35			0.0932		

$$\frac{x}{0.058} = \frac{2}{5}$$

$$X = 0.0231$$

$$P = 0.0622$$

$$P = p' \gamma_1 L^2 = (0.0622)(100)(13.5)^2 = 1130 \text{ \#/FT.}$$

6.) THE PERCENT DIFFERENCE BETWEEN THE RESULTS USING THE DESIGN CHARTS WITH THE RESULTS OF THE HAND CALCULATIONS ARE:

PENETRATION DEPTH	:	-1.3 %
BENDING MOMENT	:	6.3 %
TIE-ROD PULL	:	13.0 %

<u>EXAMPLE 4:</u> CONSIDER THE SITE GEOMETRY OF EXAMPLE 1 AND THE FOLLOWING SOIL CONDITIONS AND COMPUTE THE PENETRATION DEPTH, BENDING MOMENT AND TIE-ROD PULL:

$\phi_1 = 30°$	$\gamma_1 = 100$ pcf
$\phi_2 = 30°$	$\gamma_2 = (120 - 62.4) = 57.6$ pcf
$\phi_3 = 0$	$\gamma_3 = (110 - 62.4) = 47.6$ pcf
$C_3 = 300$ psf	

1.) <u>DETERMINE</u> STABILITY NUMBER AND SOIL PARAMETERS:

$$St = \frac{Cr}{(\gamma_1 t_1 + \gamma_2 t_2)} = \frac{(300)(1.25)}{[(100)(4) + (57.6)(8)]} = 0.435 > .25, \text{ OK}$$

$$\phi_1 = \phi_2 = 30°, \quad Ka = 0.279 \text{ (UNFACTORED)}$$

$$c' = \frac{1}{1.5}C = \left(\frac{1}{1.5}\right)(300) = 200 \text{ psf}$$

2.) <u>COMPUTE</u> RESULTANT FORCES AND SUM MOMENTS ABOUT TIE ROD:

$$\frac{1}{2} Ka\,\gamma_1 t_1^2 \left(\frac{2}{3} t_1 - H_A\right) + \frac{1}{2} Ka\,\gamma_2 t_2^2 \left(\frac{2}{3} t_2 + t_1 - H_A\right) + Ka\,\gamma_1 t_1 t_2$$

$$\left(\frac{1}{2} t_2 + t_1 - H_A\right) - (4c'r - \gamma_1 t_1 - \gamma_2 t_2)\, D \left(\frac{1}{2} D + H - H_A\right) = 0$$

$$149 + 3770 + 3360 - (139.2)\, D \left(\frac{1}{2} D + 8\right) = 0$$

$$69.6\, D^2 + 1114\, D - 9280 = 0$$

$$D = \frac{-b + \sqrt{b^2 - 4ac}}{2a}, \quad \text{WHERE } a = 69.6,\ b = 1114,\ c = 9280$$

$$D = 6.02$$

3.) FIND TIE-ROD LOAD BY SUMMING MOMENTS ABOUT $\frac{1}{2}$ D:

$$\frac{1}{2} K_a \gamma_1 t_1^2 \left(\frac{1}{3}t_1 + t_2 + \frac{1}{2}D\right) + \frac{1}{2} K_a \gamma_2 t_2^2 \left(\frac{1}{3}t_2 + \frac{1}{2}D\right) +$$

$$K_a \gamma_1 t_1 t_2 \left(\frac{1}{2}t_2 + \frac{1}{2}D\right) - P\left(\frac{1}{2}D + H - H_A\right) = 0$$

$$2750 + 2910 + 6250 - P(13) = 0$$

$$P = 916 \text{ \#/ft.}$$

4.) FIND THE POINT OF ZERO SHEAR

$$P - F_{T1} - \frac{1}{2} K_a \gamma_2 X^2 - K_a \gamma_1 t_1 X = 0$$

$$916 - 223 - 8.04x^2 - 112x = 0$$

$$X = \frac{-b + \sqrt{b^2 - 4ac}}{2a} \qquad \text{WHERE } a = 112$$

$$b = 8.04$$

$$X = 4.65 \qquad c = -(916 - 223) = -693$$

5.) COMPUTE M_{MAX}:

$$M_{MAX} = P_1(t_1 + x - H_A) - F_{T1}\left(\frac{1}{3}t_1 - x\right) - \frac{1}{6}ax^3 - \frac{1}{2}bx^2$$

$$= (916)(6.65) - (223)(5.98) - (1.34)(4.65)^3 - (56)(4.65)^2$$

$$= 3412 \text{ ft.-\#/ft.}$$

6.) COMPUTE BENDING MOMENT

a.) $T_{MAX} = (12) M_{MAX}/H_D^3 = (12)(3412)/(18)^3$

$$\alpha = \frac{H}{H_D} = \frac{12}{18} = 0.67$$

$$\beta = \frac{H_A}{H_D} = \frac{2}{18} = 0.11$$

b.) GENERATE OPERATING AND STRUCTURAL CURVES:

$T_{op} = T_{MAX} \cdot rd$ (VALUES OF rd ARE FROM FIG. 3.3a)

$T_{STR} = \dfrac{\psi}{(H_o\rho^2)^{\frac{1}{3}}}$; USE $\psi = 0.305$ (WOOD)
$\psi = 0.260$ (A328)
$\psi = 0.400$ (A690)

$\log \rho$	-3.1	-2.6	-2.1
rd	0.79	0.76	0.71
T_{op}	5.55	5.33	4.98
$(H_o\rho^2)^{\frac{1}{3}}$	44.5	20.6	9.58
T_{STR} (WOOD)	13.6	6.28	2.92
(A328)	11.6	5.36	2.49
(A690)	17.8	8.24	3.83

c.) RECOMPUTATION OF T_{STR} IS NOT NECESSARY. INSPECTION OF THE GRAPH SUGGESTS THAT LITTLE CHANGE IN T WILL RESULT.

d.) $M = T \cdot H_o^3$

MATERIAL	T	M(in#/ft.)	f_{all} (psi)	S(in³/ft.
WOOD	5.12	29,400	2,000	15.0
A328	5.15	30,000	25,000	1.19
A690	5.05	29,500	32,000	0.92

