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# CHAPTER 15

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## CELLULAR COFFERDAMS

### 15-1 CELLULAR COFFERDAMS: TYPES AND USES

Cellular cofferdams are constructed of steel sheetpiling and used primarily as water-retaining structures. They depend for stability on the interaction of the soil used to fill the cell and the steel sheetpiling. Either material used alone is unsatisfactory; both materials in combination provide a satisfactory means to develop a dry work area in water-covered sites such as ocean- or lakefront or river area construction projects.

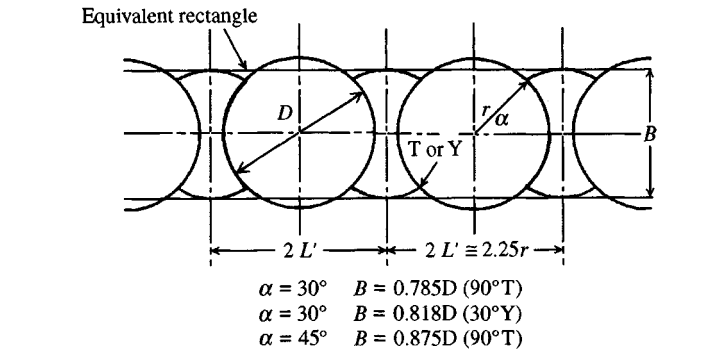
We will define the land, inside, or dry side of the cofferdam as the *basin side* and the outside as the *water side* since the cofferdam is usually used to keep water out of the *basin*.

Cellular cofferdams are not intended to be completely impervious but rather provide sufficient resistance to water flow that the quantity of water that does seep through can be readily pumped.

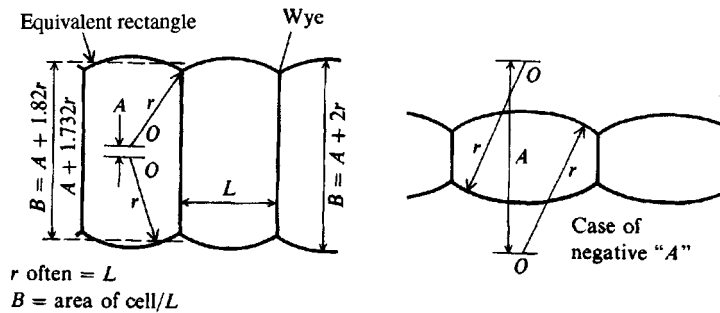
Cellular cofferdams are of three basic types: circular, diaphragm, and cloverleaf (see Fig. 15-1). These structures are usually constructed of straight-web sheetpiling since a cell full of soil and/or water tends to split so that tension stresses are produced in the web. A straight web will have essentially in-plane tension stresses; the out-of-plane thin webs of **Z** piles would develop large moments and very high bending stresses from cell-bursting forces. Tension forces would also produce large pile distortion as the pile attempted to straighten and for these two reasons **Z** piles are not used for the type of construction considered in this chapter.

Cofferdams are most commonly constructed using circular cells with smaller connecting partial cells as shown in both Figs. 15-1 and 15-4. Sometimes for waterfront structures the back side of the connecting circular cell or diaphragm (see Fig. 15-2) is omitted, however, if this is done one should consider using an anchored bulkhead.

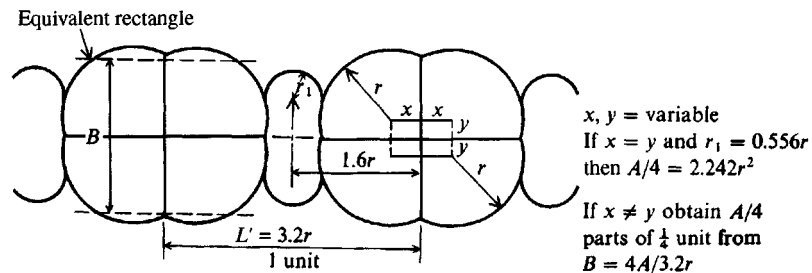
For working out into a river or for certain types of near-shore marine work a combination of circular cells is used to create a basin as in Fig. 15-3. Figure 15-1 (see also Fig. 15-4) illustrates the usual method of joining several circular cells for this purpose. There have been a few cases where the modified full circular and diaphragm cell types of Fig. 15-2 have been successfully used.



(a) Approximate dimensions of circular cells.



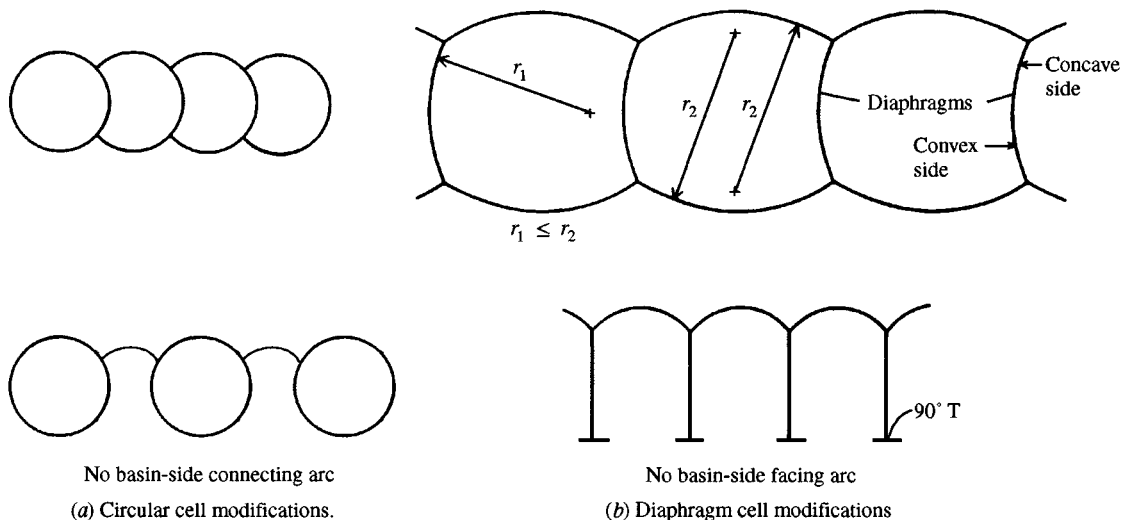
(b) Dimensions of diaphragm cofferdams.



(c) Cloverleaf cell.

**Figure 15-1** Types of conventional cellular cofferdams. Typical dimensions are shown for analysis. Alternative cell configurations are shown in Fig. 15-2.

River dams commonly use a form of Fig. 15-3 where approximately half the river is blocked with the cell line 1-2 and the work area enclosed by cells along lines 1-2-3-4 as shown in Fig. 15-3. A part of the dam is then built in this area and when completed, cells 3A-4 and 2A-1 are removed, leaving cell line 2-3 with the sheetpiling reused to construct a cofferdam from cell 2 to the far shore and from cell 3 to the far shore. When that section of dam is complete the cofferdam is removed and most of the piling is salvaged. The dam connection where the cell line 2-3 is located is done as most convenient to the contractor. For example an alternative line of cells might be set from 2B to 3B before removing piling from 2B to 1 and 3B to 4 and the far shore cofferdam then extended from 2B to shore and from 3B to shore, etc.



**Figure 15-2** Cofferdam cell modifications. Not shown is modification of height of basin-side pilings (sometimes 1 to 2 m shorter than river-side piles).

Occasionally single cells may be used as offshore mooring structures for barges and other marine equipment. In this case extension walls to provide shore access may consist of one or more arcs of sheeting, however, again it may be more economical to use a double line of anchored walls using a common anchor rod between opposite wales as shown in Fig. 13-1d.

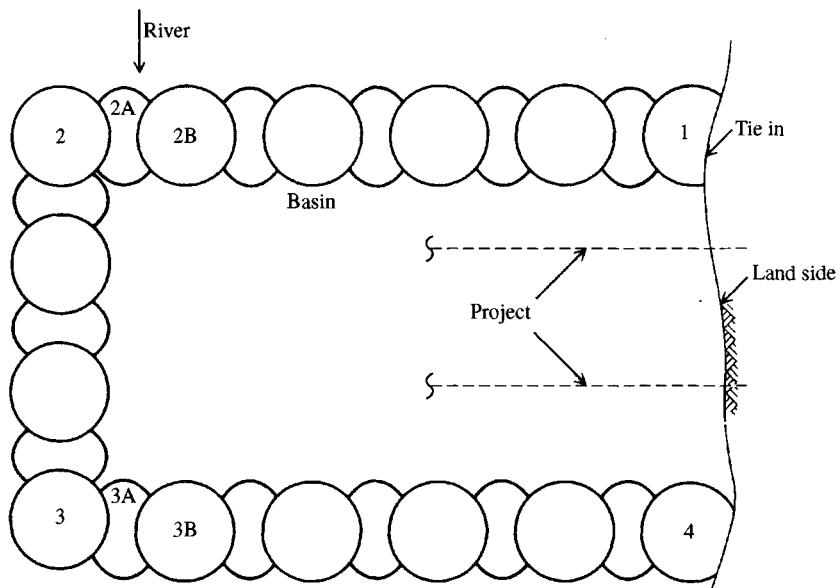
Cellular cofferdams may also be used for structures such as breakwaters and retaining walls, or the cells may be built out into the water and capped with concrete or asphalt pavement to function as piers for boats to load or unload cargo.

### 15-1.1 Cell Construction

Cofferdam cells are constructed by assembling the necessary number of sheet piles around a wooden template consisting of two rings (or other shape) spaced vertically about 3 m apart that have been anchored into correct position (usually with four or more steel **HP** piles). The sheetpiling is then placed into position with the pile sections, which have been fabricated<sup>1</sup> to connect the cells (wyes or tees), set into position first and as accurately as possible.

These become *key piles*; in deep water it may be necessary to add additional key piles made using regular sheets to which light beams or angles (or thicker plates as shown in Fig. 15-4) have been bolted to the interior part to increase their stiffness. One can also use one of the **HP** pile sections of Fig. 13-6b. The remainder of the piles are then set both ways from the key piles to close the cell. At this point the pile tips are resting in the overburden at the bottom of the river. If the closure piles do not slip easily in the interlocks to the bottom, the adjacent piling is picked up in multiples and “shaken out” until all the sheets in the cell perimeter are

<sup>1</sup>If there is sufficient quantity of intersection units it may be possible to have the producer extrude them rather than use shop fabrication.

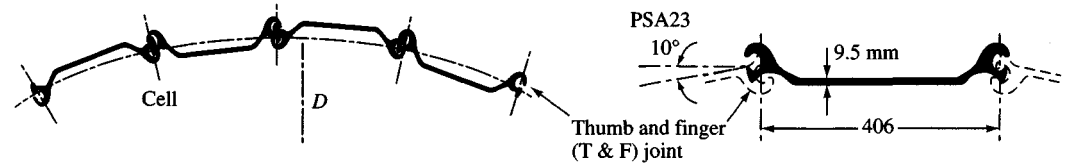


(a) General layout of cellular cofferdam.

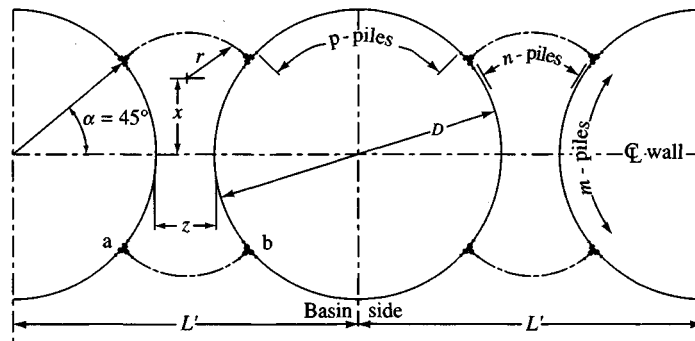


(b) Cofferdam under construction showing initial part of cell line 1–2 with cells being partially filled for stability.

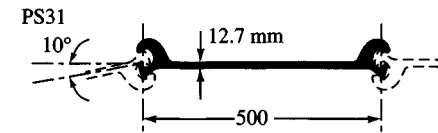
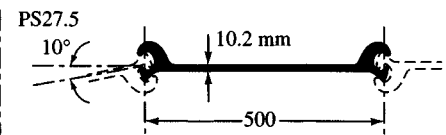
**Figure 15-3** Cofferdam work (or basin) area. This “dry” area may be in the range of 5000 to 30 000 m<sup>2</sup>.



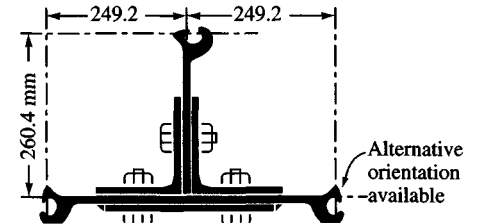
(a) Circular cell using straight web sections



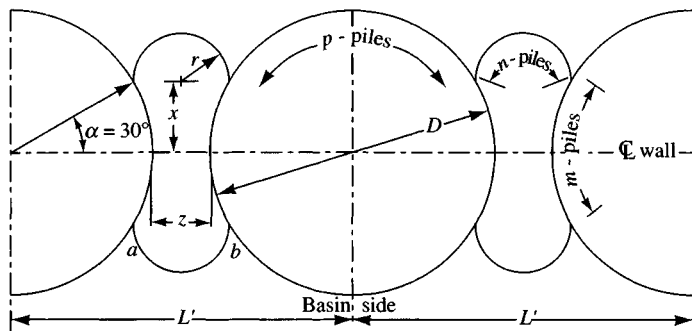
(b) Circular cell using 90° T connections



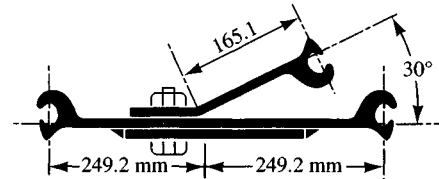
Sections for cofferdams



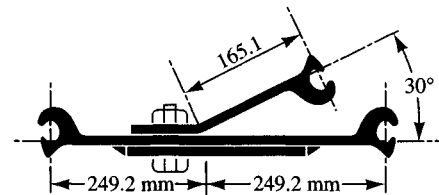
**Figure 15-4** Sheetpiling and connections used in cellular cofferdam construction. Bolts are A325 with washers (usually 22-mm diam) at 115-mm spacing except 600-mm end zones where spacing is 75 mm. (Figure is a composite from Bethlehem Steel Corporation booklet No. 2001).



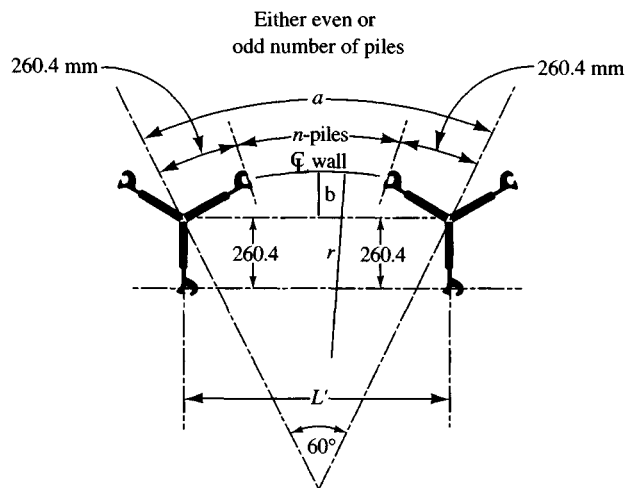
(c) Circular cell using 30° Y connections.



