CHAPTER 13

SHEET-PILE WALLS: CANTILEVERED AND ANCHORED

13-1 INTRODUCTION

Sheet-pile walls are widely used for both large and small waterfront structures, ranging from small pleasure-boat launching facilities to large dock structures where ocean-going ships can take on or unload cargo. A pier jutting into the harbor, consisting of two rows of sheetpiling to create a space between that is filled with earth and paved, is a common construction.

Sheetpiling is also used for beach erosion protection; for stabilizing ground slopes, particularly for roads (instead of using the walls of Chap. 12); for shoring walls of trenches and other excavations; and for cofferdams. When the wall is under about 3 m in height it is often cantilevered (Fig. 13-1*a*); however, for larger wall heights it is usually anchored using one or more anchors. The resulting wall is termed an *anchored sheet-pile wall* or *anchored bulkhead*. Several of the more common wall configurations are illustrated in Fig. 13-1. The alternative shown in Fig. 13-1*d* of using continuous rods for parallel sheet-pile walls may be considerably more economical than driving pile anchorages—even for tie rod lengths of 30 to 40 m.

There are several methods of analyzing cantilever and anchored sheet-pile walls. Two of the early methods were (*a*) the *free-earth* support and the (*b*) *fixed-earth* support, as shown in Fig. 13-2 along with the simplified assumptions of active (from filled side) and passive pressure on the free side below the dredge line. The design was based primarily on taking moments about the anchor rod, increasing the depth of embedment D until $\sum F_h$ was satisfied, and then computing the resulting bending moments in the piling. A safety factor was incorporated by using a reduced K_p for passive pressure or by increasing the embedment depth D some arbitrary amount such as 20 or 30 percent. Two of the simplifications could result in errors:

1. Unless the anchor rod elongates sufficiently, the active pressure may not fully develop, resulting in a computed anchor rod force that is too small.

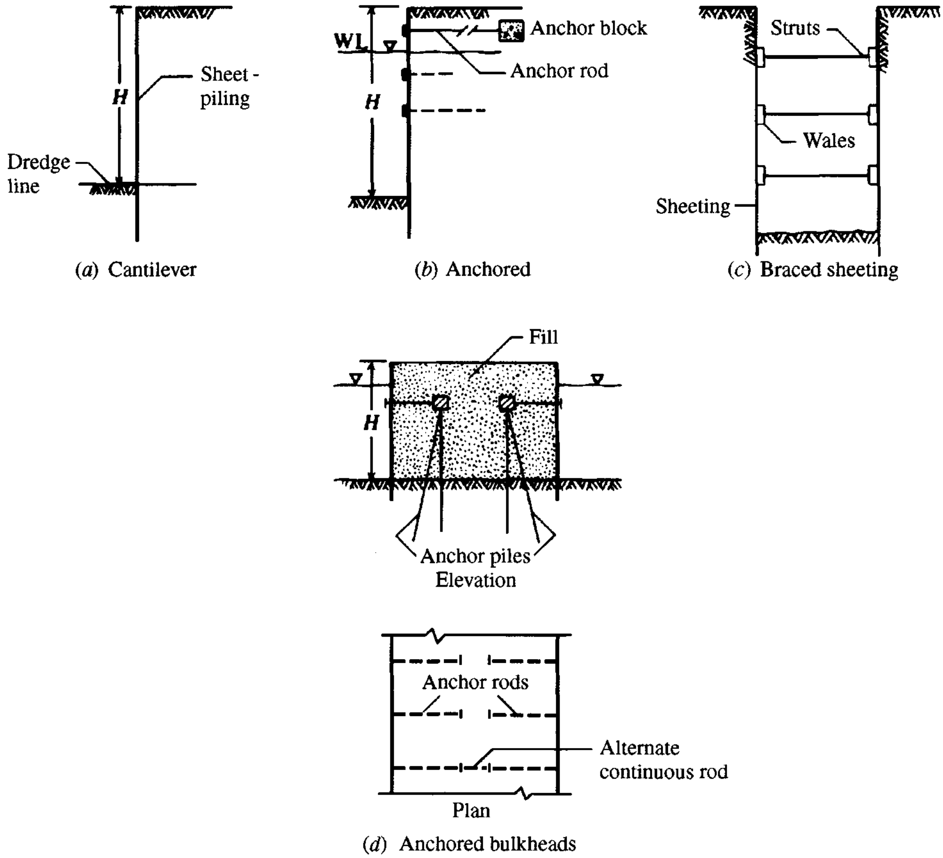


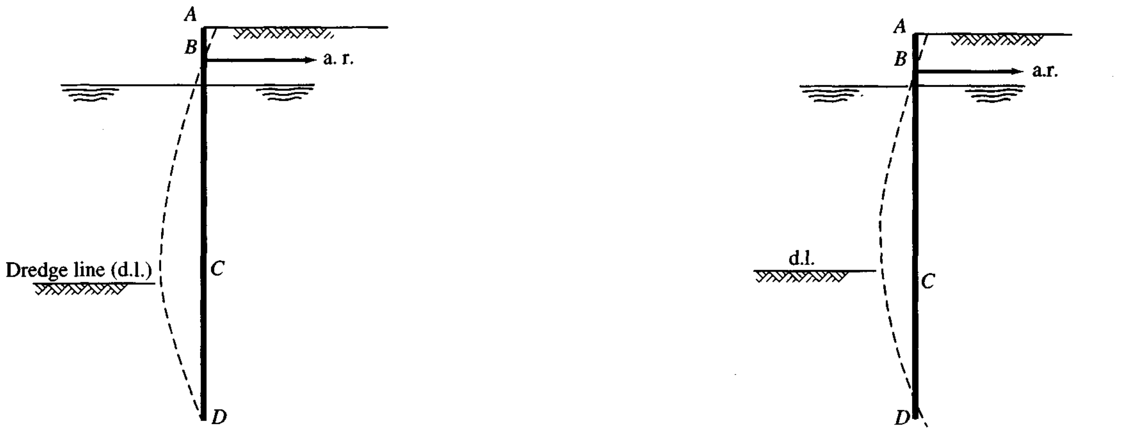
Figure 13-1 Sheet-pile structures.

2. The center of pressure below the dredge line is qualitatively shown by the dashed lines of Fig. 13-2*c* and *d* and is closer to the dredge line than assumed using the passive pressure profiles shown. The erroneous location of the center of pressure usually results in moments that are too large.