7.) SELECT MEMBER SIZE:

a.) WOOD: $t = \sqrt{\frac{S}{2}} = \sqrt{\frac{15}{2}} = 2.73$ in ; USE 4×12 (NOMINAL)

b.) A328 ; USE PS 28; $S = 1.9 > 1.19$

c.) A690 ; USE PS 28; $S = 1.9 > 1.19$

8.) TIE-ROD LOAD

a.) FROM FIG. 3.3b, VALUES OF f_c FOR St = 0.435 ARE:

$\log \rho$	-3.1	-2.6	-2.0
f_c	1.40	1.25	1.05

b.) VALUES OF $\log \rho$ CAN BE ESTABLISHED FROM

$$\rho = \frac{H^4}{EI}$$

AND f_c CAN THEN BE INTERPOLATED. THE TIE-ROD LOAD FOR SPACING OF 7'-6" IS THEN COMPUTED BY

$$T = (7.5)\, f_c\, P_{res}$$

MATERIAL	E (psi)	I (in^4/ft.)	ρ	f_c	P_{335}	T (#)
WOOD	1.5×10^6	38.4	-2.74	1.29	916	8850
A328	30×10^6	2.8	-2.90	1.34	916	9230
A690	30×10^6	2.8	-2.90	1.34	916	9230

EXAMPLE 5: USING THE CONDITIONS GIVEN IN EXAMPLE 3, FIND THE PENETRATION DEPTH, BENDING MOMENT AND TIE ROD PULL, USING THE DESIGN CHARTS.

1.) COMPUTE RD

$$R = \frac{\gamma_1 t^3 + \gamma_2 t_2^3}{(5C - \gamma_1 t_1 - \gamma_2 t_2) H^2} = \frac{(100)(4)^3 + (57.6)(8)^3}{[(5)(300) - (100)(4) - (57.6)(8)](12)^2}$$

$= 0.390$

$$C_D = \frac{H_w}{(H - H_A) S_t} = \frac{8}{(12-2)(0.435)} = 1.84$$

$$R_D = R \cdot C_D = (0.390)(1.84) = 0.717$$

2.) COMPUTE D: ENTER FIGURE 4-7 @ $R_D = 0.717$ AND READ OFF D' FOR $\frac{C}{\gamma H} = 0.25$:

$D' = 0.488$

$D = D'H = (0.488)(12) = 5.86'$

3.) COMPUTE R_M & FIND M

$C_M = 1$

$R_M = R \cdot C_M = 0.390$

ENTER FIGURE 4-9 @ $R_M = 0.390$ AND READ OFF M' FOR $\frac{C}{\gamma H} = 0.25$

$M' = 2.99 = \frac{M}{CL}$ (C = COHESION ; L = D)

$M = M' C D^2$

$M = (2.99)(300)(5.86)^2 = 30,800$ in - #/ft.

4.) COMPUTE R_p & FIND P:

$$C_p = \left(\frac{H_A}{D \cdot S_t}\right)^3 = \left[\frac{(2)}{(5.86)(0.435)}\right]^3 = 0.483$$

$$R_p = R \cdot C_p = (0.390)(0.483) = 0.188$$

ENTER FIGURE 4-8 @ R_p = 0.270 AND READ OFF P' FOR:

$$\frac{c}{\gamma H} = 0.25$$

$$P' = 0.554$$

$$P = P'CD = (0.554)(300)(5.86) = 974\ \#/ft.$$

5.) COMPARING THE RESULTS WITH EXAMPLE 4:

DEPTH : −2.7 % DIFFERENCE
BENDING MOMENT: 2.7% DIFFERENCE (A328 STEEL)
TIE-ROD PULL: −21 % DIFFERENCE

THE SIGNIFICANCE OF THE TIE-ROD LOAD CAN BE EXAMINED BY COMPARING THE REQUIRED DIAMETERS.

DESIGN CHART VALUES:

$$T = (974)(7.5) = 7305\ \#$$

$$A_{req} = \frac{7305}{22{,}000} = 0.332\ in^2$$

$$d = \sqrt{\frac{4A}{\pi}} = \left[\frac{(4)(.332)}{\pi}\right]^{\frac{1}{2}} = 0.65$$

HAND CALCULATION:

$$T = 9230\ \#$$

$$A_{req.} = \frac{9230}{22{,}000} = 0.420\ in^2$$

$$d = \left[\frac{(4)(0.420)}{\pi}\right]^{\frac{1}{2}} = 0.73$$

EXAMPLE 6: ATTERBERG LIMIT TESTS PERFORMED ON THE CLAY FRACTION OF THE SUBGRADE MATERIAL IN EXAMPLE 5 REVEALED:

WATER CONTENT: $W = 40\%$
LIQUID LIMIT: $LL = 55\%$
PLASTIC LIMIT: $PL = 34\%$

1.) DETERMINE PLASTICITY INDEX & LIQUIDITY INDEX:

$$PI = LL - PL = 55 - 34 = 21$$

$$IL = \frac{w - PL}{PI} = \frac{40 - 34}{21} = 0.29$$

2.) DETERMINE ACTIVITY (60% CLAY):

$$A = \frac{PI}{\%\,CLAY} = \frac{21}{60} = 0.35$$

THE INDICATORS SUGGEST THAT THIS CLAY SOIL WILL CAUSE NO TROUBLES (LOW ACTIVITY, LOW PLASTICITY AND LOW LIQUIDITY INDEX.) SEE WU, 1976

3.) THE DRAINED STRENGTH CAN BE ESTIMATED AS:

$\phi = 26°$ (WU, 1976)