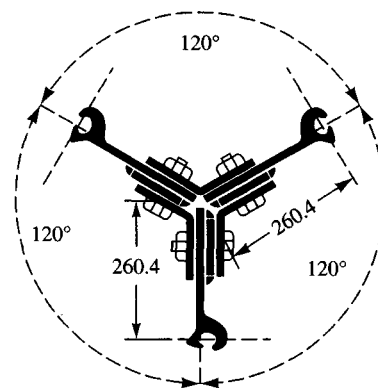
T & F joint orientations



30° cell connection



(d) Diaphragm cell using 120° Y connections.



Typical Y connection.

Figure 15-4

free-running in the interlocks. Driving can now commence and the cell piles are driven—usually in pairs to a depth of about 1 to 2 m, then the next pair, etc., around the cell perimeter. This process requires an even multiple of piles between the key piles. The operation is then repeated, either using a new starting pair of piles or going in reverse, to avoid distortion of the cell from systematic accumulation of driving effects to one side.

Two or more piles are set for the start of the connecting arcs before the key piles are driven to final grade. Long piles may require splicing, for the driving equipment will have some kind of length limitation. Splices are made by cutting the first piles in staggers so that the splice will vary up and down some 1 to 2 m and fall above basin side cell water line and side ground level on either side.

In fast-moving water or high winds, pile-setting operations are greatly slowed as it is difficult to maintain pile alignment. A movable fender or breakwater may be used in fast water to protect the piles during driving but for wind there is little that can be done.

The cell template should be positioned within about 150 to 300 mm of alignment for circular cells and to less than 150 mm for diaphragm cells. Closer tolerance than this is not often possible and is usually unnecessary owing to cell distortion during filling and dewatering operations.

### 15-1.2 Cells and Number of Piles Required for a Cell

A series of connecting soil-filled cells (Fig. 15-2) around the perimeter of a work area is termed a *cofferdam*. The basin side is usually provided with a drainage ditch emptying into sump pits where the water can be pumped back to the wet side.

The circular cell cofferdam of Fig. 15-1a consists of circles of different radii intersecting as shown. The cell intersection angle  $\alpha$  is usually either  $30^\circ$  or  $45^\circ$  (Fig. 15-1a). The joint is either a  $90^\circ$  T or a  $30^\circ$  Y, but other angles might be used for special cases. The  $30^\circ$  Y is claimed to produce smaller stresses in the connection than other angles for connecting deep, large-diameter cells.

Sheetpiling interlocks allow a maximum of about  $10^\circ$  deflection between pieces. This results in a minimum cell radius of

$$r \approx \frac{\text{Driving distance, m or ft}}{2 \sin 10^\circ}$$

For a PS31 section,  $r \approx 0.50/(2 \times \sin 10^\circ) = 1.44$  m for a minimum cell diameter  $\approx 2.88$  m.

The number of sheet piles  $N_s$  in a cell (or circle) of radius  $r$  and driving distance  $D_d$  (given in sheet-pile tables such as Appendix Tables A-3a, A-3b) is

$$N_s = \frac{2\pi r}{D_d} \quad (15-1)$$

For a cell diameter of 6.05 m and using PS27.5 sections, we find that the driving distance (Appendix A-3a)  $D_d = 0.500$  m requires

$$N_s = \frac{2\pi 6.05/2}{0.500} = 38.013 \text{ piles}$$

Round off and use 38 piles. If the decimal fraction were much over 0.01 it would be necessary to round up to 40 piles (must use integer multiples of 2 for driving in pairs). Forty piles would require that the diameter be slightly increased to

$$\text{Diam} = \frac{40 \times 0.500}{\pi} = 6.37 \text{ m} \quad (\text{vs. original } 6.05 \text{ m})$$

Pile producers generally will provide free tables for calculating the number of piles needed for cells and diaphragms of varying practical dimensions.

### 15-1.3 Diaphragm Cells

Diaphragm cells are made of a series of circular arcs connected by crosswalls (diaphragms) using  $120^\circ$  intersection pieces (Figs. 15-1*b*, 15-4*d*). The radius of the arc is often made equal to the cell width  $L'$  (Fig. 15-1*b*) so that the interlock tension in the arcs and diaphragms may be equal. The distance  $A$  shown in Fig. 15-1*b* may be either positive for high, wide cells or negative for low, narrow cells. A wide cell (large  $B$ ) will be necessary for stability when a large head of water is to be resisted.

Other cell types, such as cloverleafs (Fig. 15-1*c*) and ellipsoidal shapes (Fig. 15-2*b*), may be assembled from sheetpiling shapes and fabricated connections, depending on the purpose, cell height (head of water), type of fill, amount of tolerable distortion, and location.

The cloverleaf type has been used considerably as a corner, or anchor, cell in conjunction with circular cells. This cell can also be used to reduce the effective diameter of a cell when a large cell width is required for stability against a high head of water.

### 15-1.4 General Cell Details

The basin-side piling may be 1 to 2 m shorter than the wet side, producing a slope across the cell top (and fill) for some savings in steel mass. It is also possible to use a **Z** pile anchored wall for the wet side of the diaphragm wall of Fig. 15-1*b* since water side piles are in compression. The anchor rods, primarily for alignment, are attached to exterior wales and extend back into the cell fill (or attach to the dry-side walls). Swatek (1967) described a wide range of cofferdam configurations and heights; one should look at this publication prior to making a design—particularly for unusual site conditions of large water head  $H_w$  or poor base soil.

The circular cell is generally preferable to the other cellular types for the following reasons:

1. It is stable as a single unit and can be filled as soon as it is constructed.
2. The diaphragm-type cell will distort unless the various units are filled essentially simultaneously with not over 0.5 to 0.75 m of differential soil height in adjacent cells; the use of a circular diaphragm cell (Fig. 15-2*b*) reduces this requirement if filling is first against the concave wall side of Fig. 15-2*b* [Cushing and Moline (1975)].
3. The collapse of a diaphragm cell may cause the entire cofferdam to fail, whereas the collapse of a circular cell is generally a local cell failure.
4. The circular cell is easier to form using templates.
5. The circular cell usually requires less sheetpiling, but this need depends somewhat on the diaphragm crosswall spacing.



Increasing the size of a circular cofferdam cell does not necessarily increase the total quantity of sheetpiling for the cofferdam, since the total number of cells will be reduced. This is not true for the diaphragm-type cell. The quantity of cell fill depends directly on the cell dimensions for all types of cofferdams.

### 15-1.5 Sand Islands

*Sand islands* are large circular cells, generally using fairly short sheetpiling, that are filled with sand and sometimes capped with concrete. They provide a dry work area where the water table is at or only 0.5–1 m above the existing ground surface.

A smaller sheeted excavation such as for a bridge pier may be constructed through this small artificial island. Sand islands are usually left in place after construction; however, most other cofferdams are removed and the sheeting stored for reuse or sold as used material.

### 15-1.6 Connections

The connections shown in Fig. 15-4 are all bolted. This step is necessary for economy and so that driving does not cause separation of the parts. Fillet welds tend to fracture from driving stresses and are seldom used. Fracture of built-up welded sections can be avoided by

1. Preheating the parts to be welded to about 540°C (about 1000°F) so that the parts are essentially fused together in welding. This high heat is seldom practical.
2. Using both longitudinal fillet welds and transverse slot welds. The slots should be spaced on about 1-m staggered spacings and of the filled type (not filleted). This approach is costly both because of the substantial amount of welding required and the great effort needed to achieve proper notching of pieces so the slots are staggered. The slots would tend to keep the fillet welds in joined pieces from slipping with respect to each other, and fracturing during driving.

## 15-2 CELL FILL

The cell fill provides mass (or weight) for stability and a reduced coefficient of permeability  $k$  for retaining water without excessive pumping. These advantages must be balanced against the lateral pressure effects of the soil-water mixture and the resulting stresses that the sheet-pile interlocks must resist before rupture and/or cofferdam failure.

For mass, it would be preferable to use a soil with a high density. For permeability considerations alone, clay is the best possible fill. The earth-pressure coefficient of sand with a high angle of internal friction  $\phi$  gives the minimum lateral pressure that must be resisted by *hoop tension* in the interlocks, which usually controls cell design. Considering all these factors, the best cell fill:

1. Is free-draining (large coefficient of permeability,  $k$ )
2. Has a high angle of internal friction,  $\phi$
3. Contains small amounts of No. 200 sieve material—preferably less than 5 percent
4. Is resistant to scour (nonsilty or clayey)—requires presence of some gravel

Cell fills that do not meet these criteria are sometimes used, but the closer the fill material approaches these criteria the more economical the design in terms of sheetpiling, which is usually the most expensive portion of the cofferdam.

Cell fill is often placed hydraulically; i.e., the material is obtained from the river bottom if at all possible. The material is dredged and pumped through a pipe system and discharged into the cells, which are already driven, with the river level being the inside water line. This operation may substantially reduce the fines, which are often present in river-bottom material and which are temporarily suspended in the water and wash overboard. Of course, if material is not available close by, fill may have to be brought in by barge, truck, or rail. In any case the cell fill is generally deposited under water so the angle of internal friction  $\phi$  may not be very large. It appears that this method of soil deposition seldom produces an angle of internal friction over about  $30^\circ \pm 2^\circ$ .

Unless satisfactory drained triaxial tests can be performed on the soil and at the expected cell density, the  $\phi$ -angle should be limited to  $28$  to  $30^\circ$  for design (or preliminary design). It is possible to increase the cell fill density and  $\phi$  by using some type of compaction with vibratory equipment such as the Vibroflot or Terra-probe described in Sec. 6-5. If this is carefully done (*before any drawdown of water on the basin side*) relative densities  $D_r$  on the order of  $0.75$  to  $0.85$  can be obtained with  $\phi$ -angles in the range of  $35$  to  $40^\circ$ .

### 15-3 STABILITY AND DESIGN OF CELLULAR COFFERDAMS

The design of a cofferdam requires providing an adequate margin of safety against the following:

1. Cell sliding (Fig. 15-5a)
2. Cell overturning (Fig. 15-5b)
3. Cell bursting of Fig 15-5c, which is usually critical since the interlock (thumb and finger joints) are the weakest part of the system.
4. Cell shear along the centerline and including a component of interlock friction as illustrated in Fig. 15-5d.
5. Bearing capacity and settlement (not shown)

There are no theoretical solutions for any of these five factors owing to the complex interaction of the cell geometry, sheet piles, and cell fill. Further complicating the analysis is the transient state of water level outside and against the cell and the saturation line inside the cell fill. Finally, in river environments there is the ever-present possibility of flooding and overtopping. As a consequence of these uncertainties, cofferdam design is semi-empirical and there are at least three design approaches to the problem, all of which have had a reasonably successful design history. These methods are as follows:

1. Former Tennessee Valley Authority (TVA) methods, also called Terzaghi's method
2. Cummings method
3. Hansen's (or Danish) method

Of these the TVA (1966, but publication now out of print) and Cummings (1960) methods are commonly used in the United States and elsewhere. The Hansen method as modified by

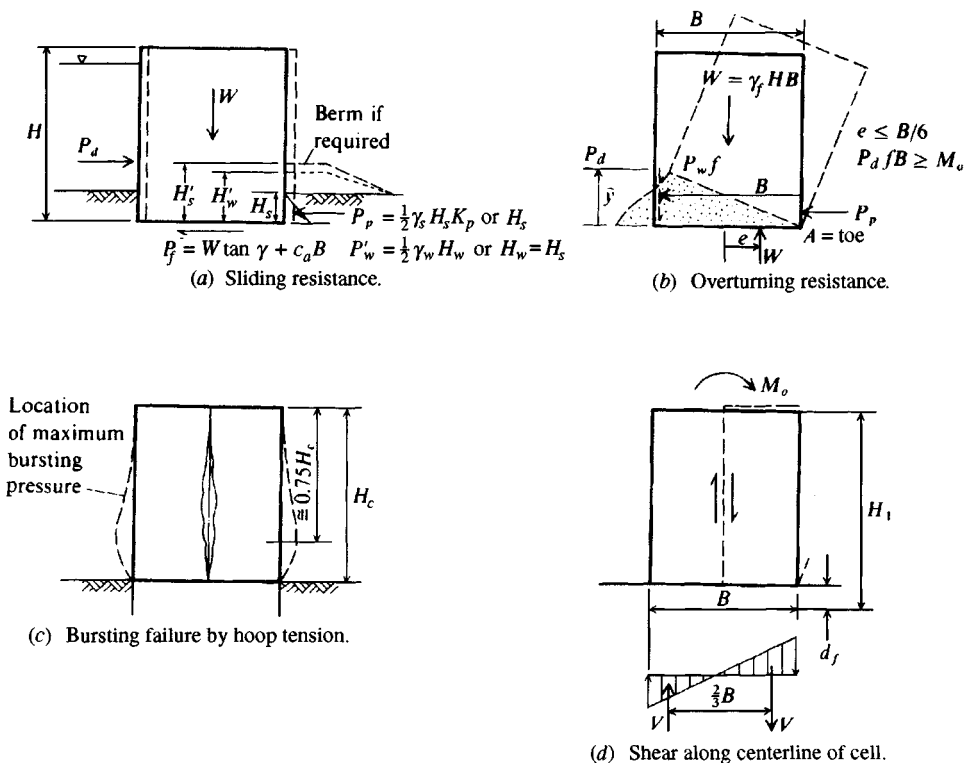


Figure 15-5 Stability of cofferdams.

