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,

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

in which \( \phi_f \) = factored soil parameter, \( \phi \) = 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:

\[ aD^3 + bD^2 + cD + d = 0 \]  

(4-1)
Figure 4-2. Stress distribution and resultants
in which: \( a = \frac{1}{3} (K_{a3} - K_p') \gamma_3, b = \frac{1}{2} (K_{a3} - K_p') \gamma_3 (H - H_A) + \frac{1}{2} K_a' \gamma_1 (q + \gamma_1 t_1 + \gamma_2 t_2), c = K_{a2}' (q + \gamma_1 t_1 + \gamma_2 t_2) (H - H_A), \) 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 \( \phi_f \) 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:

\[
T_s = (F_{T1} + F_{T2} + F_{T3} + F_{R1} + F_{R2} + F_{R3} - F_{T4}) \tan^2 (\phi_f) + W_p H D \tan (\phi_f)
\]

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:

\[
P_{FES} L = M_{R1} + M_{R2} + M_{R3} + M_{T1} + M_{T2}, \quad \text{and} \quad (4-3a)
\]

\[
L = \frac{2}{3} D + H - H_A \quad (4-3b)
\]
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,

\[
x = \frac{-b + \sqrt{b^2 - 4ad}}{2a}
\]  

(4-4)

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:

\[
M_{\text{MAX}} = F_{\text{FES}} (t_1 + x - H_A) - F_{T1} \left( \frac{1}{3} t_1 + x \right) - F_{R1} \left( \frac{1}{2} t_1 + x \right) - \frac{1}{6} K_{a2} \gamma_2 x^2 - \frac{1}{3} K_{a2} \gamma_1 t_1 x^2.
\]  

(4-5)

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:

\[
\alpha = \frac{H}{H_D}
\]  

(4-6)

\[
\beta = \frac{H_A}{H_D}
\]  

(4-7)

\[
\tau_{\text{MAX}} = \frac{12 M_{\text{MAX}}}{H_D^3}
\]  

(4-8)
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

\[
P = f_c P_{FES}
\]

or, where appropriate

\[
P = f_c r_t P_{FES}
\]

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

\[
\tau_{op} = \tau_{MAX} \tau_d
\]

Values of \( \tau_d \) are taken from the moment reduction chart in Figure 2-17a for values of \( \log \rho \).

The structural curve is generated by values of

\[
\tau_s = \frac{\psi}{(H_d^2)^{2/3}}
\]

in which \( \psi \) = the flexibility characteristic and

\[
\psi = \frac{f_b}{(EI)^{2/3}}
\]

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,

\[
\psi = \frac{2 f_b}{g^{2/3}} \tag{4-14}
\]

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

\[
M = \tau H^3_D \tag{4-15}
\]

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, \( \psi \), 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, $S_{c}$, is the prime indicator of the ability of a wall to stand, where

$$S_{c} = \frac{2cr}{q + \gamma_1 t_1 + \gamma_2 t_2}$$

(4-17)

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 $4cr$ for any depth of penetration, or when $S_{c}$ 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 $S_{c}$ 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
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

\[ a^2D^2 + bD + d = 0 \]  \hspace{1cm} (4-18a)

in which: \[ a = \frac{1}{2} (4 cr - q - \gamma_1 t_1 - \gamma_2 t_2), \]
\[ b = (4 cr - q - \gamma_1 t_1 - \gamma_2 t_2) (H - H_A), \]
\[ d = \frac{1}{2} K_a \gamma_1 t_1^2 (\frac{2}{3} t_1 - H_A) + \frac{1}{2} K_a \gamma_2 t_2^2 (\frac{2}{3} t_2 + t_1 - H_A) + K_a (\gamma_1 t_1 + q) (\frac{1}{2} t_2 + t_1 - H_A). \]

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

\[ D = \frac{-b \pm \sqrt{b^2 - 4ad}}{2a} \]  \hspace{1cm} (4-18b)

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:

\[ \log \sigma = -3.6 \] (stiff walls),
\[ \log \sigma = -2.6 \] (working stress), and
\[ \log \sigma = -2.0 \] (first yield).

Each selection of \( r_d \) is based upon the stability number, \( S_t \), and the relative wall height, \( a \).
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)

\[
R = \frac{\gamma_1 t_1^3 + \gamma_2 t_2^3}{\gamma_3 h^3}
\]

(3-4)

and for clay (undrained)

\[
R = \frac{\gamma_1 t_1^3 + \gamma_2 t_2^3}{(5c - \gamma_1 t_1 - \gamma_2 t_2) h^2}
\]

(3-15)

in which \( \gamma_i \) = unit weight of the \( i^{\text{th}} \) layer, \( t_i \) = thickness of the \( i^{\text{th}} \) 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

\[
R_D = R \cdot C_D
\]

(3-17a)

\[
R_P = R \cdot C_P, \text{ and}
\]

(3-17b)

\[
R_M = R \cdot C_M
\]

(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

<table>
<thead>
<tr>
<th>Normalizing Parameter</th>
<th>Sand</th>
<th>Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimensionless Depth: D'</td>
<td>( \frac{D}{H} )</td>
<td>( \frac{D}{H} )</td>
</tr>
<tr>
<td>Dimensionless Tie-Rod Load: P'</td>
<td>( \frac{P}{\gamma_1 L^2} )</td>
<td>( \frac{P}{cD} )</td>
</tr>
<tr>
<td>Dimensionless Moment: M'</td>
<td>( \frac{M}{\gamma_2 L^3} )</td>
<td>( \frac{M}{cD^2} )</td>
</tr>
</tbody>
</table>
\[ L = \frac{2}{3} D + H - H_A, \quad \text{and} \tag{4-14} \]

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

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 multiplied 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 $\phi = 30^\circ$, $\gamma_{\text{moist}} = 100$ pcf, $\gamma_{\text{sat}} = 120$ pcf;
- **medium:** in which $\phi = 35^\circ$, $\gamma_{\text{moist}} = 105$ pcf, $\gamma_{\text{sat}} = 125$ pcf; and
- **dense:** in which $\phi = 40^\circ$, $\gamma_{\text{moist}} = 110$ pcf, $\gamma_{\text{sat}} = 130$ 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 \( \gamma \) = the unit weight of the fill, taken as 100 pcf \((15.7 \text{ kN/m}^3)\), \( c \) = 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,

\[
\sigma_H = \gamma H - 2c
\]  \( (4-21) \)

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. $M' \text{ vs. } R_M$: sand
Figure 4-7. $D' \text{ vs. } R_D$: clay (undrained)
Figure 4-8. $P' \text{ vs. } R_p$: clay (undrained)
Figure 4-10. \( D' \) vs. \( R_D \): clay (drained)
Figure 4-11. $P' \text{ vs. } R_p$: clay (drained)
Figure 4-12. $M'$ vs $R_M'$: clay (drained)
Figure 4-13. $D'$ vs $R_D$: sand

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

Loading Ratio $R_D = R \cdot C_D$
Figure 4-14. M vs. $R_m$; sand
Figure 4-15. \( D' \) vs. \( R_D \); clay (undrained)
Figure 4-16. $M^* \text{ vs } R_H^*$: Clay (undrained)
Figure 4-17. $D' \text{ vs } R_D$; clay (drained)
Figure 4-18. $M'$ vs. $R_M$: clay (drained)