4.) RECALCULATE PENETRATION DEPTH:

$$R = \frac{\gamma_1 t_1^3 + \gamma_2 t_2^3}{\gamma_3 H^3} = \frac{(100)(4)^3 + (57.6)(8)^3}{(47.6)(12)^3} = 0.436$$

$$C_D = \left(\frac{HW}{H}\right)^2 \left(\frac{HA}{H - HA}\right) = \left(\frac{8}{12}\right)^2 \left(\frac{2}{10}\right) = 0.0889$$

$$R_D = R \cdot C_D = (0.436)(0.0889) = 0.0388$$

ENTER FIGURE 4-10 @ $R_D = 0.0388$ AND READ OFF D' FOR "SANDFILL / PHI = 26°":

$$D' = 0.719$$

$$D = D' \cdot H = (0.719)(12) = 8.62 \text{ ft.}$$

5.) RECALCULATE BENDING MOMENT

$$C_M = \left(\frac{D \cdot HA}{H \cdot HW}\right) = \left[\frac{(8.63)(2)}{(12)(8)}\right] = 0.179$$

$$R_M = R \cdot C_M = (0.436)(0.179) = 0.0780$$

ENTER FIGURE 4-12 @ RM = 0.0780 AND READ OFF M FOR "SAND FILL / PHI = 26"

$$M' = 0.100$$

$$\text{FOR } L = \frac{2}{3}D + H - HA = \left(\frac{2}{3}\right)(8.63) + 10 = 15.75$$

$$M = M'\gamma_s L^3 = (0.098)(47.6)(15.8)^3 = 18{,}400 \text{ in-lb./ft.}$$

6.) RECALCULATE TIE-ROD PULL:

$$C_o = C_M = 0.179$$

$$R_p = R \cdot C_p = RM = 0.078$$

ENTER FIGURE 4-11 @ Rp = 0.0780 AND READ OFF P FOR "SAND FILL / PHI = 26"

$$P = 0.0334$$

$$P = P'\gamma_i L^2 = (0.0334)(100)(15.8)^2 = 334 \text{ lb./ft.}$$

EXAMPLE #7: DETERMINE THE DIAMETER OF THE TIE-ROD BASED UPON THE LOAD GIVEN IN EXAMPLE #1:

1.) GIVEN: T = 7,500 #

2.)

$$d = \sqrt{\frac{4T \cdot LF}{\pi f_t}} \qquad \text{(EQ. 5-17)}$$

a.) CHOOSE LF = 1.2 (SEC. 5.2.6)

b.) $f_t = 0.60 f_y$ (EQ. 5-12c)
$= (0.60)(36{,}000)$ (TAB 5-5)
$= 21{,}600$ psi

c.)
$$d = \sqrt{\frac{(4)(7500)(1.2)}{\pi (21{,}600)}}$$
$= 0.728$ IN.

3.) ADD 1/8 IN. FOR FRESH WATER (TAB 5-9)
(d = 0.853 IN., USE 7/8 IN.)

ADD 1/4 IN. FOR SALT WATER
(d = 0.978 IN., USE 1 IN.)

4.) USE A 1 1/8 HOLE FOR THE TIE-ROD BEARING PLATE,
A 1 1/32 INCH HOLE FOR THE WALE AND PILE (WOOD WALES)

USE A 1 1/8 HOLE FOR THE TIE-ROD PASSING THROUGH STEEL SHEET PILES.

EXAMPLE #8: GIVEN THE LOADS IN EXAMPLE #1, DESIGN A WALE FOR STEEL AND WOOD SHEET PILES.

1.) GIVEN: P = 1000 #/ft., L = 7.5 ft.

2.) DETERMINE MOMENT AND SECTION MODULUS REQUIRED

$M = \frac{1}{9} PL^2$

$= (\frac{1}{9})(1000)(7.5)^2(12)$ (EQ. 5-18)

$= 75{,}000$ IN.-LB.

$S = \frac{M}{f_b}$ (EQ. 5-9)

$= (75{,}000)/(22{,}000)$ (A36 STEEL)

$= 3.41$ IN.^{3}, USE 2 EA. C4×5.4 CHANNELS

$S = 1.93$ IN.^{3}/ PER CHANNEL × 2 CHANNELS (TAB 5-3)

$= 3.86$ IN.$^3 > 3.41$ IN.^{3}

3.) $S = \frac{M}{f_b} = \frac{75000}{2000}$ (SOUTHERN PINE)

$= 37.5$ IN.^{3}

4.) USE 4×10 MEMBER, $S = 54.53$ IN.^{3} (TAB 5-46)
A 3×10 SECTION HAS ADEQUATE SECTION MODULUS, HOWEVER A 1 1/32 IN. HOLE LEAVES ONLY 0.8 IN. OF WOOD BETWEEN BOLT AND EDGE OF WALE.

EXAMPLE #9: DETERMINE THE SIZE AND NUMBER OF NAILS REQUIRED TO FASTEN THE PILES DESIGNED IN EXAMPLE #1 TO

1.) GIVEN: $P = 1000$ #/ft.
$t = 2\frac{5}{8}$ in. (3×12 NOMINAL)
TIMBER MATERIAL IS SOUTHERN PINE

2.) FIND G:
$G = 0.59$ (TAB 5-6)

3.) TRY A 40 PENNY NAIL (40d)

$\ell = 5$ in. (TAB 5-7)
$p = 83$ #/in.

$\ell_e = 5\text{ in.} - 2\frac{5}{8}\text{ in.} = 2\frac{3}{8}$ in.

$W_r = p\ell_e$ (EQ. 5-13)
$= (83)(2,375)$
$= 197$ #/NAIL

4.) NUMBER OF NAILS

$n = \frac{P}{W_r}$ (EQ. 5-19)

$= \frac{1000}{97}$

$= 5.08$, USE 6 NAILS/PILE

5.) TRY A 40d SPIKE

$\ell = 5$ in.
$p = 97$ #/in.

$W_r = p\ell_e$
$= (97)(2.375)$
$= 230$ #/SPIKE

$n = \frac{P}{W_r} = \frac{1000}{23} = 4.35$, USE 5 SPIKES/PILE

EXAMPLE #10: DETERMINE THE NAIL SIZE REQUIRED TO FASTEN SHEET PILES TO AN OUTSIDE WALE.

1.) GIVEN: $t = 2\,5/8$ IN.

2.) $\ell = 5/3\, t$ (EQ. 5-20b)
$= (5/3)(2.625)$
$= 4.375$ IN.