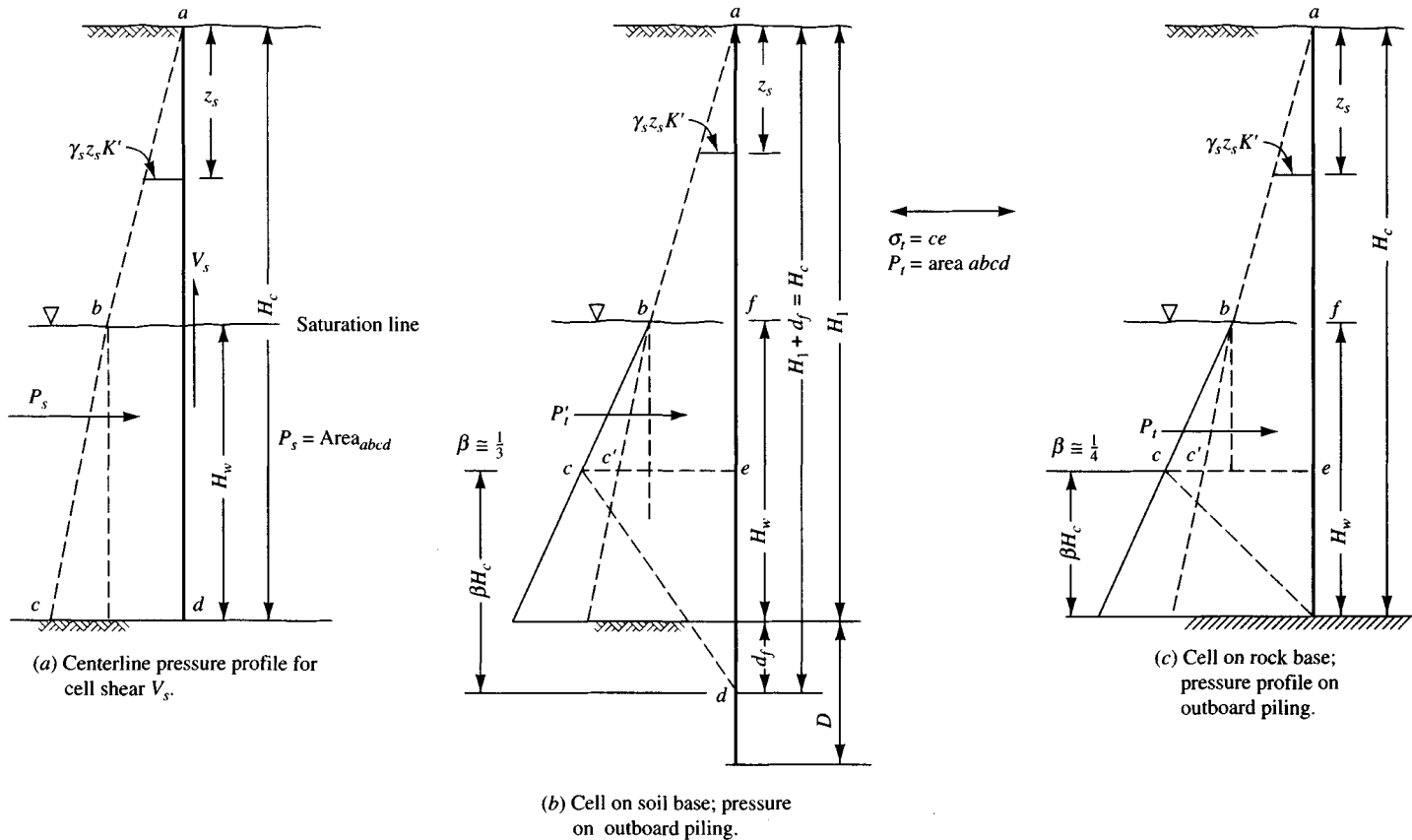
Ovesen (1962) is used much less—primarily in Europe. More cofferdams have been built by TVA and the U.S. Corps of Engineers than by any others. Thus, the TVA and Cummings methods have much to commend them since, if one must use empirical methods, the simpler ones are preferable. This is a major drawback of using the Hansen method—aside from there being less construction experience to validate it. For these reasons it is not considered further in this text; however, for the interested reader the method is outlined in Lacroix et al. (1970).

Dismuke (1975) provides a summary of the several design methods in use in the United States, and Sorota and Kinner (1981) describe a recent use of the several U.S. design methods in a major cofferdam installation. This latter reference provides instrumented data comparing design to the as-built stresses and deformations; particularly valuable since there is not a great deal of published postdesign verification available.

### 15-3.1 TVA Method of Cellular Cofferdam Design

Terzaghi (1945) presented a paper on cellular cofferdam design in which the methods used by TVA in dam building along the Tennessee River since about 1935 were outlined. TVA (1966) later published a monograph, with the first printing in 1957 having outlined in some detail their design methods.

In the following discussion the unit weight of soil for all states will generally be used as  $\gamma_s$ , but its numerical value will depend on its location in Fig. 15-6 (it may be dry, damp,



**Figure 15-6** Cell pressure profiles for centerline shear, interlock friction, and interlock tension.

or saturated, with the effective value  $\gamma'$ ). The TVA method considers the following (refer to Fig. 15-5 for factors and to Fig. 15-6 for terms).

**SLIDING STABILITY.** A cofferdam must provide adequate resistance to sliding on the base caused by the unbalanced hydrostatic pressure. A sliding stability number  $N_s$  is defined (see Fig. 15-5a), neglecting any active soil pressure, as

$$N_s = \frac{P_p + P_f + P'_w}{P_d} > 1.10 \quad (15-2)$$

where  $P_f$  = friction on base as  $W \tan \delta + c_a B$

$P_d$  = driving force (usually outside water with  $P_w = \frac{1}{2} \gamma_w H_w^2$ )

$P_p$  = passive resistance ( $\frac{1}{2} \gamma'_s H_s^2 K_p$ ) but may include a berm.

With a berm it is also necessary to estimate the location of the water surface since the passive pressure is an *effective* stress computation but there is also a berm water force of  $\frac{1}{2} \gamma_w H_w'^2$  where  $H_w'$  = water depth on basin side (may be from near top of berm or near top of excavation of depth  $H_s$  below existing dredge line)

In this equation the active earth force  $P_a$  on the water side (not shown) is usually neglected unless the embedment depth is more than about 1.75 m.

Use a  $N_s \geq 1.25$  if this analysis controls the size of the cell.

Berms (Fig. 15-5a) may be used to increase sliding resistance. The berm, being limited in plan, may not fully develop passive pressure, so it might be best analyzed using the trial wedge method of Sec. 11-12.1. For a sloping berm one can use the Coulomb  $K_p$  with a negative  $\beta$  angle. If the berm has a shelf (broken backslope) and a slope on the order of about 3H:1V or larger, the berm may be analyzed as a sliding mass of some weight and appropriate friction coefficient  $\tan \delta$  between berm and base.

A problem with berms is the location of the passive resistance. Although one may take this as  $H_s/3$  or  $H_s/3$  from the bottom, this is not likely correct. One might use the sheet-pile program FADSPABW to locate the center of resisting pressure and its magnitude.

It is often better to increase the cell diameter rather than to use a berm (an increase in diameter is not directly related to the increase in number of sheet piles required). The berm increases the required basin space, so some economy is achieved by increasing the cell diameter and possibly using a smaller basin.

If a berm must be used it is preferable to use the existing soil, i.e., leave that part of the basin unexcavated rather than excavating and backfilling—unless the dredged soil is totally unsuitable. If the excavation does not produce sufficient berm height, use as much of the excavated soil as practical to increase the berm height.

**OVERTURNING STABILITY.** The cofferdam must be stable against overturning. Two possibilities, or types of analysis, can be made when considering this type of stability. To avoid overturning, and reasoning that soil cannot take tension forces, we see that the resultant weight  $W$  should lie within the middle one-third of the base (see Fig. 15-5b) giving

$$e = \frac{P_d \bar{y}}{\gamma H B} = \frac{P_d \bar{y}}{W} \quad (a)$$

Thus, larger cell heights  $H$  require wider *average* cell widths  $B$  defined by the *equivalent rectangle* of Fig. 15-1. The unit weight  $\gamma$  used in Eq. (a) is understood from previous discussion as the **average** for the cell.

Alternatively, one may reason that as the cell tends to tip over, the soil will pour out at the heel. For this to occur the friction resistance between the cell fill and the water-side sheet-piling must develop from the water force  $P_d = P_w$ . Summing moments about the toe of the cell (point A of Fig. 15-5b) gives

$$BP_w \tan \delta = P_w \bar{y}$$

and the required average width  $B$  is

$$B = \frac{\bar{y}}{\tan \delta} \quad (b)$$

where  $\delta$  = angle of friction between cell fill and steel and may be estimated at about 0.6 to 0.7 $\phi$  or from Table 11-6. The stability number  $N_{ot}$  is

$$N_{ot} = \frac{B \tan \delta}{\bar{y}} \quad (\text{about } 1.1 \text{ to } 1.25) \quad (15-3)$$

If the sheetpiling is embedded to some depth in the soil, the effects of the active  $P_a$  and passive  $P_p$  soil pressures on the overturning moment and friction resistance should be included in summing moments about point A in Eq. (b).

This  $N_{ot}$  stability check is not now used by TVA (1966, see Foreword) since the mode is highly unlikely, but it is not a difficult check and probably should be continued.

**CELL SHEAR.** Shear along a plane through the centerline of the cell is another possible mode of failure (Fig. 15-6a). For stability, the shearing resistance along this plane, which is the sum of soil shear resistance and resistance in the interlocks, must be equal to or greater than the shear due to the overturning effects. Referring to Fig. 15-5d and assuming a linear pressure distribution across the base of the cell, we have

$$M_o = \frac{2}{3}BV$$

and solving for the vertical shear force  $V$  we obtain

$$V = 1.5M_o/B \quad (15-4)$$

For stability the resisting shear  $V_r \geq V$ .

Since  $V_r$  depends on both interlock resistance  $R_{il}$  and cell shear along the center line  $V_s$ , it is necessary to obtain their values so that

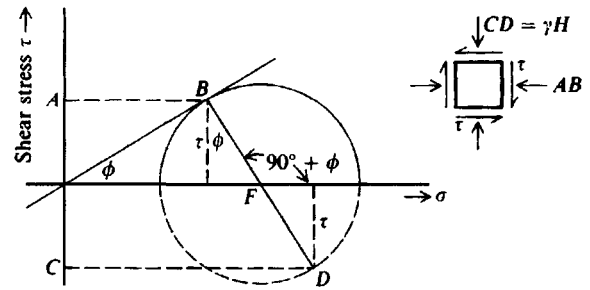
$$V_r = V_s + R_{il} \geq V \quad (15-5)$$

**Soil shear resistance.** The soil shear resistance part of the total cell shear resistance is computed from Fig. 15-6a as

$$V_s = P_s \tan \delta \quad (15-5a)$$

with  $P_s$  = area of pressure profile  $abcd$ . The location of the resultant  $\bar{y}$  is not required.

The earth-pressure coefficient  $K'$  may be computed using the Mohr's circle construction of Fig. 15-7—which may not be correct. Figure 15-7 was developed by Krynine in his discussion of Terzaghi's 1945 paper [Terzaghi (1945)] and was based on the idea that the cell centerline



**Figure 15-7** The method suggested by Krynine in his discussion of the Terzaghi (1945) paper to compute the earth pressure coefficient  $K'$  for vertical shear along the cell centerline.

$$\begin{aligned}\tau &= AB \tan \phi = AB \frac{\sin \phi}{\cos \phi} & FB &= \frac{\tau}{\cos \phi} = AB \frac{\sin \phi}{\cos^2 \phi} \\ CD &= AB + 2\tau \tan \phi = AB + 2AB \frac{\sin \phi}{\cos \phi} \frac{\sin \phi}{\cos \phi} \\ &= AB \left(1 + 2 \frac{\sin^2 \phi}{\cos^2 \phi}\right) \text{ and } \sin^2 \phi = 1 - \cos^2 \phi \\ K' &= \frac{AB}{CD} = \frac{\cos^2 \phi}{2 - \cos^2 \phi}\end{aligned}$$

is not a principal stress plane but at any centerline depth  $z_i$  there is a vertical stress  $CD = \gamma z_i$ , which produces a horizontal stress of  $\gamma z_i K'$  and a shear stress  $\tau$ . From a Mohr's circle and using the trigonometric relationships shown in Fig. 15-7 we produce the desired equation for  $K'$  as

$$K' = \frac{\cos^2 \phi}{2 - \cos^2 \phi} \quad (15-6)$$

Since this equation gave  $K' > K_a$  for the same angle, the readers (and Terzaghi) assumed it was correct. No one immediately noticed that although it gave  $K' > K_a$  for increasing  $\phi$ -angles, it also gave smaller  $K'$ -values for increasing  $\phi$ -angles, which was clearly not correct. It has since been postulated that Eq. (15-6) is not correct since the lateral stress on the cell centerline was developed both by cell soil and the lateral force  $P_d$  of Fig. 15-5a.

The following table is instructive. It tabulates the foregoing discussion for a rapid comparison. In the table we will use a constant  $H$ , vary both  $\phi$  and  $\gamma_s$ , and compute both the Rankine  $K_a$  and  $K'$  as shown:

$\phi$ , degrees	$\gamma$ , kN/m <sup>3</sup>	$K_a$	$K'$	$\frac{1}{2} \gamma H^2 K'$	$\frac{1}{2} \gamma H^2 K' \tan \phi$
30	15.7	0.333	0.600	$4.71H^2$	$2.72H^2$
34	16.7	0.283	0.524	$4.38H^2$	$2.95H^2$
38	17.7	0.238	0.450	$3.98H^2$	$3.11H^2$
42	18.7	0.198	0.381	$3.56H^2$	$3.21H^2$

The table uses unit weights consistent with the increase in the angle of internal friction  $\phi$ . In the commonly used range of  $\phi$  from 30 to 34° there is only about an 8 percent increase in shear resistance on the centerline. This table indicates that the earth-pressure coefficient  $K'$  from Eq. (15-6) is adequate if correctly used.

The use of  $\phi = 30^\circ$  is probably not realistic for a sandy soil deposited under water and given a modest amount of vibration by a Terra-probe [a round pipe firmly attached to a vibrat-

ing pile driver, which is slowly driven into the cell soil (including the base soil if possible) and pulled with a crane while still vibrating] or the like. An ordinary concrete vibrator can be used if it has a sufficiently long probe.

It is suggested that using  $K' = 0.45$  to  $0.50$  is a good compromise instead of using Eq. (15-6). The author suggests values of  $K'$  between  $0.45$  and  $1.0$  ( $0.45$  at  $30^\circ$  and  $1.0$  at about  $40^\circ$ ).

Maitland and Schroeder (1979) suggested using  $K' = 1$ ; however, the author found that using  $0.56$  gave the best moment resistance comparison for that case. Sorota et al. (1981) suggest  $K' = 0.35$  to  $0.40$  for compacted well-graded granular soils, which seems somewhat low. Since there is no universal agreement on what to use for the effective cell height (see  $\beta$ , Fig. 15-6b, c) it is reasonable that there is no agreement on what to use for  $K'$ .

The cell environment is such that precise attempts to identify the soil state are not justified. A rain can easily saturate the top zone (unless capped). In flood stages the cells may become overtopped regardless of freeboard (see Fig. 15-11); however, overtopping may not be critical since the interior, or dry side, is likely to fill before cell saturation occurs. If it is possible that cell saturation could occur first, some systems have deliberately provided a flood gate to ensure the basin fills before the cells saturate and possibly burst.

**Interlock friction/shear.** Friction in the interlock joints (see Fig. 15-8) occurs simultaneously with soil shear resistance for vertical shear distortion along the centerline to take place. Conventional design uses the average interlock tension based on using  $P_t$  of Fig. 15-6a or b. Here the lateral force (for a unit of width) is

$$P_t = \text{area } abcd \quad (15-7)$$

Note the use of  $K_a$  for the lateral pressure here and not  $K'$ . The interlock friction resistance contribution is

$$R_{il} = P_t f_i \quad (15-5b)$$

**Figure 15-8** Cell interlock tension force computations. Suggest using  $K' = K_a$  for these computations to obtain force  $P_s$  shown in Fig. 15-6b, c.

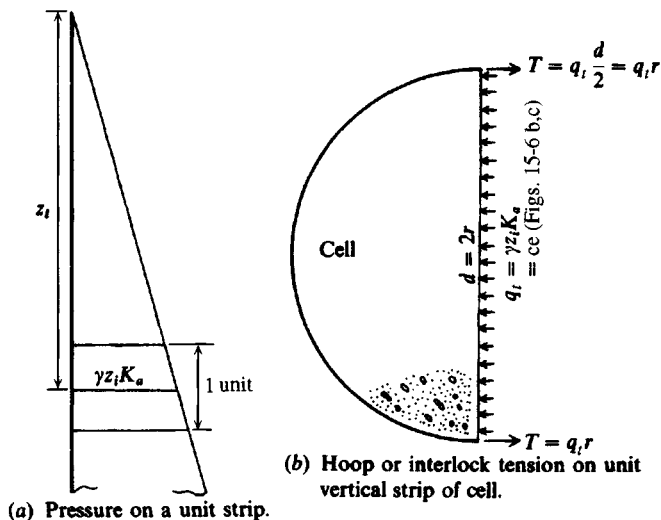




TABLE 15-1

**Sheet-pile sections commonly used for cellular cofferdams with interlock tension and suggested SF.**Recommended allowable stress for sheet pile web tension  $f_a = 0.65f_y$ .