Cantilever sheet-pile walls were analyzed similarly to anchored walls, except the soil pressure profiles were slightly different and moments were usually taken about the base since there was no anchor rod.

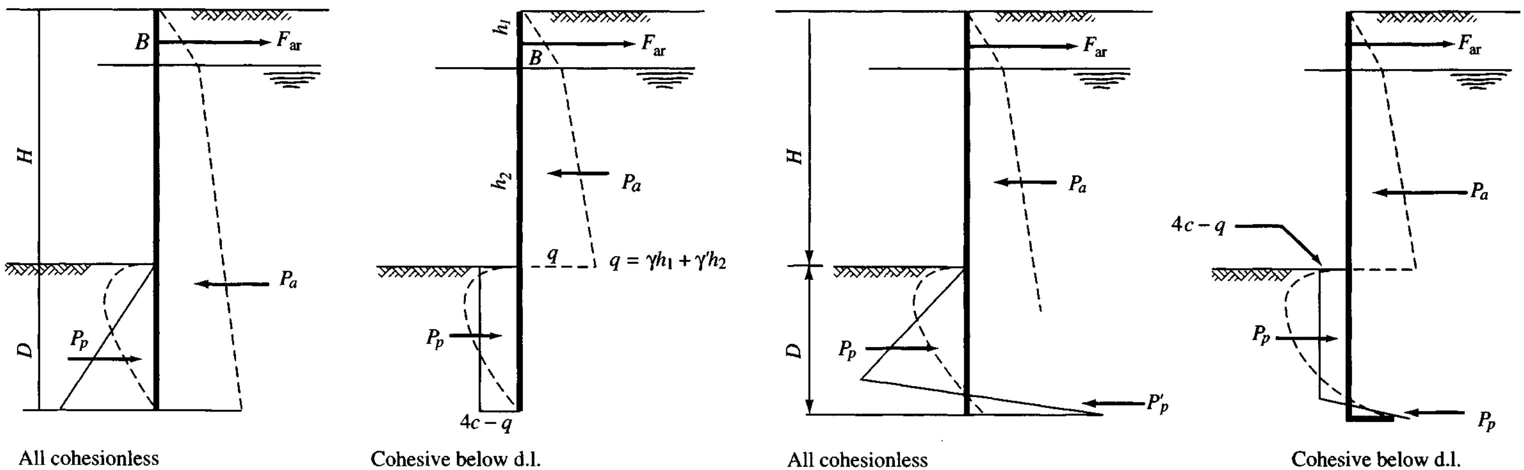
These were the only methods used in the United States and elsewhere until the mid-1960s when Haliburton (1968) described a finite-difference method he and his coworkers had developed. Bowles (1974*a*, and included in the second and later editions of this textbook) used the finite-element method (FEM) for sheet-pile wall analysis. As of this edition the *free-* and *fixed-earth* support methods will no longer be presented.[1] Although these two methods were widely used, so many of the author's FEM programs are available (worldwide) and use of personal computers is so widespread their continued inclusion is no longer warranted.

[1]The reader can still access them in the first through fourth editions of this text.



(*a*) Free-earth support deflection line (qualitative).

(*b*) Fixed-earth support deflection line (qualitative).



(*c*) Assumed and probable (dashed) soil resistance and active earth pressure profile for "*free-earth*" support method.

(*d*) Assumed and probable (dashed) soil resistance and active earth pressure profile for "*fixed earth*" support method.

Figure 13-2 General assumptions and earth pressure profiles for anchored walls. Essential difference between anchored and cantilevered walls is there is no anchor rod in the cantilever wall design. Active and passive pressure profiles are similar (but not exactly same).

There is no "exact" method to analyze/design a sheet-pile type of wall. Both field observations and laboratory model tests show that there is a complex interaction of (as a minimum) construction method (install and backfill, or install and excavate the free side), excavation depth, stiffness of wall material, type and state of retained soil, and passive pressure resistance. With anchored walls there is also the anchor geometry, initial anchor prestress (or load), construction stage when anchor rod is installed, and behavior of that part of the wall above the anchor rod (into or away from the backfill).

The two original methods named were oversimplifications of an extremely complex problem, relied totally on rigid body statics, and were based entirely on the assumptions of an active earth pressure above the dredge line and passive earth pressure below. Wall and anchor rod stiffness did not enter into the equation. As a result of substantial overdesign, few walls failed.

The FEM is somewhat less of an approximation. Additionally, it allows for better modeling of the problem and gives more useful design information as part of the output. It requires a computer program, but this is provided as program B-9 (FADSPABW) on your computer diskette. Section 13-6 will present considerable detail on this method so it can be used in design with reasonable confidence.

The finite-difference method (FDM) is not considered further because it offers no advantage over the FEM and is more difficult to use. Indeed, it has these disadvantages: Constant-length elements are required over the full pile length; the stiffness matrix cannot be banded; and modeling boundary conditions of zero displacement and rotation is difficult.

The several materials and material configurations used for sheet piles will be given in Sec. 13-2 since they are used for walls in both this and the next two chapters.

13-2 TYPES AND MATERIALS USED FOR SHEETPILING

Sheetpiling materials may be of timber, reinforced concrete, or steel. Allowable design stresses are often higher than in general building construction and may be from about 0.65 to $0.90 f_y$ for steel[2] and wood. Reinforced concrete design stresses may be on the order of $0.75 f_c'$ for unfactored loads. The design stress actually used will depend on engineering judgment, effect of wall failure (site importance factor), and the local building code.

13-2.1 Timber Sheetpiling

Timber piling is sometimes used for free-standing walls of $H < 3$ m (see Fig. 13-1*a*). It is more often used for temporarily braced sheeting to prevent trench cave-ins (see Fig. 13-1*c*) during installation of deep water and sewer lines. If timber sheeting is used in permanent structures above water level, preservative treatment is necessary, and even so the useful life is seldom over 10 to 15 years. At present timber is little used except in temporary retaining structures owing to both the scarcity of timber—particularly of large cross section—and cost.

Several timber piling shapes are shown in Fig. 13-3, of which the Wakefield and **V** groove piling have been and are the most used. Dimensions shown are approximate and you will have to use what is currently available.

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[2]Value recommended by Bethlehem Steel Corporation, the principal producer of rolled sheetpiling in the United States at present.