3.) USE 30d NAIL ($\ell = 4\,1/2$ IN.) (TAB 5-7)

EXAMPLE #11: DESIGN A BEARING PLATE FOR THE TIE-ROD DESIGNED IN EXAMPLE #7

1.) GIVEN: $T = 7500$ #
$d = 1$ IN. ($1\,1/8$ IN. HOLE)

2.) DETERMINE AREA REQUIRED

$A = T/F_p$ (EQ. 5-21)
$= (7500)/(455)$ (TAB 5-4)
$= 16.48$ IN.^{2}

3.) SIZE THE PLATE
$A = bh - A_{HOLE}$, USE $b = 3\,1/2$ IN.
$h = (A + A_{HOLE})/b$

$A = \frac{\pi}{4} d^2$

$= (\frac{\pi}{4})(1.125)^2$
$= 0.99$ IN.^{2}
$h = (16.48 + .99)/(3.5)$
$= 4.99$, USE $3\,1/2 \times 5$ IN. PLATE

4.) DETERMINE F_p; N AND t

$F_p = T/(A_{PL} - A_{HOLE})$
$= (7500)/[(3.5)(5) - (0.99)]$
$= 454$ psi

$N = 1/2\,(b - d_{HOLE})$
$= 1/2\,(3.5 - 1.125)$
$= 1.19$

$t = \sqrt{\dfrac{3 F_p N^2}{F_b}}$ (EQ. 5-22)

$= \left[\dfrac{(3)(454)(1.19)^2}{22{,}000}\right]^{\frac{1}{2}}$

$= 0.30$

USE PL $3/8 \times 3.5 \times 5$

EXAMPLE # 12 : A UNIFORMLY DISTRIBUTED SURCHARGE LOAD OF 200 LB. PER SQ. FT. IS TO BE PLACED UPON THE BACKFILL OF THE SITE DESCRIBED IN EXAMPLE #1. DETERMINE THE REQUIRED PENETRATION DEPTH, TIE-ROD LOAD, AND MAXIMUM BENDING MOMENT.

1.) GIVEN: $q = 200\ \#/ft^2$
GEOMETRY AND SOIL CONDITIONS GIVEN IN EX. #1

2.) THE EFFECT OF THE UNIFORMLY DISTRIBUTED SURCHARGE IS A RECTANGULAR STRESS DISTRIBUTION IN EACH SOIL LAYER, AS SHOWN IN FIG. 4-2. COMPUTE THE RESULTING MOMENTS ABOUT THE TIE-ROD AND ADD TO THE MOMENTS COMPUTED IN EXAMPLE #1.

$$(Ka_1' q t_1)(\tfrac{1}{2}t_1 - HA) + (Ka_2' q t_2)(\tfrac{1}{2}t_2 + t_1 - HA) + (Ka_3' q D)(\tfrac{1}{2}D + t_1 + t_2 - HA) + (10{,}400 + 3{,}360 D - 713 D^2 - 58.8 D^3) = 0$$

$$(0) + (3670) + (38.2 D^2 + 764 D) + (10{,}400 + 3360 D - 713 D^2 - 58.8 D^3) = 0$$

$$14{,}070 + 4120 D - 675 D^2 - 58.8 D^3 = 0$$

$$D = 6.2'$$

3.) SUM MOMENTS ABOUT $2/3\ D$ TO DETERMINE TIE-ROD LOAD

$$(Ka_1 q t_1)(\tfrac{1}{2}t + t_2 + \tfrac{2}{3}D) + (\tfrac{1}{2}Ka_1 \gamma_1 t_1^2)(\tfrac{1}{3}t_1 + t_2 + \tfrac{2}{3}D) + Ka_2 t_2(\gamma_1 t_1 + q)(\tfrac{1}{2}t_2 + \tfrac{2}{3}D) + (\tfrac{1}{2}Ka_2 \gamma_2 t_2^2)(\tfrac{1}{3}t_2 + \tfrac{2}{3}D) + Ka_3 D(\gamma_1 t_1 + \gamma_2 t_2 + q)(\tfrac{1}{6}D) - P(H - \tfrac{2}{3}D - HA) = 0$$

$$3150 + 3010 + 9990 + 3340 + 1770 - 14.1 P = 0$$

$$P = 1510\ \#/FT.$$

4.) FIND POINT OF ZERO SHEAR, X FT BELOW THE WATER LEVEL (t_1):

$$P - \tfrac{1}{2}Ka_1(\gamma_1 t_1 + q)t_1 - \tfrac{1}{2}Ka_2\gamma_2 X^2 - Ka_2(\gamma_1 t + q)X = 0$$

$$7.68x^2 + 154x - 1175 = 0$$

$$x = \frac{-b + \sqrt{b^2 - 4ac}}{2a} \qquad \text{WHERE } a = 7.68,\ b = 154,\ c = 1175$$

$x = 5.90'$ BELOW t_1

5.) FIND MAXIMUM MOMENT

$$M_{MAX} = P(t_1 + x - H_A) - \tfrac{1}{2} K_{a1} \gamma_1 t_1^2 (\tfrac{1}{3} t_1 + x) - K_{a1} q (\tfrac{1}{2} t_1 + x) - \tfrac{1}{2} K_{a2} (\gamma_1 t_1 + q) x^2 - \tfrac{1}{6} K_{a2} \gamma_2 x^3$$

$$= 11{,}930 - 1610 - 440 - 2670 - 530$$

$$= 6680 \text{ ft.-lb./ft.}$$

6.) COMPUTE α & β:

$$\alpha = \frac{H}{H_O} = \frac{12}{(12+6.2)} = 0.66$$

$$\frac{H_A}{H_O} = \frac{2}{(12+6.2)} = 0.11$$

7.) TIE-ROD LOAD:

$f_c = 0.95$ (FIG 2-17b)

$P = (0.95)(1510) = 1435$ #/ft.

8.) COMPUTE REDUCTIONS FROM OPERATING AND STRUCTURAL CURVES FOR WOOD, AS IN EX. #1:

$$\rho_{MAX} = (12)(6680)/(18.2)^3 = 13.3$$

$$\rho = 4.56$$

$$m = \rho H_O^3 = (2.95)(18.2)^3$$
$$= 27{,}510 \text{ in-\#}$$

EXAMPLE #13: USE THE SIMPLIFIED METHOD FROM THE PRECEDING SITUATION

1.) DETERMINE THE EQUIVALENT HEIGHT OF SOIL FOR q AND ADD THIS TO THE FREE STANDING WALL HEIGHT, H':

$$H_{eq} = \frac{q}{\gamma_1} = \frac{200}{100} = 2 \text{ FT.}$$

$$H = 12 + 2 = 14 \text{ FT.}$$

2.) FROM EX. #3, $\frac{D}{H} = 0.436 = D'$

$\therefore D = D'H = (0.436)(14) = 6.1$ FT.

