CHAPTER 6

CONSTRUCTION CONSIDERATIONS

The construction of bulkheads is less complicated than the design process. 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. General Construction Procedure

6.1.1. Pile Installation

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
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.

SACRFILLING to the desired height conceals the ROCKS and provides for a level property to the waterfront.

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/m²) 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

\[
LF = \frac{1}{\cos \theta}
\]  

(6-1)
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
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 $\theta$ = 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 overstrengthening.
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 over-stressing 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 6-3. 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

$$P_f = 1 - R$$

(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/m²) 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/m²) and 99 percent of the same material can be assumed to possess an ultimate strength of 2,500 psi (17.2 MN/m²). Tie-rods made from grade A36 steel must possess a minimum yield strength of 36,000 psi (248 MN/m²). 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/m²). 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/m²).

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

\[
FS = \frac{C}{D} \quad (7-2)
\]

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

\[
SM = C - D \quad (7-3a)
\]

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

\[
\text{SM} = \bar{C} - \bar{D}, \quad \text{and} \tag{7-3b}
\]

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

\[
z = \frac{\text{SM}}{S_{SM}} \tag{7-4}
\]

From this value can be determined the probability that \( \text{SM} \leq 0 \), 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

\[
\begin{align*}
H &= 10' \text{ (3.05 m)} \\
H_W &= 6' \text{ (1.83 m)} \\
H_A &= 2' \text{ (0.61 m)} \\
t_1 &= 4' \text{ (1.22 m)}, \text{ and} \\
t_2 &= 6' \text{ (1.83 m)}
\end{align*}
\]

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:

\[
\begin{align*}
\gamma_1 &= 100 \text{ pcf} \text{ (15.8 kN/m}^3\text{)} \\
\phi_1 &= 30 \text{ degrees} \\
\gamma_2 &= 120 - 62.4 \\
&= 57.6 \text{ pcf} \text{ (9.09 kN/m}^3\text{), and} \\
\phi_2 &= 30 \text{ degrees}
\end{align*}
\]

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

\[
\phi_f = \tan^{-1} \left( \frac{1}{SF} \cdot \tan \phi \right) \quad (3-1)
\]

in which \(SF\) = an appropriate safety factor, taken as 1.5, \(\phi = \) angle of
internal friction, unfactored, and \( \phi_f \) = angle of internal friction, factored. The tie-rod diameter is then calculated based on an allowable tensile strength, \( f = 22,000 \) psi (151 MN/m²). Finally, the sheet pile member thickness is selected based upon an allowable flexural stress of \( f = 2,000 \) psi (13.8 N/m²). 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

\[
M = \frac{1}{2} K_a y_1 t_1^2 \left( \frac{2}{3} t_1 - H_a \right) \\
+ \frac{1}{2} K_a y_2 t_2^2 \left( \frac{2}{3} t_2 + t_1 - H_a \right) \\
+ \frac{1}{2} K_a y_3 b^2 \left( \frac{2}{3} D + H - H_a \right) \\
+ K_a y_1 t_1 t_2 \left( \frac{1}{2} t_2 + t_1 - H_a \right) \\
+ K_a y_1 t_1 + y_2 t_2 \right) D \left( \frac{1}{2} D + H - H_a \right).
\] (7-6a)

For the geometry of this situation and for \( y_2 = y_3 \), and \( K_a = K_{a_1} = K_{a_2} = K_{a_3} \),

\[
M = K_a \left[ (318) y_1 + (517) y_2 \right].
\] (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
\[ R = \frac{1}{2} K_p \gamma_3 D^2 \left( \frac{2}{3} D + H - H_A \right), \text{ or} \]  
(7-7a)

\[ R = K_p \gamma_3 (121). \]  
(7-7b)

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

\[ V = \frac{\bar{x}}{S_x} \times 100\% \]  
(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

\[ V_{K_A} = 1.15 V_\phi, \text{ and} \]  
(7-9a)

\[ V_{K_P} = 1.10 V_\phi. \]  
(7-9b)

For example, for an angle of internal friction, \( \phi = 30 \) degrees, \( V_{K_A} = 16.1 \) percent and \( V_{K_P} = 15.4 \) percent.