AISI designation after 1971	Current* sections (1995)	Guaranteed† interlock tension, kN/m, k/in.		SF	$f$
PSA23, PSA28	PSA23	2100	(12) A328	4	0.3
PS28, PS32	PS27.5, PS31	2800	(16)	2	0.3
		3500	(20) A572-50	2	0.3
		4900	(24) A572-60	2	0.3

\*Bethlehem Steel Corporation (only U.S. producer as of 1995).

†Normal steel grade is A328 ( $f_y = 270$  MPa).

Notes: 1. A572 steel grade is available in 350 and 400 MPa.

2. A690 corrosion-resistant steel grade 400 MPa.

3. Use high SF of 4 for PSA23 section as the web may tend to straighten under a high-tension stress.

where  $f_i$  = interlock friction coefficient, usually taken as 0.3 (values of 0.25 to 0.4 have been measured, and higher values are obtained when the steel is wet) as given in Table 15-1.

The total cell shear resistance combines Eqs. (15-5a) and (15-5b) for the *circular* cell to obtain Eq. (5-5), given earlier and repeated here:

$$V_r = V_s + R_{if} < V \quad (15-5)$$

Carefully note that  $V_s$  in Eq. (15-5) is per unit of width, and the computation for  $R_{if}$  is also in per unit of width.

The stability number against cell shear  $N_{cs}$  is defined as

$$N_{cs} = \frac{V_r}{V} = \frac{V_s + R_{if}}{V} \geq 1.1 \text{ to } 1.25 \quad (15-8)$$

**BURSTING STABILITY.** The cells must be stable against bursting pressures (pulling apart of the piles at the joints).

**Interlock tension.** Experiences at the TVA and elsewhere indicate that during filling of the cell, lateral pressures develop during filling and increase during subsequent consolidation of the fill (on the order of 10 days or so). The cell expands in proportion to the lateral pressure but expansion is dependent on the base restraint—whether the cell is founded on rock or embedded in the ground. The expanded cell takes on a modified barrel or bulged shape, and field observation finds the bulge most pronounced at from one-fifth to one-third of the free height of the cell above the dredge line or rock. On this basis the TVA uses  $H_c/4$  for cells on rock to the *maximum bulge*. This point is also most critical for interlock tension.

Maitland and Schroeder (1979) suggest finding this point as one-third of the modified cell height  $H_1$ , which is based on the depth of fixity  $d_f$  below the dredge line when the cell is embedded in soil (or not driven to rock), to obtain

$$H_1 = H_c + d_f$$

where  $H_c$  = free cell height (see Fig. 15-6b for  $H_c$  and  $d_f$ ).

The location of the depth of fixity  $d_f$  can be estimated two ways:

### Method 1.

1. Compute the inside effective lateral pressure at the dredge line inside the cell as  $p_a$ .
2. Compute the estimated depth of fixity  $d_f$  in sand as the point of zero pressure (also zero deflection) using

$$d_f = \frac{P_a}{\gamma'(K_p - K_a)} \quad (15-9)$$

where  $\gamma'$  = effective unit weight below the outside dredge line

$K_p, K_a$  = Rankine passive and active earth-pressure coefficients for the soil below the dredge line

If the dredge line soil has  $\phi = 0$ , use  $d_f \approx 0.3$  to  $0.5$  m.

**Method 2.** Use your program FADSPABW and a pressure profile computed as  $q_h = \gamma_{\text{eff}} \times z_i$  for the depth  $H_c$ . Now adjust  $q_h$  for the pile width (not a unit width); use the moment of inertia for the single sheet pile. Now code the piling (use fairly long elements above the dredge line, because they are not critical; define the JT Soil node and use short nodes below the dredge line. Make a trial and see what you get. You may have to increase the pile depth or let the program increment it. If this analysis is done, you may find that the initial trial cell embedment depth should be increased—if so, increase the depth and try again. Now from the output sheets plot the location of zero displacement and measure this from the dredge line as  $d_f$ .

An estimate of the lateral subgrade modulus in the embedment part of the pile must be made. Note, however, that the  $H_c/4$  location shown in Fig. 15-6c is commonly used and is probably as accurate as the  $\phi$  and  $\gamma$  being used; there is not a great deal of difference in the design whether you use the  $H_c/4$  or the  $H_c/3$  locations. If you do not elect to find  $d_f$ , then use  $H_1 = H_c$ .

The pressure intensity  $q_t = ce$  of Figs. 15-6b or 15-6c is used for the critical interlock tension  $t_i$  and with reference to Fig. 15-8b is computed as follows:

$$t_i = \frac{q_t r}{C_1} \leq \frac{t_u}{SF} \quad (15-10)$$

where  $q_t$  = pressure intensity  $ce$  of Figs. 15-6b,  $c$ , kPa or k/ft<sup>2</sup>

$C_1$  = constant: use 1 if  $q_t$  in kPa; use 12 if in k/ft<sup>2</sup>

$t_u$  = ultimate interlock value from Table 15-1, kN/m or k/in.

SF = value from Table 15-1

Appendix A Tables A-3a and A-3b give the profiles and additional section properties of the sections rolled in the United States and select sections rolled in Europe to supplement the data in Table 15-1.

The designer can, of course, use any other pressure profile deemed more suitable than those of Fig. 15-6b, *c* along with whatever value is selected for the earth pressure coefficient  $K_a$ . One possible choice is to use a parabolic distribution, which gives approximately a one-third increase in the hoop tension.

Both the model cells of Maitland and Schroeder (1979) and the prototype cells of Sorota et al. (1981) show that when the basin (work area) side is dewatered the interlock tensions increase in the range of 20 to 25 percent on the basin side. Simultaneously the interlock tensions decrease on the river side. The reason is probably that the cell, acting as a large gravity-retaining structure, produces a compression arch on the river side, which tends to open the basin side sheets. Since this loading stage is only one of several to which the cell is subjected, it is not generally feasible to use, say, lower strength interlocks on the river side.

The cell location during filling also affects the bursting pressure  $q_t$ ; a cell near the shore will—during and shortly after filling—be subjected to both the maximum  $q_w$  and a maximum effective earth pressure  $q_{sh}$ . This increase is compensated somewhat by the near-shore cell's usually having smaller  $H_c$ . Cells in the water will undergo only active effective earth pressure until dewatering.

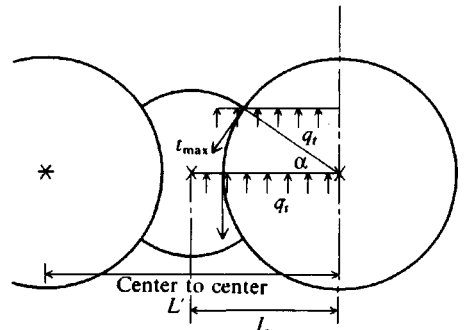
Most cofferdam failures result from failure of the connecting tee from either a fabrication failure or interlock failure [Swatek (1967), Grayman (1970)]. According to the TVA (1966) the interlock tension of the connection pieces can be computed from the free body of the cell as shown in Fig. 15-9. Summation of forces gives the interlock tension in the connection as

$$T_{it} = q_t L / \cos \alpha \quad (15-11)$$

In this equation  $L = \frac{1}{2} L'$  of Fig. 15-1a and Fig. 15-9. The maximum interlock tension can be reduced by decreasing  $\alpha$ , which may require use of a 30° Y instead of a 90° T in order to obtain a reasonable width of connecting arc.

One may obtain the maximum tension force from a free body diagram that considers hoop tension in both the main and connecting cells; however, both TVA (1966, p. 112) and Dismuke (1970) show that approximately the same value is obtained from using Eq. (15-11). Rossow (1984) made a theoretical analysis of the interlock tension at the connection joint, but the results were of little value because too many assumptions were used.

There is some opinion that Eq. (15-11) may be overly conservative, but the results reported by Sorota et al. (1981) did not indicate this. Also note that there is a wide range in possible  $T_{it}$  values, depending on what is assumed for the instant depth of water in the cell and what is used for the active earth pressure coefficient  $K_a$ . For example, using a good-quality granular cell fill with a modest amount of compaction to increase  $\phi$  from 30 to 36° (probably more



**Figure 15-9** Connecting Y or T stresses according to TVA.

nearly correct) produces a 28 percent stress reduction between the Rankine  $K_a$  for  $30^\circ$  and  $K_a$  for  $36^\circ$ .

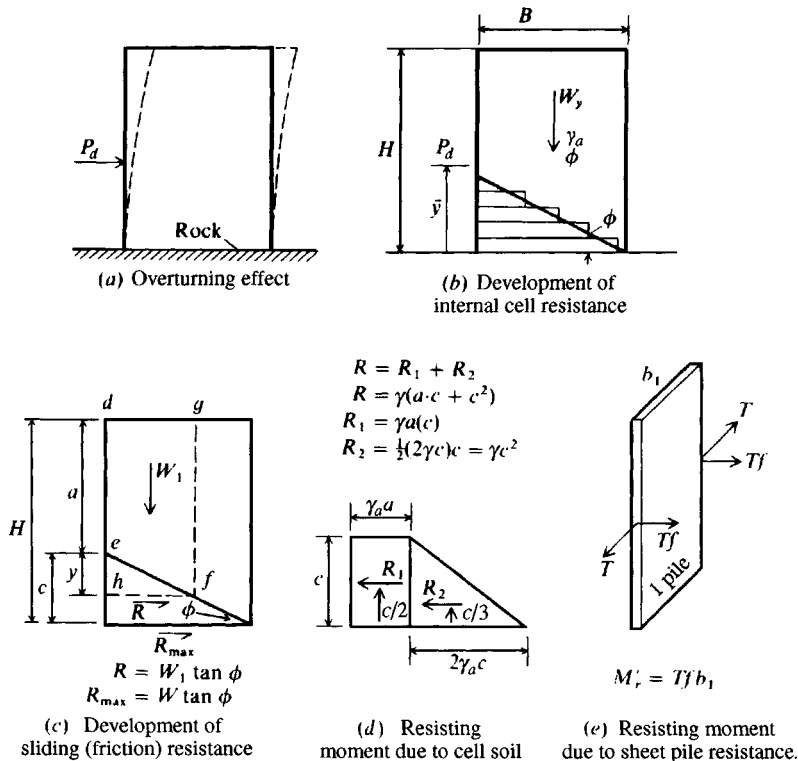
### 15-3.2 The Cummings Method (Currently Used by TVA)

Cummings (1960) proposed a method of analysis of cellular cofferdams based on model studies for the tilting of a cofferdam on rock, as shown in Fig. 15-10. The method provides a simple analysis; however, the models were constructed of relatively stiff material for the size of the model, which may not be realistic when related to the flexible sheetpiling sections and dimensions of a field structure.

According to TVA (1966), they had made some (unpublished) model studies similar (and prior) to Cummings and observed the same type of failures. It remained for Cummings to develop the analytical method presented here. The method has been successfully used in the design of several cofferdams and is extremely simple.

The analysis is based on the premise that the cell soil will resist lateral distortion of the cell through the buildup of soil resistance to sliding on horizontal planes (Fig. 15-10*b*). This resistance will be developed in a triangle as shown, forming an angle of  $\phi$  to the horizontal. The triangle of soil will be in a passive pressure state and stabilized by the overlying soil, which acts as a surcharge. The weight of this soil is termed  $W_y$ . The derivation is complete when we can write an expression for the cell resistance in terms of the triangular zone of passive resistance, with shear on the horizontal planes and including the surcharge effect of  $W_y$ .

**Figure 15-10** Cummings method of cell analysis. [After Cummings (1960).]



Referring to Fig. 15-10c, we see that the weight of soil overlaying the triangle, in zone *defg*, plus the weight of the soil included in the triangle *efh* is

$$W_1 = \gamma_a(a + y)y \cot \phi \quad (a)$$

where  $\gamma_a$  = average effective unit weight of cell soil as a computational convenience; it can have two values if the cell is embedded in the dredge line soil (refer to Example 15-4).

The shear resistance developed by  $W_1$  along the horizontal plane *hf* with  $\delta = \phi$  is

$$R = W_1 \tan \phi = \gamma_a(ay + y^2) \quad (b)$$

The maximum value of  $R$  occurs when  $y$  is a maximum. This occurs when

$$y = c = B \tan \phi \quad (c)$$

The geometry of the problems yields, by inspection,

$$a = H - c \quad (d)$$

Now substituting the values from Eq. (c) for  $y$  and Eq. (d) for  $a$ , and Eq. (a) for  $W_1$  into Eq. (b) and defining  $R_{\max}$  = maximum force, we obtain

$$R_{\max} = \gamma_a B H \tan \phi \quad (e)$$

The force  $R$  of Eq. (b) can be interpreted as consisting of two parts,  $R_1$  and  $R_2$  (see Fig. 15-10d) and from Eq. (b) and using Eq. (c) for  $c = B \tan \phi$  for  $y$  these two forces are

$$R_1 = \gamma_a a \cdot c \quad R_2 = \gamma_a c^2 \quad (f)$$

The force  $R_1$  is taken as the area of a rectangle of height  $c$  and base  $\gamma_a a$ . Force  $R_2$  is the area of a triangle of height  $c$  and base  $2\gamma_a c$ . This concept is used so that resisting moments can be computed for these two forces as

$$M_1 = R_1 \bar{y}_1 = R_1 \frac{c}{2} \quad M_2 = R_2 \bar{y}_2 = R_2 \frac{c}{3} \quad (g)$$

and the total soil resisting moment  $M_r$  is

$$M_r = M_1 + M_2 \quad (h)$$

Rewriting and substituting Eqs.(f) and (g) into Eq. (h), we find the total soil resisting moment is

$$M_r = \gamma_a c^2 \left( \frac{a}{2} + \frac{c}{3} \right) \quad (15-12)$$

The bending resistance of the piles due to interlock effects (Fig. 15-10e) is computed from the bursting pressure

$$T = \frac{1}{2} \gamma_a H^2 K_a r = Pr \quad (i)$$

For a unit strip (or for cell width  $L$ ) the width is number of piles  $n \times$  pile width  $b_1$ , giving the total resisting moment from cell fill on either a unit strip or width  $L$  as

$$M_r'' = \frac{Prf(nb_1)}{r} \quad \text{but} \quad nb_1 = L \quad \text{or} \quad B \rightarrow M_r'' = PfB$$

This expression gives the total resisting moment  $M_{tr}$  from soil and pile as

$$M_{tr} = M_r + M_r''$$

$$M_{tr} = \gamma_a c^2 \left( \frac{a}{2} + \frac{c}{3} \right) + PfB \quad (15-13)$$

The stability number against overturning  $N_{oc}$  is the ratio of the cell resisting moments to the overturning moments

$$N_{oc} = \frac{M_{tr}}{P_d \bar{y}} \rightarrow \frac{M_{tr}}{M_o} \quad (15-14)$$

Stability against sliding in the Cummings method is computed the same as by Eq. (15-2) and for interlock tension by Eqs. (15-10) or (15-11).