3.) FROM EX. #3, $M' = \frac{M}{\gamma_3 L^3} = 0.103$

$L = \tfrac{2}{3} D + H - H_A = (\tfrac{2}{3})(6.1) + (14) - (2) = 16.1$ FT.

$M = M' \gamma_3 L^3 = (0.103)(60)(16.1)^3 = 25{,}800$ IN. #/FT.

4.) FROM EX. #3, $P' = \frac{P}{\gamma_1 L^2} = 0.0622$

$P = P' \gamma_1 L^2 = (0.0622)(100)(16.1)^2 = 1612$ #/FT.

EXAMPLE #14: DETERMINE THE PENETRATION DEPTH, BENDING MOMENT, AND TIE-ROD LOAD FOR THE WALL IN THE PREVIOUS EXAMPLE. INSTEAD OF A POINT LOAD, CONSIDER A CONTINUOUS FOUNDATION FOOTING 10 FT. FROM THE SHEET PILES WITH A LOAD OF 5 KIPS/FT.

1.) GIVEN: Q_l = 5000 #/FT.
X = 10 FT.
GEOMETRY AND SOIL CONDITIONS REMAIN UNCHANGED.

2.) $M = \frac{X}{H} = \frac{10}{12} = 0.83$

$P_H = \frac{0.64\,Q_l}{(M^2+1)} = \frac{(0.64)(5,000)}{(0.83)^2+1} = 1890$ #/FT. (FIG 5-1a)

3) EXTRAPOLATE L FROM FIGURE 5-1b. FOR M = 0.83,

$L = 0.43\,H = 5.16$ FT.

4) SUM MOMENTS ABOUT TIE-ROD, AS IN PREVIOUS EXAMPLE:

$P_H\,(H - L - H_A) = (1890)(12 - 5.16 - 2) = 9150$

$(9150) + (10{,}400) + 33600 - 713\,D^2 - 58.8\,D^3 = 0$

$D = 6.2$

5) SUM MOMENTS ABOUT ⅔ D. P_H ACTS @ (L + ⅔ D) FROM ⅔ D:

$P_H\,(L + \tfrac{2}{3}D) = (1890)\left[12 + (\tfrac{2}{3})(6.2)\right] = 17{,}560$

$(17{,}560) + (3000 + 3340 + 1440) = 14.1\,P$

$P = 1800$ #/FT.

6) FIND POINT OF ZERO SHEAR:

$C = \tfrac{1}{2}\,K_{a_1}\,\gamma_1\,t_1^2 - P_H - P = 313$

THE VALUE OF C IS POSITIVE, WHICH INDICATES THAT THE SHEAR FORCE DIAGRAM CHANGES ABRUPTLY (AT THE POINT OF PH) FROM POSITIVE TO NEGATIVE. THIS IS WHERE THE MAXIMUM MOMENT WILL OCCUR.

$X = H - L - t_1 = 2.84'$ BELOW THE WATER LEVEL.

7) FIND M_{MAX}

$M_{MAX} = (1800)(4.84) - (223)(4.17) - (60) - (413) = 7310$ FT.-#/FT.

8) COMPUTE THE TIE-ROD LOAD

$\beta = 2/18.2 = 0.11 \quad \alpha = 12/18.2 = 0.66, \; R_c = 0.95$ (FIG. 2-17b)

$P = R_c P_{FES} = (0.95)(1800) = 1710$ #/FT.

9) COMPUTE BENDING MOMENT REDUCTIONS

$\tau_{max} = (12)(7310)/(18.2)^3 = 14.6$

GENERATING NEW τ_{ap} VALUES USING THE SAME REDUCTION FACTORS WILL GIVE

$\tau = 5.90$

$M = \tau H_0^3 = (5.90)(18.2)^3 = 35,600$ IN. #/FT.

EXAMPLE #15 : USE THE SIMPLIFIED METHOD FROM THE PRECEDING SITUATION.

1.) DETERMINE AN EQUIVALENT HEIGHT OF SOIL FOR PH AND ADD THIS TO THE FREE STANDING WALL HEIGHT, H:

$$H_{eq.} = \frac{PH}{\gamma_1 (H-L)} = \frac{1890}{(100)(12-5.16)} = 2.77'$$

$H = 12 + 2.76 = 14.8$

2) FROM EX. #3, $\frac{D}{H} = 0.436 = D'$

$\therefore D = D'H = (0.436)(14.8) = 6.5'$

3) FROM EX. #3, $M' = \frac{M}{\gamma_3 L^3} = 0.103$

$L = 2/3\,D + H - HA = (2/3)(6.5) - (14.8) - (2) = 17.13'$

$M = M' \gamma_3 L^3 = (0.103)(60)(17.13)^3 = 31{,}100$ IN. #/FT.

4) FROM EX. #3, $P' = \frac{P}{\gamma L^2} = 0.0622$

$P = P' \gamma_1 L^2 = (0.0622)(100)(17.1)^2 = 1820$ #/FT.

EXAMPLE # 16: A 10,000 LB. LOAD IS TO BE LOCATED 5 FT. FROM THE SHEET PILES OF THE WALL GIVEN IN EX. #1. DETERMINE THE REQUIRED PENETRATION DEPTH, TIE-ROD LOAD, AND MAXIMUM BENDING MOMENT.

1.) GIVEN: $Q_P = 10{,}000$ # (FIG 5-1d)

$x = 5$ ft.

GEOMETRY AND SOIL CONDITIONS GIVEN IN EX. #1

2) $m = \frac{x}{H} = \frac{5}{10} = 0.5$

$P_H = 0.45 \frac{Q_P}{H} = 450$ #/ft.

3.) INTERPOLATE L FROM FIG 5-1b

$L = 0.54H = 6.48$ ft.