The standard deviations associated with stress coefficients \( K_A = 0.279 \) and \( K_p = 5.74 \) are \( S_{K_A} = 0.0449 \) and \( S_{K_P} = 0.884 \) 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

\[
\gamma = \frac{(G_s + e)}{(1 + e)} \gamma_w
\]

in which \( \gamma_w \) = unit weight of water.

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

\[
S_y^2 = \sum_{i=1}^{n} \left( \frac{\partial y}{\partial x_i} \right)^2 \left( S_{x_i} \right)^2
\]

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

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

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

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

\[
S_y = 3.30 \text{ lb/ft}^3 (0.521 \text{ kN/m}^3).
\]

Using Equations 7-7 through 7-ll 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

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Penetration</th>
<th>Tie-Rod Pull</th>
<th>Bending Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D (ft-lb)</td>
<td>C (ft-lb)</td>
<td>D (lb)</td>
</tr>
<tr>
<td>Mean</td>
<td>17,200</td>
<td>40,000</td>
<td>7,100</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>2,830</td>
<td>6,600</td>
<td>1,162</td>
</tr>
<tr>
<td>Standard Score</td>
<td>3.17</td>
<td></td>
<td>5.74</td>
</tr>
<tr>
<td>Probability of Failure</td>
<td>$8.00 \times 10^{-4}$</td>
<td>$5.10^{-9}$</td>
<td>$1.20 \times 10^{-3}$</td>
</tr>
<tr>
<td>Factor of Safety</td>
<td>2.33</td>
<td></td>
<td>2.04</td>
</tr>
</tbody>
</table>

Note: 1 ft-lb = 1.356 N·m  
1 lb = 0.00444 kN  
1 psi = 0.00689 MN/m²
Similar approaches can be taken with the tie-rod and bending stress demands. Tie-rod load is given by

\[ T = K_a \left[ (200) \gamma_1 + (9417) \gamma_2 \right]. \]  

(7-12)

Hence

\[ S_T^2 = \left( \frac{3T}{3\gamma_1} \right)^2 (S_{\gamma_1})^2 + \left( \frac{3T}{3\gamma_2} \right)^2 (S_{\gamma_2})^2 \]

\[ + \left( \frac{3T}{3K_a} \right)^2 (S_{K_a})^2, \text{ and} \]

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

The maximum bending moment for this situation is given by

\[ M_{\text{MAX}} = 5.88 P - K_a \left[ (71.8) \gamma_1 + (12.6) \gamma_2 \right] \]

\[ - K_a \left[ (71.8) \gamma_1 + (9.74) \gamma_2 \right] \]

(7-13b)

\[ = K_a \left[ (67.2) \gamma_1 + (50.8) \gamma_2 \right]. \]

(7-13c)

For a reduction factor in bending of 0.30 and section modulus of 6.55 in³/ft in this case, the maximum bending stress is

\[ \sigma = (0.304) (12) \frac{M_{\text{MAX}}}{(6.55)} \]

(7-14a)

\[ = K_a \left[ (47.5) \gamma_1 + (35.9) \gamma_2 \right]. \]

(7-14b)

The standard deviation for bending stress is given by

\[ S_{\sigma}^2 = \left( \frac{3\sigma}{3\gamma_1} \right)^2 (S_{\gamma_1})^2 + \left( \frac{3\sigma}{3\gamma_2} \right)^2 (S_{\gamma_2})^2 \]

\[ + \left( \frac{3\sigma}{3K_a} \right)^2 (S_{K_a})^2, \text{ and} \]
\[ S = 311 \text{ psi (2.14 MN/m}^2). \]

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

\[ T_{ULT} = \frac{\pi}{4} d^2 f_y \]

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

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

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 \( S \).