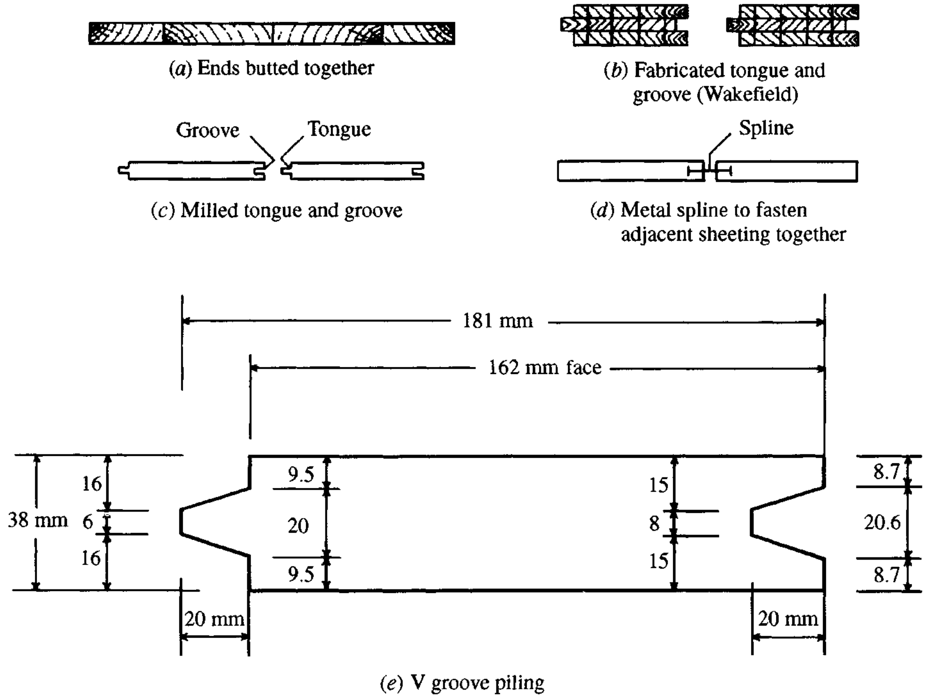


Figure 13-3 Wood sheet piles.

It is common to see low timber walls treated with wood preservative in use along waterfronts. A substantial amount of timber piling—mostly fast-growing pine—is still used for protection where the piling is driven, then surrounded with stabilizing blocks or boulders (termed *groins*) to catch sand from the ocean side to maintain beaches. Here the intent is for the wall eventually to become covered with sand from tidal action. Strength is not the primary concern for this use, so if the wood lasts long enough to become buried, the purpose of the wall has been accomplished.

If wood sheetpiling is being considered, the soil type is a major factor. Almost any driving requires interfacing the pile hammer with a driving cap over the timber to minimize top damage. Driving in hard or gravelly soil tends to damage or even split the pile tip. Damage can sometimes be avoided by driving and pulling a steel mandrel or the like or by using a water jet to create a "predrilled" hole to reduce the driving resistance. The sheeting may be pointed, generally as shown in Fig. 13-4, and placed so that the pile being driven tends to wedge against the previously driven pile.

13-2.2 Reinforced Concrete Sheetpiling

These sheet piles are precast concrete members, usually with a tongue-and-groove joint. Even though their cross section is considerably dated (see Fig. 13-4), this form is still used. They are designed for service stresses, but because of their mass, both handling and driving stresses must also be taken into account. The points are usually cast with a bevel, which tends to wedge the pile being driven against the previously driven pile.

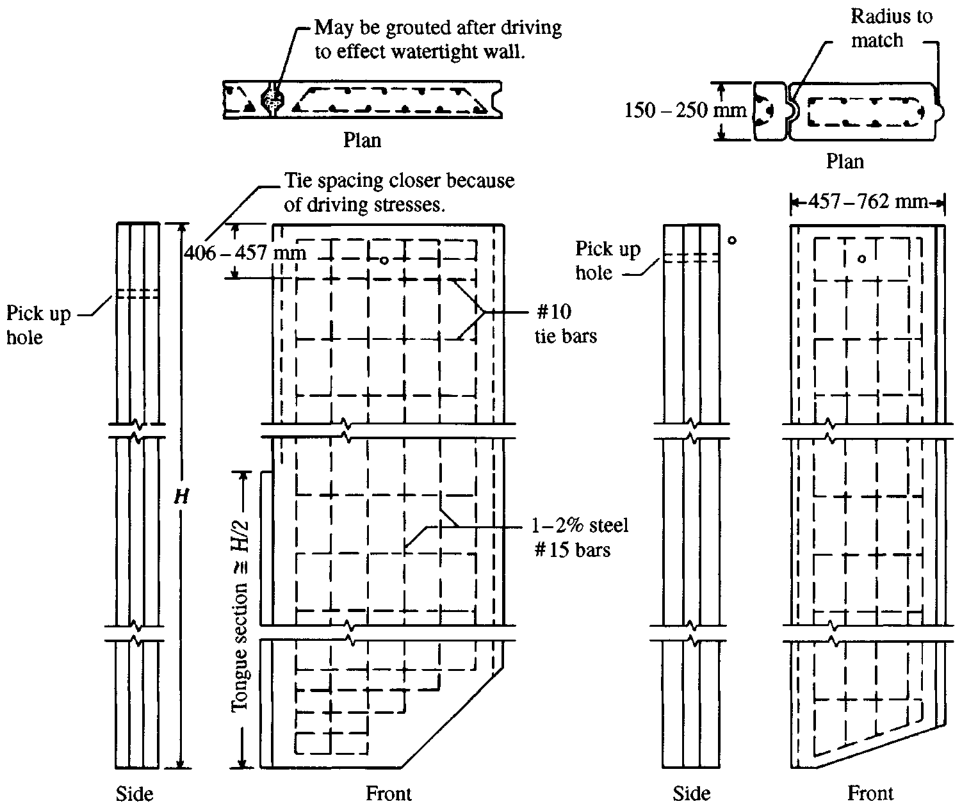


Figure 13-4 Typical details of reinforced concrete sheet piles. [*After PCA (1951).*]

The typical dimensions[3] shown in Fig. 13-4 indicate the piles are relatively bulky. During driving they will displace a large volume of soil for an increase in driving resistance. The relatively large sizes, coupled with the high unit weight ($\gamma_c = 23.6$ kN/m^3) of concrete, mean that the piles are quite heavy and may not be competitive with other pile types unless they are produced near the job site.

Dimensions and reinforcing bars shown in Fig. 13-4 are typical, but currently produced piles will contain bars that are available to the producer at casting time.