4.) SUM MOMENTS ABOUT TIE-ROD:

PH ACTS AT 5.4 FT. FROM DL OR (H-L-HA) = 3.52 FT. FROM TIE-ROD.

ADD PH (H-L-HA) TO MOMENTS COMPUTED IN STEP 4, EX. #1:

$(450)(3.52) + 10{,}400 - 3360D - 713D^2 - 58.8D^3 = 0$

$D = 5.5$ FT.

5.) SUM MOMENTS ABOUT 2/3 D. PH ACTS AT A DISTANCE $(L + 2/3\,D) = 15.7$ FT. FROM 2/3 D

ADD PH $(L + 2/3\,D)$ TO MOMENTS COMPUTED IN STEP 5, EX. #1:

$(450)(15.7) + (2900 + 3110 - 6280 + 1140) = 13.7\,P$

$P = 1500$ #/FT.

6.) FIND POINT OF ZERO SHEAR AS IN STEP 6, EX. #1, EXCEPT THAT:

$C = 1/2\ K_{a_1} \gamma_1 t_1^2 + P_H - P = -823$

$X = 7.10$ FT. BELOW THE WATER LEVEL (i.e., BELOW t_1)

7.) FIND THE MAXIMUM MOMENT, AS IN STEP 7, EX. #1 INCLUDING THE MOMENT CAUSED BY PH $(L + t_1 + X - H)$

$$M_{MAX} = -(450)(6.48 - 4 + 7.10 - 12) + (1500)(9.10) - (223)(8.43) - (916) - (2580)$$

$$= 5760 \text{ FT.}\#/\text{FT.}$$

8.) COMPUTE THE TIE-ROD LOAD, AS IN STEP 8, EX. #1:

$\beta = 2/17.5 = 0.11$, $\alpha = 12/15.5 = 0.69$, $f_c = 1.0$ (FIG 2-17b)

$P = f_c\, P_{FES} = (1.0)(1500) = 1500\ \#/\text{FT.}$

9.) COMPUTE BENDING MOMENT REDUCTION AS IN STEP 9, EX. #1:

$\Upsilon_{MAX.} = (12)(5760)/(17.5)^3 = 12.90$

GENERATE NEW Υ_{op} VALUES USING THE SAME REDUCTION FACTORS AS IN EX. #1.

$\Upsilon = 3.48$

$m = \Upsilon H_D{}^3 = (3.48)(17.5)^3 = 18{,}650$ IN. LB./FT.

EXAMPLE #17: USE THE SIMPLIFIED METHOD FOR THE PRECEDING SITUATION.

1.) DETERMINE AN EQUIVALENT HEIGHT OF SOIL FOR PH AND ADD THIS TO THE FREE STANDING WALL HEIGHT, H:

$$H_{eq} = \frac{PH}{\gamma_1 (H-L)} = \frac{450}{(100)(12-6.48)} = 0.82$$

$H = 12 + 0.82 = 12.82$

2) FROM EX. #3, $\frac{D}{H} = 0.436 = D'$

$\therefore D = D'H = (0.436)(12.82) = 5.59 \approx 5.6'$

3) FROM EX. #3, $M' = \frac{M}{\gamma_3 L^3} = 0.103$

$L = 2/3\, D + H - HA = (2/3)(5.6) + (12.82) - (2) = 14.6$

$M = M' \gamma_3 L^3 = (0.103)(60)(14.6)^3 = 19{,}800$ IN. #

4.) FROM EX. #3, $P' = \frac{P}{\gamma_1 L^2} = 0.0622$

$P = P' \gamma_1 L^2 = (0.0622)(100)(14.6)^2 = 1330\ \#/\text{FT.}$

EXAMPLE 18: DESIGN A 2 MEMBER SPLICE FOR AN INSIDE WALE HAVING THE DIMENSIONS AND LOADS AS IN EXAMPLE #8.

1.) GIVEN: WALE IS 4×10
$M = 75000$ in-lb.

2) SELECT L_B, FIND V
TRY $L_b = 24$ in.

$$V = \frac{T}{2} - \frac{PL_b}{4}$$

$$= \left(\frac{7500}{2}\right) - \left(\frac{1000}{4}\right)\left(\frac{24}{12}\right) \qquad \text{(EQ. 5-23)}$$

$$= 3250\ \#$$

3.) USE THE SAME SIZE MEMBER AS THE WALE FOR THE SPLICE PLATE, SELECT d AND Q BASED ON b

FOR 4×10, $b = 3\frac{5}{8}$,

$Q = 1620\ \#$ FOR $d = 1$ in, (TAB 5-6)

FOR 2-MEMBER JOINTS OF EQUAL b, USE $\frac{1}{2}$ Q

$\therefore Q = 810\ \#$

4) NUMBER OF BOLTS REQUIRED @ EACH END

$N = \frac{3250}{810} = 4.01$ ∴ USE 2 ROWS OF 2 BOLTS

5.) DETERMINE DISTANCE REQUIREMENTS FOR BOLT DIAMETER OF 1 in ($l/d = 3.625/1 = 3.625$)

EDGE = 4 in. BOLT SPACING = 4 (TAB 5-10)
END = $1\frac{1}{2}$ in. ROW SPACING = 3.5

6.) THE DISTANCE REQUIREMENTS FOR EDGE AND ROWS OF BOLTS EXCEED THE DIMENSION OF THE MEMBER. ∴ REPEAT STEPS 2 THRU 5, USING $L_B = 21$ INCHES. THIS WILL PERMIT OVERALL LENGTH OF THE SPLICE PLATE OF 24 IN. ALLOWING END DISTANCES OF $1\frac{1}{2}$ IN. @ EACH END.