## 15-4 BEARING CAPACITY

Bearing capacity does not have to be considered when cofferdams are founded on rock. When cells are based on soil, bearing capacity may be a problem. This can be investigated as follows:

1. Convert the cofferdam to an equivalent rectangle (see Fig. 15-1).
2. Use a unit width for foundation width  $B (= 1)$  and an initial length  $L$  equal to *equivalent rectangle B*.
3. Using the overturning stability computations find the base eccentricity  $e$ .
4. Compute the effective foundation  $L' = L - 2e$ .
5. Use the Hansen bearing-capacity equations with the effective base dimensions of  $B \times L'$  to compute the bearing capacity, and compare to the actual bearing pressure *under the toe half of the base*. Compute depth  $d_i$  and inclination factors  $i_i$ , but note that all  $s_i = 1$ .

What you are doing is computing a bearing capacity that makes some allowance for the increase in soil pressure from the overturning effect of the water.

## 15-5 CELL SETTLEMENT

Cells on rock do not settle. Probably cells on soil do not settle unless the base soil is extremely poor, or the cell is in place so long that consolidation settlements occur.

The apparent settlement, however, can be very large and will be illustrated by the following example.

**Example 15-1.** Estimate the apparent settlement of the cofferdam cell shown in Fig. E15-1. It is assumed that we can somehow measure the increases in cell diameter as given so that we can make an approximate analysis.

**Solution.** From the new diameters compute an average diameter and (as done in unconfined compression tests) assume the volume remains constant. The initial diameter is the as-driven cell diameter of 6.0 m and cell height  $H_c = 6$  m (above dredge line). Compute the new cell diameter (after it expands from dewatering the basin side) as

$$D_{av} = \frac{6.4 + 6.6 + 6.8 + 7.0 + 7.4 + 7.5 + 6.6 + 2 \times 6.0}{9}$$

$$= 60.3/9 = 6.7 \text{ m (average bulged diameter)}$$

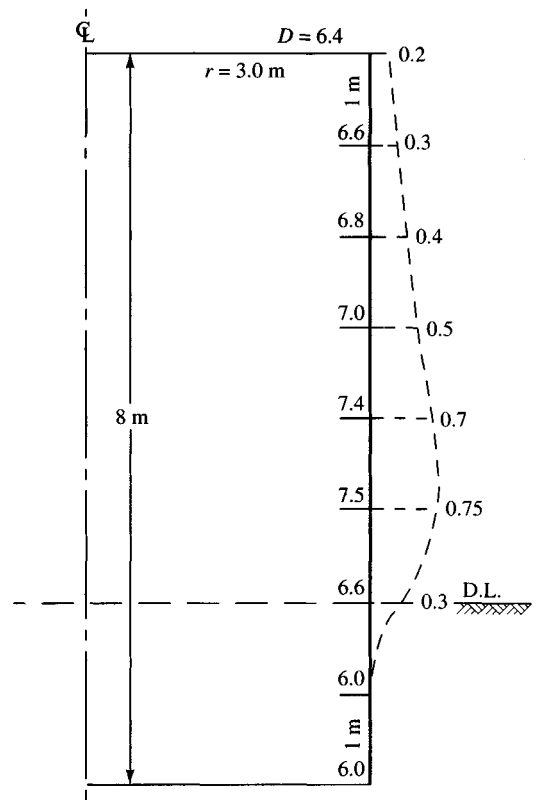


Figure E15-1

The initial cell volume based on  $H = 8$  m is

$$V_i = 0.7854 \times 6^2(8) = 226.2 \text{ m}^3$$

The final volume is assumed to equal  $V_i$  or

$$0.7854 \times 6.7^2 H_f = 226.2$$

$$H_f = \frac{226.2}{0.7854 \times 6.7^2} = 6.42 \text{ m}$$

The apparent settlement is

$$\Delta H = 8.00 - 6.42 = 1.58 \text{ m}$$

Not all of this apparent settlement would occur at once—part would occur during cell filling. It should also be evident that one could go from one cell expansion to another using the same procedure to obtain “settlements” at various stages of dewatering or other activities.

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## 15-6 PRACTICAL CONSIDERATIONS IN CELLULAR COFFERDAM DESIGN

The 10-mm (0.40-in., and formerly  $\frac{3}{8}$ -in.) web sheetpiling is widely used for cofferdam design, providing a guaranteed tension of 2800 kN/m (16 kips/in.) and an interlock stress of 280 MPa (40 ksi)—approximately  $f_y$  for A328 steel. Using a nominal SF = 1.5 on the

interlock tension, we obtain  $2800/1.5 = 1870$  kN/m and the corresponding web stress  $= T/t_w = 1870/0.01 = 187\,000$  kPa  $= 187$  MPa. The latter is close to the value of  $f_a = 0.65 f_y$  given in the heading of Table 15-1.

For substantial embedment depths, such as for cellular cofferdams not on rock, it may be necessary to increase the web thickness to 12.7 mm. It is not usually recommended to drive sheetpiling much over 3 to 5 m (10 to 15 ft) with 6 m as an upper practical limit owing to driving damage since soil in river beds at these depths usually becomes sandy and dense so that driving becomes difficult. It is usually desirable to excavate 1 to 2 m of overburden to remove surface debris such as stumps, logs, tires, etc., which may damage the sheetpiling if large embedment depths are necessary.

Secondhand sheetpiling is widely used. It may be reused as many as four times, which represents about 25 percent loss from each use. It is for this reason that former as well as current designations for sheet-pile sections are given in Table 15-1. A major consideration with used sheetpiling is damage to the thumb and finger elements that produce the interlock groove. It is absolutely essential that the interlock be correctly done as illustrated in Fig. 15-4a (dashed), for a thumb reversal greatly reduces the interlock tension and may open up the groove. Other damage may also occur from rust, wedging a stone, kinking the pile, hard driving, and so forth.

For important projects it may be advisable to require either new piling or that the contractor be responsible for any cofferdam failure that arises from driving used piling.

Cofferdam dewatering is necessary to reduce the hoop tension stresses—which usually control the design—and it is standard practice to burn holes of about 35- to 50-mm diameter through several of the cell piles in each cell on the basin side. Practice is to burn the *weep* holes at about 1.5- to 2-m centers vertically on every third to sixth sheet pile (weep holes on the same pile result in maximum salvage of piling). Holes are made to the top of the berm or to the inside ground surface if no berm is used. During dewatering operations it is necessary that the drain or weep holes be systematically rodded to maintain drainage.

It appears that one cannot rely on drainage through the interlocks to dewater the cells adequately; the interlocks tend to “silt up” during the cell-filling operation. They also tighten when the cell bulges, which also reduces water flow. If the dewatering is carefully done and the cell fill is free-draining, TVA experience (1966, p. 118) indicates that it is satisfactory to assume a horizontal saturation line at one-half the free interior cell height.

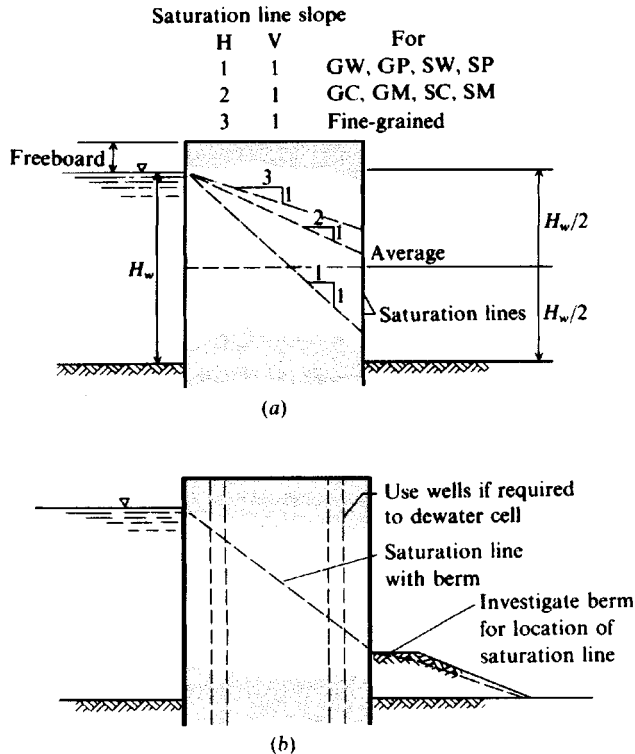
Assuming a horizontal saturation line location greatly simplifies the design computations (Fig. 15-11a). Wells may be installed in the cells adjacent to the dry-side sheeting to collect water, which can then drain through any nearby weep holes. Alternatively, wells of 200- to 300-mm diameter (Fig. 15-9b) may be installed on both the river and basin side of the cell together with well pumps to aid in dewatering and to depress the saturation line further. Pump capacities on the order of 10 to 40 gal/min—depending on the drainage characteristics of the cell fill—are commonly required.

If the cells are located in very poor soils it may be possible to use stone columns (described in Sec. 6-8) to stabilize both the base and cell soil.

If cells are high, it may be possible to drive soldier piles of the type shown in Fig. 13-6b on a spacing to accommodate anywhere from three to six straight web piles. The cell template would have to be redesigned to accommodate the **H** pile sections.

Cell bulge may be avoided by cutting undersized circles of geogrids and using special lowering devices to position a layer until it is well-covered with fill, raising the positioning device to a higher level, installing a second geogrid, . . . , etc. If the grid openings are properly aligned, a Terra-probe or concrete vibrator may be used to densify the soil.





**Figure 15-11** Location of the saturation (phreatic) line inside cofferdams. The average saturation line in (a) is probably as accurate as any, however, the alternative saturation line slopes (use an average for them also) may be required in some designs. Cell wells as in (b) may lower the saturation line close to the cell base if properly constructed and in sufficient number.

It was previously discussed that an inside berm may be used to obtain additional sliding resistance. It may also be used to increase the length of flow path to reduce the possibility of piping or excessive flow beneath the cofferdam. In this case, however, it is usually preferable to put wells in the cell and pump the head down so that the flow through and beneath the cofferdam is acceptable.

Cofferdams are built with a freeboard depth on the order of 1.5 to perhaps 2 m (Fig. 15-11a), usually based on a 5-year design period. It is usually considered more economical to allow for overtopping during an extreme flood than to design for a longer flood period (10 to 50 years).

Inclusion of overtopping considerations can quickly change design parameters unless provision is made for rapid flooding of the basin. Also overtopping can result in severe erosion of the cell fill. The cell fill may be capped with a lean (14–21 MPa) concrete mix (on the order of 150 to 200 mm thick) or with an asphalt mix. Since cell fill settlement/subsidence can approach 600 mm (or more) asphalt, being flexible, may be a better capping material than concrete to control both cracking and surface water infiltration.

Even if overtopping does not occur, the river bed tends to *scour*, or erode, during floods. If this occurs beneath the cell piling on the river side, the cell(s) tip into the river. Some means to monitor toe scour should be provided because equipment is often stored on top of the cells and could be lost if the cells tip over.

The cofferdam often will carry construction equipment such as cranes and otherwise be surcharged with stockpiles of cell fill, sheetpiling for later cells, etc. Very large surcharges should be considered in the cell design.

Most reported cofferdam failures appear due to failure of the connection element (tee or wye) between the main cell and the connecting arc. Some of the earlier failures were from using welded connections that fractured. Grayman (1970) summarized a number of cofferdam failures as to cause, and only one failure was attributed to sliding and one to overturning.

Finally, the cells cannot be aligned during driving to much more than about 150 mm owing to the flexible sheeting involved and to the problems in driving and the process of filling and dewatering. Later cell distortions may produce vertical settlement/subsidence already noted of 300 to 1000 mm or more. The system always moves into the basin some amount ranging from almost zero to perhaps 150 mm at the base and from 75 to perhaps 300+ mm at the top. Noticeable bulges nearly always develop in the cell—probably all around but visible on the basin (or cofferdam side). All of these happenings are considered acceptable practice.

**COMPUTER PROGRAMMING.** It should be evident that there is a substantial amount of busywork involved with a cofferdam design. Thus, there is a high possibility of errors, so a computer program should be used for the analysis if one is available. Program *COFERDAM* is one such program. These types of programs generally require an interactive mode since there are a number of options such as absence or presence of a berm, water height in berm, location of saturation line, use of passive and/or active pressures, and so on.

## 15-7 DESIGN OF DIAPHRAGM COFFERDAM CELL

This section will consider the design of a diaphragm cell. Both design and required as-built values and volume of cell fill are examined.

**Example 15-2.** Design a diaphragm cofferdam cell. Assume the cell saturation line to be at one-half the cell height from the rock base to the top. This assumption allows for a small flood rise of 0.6 m (0.6 m = freeboard) at incipient overtopping as shown in Fig. E15-2a. The soil data are also shown in this figure, along with select initial dimensions and depth of dredge line soil to the rock base.

Other data: Use either PS27.5 or PS31 piling of grade A 328.

$$f_y = 270 \text{ MPa} \quad (\text{if possible})$$

$$\text{Interlock tension } T_i = 2800 \text{ kN/m}$$

$$\text{SF} = 2.0 \text{ (Table 15-1)}$$

$$\text{Interlock friction } f_i = 0.3$$

$$\text{Friction of pile-soil } f_s = 0.4 \text{ (Table 11-6)}$$

### **Solution.**

**Step 1.** Find a width  $B$  of Fig. 15-1b to satisfy sliding stability.

There will be a water force based on the cell height and, since the embedment depth is 5.5 m, a small active earth-pressure force. The water has  $K_a = 1.0$ ; for a soil of  $\phi = 30^\circ$  the Rankine  $K_a = 0.333$  (Table 11-3). The lateral forces are as follows:

$$P_w = \frac{1}{2} \gamma_w H_c^2 (1) = \frac{1}{2} \times 9.807 \times 15.25^2 = \mathbf{1140.4 \text{ kN}}$$

$$\bar{y}_w = 15.25/3 = 5.08 \text{ m} \quad \text{above base}$$

$$P_a = \frac{1}{2} \gamma'_s H_s^2 (0.333) = \frac{1}{2} \times 7.5 \times 5.5^2 \times 0.333 = \mathbf{37.8 \text{ kN}}$$

$$\bar{y}_s = 5.5/3 = 1.83 \text{ m} \quad \text{above base}$$

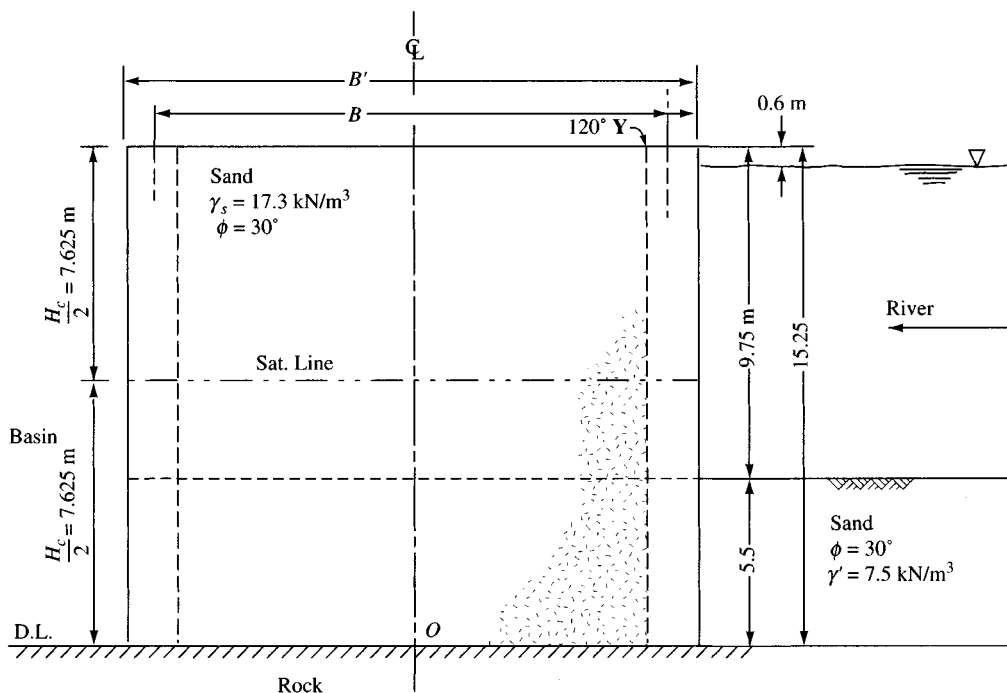


Figure E15-2a Diaphragm Cell.