If the joints are cleaned and grouted after they have been driven, a reasonably watertight wall may be obtained. However, if the wall is grouted, expansion joints may be required along the wall at intervals that are multiples of the section width.

13-2.3 Steel Sheetpiling

Steel sheetpiling is the most common type used for walls because of several advantages over other materials:

[3]Soft-converted since only Fps units were used by U.S. industry in 1951.

1. It is resistant to the high driving stresses developed in hard or rocky material.
2. It is relatively lightweight.
3. It may be reused several times.
4. It has a long service life either above or below water if it is provided with modest protection according to NBS (1962), which summarizes data on a number of piles inspected after lengthy service. Watkins (1969) provides some guidance for considering corrosion of sheetpiling in sea water. There is no available information on corrosion of steel piling in chemically contaminated soil. There is a resistance probe [see Roy and Ramaswamy (1983)] utilizing a set of electrodes, one of which is magnesium and the other is steel, that can measure the resistance of the soil between them. The soil resistance is related to the amount (in terms of "high" or "low" amount/likelihood) of expected steel pile corrosion.
5. It is easy to increase the pile length by either welding or bolting. If the full design length cannot be driven, it is easy to cut the excess length using a cutting torch.
6. Joints are less apt to deform when wedged full with soil and small stones during driving.
7. A nearly impervious wall can be constructed by driving the sheeting with a removable plug in the open thumb-and-finger joint. The plug is pulled after the pile is driven, and the resulting cavity is filled with a plastic sealer. The next pile section is then driven with the intersecting thumb or ball socket displacing part of the plastic sealer from the prefilled cavity. When the piling is driven in pairs, sealing the intermediate joint by prefilling may not provide a 100 percent impervious joint. Sellmeijer et al. (1995) describe an experimental wall project using this general approach but with European-produced piling, which has a slightly different joint configuration than the standard "thumb-and-finger" or "ball-and-socket" interlocks of piling produced in the United States (see Fig. 13-5).

Figure 13-5 illustrates several angle sections and joints that can be fabricated from cut pieces of sheetpiling; these are for illustration, as other joints can be produced. The crosses and wyes shown are used in cellular cofferdams (of Chap. 15); the angles and bends are used for direction changes in the wall.

Several steel sheet-pile cross sections currently available are given in Tables A-3*a* and A-3*b* in the Appendix. The straight-web sections are used in situations where the web is in tension; the **Z** sections are used where large bending moments require a substantial moment of inertia or section modulus.

When the stiffness capacity of the available **Z** piles is insufficient, the box sections of Table A-3 (also as Fig. 13-6*a*) or the soldier-**Z**-pile combination of Fig. 13-6*b* might be used.

13-2.4 Composite Sheet-Pile Walls

Walls may be constructed using composite construction. The soldier beam–wood lagging combination of Chap. 14 (Fig. 14-1*a*) is an example.

Other examples include use of soldier beams[4] on some spacing with sheetpiling used between the spacings. For corrosion protection one might encase the upper part of steel sheetpiling in concrete after it is driven, with the concrete extending from below the water line

[4]Rolled pile or structural sections with a moment of inertia I_p that is several times the moment of inertia I_{sp} of the sheetpiling ($I_p \gg I_{sp}$).