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

\[ S_T = (0.10) \times (7100) \]

\[ = 710 \text{ lb } (3.16 \text{ kN}) . \]

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)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Penetration</th>
<th>Tie-Rod Pull</th>
<th>Bending Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D (ft-lb)</td>
<td>C (ft-lb)</td>
<td>D (lb)</td>
</tr>
<tr>
<td>Mean</td>
<td>17,200</td>
<td>40,000</td>
<td>7,270</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>530</td>
<td>4,750</td>
<td>511</td>
</tr>
<tr>
<td>Standard Score</td>
<td></td>
<td></td>
<td>4.77</td>
</tr>
<tr>
<td>Probability of Failure</td>
<td>$\approx 10^{-6}$</td>
<td>$\approx 10^{-22}$</td>
<td>$1.30 \times 10^{-3}$</td>
</tr>
</tbody>
</table>

Note: 1 ft-lb = 1.356 N·m
1 lb = 0.00444 kN
1 psi = 0.00689 MN/m²
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

\[ R_s = \Pi_{i=1}^{n} R_i \]  

(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

\[ P_f = \Pi_{i=1}^{n} 1 - (1 - P_i) \]  

(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/m²), 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)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Probability of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration</td>
<td>$8.00 \times 10^{-4}$</td>
</tr>
<tr>
<td>Tie-Rod Pull</td>
<td>$&lt;10^{-10}$</td>
</tr>
<tr>
<td>Bending Stress</td>
<td>$2.50 \times 10^{-3}$</td>
</tr>
</tbody>
</table>
D = 5.54 feet (1.69 m)

P = 6,330 pounds (28.2 kN)

d = 0.615 inches (15.6 mm)

σ = 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)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Penetration</th>
<th>Tie-Rod Load</th>
<th>Bending Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D</td>
<td>C</td>
<td>D</td>
</tr>
<tr>
<td>Mean</td>
<td>5,230</td>
<td>30,500</td>
<td>6,530</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>920</td>
<td>13,800</td>
<td>1,070</td>
</tr>
<tr>
<td>Standard Score</td>
<td>1.83</td>
<td></td>
<td>4.45</td>
</tr>
<tr>
<td>Probability of Failure</td>
<td>$3.40 \times 10^{-2}$</td>
<td>$10^{-6}$</td>
<td>$1.60 \times 10^{-3}$</td>
</tr>
<tr>
<td>Factor of Safety</td>
<td>5.83</td>
<td></td>
<td>1.76</td>
</tr>
</tbody>
</table>

Note: 1 ft-lb = 1.356 N·m
1 lb = 0.00444 kN
1 psi = 0.00689 MN/m²
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

$$\overline{SM} = \bar{c} - \bar{D},$$  \hspace{1cm} (7-3a)

$$S_{SM} = \sqrt{S_c^2 - S_D^2}, \text{ and}$$ \hspace{1cm} (7-3b)

$$P_f = \left( \frac{\overline{SM}}{S_{SM}} \right),$$  \hspace{1cm} (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

\[
t = \frac{\bar{S}_M}{S_{SM}}
\]  

(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

<table>
<thead>
<tr>
<th>No. Data Points</th>
<th>$t$ Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>31.821</td>
</tr>
<tr>
<td>4</td>
<td>6.965</td>
</tr>
<tr>
<td>5</td>
<td>4.451</td>
</tr>
<tr>
<td>6</td>
<td>3.747</td>
</tr>
<tr>
<td>7</td>
<td>3.365</td>
</tr>
<tr>
<td>8</td>
<td>3.143</td>
</tr>
<tr>
<td>9</td>
<td>2.998</td>
</tr>
<tr>
<td>10</td>
<td>2.896</td>
</tr>
<tr>
<td>11</td>
<td>2.821</td>
</tr>
<tr>
<td>12</td>
<td>2.764</td>
</tr>
<tr>
<td>13</td>
<td>2.718</td>
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<tr>
<td>14</td>
<td>2.681</td>
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<tr>
<td>15</td>
<td>2.650</td>
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<tr>
<td>16</td>
<td>2.624</td>
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<tr>
<td>17</td>
<td>2.602</td>
</tr>
<tr>
<td>18</td>
<td>2.583</td>
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<tr>
<td>19</td>
<td>2.567</td>
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<td>20</td>
<td>2.552</td>
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<td>2.539</td>
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<td>22</td>
<td>2.528</td>
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<td>2.518</td>
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<td>2.508</td>
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<td>26</td>
<td>2.492</td>
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<tr>
<td>27</td>
<td>2.485</td>
</tr>
<tr>
<td>28</td>
<td>2.479</td>
</tr>
<tr>
<td>29</td>
<td>2.473</td>
</tr>
</tbody>
</table>
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 Frey 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.