With the saturation line at one-half the cell height, or  $15.25/2 = 7.625$  m, the weight  $W$  of a strip 1 unit (1 m) wide in terms of cell width  $B$  is

$$\begin{aligned} W &= B\gamma_s \times 15.25/2 + B\gamma'_s \times 15.25/2 \\ &= B(17.3 + 7.5) \times 7.625 = \mathbf{189.1B \text{ kN}} \end{aligned}$$

The sliding resistance is  $W \tan \delta \rightarrow W \tan \phi$ , giving

$$F_{sr}(189.1B \tan 30^\circ) = 189.1B \times 0.577 = \mathbf{109.1B}$$

For a sliding stability number  $N_s = 1.25$ , we obtain the effective cell width

$$\begin{aligned} 109.1B &= \text{SF}(P_w + P_a) \\ B &= \frac{1.25(1140.4 + 37.8)}{109.1} = \mathbf{13.50 \text{ m}} \end{aligned}$$

**Step 2.** Find the width  $B$  necessary for overturning stability. Take moments about cell base at point  $O$ :

$$M_o = P_w \bar{y}_w + P_a \bar{y}_s = 1140.4 \times 5.08 + 37.8 \times 1.83 = \mathbf{5862.4 \text{ kN} \cdot \text{m}}$$

We will arbitrarily keep the base eccentricity within the middle one-third, giving

$$e = \frac{B}{6} \quad \text{and} \quad We = M_o \times \text{SF}$$

With  $W = 109.1B$  and  $\text{SF} = 1.25$ , we obtain on substitution into the foregoing

$$\begin{aligned} 189.1B \cdot B/6 &= 5862.4 \times 1.25 \\ B^2 &= \frac{6 \times 5862.4 \times 1.25}{189.1} \\ B &= \sqrt{232.5} = \mathbf{15.25 \text{ m} > 13.50} \end{aligned}$$

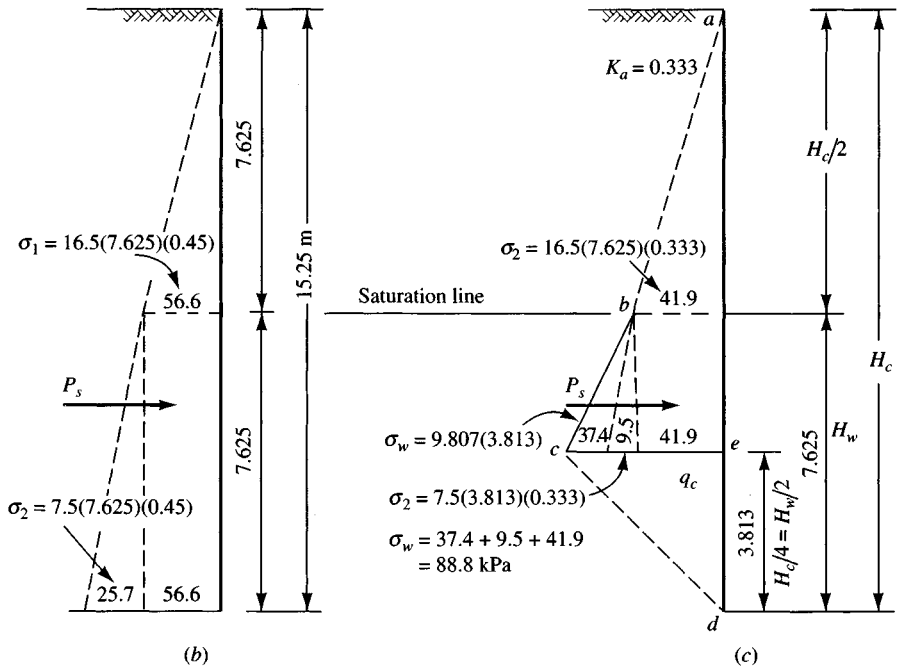


Figure E15-2b, c

Checking the overturning with friction on heel, we have  $P_w = 1140.4$  kN,  $P_a = 37.8$  kN, and  $f = 0.40$ , giving

$$B(P_w + P_a)f_s = M_o \times SF$$

$$B = \frac{5862.4 \times 1.25}{1178.2 \times 0.40} = 15.55 \text{ m} > 15.29 \quad \text{Use } B = 15.55 \text{ m}$$

**Step 3.** Check centerline shear. For this we need to refer to Fig. E15-2b and c, which gives the necessary pressure profiles for this check. We will assume  $r = L$  (see Fig. 15-1b).

The pressure profiles show the necessary computations using the method given in Chap. 11 for pressure at critical depths. Note, however, that Fig. E15-2b uses a lateral earth-pressure coefficient  $K'$ . Equation (15-6) gives  $K' = 0.60$  for  $\phi = 30^\circ$  [see table following Eq. 15-6]; the author suggested 0.45. The average of  $K'$  and  $K_a \approx 0.45$  (a coincidence for  $\phi = 30^\circ$ ). We will use  $K' = 0.45$  to compute the pressures in Fig. E15-2b (but  $K_a = 0.333$  for Fig. E15-2c).

The centerline shear is computed using Eq. (15-4) and  $M_o$  from step two as

$$V = 1.5M_o/B = 1.5(5862.4)/15.55 = 565.5 \text{ kN}$$

The resisting shear is made up of  $P_s \tan \phi + R_{il}$ . From Fig. E15-2b we compute  $P_s$  as follows:

$$P_s = 56.6 \frac{7.625}{2} + (56.6 + \frac{25.7}{2}) \times 7.625$$

$$= 215.8 + 529.6 = 745.4 \text{ kN/m}$$

$$V_s = P_s \tan \phi = 745.4 \times 0.577 = 430.1 \text{ kN/m}$$

For the interlock resistance  $R_{il}$  we use Fig. E15-2c and compute the area  $abcd = P_t$  so we can use Eq. (15-5b) [see after Eq. (15-7)]:

$$P_t = \sigma_1 \frac{7.625}{2} + \frac{\sigma_t + \sigma_1}{2} \frac{7.625}{2} + \sigma_t \frac{3.813}{2}$$

Substituting values, we obtain

$$\begin{aligned} P_t &= 41.9 \times 3.813 + \frac{(88.8 + 41.9)}{2} \times 3.813 + 88.8 \times 1.91 \\ &= 159.8 + 249.2 + 169.3 = 578.3 \text{ kN} \end{aligned}$$

and using Eq. (15-5b), we have

$$\begin{aligned} R_{il} &= P_t f_i = 578.5 \times 0.3 = \mathbf{173.6 \text{ kN}} \\ V_r &= V_s + R_{il} = 430.1 + 173.6 = \mathbf{603.6 \text{ kN}} > 565.5 \end{aligned}$$

The resulting SF =  $603.6/565.5 = 1.07 < 1.25$

Noting the sliding and overturning stability numbers are satisfactory for the  $B$  value being used, we see that any larger  $B$  will only increase those stability numbers. Let us increase  $B$  so the cell shear stability is at least 1.25. We can do this by increasing  $B$  as follows:

$$\frac{1.5M_o}{B} = \frac{V_r}{\text{SF}} \rightarrow B = \frac{1.5 \times 5862.4 \times 1.25}{603.6} = \mathbf{18.2 \text{ m}} > 15.55$$

**Step 4.** Check interlock tension using  $\sigma_t$  of Fig. E15-2c and Eq. (15-10). We do not need to check Eq. (15-11) since a  $120^\circ$  Y in a diaphragm cell produces the same interlock tension in any part of the cell.

Using Eq. (15-10) we will back-compute to find a suitable wall spacing  $r = L$ ,

$$t_i = \frac{\sigma_t r}{C} = \sigma_t r \leq \frac{2800}{\text{SF}}$$

Substituting values, we obtain ( $\sigma_t = 88.8 \text{ kPa}$  on Fig. E15-2c)

$$88.8r = 2800/2 \rightarrow r = 1600/88.8 = 15.8 \text{ m}$$

We will arbitrarily reduce this value and use  $r = L = \mathbf{15.0 \text{ m}}$ .

We now have design dimensions for this cell as follows:

$$B = 18.2 \text{ m} \quad r = L = 15.0 \text{ m} \quad \text{Cell height } H_c = 15.25 \text{ m}$$

**Step 5.** Compute the required number of piles and final cell dimensions (we cannot use fractions of piles, and we must use what is available both for piles and the Y piece). From Fig. 13-4d the legs of a typical Y = 260.4 mm  $\rightarrow$  **0.260 m**. The central angle of all diaphragm cells is  $60^\circ = 1.047$  radians. Both PS27.5 and PS31 piles have a driving distance (width)  $b_p = 500 \text{ mm}$  (0.50 m).

a. Plot the computed dimensions to a large scale (as in Fig. E15-2d) and scale the wall length  $\approx 17.3 \text{ m}$ . This is reduced by two Y legs of 0.260 m

$$\begin{aligned} \text{No. of piles} &= [17.3 - 2(0.260)]/0.5 = 16.78/0.5 = \mathbf{34 \text{ piles}} \\ \text{Side wall } L_w &= 34 \times 0.5 + 0.52 = \mathbf{17.52 \text{ m}} \quad (\text{actual distance}) \end{aligned}$$

b. Get piles in the arc. The initial arc length is

$$\begin{aligned} L_{\text{arc}} &= r\theta = 15.0(1.047) = 15.70 \text{ m} \\ \text{No. of piles} &= (15.70 - 0.52)/0.5 = 30.36 \quad \text{use } \mathbf{31 \text{ piles}} \\ \text{Actual } L_{\text{arc}} &= 31 \times 0.5 + 0.52 = 16.02 \text{ m} \\ \text{Actual } r &= L_{\text{arc}}/\theta = 16.02/1.047 = \mathbf{15.3 \text{ m}} \end{aligned}$$

c. Actual effective  $B$  (refer to Fig. 15-1b) is approximately

$$\begin{aligned} B &= A + 1.820r \\ L_w &= A + 1.732r = 17.52 \text{ m} \end{aligned}$$

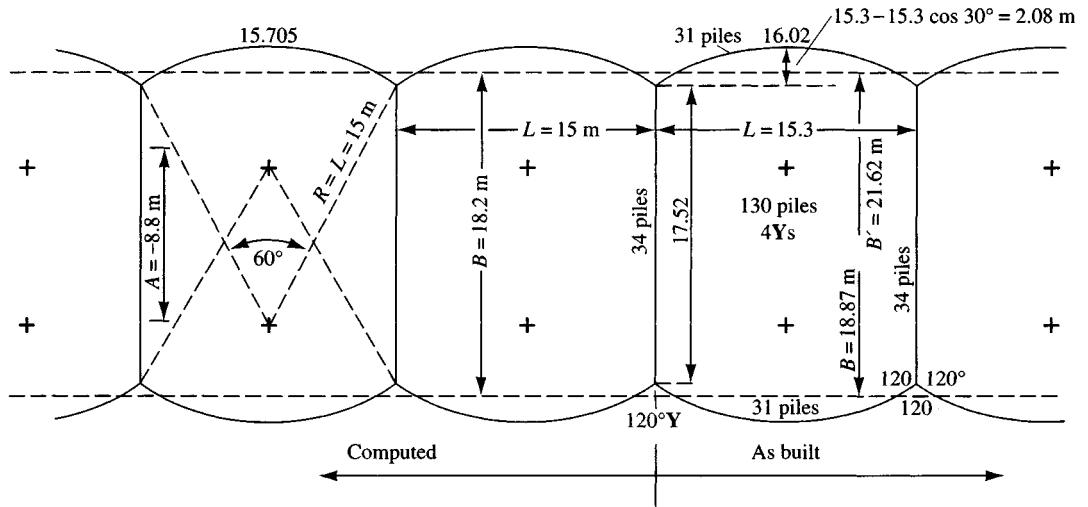


Figure E15-2d

Eliminating  $A$ , we obtain  $0.088r = B - 17.52$ . Solving, we see that

$$B = 17.52 + 0.088(15.3) = 18.87 \text{ m}$$

Since this is larger than the last computed  $B = 18.82$  it appears that the computations are satisfactory. The full cell width

$$\begin{aligned} B' &= L_w + 2(r - r \cos 30^\circ) \\ &= 17.52 + 2(15.3 - 15.3 \cos 30^\circ) \\ &= 17.52 + 4.10 = 21.62 \text{ m} \end{aligned}$$

d. The number of piles/cell is based on 1 side wall + 2 end arcs, giving

$$\begin{aligned} \text{Side wall} &= 34 \text{ piles} \\ 2 \text{ end arcs} &= 2 \times 31 = 62 \text{ piles} \\ \text{Total} &= 96 \text{ piles} + \text{two } 120^\circ \text{Ys} \end{aligned}$$

e. The approximate cell fill volume above the dredge line is

$$V_{\text{fill}} = BrH = 18.87 \times 15.3 \times 9.75 \approx 2815 \text{ m}^3$$

////

## 15-8 CIRCULAR COFFERDAM DESIGN

This section considers the design of a circular cell cofferdam on a soil base using the TVA method. The following example will illustrate both the current TVA and the Cumming's methods for analysis.