7.) $V = \left(\frac{7500}{2}\right) - \left(\frac{1000}{4}\right)\left(\frac{21}{12}\right) = 3312$

FOR $\frac{5}{8}$ IN BOLT, $Q = \frac{1210}{2} = 605$ #

$n = 3312 = 5.48$ ∴ USE 2 ROWS OF 3 BOLTS

FOR A $\frac{5}{8}$ IN. BOLT, $l/d = 5.8$

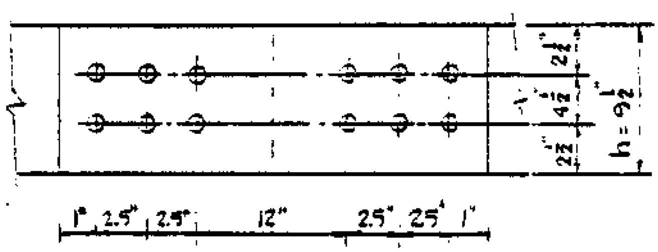
EDGE = 2.5"   BOLT SPACING = 2.5"
END = 0.938, USE 1"

ROW SPACING : l/d = 3.625/0.625 = 5.8

$nd = (\frac{5}{8})(5.8) - 1\frac{1}{4} = 4.375$

ROW SPACING = $(4.875)(\frac{5}{8})$ = 3.05, SAY 3 IN.

8) USE 6 EACH $\frac{5}{8}$ IN. BOLTS IN EACH END



EXAMPLE 19: DESIGN A 3 MEMBER SPLICE USING THE DATA FROM THE PREVIOUS EXAMPLE.

1.) GIVEN: WALE IS 4×10
m = 75000 IN-LB

2.) USE SAME L_a AS PREVIOUS EXAMPLE

FOR L_a = 21 IN., V = 3312

3.) SELECT A SPLICE DIMENSIONS:
THE SECTION OF EACH PLATE MUST BE ½ THE REQUIRED.

REQUIRED S = 37.5 IN^3; ½S = 18.75

USE 2×10 (S = 24.44 > 18.75) (TAB 5-4b)

a = 1.625 (FIG. 5-11)

TAKE b = 2a = (2)(1.625) = 3.25

USE ℓ = b = 3.0 IN TABLE 5-8

4) SELECT d AND Q

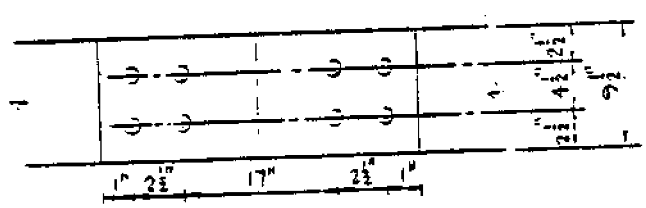
FOR 5/8 IN. BOLT, Q = 1000

$n = \frac{3312}{1000} = 3.31$ ∴ USE 2 ROWS OF 2 BOLTS

5.) DETERMINE SPACING FOR $\ell/d = 3/\frac{5}{8} = 4.8$

EDGE = 2.5
END = 0.938

BOLT SPACING = 2.5
ROW SPACING = (4.25) d
= 2.656

6.) USE 4 EA. 5/8 IN. BOLTS @ EACH END

EXAMPLE #20: DETERMINE THE FASTENERS REQUIRED FOR THE STEEL SHEET PILE WALL IN EXAMPLE #1 AND THE WALES IN EXAMPLE #8.

1.) GIVEN: PS28 SECTION
C3×5 WALES
P = 1000 #/FT.

2) DETERMINE THE NUMBER OF BOLTS REQUIRED FOR AN INSIDE WALE.

USE W = 15 IN. (TAB 5-2)
SELECT A 5/8 IN. BOLT (SMALLEST BOLT)

$$n = \frac{4PW}{\pi d^2 f_t}$$

$$= \frac{(4)(1000)(\frac{15}{12})}{\pi (0.625)^2 (40{,}000)}$$

$$= 0.102$$

USE 1 BOLT EVERY OTHER SECTION

3.) DIMENSION THE FIXING PLATE

USING PIPE SEPARATORS 2 IN. LONG GIVES A SPAN BETWEEN CHANNELS OF 2 IN.

USING 1 EA. 5/8 IN. BOLT EVERY OTHER SECTION EXERTS A TENSILE FORCE IN THE BOLT OF

$$F = 2PW = (2)(1000)\left(\frac{15}{12}\right)$$

$$= 2500 \#$$

THE MOMENT IN THE FIXING PLATE IS

$$M = \frac{1}{4} PL = (\tfrac{1}{4})(2500)(2)$$

$$= 1250 \text{ IN. LB.}$$

$S = \frac{M}{f_b}$ AND $S = \frac{1}{6}bt^2$ FOR RECTANGULAR SECTIONS

$$t = \sqrt{\frac{6M}{bf_b}}$$

FOR $b = 3$ IN.

$$t = \sqrt{\frac{(6)(1250)}{(4)(22{,}000)}} = 0.29, \quad \text{USE } t = \frac{3}{8} \text{ IN.}$$

EDGE DISTANCE $= 1.25\,d = (1.25)(5/8) = 0.78$ IN.

REQUIRED MINIMUM DIMENSION IS TWICE THE EDGE DISTANCE PLUS THE BOLT HOLE. THE BOLT HOLE IS 1/8 IN. LARGER THAN THE BOLT.

$$\ell = (2)(0.78) + (5/8) + (1/8) = 2.31 \text{ IN. MIN.}$$

USE ℞ 3/8 × 3 × 3

EXAMPLE #21: DESIGN SPLICE PLATES FOR THE WALLS DESIGNED IN EXAMPLE #6.

1.) GIVEN: M = 75000 IN. LB.
C4 x 5.4 CHANNELS

2) THE PLATE WIDTH IS LIMITED BY THE FLANGE-TO-FLANGE WIDTH OF THE CHANNELS, EDGE DISTANCE AND BOLT HOLE DIAMETER.

$b = d - 2t_f$ (TAB 5-3)

$= (4.00) - (2)(0.296)$
$= 3.41''$

FOR A 5/8 IN. BOLT, THE EDGE DISTANCE AND BOLT HOLE REQUIREMENTS GIVE A MINIMUM b OF 2.31 IN. (EX. #13)
∴ USE b = 3 1/4 IN.