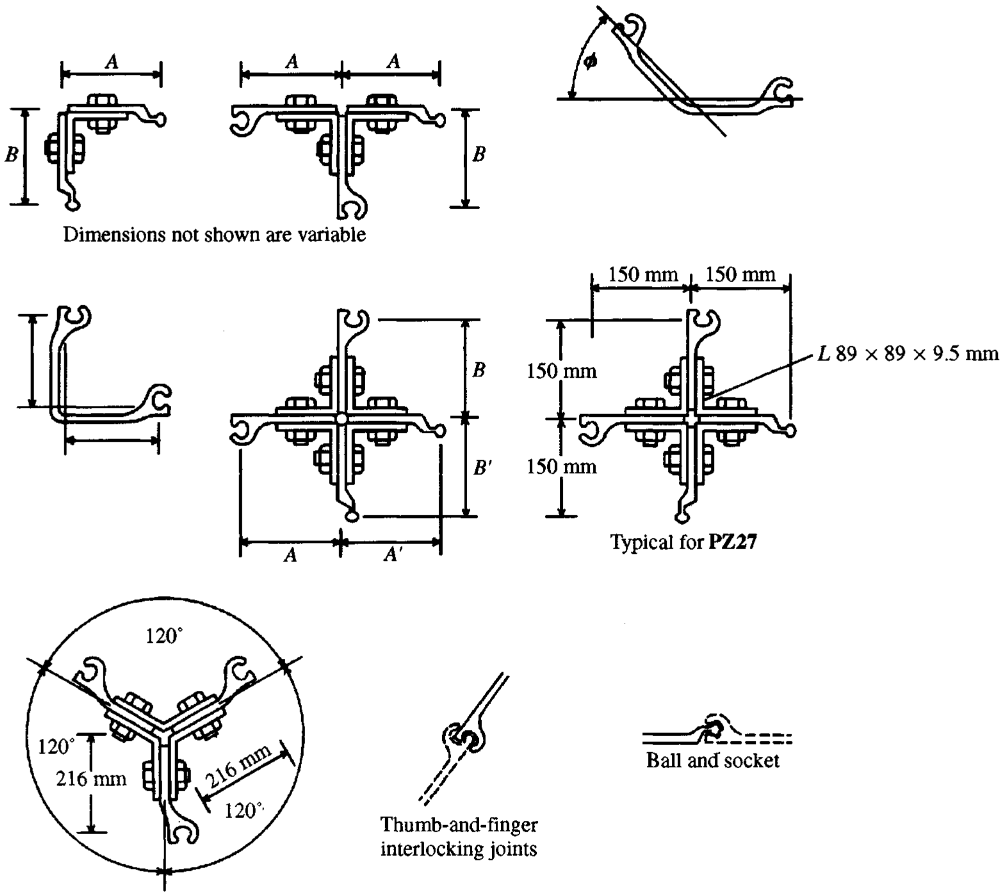


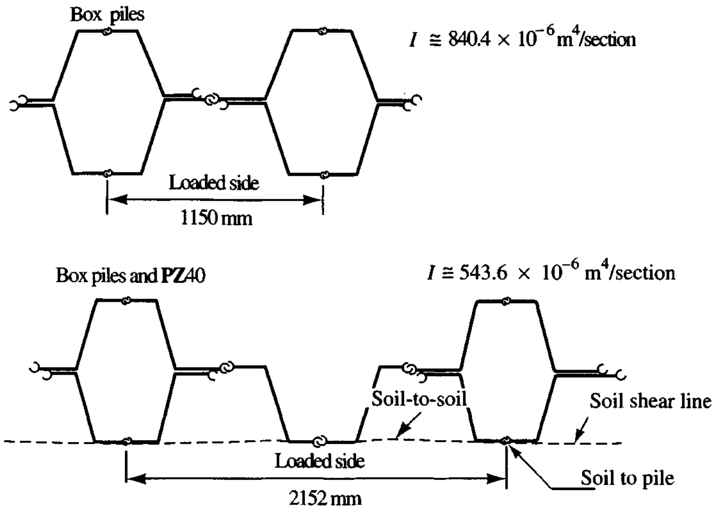
Figure 13-5 Typical fabricated or rolled sheet-pile joints. All dimensions shown are millimeters. Bolts are high-strength 22-mm diameter on 150-mm centers except at end 610-mm where they are on 75-mm centers.

to the pile top. A wood facing might also be used, or the lower part of the sheeting could be made of steel and the upper part of a different material—wood or concrete.

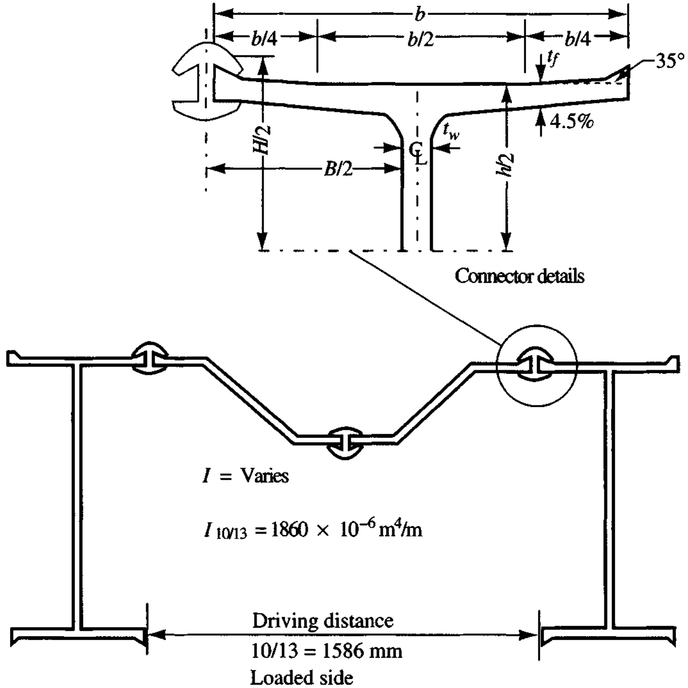
Since steel is relatively durable in most waterfront installations, the principal composite construction consists in using a mix of soldier beams and sheet piles or built-up box pile sections.

13-3 SOIL PROPERTIES FOR SHEET-PILE WALLS

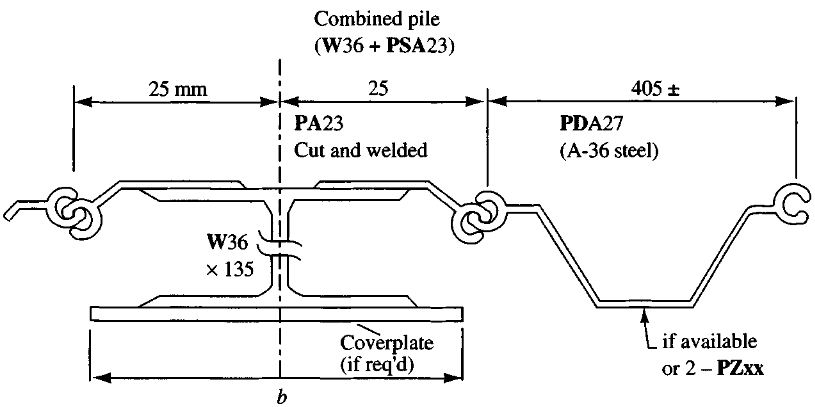
Referring to Fig. 13-2, we see that lateral earth pressures are involved with active pressures approximately developed behind the walls from the fill (or backfill) and passive pressures in front of the wall below the dredge line. Either the Rankine or Coulomb lateral earth-pressure coefficients may be used for the earth pressures, however, the Coulomb values are generally preferred. Because a sheet-pile wall is not very rigid, relatively large lateral displacements (and resulting relative movement between soil and wall) often occur between points of assumed fixity. Relative soil-wall movement produces adhesion and/or friction depending upon



(*a*) Bethlehem Steel sections.



(*b*) Arbed steel (European) sections using special pile sections with upset edges and special connector.



(*c*) Locally fabricated pile section. Any **W** section can be used. [From Munfakh (1990).]

Figure 13-6 Built-up pile sections used where standard rolled shapes do not have adequate bending stiffness. The Bethlehem Steel Corporation box sections of (*a*) and the Arbed sections of (*b*) can be obtained directly from the producers. The section shown in (*c*) can be fabricated locally to meet the required bending stiffness. The principal precaution in fabricating this section is that the interlocks be compatible.

the soil. Friction can be approximately accounted for by use of the Coulomb earth-pressure coefficient. If the backfill is cohesive, you have to do the best you can. You might use Fig. 11-11*c* and Example 11-4 as a guide with a Coulomb K_a. You might also consider programming Eq. (11-12) to give reduced values to account for cohesion. In this latter case obtain the lateral pressure as

$$\sigma_h = \gamma z K_{a,\phi} + c K_{a,c}$$

For passive pressure use the $K_{p,i}$-coefficients.

Any backfill cohesion would appear to reduce the lateral pressure; however, give consideration to a wall-soil tension crack, which would produce a surcharge effect on the soil below the tension crack depth and negate most of its beneficial effect.

Even though it is known that wall friction develops, the Rankine earth-pressure coefficients are often used for K_a, with the rationale being that they are slightly more conservative.

For the finite-element procedure it is necessary to use active earth-pressure coefficients behind the wall and the concept of the modulus of subgrade reaction k_s for the soil below the dredge line. The use of k_s allows one to model the dredge line soil as a series of nodal springs on the wall to assist in resisting lateral displacement.