**Example 15-3.** Design a circular cofferdam cell resting on a riverbed sand stratum approximately 25 m thick using the current TVA method. Other data are as follows (refer also to Fig. E15-3a):

$$\begin{aligned} \text{Cell fill: } \gamma_{\text{wet}} &= 17.0 \text{ kN/m}^3 \text{ (cell fill from river bottom)} \\ \gamma' &= 9.0 \text{ kN/m}^3 \end{aligned}$$

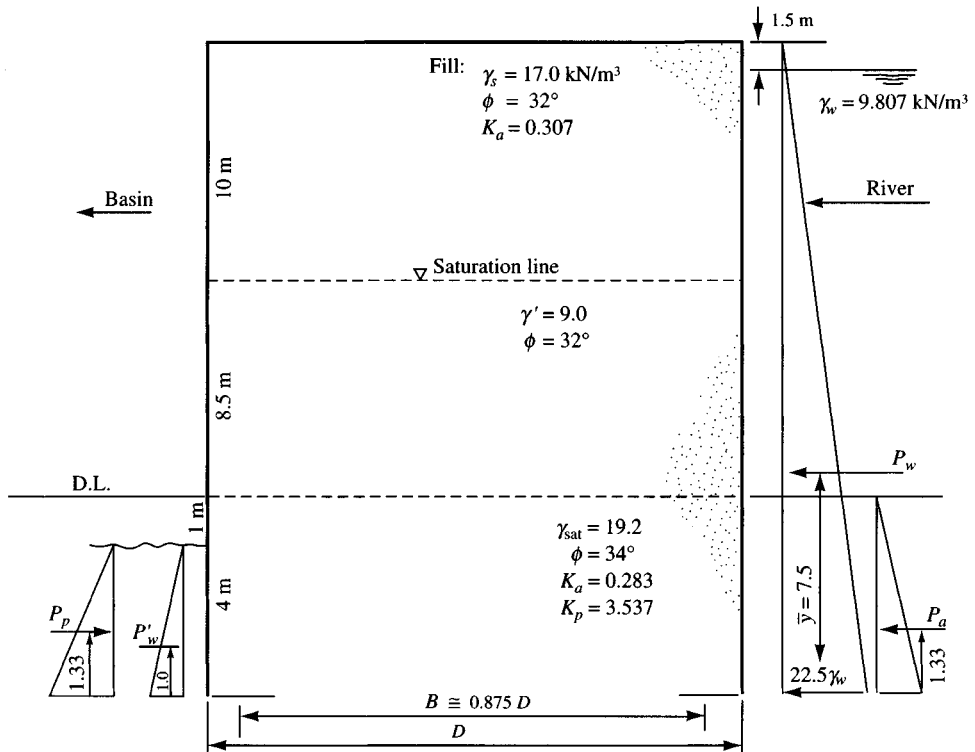


Figure E15-3a

Saturation line at  $H_c/2$  (free-draining).

$\tan \delta = \tan 32^\circ = 0.40$  (fill-to-pile)

Base soil:  $\gamma_{\text{sat}} = 19.2 \text{ kN/m}^3$   $\phi = 34^\circ$   
 Pile data: Use **PS27.5** or (if required) **PS31** piling

Interlock tension 2800 or 4900 kN/m (if required)

Interlock friction  $f_i = 0.3$

Use cell  $\alpha = 45^\circ$  (see Fig. 15-1a with  $B \approx 0.875D$ )

All SF > 1.25. Neglect dynamic force of river flow.

**Solution.** (To minimize errors, I used the program COFERDAM to check the following computations.)

**Step 1.** Compute the driving forces and the overturning moment. These several forces and  $\bar{y}$  locations, shown on Fig. E15-3a, are computed using methods given in Chap. 13:

$$P_w = \frac{1}{2} \gamma_w H^2 = \frac{1}{2} \times 9.807 \times 22.5^2 = 2482.4 \text{ kN/m} \leftarrow$$

$$P'_w = \frac{1}{2} \times 9.807 \times 3^2 = 44.1 \text{ kN/m} \rightarrow$$

The soil below the dredge line has  $\gamma' = 19.2 - 9.807 = 9.4 \text{ kN/m}^3$ , and for  $\phi = 34^\circ$  we can look up  $K_i$  values from Tables 11-3 and 4:

$$P_a = \frac{1}{2} \gamma' H^2 K_a = \frac{1}{2} \times 9.4 \times 4^2 \times 0.283 = 21.3 \text{ kN/m} \leftarrow$$

$$P_p = \frac{1}{2} \times 9.4 \times 4^2 \times 3.537 = 266.0 \text{ kN/m} \rightarrow$$

Referring to directions of the arrows and  $(+) = \leftarrow$ , we see that the net force is

$$P_{\text{net}} = 2482.4 + 21.3 - 44.1 - 266.0 = \mathbf{2193.6 \text{ kN/m} \leftarrow}$$

The net overturning moment  $M_o$  is computed using the foregoing forces with their  $\bar{y}$  values, giving

$$\begin{aligned} M_o &= 2482.4 \times 7.5 + 21.3 \times 1.33 - 44.1 \times 1.0 - 266.0 \times 1.33 \\ &= \mathbf{18\,248.4 \text{ kN} \cdot \text{m/m}} \quad (\text{counterclockwise } \curvearrowright) \end{aligned}$$

**Step 2.** Cell centerline shear usually controls, so we will compute the  $B$  required for this (keeping the eccentricity in the middle one-third) and then check overturning and sliding stability:

$$V = 1.5M_o/B \quad [\text{Eq. (15-4)}]$$

$$V_s = P_s \tan \delta \quad [\text{Eq. (15-5a)}]$$

$$V = V_s + R_{il} \quad [\text{Eq. (15-5)}]$$

$$R_{il} = P_t f_i \quad [\text{Eq. (15-5b)}]$$

Obtain  $P_s$  from the pressure profile shown in Fig. E15-3b and compute  $R_{il}$  using either  $P_t$  from Fig. E15-3c or  $P'_t$  from Fig. E15-3d. Here we will compute both values of  $P_t$  and use the smaller.

For  $P_s$  it is necessary to compute a value of  $K'$  and, based on its value, make a selection in the range of 0.45 to 1.0. Using Eq. (15-6), we obtain

$$K' = \frac{\cos^2 32^\circ}{2 - \cos^2 32^\circ} = \frac{0.719}{2 - 0.719} = 0.561$$

We will arbitrarily use  $K' = 0.60$ . With this the pressure profile of Fig. E15-3b is drawn. Select lateral pressure computations are on the diagram. Compute  $P_s$  as

$$\begin{aligned} P_s &= 102.0 \times \frac{10}{2} + \frac{(102.0 + 102.0 + 45.9)}{2} \times 8.5 \\ &= 510.0 + 1062.1 = 1572.1 \text{ kN/m} \\ V_s &= P_s \tan 32^\circ = 1572.1 \tan 32^\circ = \mathbf{982.3 \text{ kN/m}} \quad [\text{Eq. (15-5a)}] \end{aligned}$$

The value of  $P_t = \mathbf{1110.2 \text{ kN/m}}$  is shown on Fig. E15-3c, and you should be able to compute this using the pressure profile given. Let us look at Fig. E15-3d. This profile uses the depth of fixity suggested by Maitland and Shroeder (1979), as modified by the author.

The lateral pressure  $p_a$  is at the dredge line  $+dz$  where  $\phi = 34^\circ$  so  $K_a = 0.283$  and

$$\begin{aligned} p_a &= (\gamma_s \times 10 + \gamma' \times 8.5)0.283 \\ &= (17.0 \times 10 + 9.0 \times 8.5)0.283 = 69.8 \text{ kPa} \end{aligned}$$

Summing pressures, we find

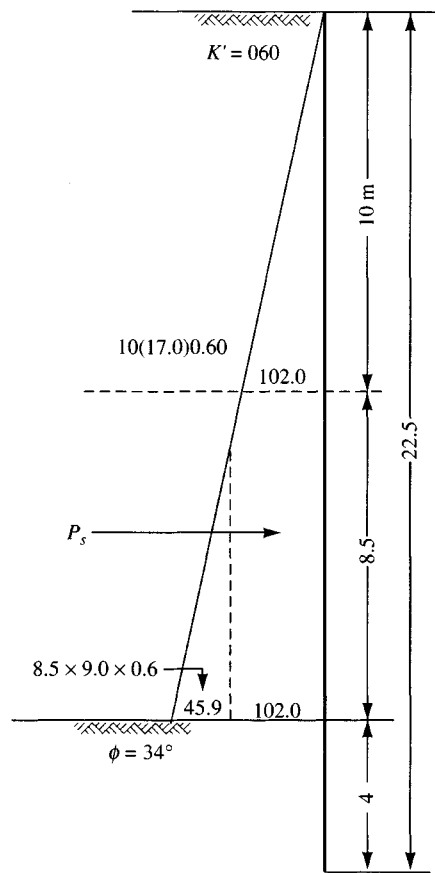
$$d_f \gamma' K_p - d_f \gamma' K_a = p_a \quad [\text{Eq. (15-9) slightly rearranged}]$$

$$d_f = \frac{p_a}{\gamma'(K_p - K_a)} = \frac{69.8}{9.4(3.537 - 0.283)} = 2.3 \text{ m}$$

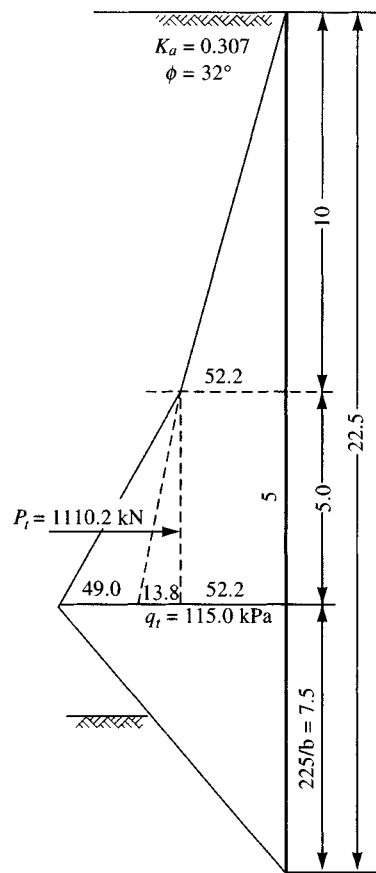
The total effective pile depth  $H_1 = 10 + 8.5 + 2.3 = 20.8 \text{ m}$ . The maximum stress is assumed to act at  $20.8/3 = 6.9 \text{ m}$  above this point, giving the dimensions and stresses shown on Fig. E15-3d. From the stresses and dimensions compute  $P'_t$  as

$$\begin{aligned} P'_t &= 52.2 \times 5 + \frac{(52.2 + 101.2)}{2} \times 3.9 + 101.2 \times \frac{6.9}{2} \\ &= 261.0 + 299.1 + 349.1 = 909.2 \text{ kN/m} \\ R_{il} &= P'_t f_i = 909.2 \times 0.30 = \mathbf{272.8 \text{ kN/m}} \\ V_r &= V_s + R_{il} = 982.3 + 272.8 = \mathbf{1255.1 \text{ kN/m}} \end{aligned}$$

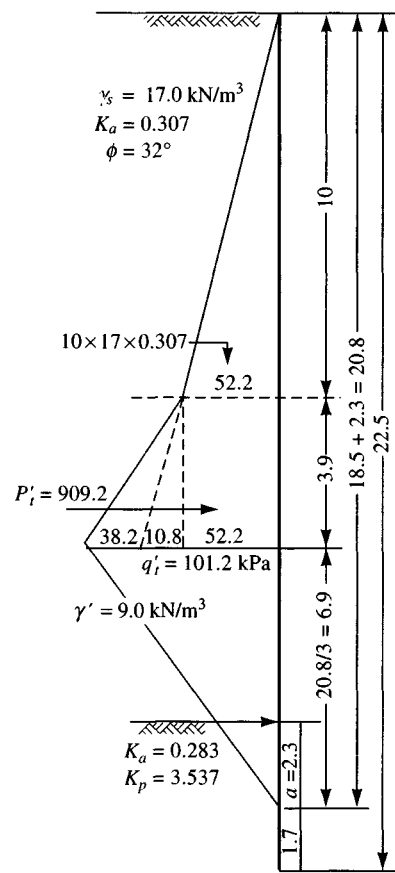




(b)



(c) TVA.



(d) Alternative.

Figure E15-3b, c, d

We can now compute the average cell width  $B$ :

$$V = \frac{1.5M_o}{B} \leq V_r$$

Replacing the  $\leq$  with an equal sign and introducing the  $SF = 1.25$ , we obtain

$$B = \frac{1.5M_oSF}{V_r} = \frac{1.5 \times 18 \times 248.4 \times 1.25}{1255.1} = 27.3 \text{ m}$$

**Step 3.** Check sliding stability. The weight  $W$  of a unit width slice  $\times B$  is

$$W = (10 \times 17.0 + 8.5 \times 9.0 + 4 \times 9.4)B = 284.1 \text{ kN/m}$$

The net driving force tending to slide the cell into the basin was computed earlier as  $P_d = 2193.6$  kN/m. The resulting stability number  $N_s$  when we insert  $B = 27.3$  m (just computed) is

$$N_s = \frac{P_R}{P_d} = \frac{W \tan \phi}{2193.6} = \frac{284.1 \times 27.3 \times \tan 34^\circ}{2193.6} = 2.38 > 1.25 \quad (\text{O.K.})$$

**Step 4.** Check the interlock tension both in the cell piles and at the Ts or Ys. For the cell piles use Eq. (15-10) with  $C = 1$ , giving

$$t_i = q_t r$$

(Obtain  $q_t = 101.2$  from Fig. E15-3d.) The diameter  $D \approx B/0.875 = 27.3/0.875 = 31.2$  m. Thus,

$$r = D/2 = 31.2/2 = 15.6 \text{ m}$$

Substitution into Eq. (15-10) now gives

$$t_i = 101.2 \times 15.6 = 1578.7 < 2800/2 \quad (\text{O.K.})$$

For Eq. (15-11) we need a value  $L$  shown on Fig. 15-1a:  $2L = 2.25r$ .

$$L' = 1.125r = 1.125 \times 15.6 = 17.55 \text{ m}$$

Substitution into Eq. (15-11) with  $L = L'$  gives

$$T_{iL} = q_t L / \cos \alpha = 101.2 \times 17.55 / \cos 45^\circ = 2511 \text{ kN/m} > 2800/2$$

We might be able to use  $4900/2 = 2400$  kN/m interlock. If we use  $\alpha = 30^\circ$ ,  $T_{iL} = 2051$  kN/m.

This result is acceptable using high-strength interlocks. Alternatively, we could use a berm or install wells and lower the saturation line to near the inside dredge line. Depending on the number of cells it may be most economical to pay a premium for high-strength interlocks. These are only needed for the  $30^\circ$  Y pieces at four per cell.

Check the web tension based on using the PS31 pile with  $t_w = 12.7$  mm (0.0127 m). Then

$$f_t = 101.2 \times 15.6 / 12.7 = 124.3 \text{ MPa} \ll 0.65 f_y \text{ of A328 steel}$$

**Step 5.** Check the bearing capacity. For the base soil  $\gamma' = 9.4$ ;  $\phi = 34^\circ$ ; from Table 4-4  $N_q = 29.4$ ;  $N_\gamma = 28.7$ ; depth factor = 0.262. Also  $H = P_d = 2194$  kN;  $V = 284.1 \times 27.3 = 7756$  kN.