3) $S = \frac{M}{F_b} = 3.41 \text{ IN.}^3$ (FROM EX. #8) (EQ. 5-9)

$S = \frac{1}{6} t b^2$ (BENDING ABOUT STRONG AXIS) (EQ 5-10b)

$t = \frac{6S}{b^2} = 1.94$ IN. FOR 2 PLATES (TOP & BOTTOM CHANNELS)

4.) USE A 12 IN. LONG PLATE, MINIMUM EDGE DISTANCE IS 1.50d, OR 0.94 IN. FOR 5/8 IN. BOLTS. USE L_b = 10 IN.

$V = \frac{T}{2} - \frac{PL_b}{4}$ (EQ 5-23)

$= \left(\frac{2500}{2}\right) - \left(\frac{1000}{4}\right)\left(\frac{10}{12}\right)$
$= 3542$ #

5.) CAPACITY OF A 5/8 BOLT IN SINGLE SHEAR IS

$F_v = (15{,}000)A = (15{,}000)\left(\frac{\pi}{4}\right)\left(\frac{5}{8}\right)^2 = 4600$#

CAPACITY IN DOUBLE SHEAR IS 9200# > 3542#

∴ USE 1 EACH 5/8 IN. BOLT @ 1 IN. FROM THE END.
USE PL 1"x3"x12" (2 EACH)

EXAMPLE #22: GIVEN THE CONDITIONS OF EXAMPLE #1, DESIGN A CONTINUOUS DEADMAN ANCHORAGE.

1.) GIVEN: $t_1 = 4$ ft. $\gamma_1 = 100$ pcf $K_P' = 3.00$
$H_A = 2$ ft. $\gamma_2 = 60$ pcf . $K_A' = 0.408$

2) SELECT $h_1 = 1$ ft., $K_P' - K_A' = 2.59$

3.) LET $h_w = H_A$, ALTHOUGH THE TIE-ROD IS LOCATED SLIGHTLY ABOVE THE WATER LINE.

4) COMPUTE THE RESULTANT FORCES ACTING ON THE ANCHORAGE (FIGURE 5-25)

a.) NET FORCES:

$(K_P' - K_A')\gamma_1 h_1 h_L = (2.59)(100)(1)h_L = 259\,h_L$

$\frac{1}{2}(K_P' - K_A')\gamma_1 (h_w - h_1)^2 = \frac{1}{2}(2.59)(100)(2-1)^2 = 129.5$

$(K_P' - K_A')\gamma_2 (h_w - h_1)(h_1 + h_L - h_w) = (2.59)(100)(2-1)(h_L - 1)$

$= 259\,h_L - 259$

$\frac{1}{2}(K_P' - K_A')\gamma_2 (h_1 + h_L - h_w)^2 = \frac{1}{2}(2.59)(60)(h_L - 1)^2$

$= 77.7\,h_L^2 - 155.4\,h_L + 77.7$

b.) SUM NET FORCES, EQUATE TO TIE-ROD PULL/UNIT LENGTH.

$P = 259\,h_L + 129.5 + 259\,h_L - 259 + 77.7\,h_L^2 - 155.4\,h_L - 77.7$

$1000 = 77.7\,h_L^2 + 358.6\,h_L - 51.8$

c.) SOLVE THE QUADRATIC FOR h_L

$77.7\,h_L^2 + 358.6\,h_L - 1051.8$

$$h_L = \frac{-358.6 \pm \sqrt{(358.6)^2 - (4)(77.7)(-1051.8)}}{(2)(77.7)}$$

$= 2.04, -6.55$

USE POSITIVE ROOT, $h_L = 2.04'$, 2.00 IS OK.

5) USING THE SAME MATERIAL AS THE WALL REQUIRES NO FURTHER DESIGN. WALES ON THE ANCHORAGE ARE THE SAME AS FOR THE WALES ON THE WALL.

6.) ENSURE THAT THE TOE OF THE WALL DOES NOT INTERSECT THE FAILURE WEDGE (FIGURE 5-8).

EXAMPLE #23: USING THE DATA OF EXAMPLE #15 DESIGN A SHORT DEADMAN.

1.) GIVEN: $(P_p - P_a) = 77.7 h_L^2 + 358.6 h_L - 51.8$

$\gamma_1 = 100 \quad K_p' = 3.00 \quad \phi_c = 21°$
$\gamma_2 = 60 \quad K_a' = 0.408 \quad K_o = 0.4$

2.) SELECT A LENGTH : L = 4 FT.

3.) INCORPORATE THE DATA OF EX. #15 INTO EQ. 5-7:

$$T_{ULT} = L(P_p - P_a) + \tfrac{1}{3} K_o \gamma (\sqrt{K_p'} + \sqrt{K_a'}) h_L^3 \tan \phi_c$$

2 VALUES OF γ, γ_1 OVER THE LENGTH $h_w - h_1$
γ_2 OVER THE LENGTH $h_1 + h_L - h_w$:

$$T_{ULT} = L(P_p - P_a) + \tfrac{1}{3} K_o (\sqrt{K_p'} + \sqrt{K_a'}) \tan \phi_c \left[(\gamma_1)(h_w - h_1)^3 + (\gamma_2)(h_1 + h_L - h_w)^3\right]$$

$$= (4)(77.7 h_L^2 + 358.6 - 51.8) + (\tfrac{1}{3})(0.4)(2.37)(0.384)\left[(100)(1)^3 + (60)(h_L - 1)^3\right]$$

$$7500 = 154.8 h_L^2 + 1434 h_L - 207.2 - 12.1 + 7.28 (h_L - 1)^3$$

$$7500 = 7.28 h_L^3 + 133 h_L^2 - 1456 h_L - 202$$

$$0 = 7.28 h_L^3 + 133 h_L^2 + 1456 - 7700$$

4) SOLVE THE CUBIC BY TRIAL AND ERROR

$h_L = 3.74'$ USE $3.75'$

5) DETERMINE REQUIREMENTS IF AN 8 IN. DIAMETER PILE IS USED

$L = \frac{8}{12} = 0.9$

$$7500 = 30.0 h_L^2 + 287 h_L - 41.4 + 12.1 - 7.28 (h_L - 1)^3$$

$$0 = 7.28 h_L^3 + 8.16 h_L^2 + 309 h_L - 7522$$

$h_L = 8.4'$, TOO LARGE. 8 IN. PILES ARE NOT FEASABLE BY THEMSELVES.

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