From this discussion it is evident that we need soil parameters of γ, ϕ, and *cohesion* for both the wall backfill and the base soil. Because the wall must survive the initial loading as well as long-term loading, the undrained strength parameters are usually used. In the case of waterfront structures the soil below the water line will always be in an undrained state, but close to the wall a small zone may be in a consolidated undrained state. For on-shore retaining structures the dredge line soil is exposed to the weather and the state varies from saturated to dry. Since the undrained state is usually the worst case, it is appropriate to use that for design.

Seldom are laboratory tests performed to obtain these parameters. It is common to use CPT or SPT data and/or simply estimate ϕ and γ. The retained material is often backfill with little to no compaction; if it is hydraulically dredged silty sand, precise soil parameters are extremely difficult to obtain. The base soil into which the sheet pile is driven is more amenable to laboratory tests on recovered samples. However, in nearly all cases either SPT or CPT data are all that are taken. When one is using the SPT in cohesive soil, field q_u tests are routinely performed on recovered (but highly disturbed) samples. In this case the values for $s_u = c = q_u/2$ are obtained for cohesive soils, and the SPT or CPT data are converted to an estimate of ϕ and γ for cohesionless soils using correlations such as those given in Chap. 3. The unit weight of cohesive soils can be obtained using the procedure of Example 2-1. For loose sand backfill a γ of 12.5 to 14 kN/m^3 (80–90 pcf) might be used, but exercise care in using these values, for sand in this state may consolidate over time and produce a great increase in the lateral pressure/force.

If the equipment is available, one should perform laboratory tests of the direct shear or direct simple shear type to obtain an approximation of the plane strain ϕ angle. In most cases, as previously stated, the angle of internal friction is simply "estimated" with conservative values in the range of 28 to 32° commonly used; any testing is likely to be isotropically consolidated compression (CIUC) triaxial tests.

13-3.1 Drained Conditions

When the dredge line soil is cohesive and **not submerged**, particularly if some soil is excavated to produce the dredge line, one should use both undrained (total stress) and drained (effective stress) strength conditions for the dredge line s_u. Cohesive soil under long-term loading tends to a drained state above the water table.

When soil is excavated to produce the dredge line, unloading occurs. For cohesive soil above the GWT this produces an initial increase in s_u as a result of negative pore pressures, but over time the suction disappears and a drained state may develop (or alternate between a total and drained state with rainfall). Figure 2-28*b* indicates that there can be a substantial decrease in shear strength in transition from the total to an effective stress state.

For **submerged** cohesive soil below the dredge line, excavation also produces soil suction, but with water available the water content slightly increases with a resulting loss in strength s_u. In this case one should use consolidated-undrained tests, which give both a small total stress ϕ and cohesion c. Below the water surface the soil consolidates under lateral pressure to a consolidated-undrained state. This might be approximated in a laboratory shear test by consolidating a sample to in situ pressure in the presence of water, then unloading it to represent the final overburden state with the water available to allow an increase in water content. When one believes enough time has passed (several days) to allow for stabilization one should perform the test without allowing drainage.

Daniel and Olson (1982) thought the use of total instead of effective strength parameters caused a major bulkhead failure. In this case one can question the conclusion that not using drained strength parameters caused the failure. Here the dredge line soil was permanently below the water table, where all that could develop is a consolidated undrained state. The dredging that took place in front of this wall after it was constructed produced a sloping dredge line. One can speculate that unloading the soil of overburden produced some expansion and an increase in water content from *suction,* causing a strength reduction. This wall was constructed in the late 1970s, and the designer used the classical method of analysis. Thus, not a great deal of design information would have been obtained to provide guidance in the design compared with using the FEM. Although Daniel and Olson (1982) also stated that there was no way to ascertain exactly what caused the wall failure, their description of the bulging (lateral wall deformation away from the backfill) before failure makes it evident that there was an increase in lateral pressure in the backfill. This may have been accompanied by some loss of dredge line soil strength (or carrying capacity) as a result of sloping the dredge line and/or soil suction.

13-3.2 Angle of Wall Friction δ

The angle of wall friction δ can be estimated from Table 11-6 or directly measured for important projects. Any direct measurements between the soil and wall material should use a pressure that is on the order of what is expected in the prototype, since δ is somewhat pressure-dependent. If $\phi < \delta$, you assume a frictionless interface (but there may be adhesion, since a $\phi < \delta$ soil would have cohesion).

For metal sheetpiling of **Z** and deep web shapes, the unit width of wall will include a minimum slip zone, part of which is soil-to-soil and part soil-to-steel as in Fig. 13-5*a*. In this case one can use an average (or weighted average) value for δ as

$$\tan\delta' = \frac{\tan\delta + \tan\phi}{2} \qquad \text{(weighting factors not included)}$$

where ϕ = angle of internal friction of contact soil and δ = the friction angle from Table 11-6 or measured in a laboratory test.

13-3.3 Modulus of Subgrade Reaction, k_s

The finite-element method uses k_s in the passive pressure region below the dredge line in front of the wall. The author has shown [Bowles (1974*a*)] that this model is reasonably correct by using it to analyze full-scale field walls and to reanalyze large model sheet-pile walls reported by Tschebotarioff (1949) and small models used by Rowe (1952). Estimates of k_s can be made using the procedures given in Sec. 9-6; however, we need the equation given there that has a depth parameter Z as

$$k_s = A_s + B_s Z^n \tag{9-10}$$

Alternative equation forms (which are in your computer program B-9) are

$$k_s = A_s + B_s \tan^{-1}(Z/D)$$
$$k_s = A_s + B_s (Z/D)^n$$

with the restriction that the exponent $n > 0$ [cannot be 0 or $(-)$].

We can approximate these equations by using

$$k_s = C(\text{SF})q_a \qquad \text{or} \qquad k_s = Cq_{\text{ult}}$$

where q_a = bearing capacity computed at several depths in the likely range of pile embedment depth D and $q_a = q_{\text{ult}}/\text{SF}$. The C factor is

$$C = \frac{1}{0.0254 \text{ m}} \quad (\text{SI}); \qquad \frac{1}{1/12 \text{ ft}} \quad (\text{Fps})$$

This expression gives $C = 40$ for SI and $C = 12$ for Fps. The safety factor is SF = 3 for cohesive soil and SF = 2 for cohesionless soils. We can then plot the several values of k_s versus depth Z and obtain a best fit for the foregoing equation.