The base eccentricity  $e$  is

$$We = M_o \rightarrow e = 18 \times 248.4 / (284.1 \times 27.3) = 2.35 \text{ m}$$

$$B' = B - 2 \times 2.35 = 22.6 \text{ m} \quad L = 1 \text{ m}$$

$$d_q = 1 + 0.262D/B = 1 + 0.262(4/22.6) = 1.05$$

$$i_q = \left(1 - \sqrt{\frac{0.5H}{V}}\right)^{2.5} = \left(1 - \sqrt{\frac{0.5 \times 2194}{7756}}\right)^{2.5} = 0.683$$

$$i_\gamma = \left(1 - \sqrt{\frac{0.7H}{V}}\right)^{3.5} = \left(1 - \sqrt{\frac{0.7 \times 2194}{7756}}\right)^{3.5} = 0.462$$

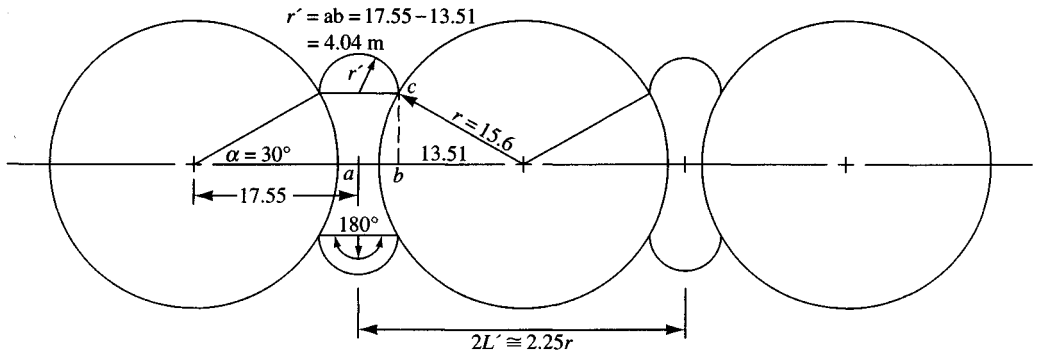


Figure E15-3e

The ultimate bearing capacity (cohesionless soil) is

$$\begin{aligned}
 q_{ult} &= \bar{q}N_q d_{q i_q} + \frac{1}{2} \gamma' B N_\gamma i_\gamma \\
 &= 4 \times 9.4 \times 29.4 \times 1.05 \times 0.683 + \frac{1}{2} \times 9.4 \times 1 \times 28.7 \times 0.462 \\
 &= 792.8 + 62.3 = 855.1 \text{ kPa}
 \end{aligned}$$

The bearing stability requires the actual bearing pressure computed as  $q = 10 \times 17.0 + 8.5 \times 9.0 + 4 \times 9.4 = 284.1 \text{ kPa}$ . Therefore,

$$N_b = \frac{q_{ult}}{q} = \frac{855.1}{284.1} = 3.0 > 2.0 \quad (\text{O.K.})$$

#### Summary.

Cell  $D = 31.2 \text{ m}$        $B = 27.3 \text{ m}$

Pile interlocks O.K.

Connection interlocks O.K. using  $30^\circ$  high-strength Ys

Use  $\alpha = 30^\circ$

Bearing capacity O.K.

**Actual pile cell data.** (refer to Fig. E15-3e for necessary geometric constructions in order to compute connecting arc data)

Cell:

$$\text{Circum} = \pi D = \pi \times 31.2 = 98.01 \text{ m}$$

$$N_{\text{piles}} = 98.01/0.5 = 196.03 \rightarrow \text{Use } \mathbf{196} \text{ piles}$$

Note the  $30^\circ$  Y has same length as pile.

Connecting arc:

$$r' = \text{distance } ab = L' - r \cos 30^\circ$$

$$= 17.55 - 15.6 \cos 30^\circ = 4.04 \text{ m}$$

$$\text{Arc length} = r' \theta \quad \text{but } \theta = 180^\circ = \pi \text{ radians}$$

$$= 4.04\pi = 12.69 \text{ m (total length cell-to-cell)}$$

The Y legs = 0.165 m each and there are two, giving 0.33 m.

$$N'_{\text{piles}} = \frac{12.69 - 0.33}{0.50} = 25.05 \rightarrow \text{Use } \mathbf{25} \text{ piles per arc}$$

Total piles:

$$\text{Cell} = 192 + \text{four } 30^\circ \text{ Ys}$$

$$2 \text{ arcs} = 2 \times 25 = 50$$

$$\text{Total piles} = 242 + \text{four } 30^\circ \text{ Ys}$$

////

**Example 15-4.** Use the data of Example 15-3 to analyze the cell shear stability by the Cummings method.

Fill: $\gamma_s = 17.0 \text{ kN/m}^3$	Base soil: $\gamma_{\text{sat}} = 19.2 \text{ kN/m}^3$
$\gamma' = 9.0 \text{ kN/m}^3$	$\gamma' = 9.4 \text{ kN/m}^3$
$\phi = 32^\circ$	$\phi = 34^\circ$

**Solution.** Refer to Fig. E15-4 (drawn from final dimensions of Example 15-3). Note the sloping line  $ef$  of Fig. 15-10c is broken here to account for two different  $\phi$ -angles as line  $BCF$ . Do not use the depth of fixity concept, as that was not a part of the Cummings method.

Compute the several distances:

$$BI = \frac{4.0}{\tan 34^\circ} = 5.9 \text{ m} \qquad IJ = \frac{8.5}{\tan 32^\circ} = 13.6 \text{ m}$$

$$KL = 27.3 - 5.9 - 13.6 = 7.8 \text{ m} \qquad FL = 7.8 \tan 32^\circ = 4.9 \text{ m}$$

**Step 1.** Compute resistance of  $DCGE$  [use Eqs. (c) and (d)]:

$$c = CG \tan 32^\circ = (27.3 - 5.9) \tan 32^\circ = 13.4 \text{ m}$$

$$a = FE = 18.5 - 13.4 = 5.1 \text{ m}$$

Find the average unit weight  $\gamma_a$  of soil in the cell above dredge line:

$$\gamma_a H = W \rightarrow \gamma_a = \frac{10 \times 17.0 + 8.5 \times 9.0}{18.5} = \frac{246.5}{18.5} = 13.3 \text{ kN/m}^3$$

$$R_1 = \gamma_a a c = 13.3(5.1)(13.4) = 909 \text{ kN} \quad [\text{see Fig. (15-10d)}]$$

$$\bar{y}_1 = \frac{c}{2} + GH = \frac{13.4}{2} + 4.0 = 10.7 \text{ m}$$

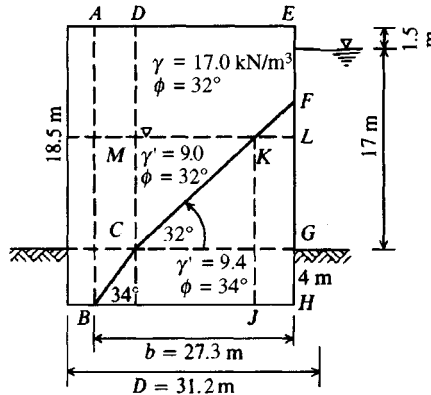


Figure E15-4

$$M_1 = R_1 \bar{y}_1 = 909 \times 10.7 = \mathbf{9726 \text{ kN} \cdot \text{m}}$$

$$R_2 = \gamma_a c^2 = 13.3 \times 13.4^2 = 2388 \text{ kN}$$

$$\bar{y}_2 = \frac{c}{3} + GH = \frac{13.4}{3} + 4.0 = 8.5 \text{ m}$$

$$M_2 = R_2 \bar{y}_2 = 2388 \times 8.5 = \mathbf{20\,298 \text{ kN} \cdot \text{m}}$$

$$M_T = M_1 + M_2 = 9726 + 20\,298 = \mathbf{30\,024 \text{ kN} \cdot \text{m}}$$

**Step 2.** Find  $M_r$  of zone  $ABID$ . First, find the average unit weight  $\gamma'_a$  of all the cell soil with  $H = 22.5$  m:

$$\gamma'_a H = W \rightarrow \gamma'_a = \frac{10 \times 17.0 + 8.5 \times 9.0 + 4.0 \times 9.4}{22.5} = \mathbf{12.6 \text{ kN/m}^3}$$

Also,

$$a = H - GH = 22.5 - 4.0 = 18.5 \text{ m} \quad c = 4.0 \text{ m}$$

From Eq. (15-12), we find that

$$M_r = \gamma'_a c^2 \left( \frac{a}{2} + \frac{c}{3} \right) = 12.6 \times 4^2 \left( \frac{18.5}{2} + \frac{4.0}{3} \right) = \mathbf{2134 \text{ kN} \cdot \text{m}}$$

**Step 3.** Find  $M_r'' = PfB$ , where  $B = b = 27.3$  m of Fig. E15-4; use  $H = 18.5$  m. For  $\phi = 32^\circ$  the Rankine  $K_a = 0.307$  from Table 11-3:

$$P = \frac{1}{2} \gamma_a H^2 K_a = \frac{1}{2} \times 13.3 \times 18.5^2 \times 0.307 = 698.7 \text{ kN}$$

$$M_r'' = PfB = 698.7 \times 0.30 \times 27.3 = \mathbf{5722 \text{ kN} \cdot \text{m}}$$

**Step 4.** Compute stability number  $N_{ot}$  against overturning using Eq. (15-14):

$$N_{ot} = \frac{M_{tr}}{M_o} = \frac{30\,024 + 2134 + 5722}{18\,248.4} = \mathbf{2.08} \quad (\text{O.K.})$$

The remainder of the Cummings design is identical to Ex. 15-3—that is, check sliding stability and bearing capacity.

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## 15-9 CLOVERLEAF COFFERDAM DESIGN

Since the cloverleaf cell contains a large amount of piling and connections it is not much used. Instead, the use of wells to dewater a circular cell to reduce the bursting pressure in the interlocks (which usually control their design) is generally more economical. When it is determined that a cloverleaf cell is required use the circular cell dimension that you will have just computed (and found inadequate) as a starting point on the cloverleaf cell dimensions.

Make an approximate scaled drawing (both plan and elevation) to select dimensions (distances  $x$ ,  $y$ , and radius  $r$ ). Also draw the required pressure diagrams similar to Figs. 15-6a and either  $b$  or  $c$  depending on whether the piling is to rock or into soil. These will not change; however, the radius may.

You will always use one  $90^\circ$  double **T** for the cell center and four  $120^\circ$  **Ys** for the cell. There will also be two, three, or four  $30^\circ$  **Ys** or  $90^\circ$  **Ts** for the connecting cells. It is usual to use the dimensions of Fig. 15-1c—that is,  $L' = 3.2r$ .

The area of the cell (usually one-fourth is computed) is computed by dividing a quadrant and the connecting arc into geometrical shapes whose areas can be directly obtained and then summing the results. The equivalent width of a rectangular cell based on the total (including connecting arc) cell area is

$$B = \frac{A}{L'} = \frac{A}{3.2r}$$

Once the equivalent width  $B$  is computed, the analysis proceeds as for a circular cell, and being checked for the following:

1. Sliding stability
2. Overturning stability
3. Cell shear—when using Eq. (15-5)

$$V = V_s + R_{il}$$

$$V_s = P_s \tan \delta$$

$$R_{il} = \frac{0.94P_t f_i}{r}$$

where select terms are identified in Fig. 15-6 or have been previously used. Note the use of  $\delta$  instead of  $\phi$  for  $V_s$  since the shear resistance is on the interior crosswalls.

## PROBLEMS

- 15-1. What is the change in  $\Delta H$  if the cell dimensions of Example 15-1 increase 10 percent (i.e.,  $0.2 \times 1.1 = 0.22$ ,  $0.3 \times 1.1 = 0.33$ , etc.)?
- 15-2. What is  $\Delta H$  if the cell dimensions of Example 15-1 *decrease* 10 percent (i.e.,  $0.2 \times 0.9 = 0.18$ ,  $0.3 \times 0.9 = 0.27$ , etc.)?
- 15-3. Redesign the diaphragm cofferdam of Example 15-2 if the cell depth of embedment  $D = 5.0$  m (instead of 5.5) and the total height is  $H = 14$  m. Assume the saturation line is at 7 m from the top, and the freeboard distance remains at 0.6 m.
- 15-4. Redesign the diaphragm cofferdam of Example 15-2 if all of the soil (fill and base soil) has a  $\phi$ -angle of  $32^\circ$ .
- 15-5. Redesign the cellular cofferdam of Example 15-3 if the saturation line is lowered to 1 m below the dredge line by using wells.
- 15-6. Redesign the circular cofferdam of Example 15-3 if the total  $H = 20$  m with the river flood stage level = 20 m and the depth to the saturation line = 8 m (it is not 10 m). All other soil data is the same.
- 15-7. Redo Example 15-3 if all of the soil (cell and base) has a  $\phi = 34^\circ$ .
- 15-8. Redo Example 15-3 using the Cummings method of Example 15-4 if the  $\phi = 34^\circ$  for all the soil (both cell and base).
- 15-9. Redo Example 15-3 using the given dimensions but using an inside berm as shown in Fig. P15-9. Assume the berm resistance is  $R_b = W_b \tan \phi$ . Note that the berm provides a surcharge for the lower passive resistance.
- 15-10. How many piles would be required in the diaphragm cofferdam of Example 15-2 if the  $r = L = 15.5$  m (we computed 15.8 and rounded to 15 in the example)?
- 15-11. How many piles would be required for the cellular cofferdam of Example 15-3 if we used  $\alpha = 45^\circ$  instead of  $30^\circ$  used in the example?

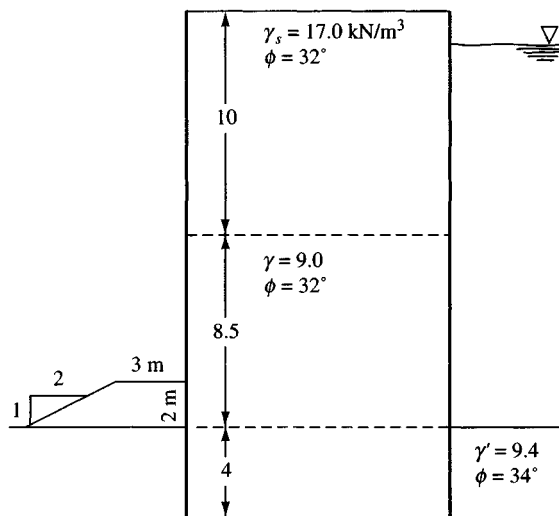


Figure P15-9

- 15-12. Design a cloverleaf cofferdam based on  $B = 35$  m, founded on rock, and able to resist a water head  $H_w = 22$  m. Neglect the embedment depth of 1.5 m. Select details are shown in Fig. P15-11.

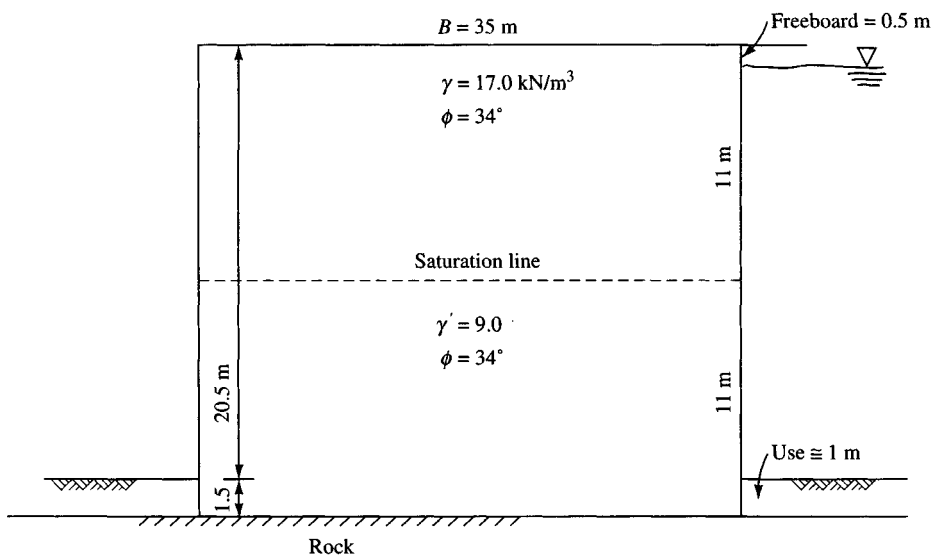


Figure P15-11