Alternatively one might use one of the bearing-capacity equations from Table 4-1, simplified (no shape, depth, inclination, **base**, or **ground** factors) to read

$$k_s = \frac{q_{\text{ult}}}{\Delta H} = C(cN_c + \bar{q}N_q + 0.5\gamma B N_\gamma)$$

where ΔH = is an *assumed* displacement of 0.0254 m ($\frac{1}{12}$ ft) when the ultimate bearing pressure q_{ult} is developed (and gives $C = 1/0.0254 \cong 40$ or 12). Separating terms, we have the following:

$$\left.\begin{aligned} A_s &= C(cN_c + 0.5\gamma \times 1 \times N_\gamma) \\ B_s Z^n &= C(\gamma N_q Z^1) \end{aligned}\right\} \tag{13-1}$$

The use of 1 in the equation for A_s is for B = unit width of wall. An upper limit can be placed on k_s by using something other than $n = 1$ in Eqs. (13-1) or using one of the previously given alternatives. We do not want k_s to become unreasonably large because driving difficulties generally limit sheet-pile embedment depths D to 5 to 6 m.

Some persons have suggested an upper limit on k_s be the passive pressure. Since there are difficulties with computing σ_p for small ϕ angles one might use computer program WEDGE on your diskette (see also Sec. 13-5) to obtain P_p. Compare this result to the sum of the computed (+) node forces [do not include any $(-)$ values] below the ground line, and if

$$\sum F_{\text{node}} > P_p$$

arbitrarily increase the depth of embedment 0.3 to 0.6 m and make another analysis.

Using the FEM and computer program B-9 allows you to make a parametric study rapidly (vary pile section I, k_s, embedment depth D, anchor rod location, and so on). You will generally find that the preceding suggestions for k_s will give reasonable values for pile bending and node soil pressure. Deflections are highly dependent on the flexural rigidity EI of the pile *and* k_s, so if you want a reliable dredge line value you have to input a carefully chosen k_s. Keep in mind that exact values are not possible, for too many variables are beyond the designer's control. What is desired is enough output data to make a design with reasonable confidence that the wall will serve its intended purpose.

The FEM allows you to consider nonlinear effects using the term X_{max} identified in Sec. 9-6 and used in Example 13-1 following. A program should do these things (as incorporated into B-9):

1. Allow adjustment of the dredge line springs to account for driving or excavation damage to the soil
2. Remove node springs when the computed $X_i > X_{max}$ and recycle

13-4 STABILITY NUMBERS FOR SHEET-PILE WALLS

13-4.1 Stability Numbers and Safety Factors

The concept of stability number (or safety factor) for sheet-pile walls is somewhat a misnomer, since it is not clear just what it means. For this discussion it is more convenient to use the term *safety factor* (SF) rather than *stability number*, which implies the ratio of system resistance/system failure effects. In classical sheet-pile wall design it has been common to do one of the following:

1. Divide the Rankine (or Coulomb) K_p by a SF for the soil below the dredge line. Some designers might use K_a larger than the Rankine or Coulomb value as well.
2. Arbitrarily increase the computed embedment depth by some factor, say, 1.2 to 1.3.

The author suggests that a more rational method is needed to estimate probable wall safety. This is done as follows:

1. Do a wall analysis using the existing conditions to find the depth required such that any depth increase does not change the dredge line deflection (at least within some tolerance of, say, 2 to 3 mm). This depth D_1 is all that is required for stability for the given load conditions.
2. Next make trial runs with the depth increased several arbitrary amounts (perhaps 0.5, 1.0, 1.5 m). Make additional analyses and make a table of dredge line displacements versus these depths and the depth from step 1.
3. From an inspection of the table from step 2, choose an arbitrary new depth of embedment D_{new}. Assume a loss of dredge line so the new depth is more than the dredge line loss, or

$$D_{new} > D_1 + \text{dredge line loss}$$

4. Now revise a copy of the original FEM data set to show the new dredge line location and new depth (compute additional active pressure values that are in the dredge line soil). Because the dredge line loss is probably attributable to erosion, it may not be necessary to reduce k_s of the first one or two nodes for driving or other damage but look at the

conditions. Make the computer analysis with this new data set. Do not recycle for depth, but do a *nonlinear* check.

5. Check this output to see if the bending moment can be carried by the sheet-pile section chosen. If not, increase the section. Also check if the toe node moves forward and how much. A large forward movement represents a soil shear failure and the embedment depth would have to be increased. If you change sheet-pile sections recycle to step 1. If you increase D recycle to step 4.
6. When step 5 is adequate, make another copy of this data set with the dredge line reset to the original location. Now add a backfill surcharge (or increase any existing surcharge) and recompute the active earth-pressure profile. Make the FEM analysis and see whether the section can carry this bending moment—if not, increase the section. Check whether the toe tends to kick out (translate forward). If it does, increase the pile embedment depth. If you change sections recycle to step 1; if you increase the embedment depth recycle to step 2.

When you have obtained satisfactory solutions from steps 3, 4, and 6, you have a suitable design. Now, what is the resulting safety factor? One possibility is that the maximum increase in depth D_{new} from steps 3 and 6 might be divided by the required depth D_1. Probably the best solution is to give the client a compact report showing the pile section and embedment depth and to indicate what loss of dredge line may produce a failure or what the maximum allowable surcharge is. File a copy and the computer printouts in case problems develop later; put the data sets on a diskette.

The finite-element method provides a relatively rapid means to analyze changed field conditions. The classical methods are much less amenable to these types of analyses and thus encourage use of an SF. If you do the analysis as outlined above and compare it to a classical design, you may find that a SF of 1.2 to 1.3 does not provide the required margin of safety for certain changed field conditions, particularly loss of dredge line.

13-4.2 Moment Reduction

From your computer output you will see that the soil node reactions below the dredge line produce a center of pressure that is closer to the dredge line than indicated by the linear Rankine/Coulomb profiles shown on Fig. 13-2*c* and *d*. This center of pressure results in computed moments that are less than those computed from the classical theories but have been confirmed by the small-scale model tests of Rowe (1952, 1957) and the larger-scale model tests of Tschebotarioff (1949). To account for this moment reduction, Rowe introduced the concept of *moment reduction* as a means to reduce moments computed by classical methods so the design would not be overly conservative (at least for bending). It is evident that the FEM directly gives the "reduced" design moment—applying Rowe's moment reduction method is not easy.

13-5 SLOPING DREDGE LINE

In many sheet-pile wall configurations the dredge line is not horizontal ($\beta = 0°$) but rather slopes away from the wall ($\beta < 0°$). How should we treat this situation? There are two cases:

1. The soil below the dredge line is a sand with $\phi > 28$ to $30°$.
2. The soil below the dredge line is a cohesive material with a small ϕ angle and cohesion c.

In case 1 we can use the Coulomb equation [Eq. (11-6)] to compute two values of K_p: one for a horizontal dredge line $K_{p,h}$ using $\beta = 0$, the other for a sloping dredge line $K_{p,s}$ using a (−) β. We can use these values to obtain a reduced $k_{s,s}$ for program input as

$$k_{s,s} = k_{s,h}\frac{K_{p,s}}{K_{p,h}} \tag{13-2}$$

where $k_{s,h}$ is your best estimate of a horizontal value that will be reduced to take into account the sloping dredge line.

For case 2 we cannot get a valid K_p from Eq. (11-6), so we will rely on the trial wedge method of Sec. 11-12.1 to obtain passive forces[5] P_p. For this, use program WEDGE on your program diskette. This program is specifically written to obtain the *passive earth force* for either a horizontal or sloping dredge line. It uses the embedment depth D for the "wall" H. We make two trials:

Trial 1: Dredge line horizontal (use only a single line) as in Fig. 13-7*a*, obtain $P_{p,h}$, and
Trial 2: Dredge line sloping as in Fig. 13-7*b* and obtain $P_{p,s}$.

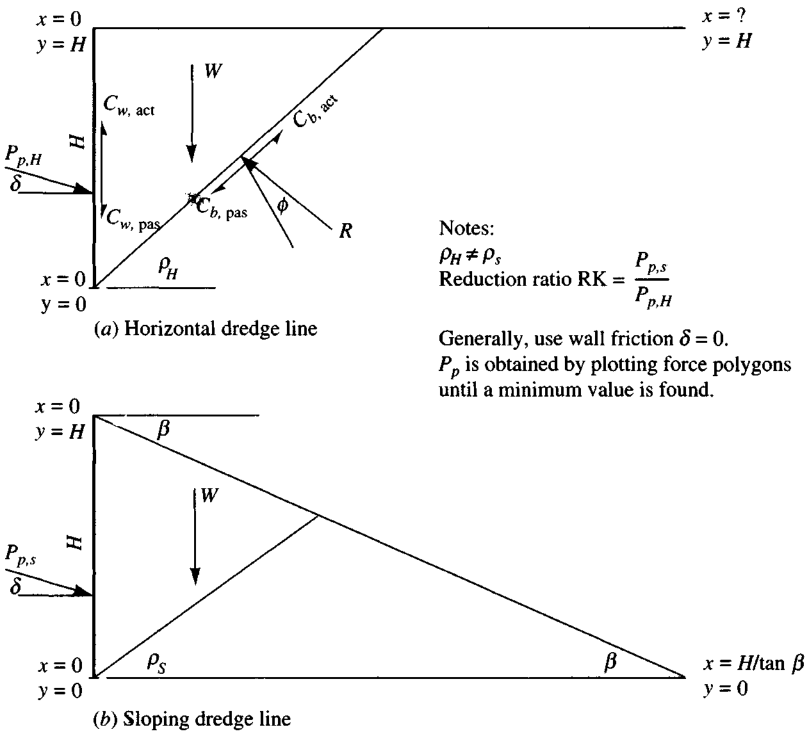


Figure 13-7 The case of sloping dredge line. Use program WEDGE from your diskette and solve both cases to obtain $P_{p,H}$ and $P_{p,s}$. Note coordinates to use. For X = ? use a value of about 4 to $5H$.

[5] Terzaghi (1954) indicated that passive earth force is a factor but did not elaborate on how to apply its effect for the sloping dredge line.

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From these two values we can compute a *reduction factor* (RF) for the several values of k_s below the sloping dredge line using

$$\mathrm{RF} = \frac{P_{p,s}}{P_{p,h}} \qquad \text{and} \qquad k_{s,s} = \mathrm{RF} \times k_{s,h} \tag{13-3}$$

where $k_{s,s}$ = sloping value
$k_{s,h}$ = best estimate of a horizontal value

For this case you must be able to input node values of k_s as a program option (allowed with program B-9).

As outlined in Sec. 13-4.1 you initiate the design of a wall for a sloping dredge line by going through design steps 1 and 2. At this point you make an initial embedment depth selection D_{init}. Now you use that D_{init} and the $k_{s,s}$ and check if D_{init} is adequate. Next check for loss of dredge line and increased surcharge.

When you check the computer output for each of the foregoing cases, you will notice that the moments are larger than for the horizontal ground case. You will also note that the nodal soil reactions will be larger near the dredge line and decrease with depth (this is similar to the horizontal case). There may even be negative values if the embedment depth is larger than needed for this analysis (but you increased it for other reasons).

Nevertheless, we need to check whether the computer program output is a possible solution and this can be determined as follows:

1. Sum by hand the node spring forces (tabulated in a table on the output sheets) and compute the passive force for the sloping dredge line as $P_{p,\text{sdl}}$.

$$P_{p,c} = \sum F_{\text{springs}} \qquad \text{then} \qquad \text{Check } P_{p,\text{sdl}} \geq P_{p,c}$$

   If you have a cohesionless dredge line soil, compute $P_{p,\text{sdl}}$ for use in the preceding as $P_{p,\text{sdl}} = \frac{1}{2}\gamma D^2 K_{p,\text{sdl}}$; if cohesive, use program WEDGE. This check assumes the limiting wall resistance is the passive force for a wall whose height is the embedment depth. The limiting passive force must be larger than that computed in the analysis [the sum of the (+) node reactions].
2. If $P_{p,c} > P_{p,\text{sdl}}$ you initially have three options to try:
   a. Try a larger pile section, because a stiffer section may even out the nodal reactions somewhat.
   b. Increase the embedment depth. [Note: This step will not improve the solution if the bottom soil nodes have (−) reactions.]
   c. Try a lower node location for the anchor rod.

If none of these produces $P_{p,\text{sdl}} \geq P_{p,c}$ consult with the geotechnical engineer who provided the soil data. It may be necessary to build up or modify the dredge line slope or use one of the walls of Chap. 12.

Schroeder and Roumillac (1983) conducted a model wall study in sand that showed that the sloping dredge line case produced less passive resistance than for horizontal ground; however, this result could have been predicted prior to any testing. Their tests showed that as the slope increased, so did the bending moments in the sheet pile. The FEM analysis using the foregoing k_s reductions does precisely that.