14-1 CONSTRUCTION EXCAVATIONS

It is a legal necessity with any new construction to provide protection to the adjacent structures when excavating to any appreciable depth. Without adequate lateral support the new excavation will almost certainly cause loss of bearing capacity, settlements, or lateral movements to existing property.

New construction may include cut-and-cover work when public transportation or public utility systems are installed below ground and the depth is not sufficient to utilize tunneling operations. The new construction may include excavation from depths of 1 to 20+ m below existing ground surface for placing any type of foundation from a spread footing to a mat, or for allowing one or more subbasements.

All of this type of construction requires installation of a lateral retaining system of some type before excavation starts.

Current practice is to avoid clutter in the excavation by using some kind of tieback anchorage (if required). The older methods of Fig. 14-1b and c produced substantial obstructions in the work area. Accidental dislodgement of these obstructions (struts and rakers) by equipment could cause a part of the wall to collapse. This mishap could be hazardous to the health of anyone in the immediate vicinity and to the contractor’s pocketbook shortly afterward.

14-1.1 Types of Walls

Until the late 1960s basically two types of walls were used in excavations. These are shown in Fig. 14-1b and c. Since then there has been a veritable explosion of wall types and/or materials used for the wall. We might group these walls as follows:

- Braced walls using wales and struts
- Soldier beam and lagging
- Braced sheeting
Bored-pile walls
Diaphragm-slurry walls

Braced walls using struts or rakers as shown in Fig. 14-1b, c were widely used up to the mid-1960s. They are seldom used today except in small projects such as bracing for water and sewer line trenches that are over about 2.5+ m deep. They are not much used for large excavations in urban areas since the struts and rakers produce too much clutter in the excavated area and increase both the labor cost and the possibility of accidents.

**Figure 14-1a** Three methods for providing lateral support for excavations. Method (a) is most popular in urban areas if trespass for anchorages is allowed.

(a) Tieback construction
The soldier beam and lagging system of Fig. 14-1a is popular for temporary construction. That is, pairs of rolled steel sections (the soldier beams) are driven to a depth slightly below the final excavation. Their spacing $s$ is on the order of 2 to 4 m so that available timber can be used for lagging. The lagging timbers, which are slightly shorter than the spacing but on the order of 50 to 100 mm thick, are installed behind the front flanges (or clipped to the front flanges using proprietary clips) to retain the soil as excavation proceeds. If the lagging is behind the flange, some hand excavation is usually required to get the lagging into place.

At depths specified by the foundation engineer—usually computed using empirical methods—excavation halts and a drill rig is used to drill the anchor holes for tiebacks. These are installed using bearing plates on the soldier beam flanges and tack welded for the vertical force component from the anchor; additional welding may be needed to hold the beams in alignment. The plates may be tilted to accommodate sloping anchorages (see Fig. E13-5 and Fig. 13-10d). It is usually more economical when using tieback slopes in the range of 15° to 20° to shop-drill the holes for the anchor rods at approximately that slope (the hole...
must be slightly oversize anyway) in the plate to produce an anchor point that costs less than cutting a channel to produce a slope. Alternatively, the anchor plate may have two holes for bolting to holes field-drilled into the outer flanges of the soldier beams in lieu of welding for easier wall disassembly.

Braced sheeting is essentially the anchored sheet-pile wall of Chap. 13 but with multiple levels of tiebacks or anchors. Construction is similar to the soldier beam lagging system in that the sheeting is driven and at selected excavation depths the wales and tiebacks are installed. When using this system it may also be necessary to tilt the anchorage assembly as shown in Fig. 13-10d.

Advantages of both the soldier beam and lagging and the braced sheeting systems are that they are easy to install (unless the excavation zone is rocky) and to remove and that the materials can be reused a number of times. The principal disadvantage is that the adjacent property owner may not allow encroachment (or request a royalty payment deemed too high) to install the anchorage. Since anchorages are not removed they represent permanent obstacles in the underground area around the perimeter of the construction site.

When the soil is rocky or the excavation is into rock, one only needs to drive the piling to the rock interface. Sometimes—especially with sheetpiling—it is impossible to drive the piling the full depth of the excavation. When this situation occurs, it may be possible to step the construction as shown in Fig. 14-2. An equation for the sheeting depth for each stage is given on the figure.

**Figure 14-2**  Critical depth $D$ (SF = 1) when soil conditions do not allow sheetpiling to be driven the full depth of excavation and it is possible to reduce lower work areas.

Using $\sigma_{1,B} = \gamma D$ and solve (2) we obtain

$$D = \frac{2c}{\gamma K_a^{0.5}} + \frac{2c}{\gamma K_a^{1.5}}$$

for $\phi = 0$; $K_a = 1$ and the critical depth is:

$$D = \frac{4c}{\gamma}$$
Pile walls are used in these circumstances:

a. It is too difficult to drive soldier beams or sheetpiling.

b. It is necessary to have a nearly watertight wall so as not to lower the GWT outside the construction perimeter.

c. The retaining wall is to be used as a permanent part of the structural system (e.g., the basement walls).

d. It is necessary to use the full site space, and adjacent owners disallow using their underground space to install tieback anchors (or there are already existing obstructions such as tunnels or basement walls).
There are a large number of pile wall configurations or modifications of existing methodology, of which Fig. 14-3 illustrates several. The diaphragm-slurry wall is shown in Fig. 14-16 and will be considered in Sec. 14-9. The particular wall configuration used may depend on available equipment and contractor experience. Terms used in the construction of these walls are shown on the appropriate figures.

When the wall must be watertight, the secant wall, consisting of interlocking piles (available in diameters ranging from 410 to 1500 mm), is most suited. This wall is constructed by first casting a concrete guide wall about 1 m thick and of a width 400 to 600 mm larger than the pile diameters and preferably with the casing preset for the primary piles. The primary (or female) piles are then drilled (they may be cased, but the casing must be pulled) and the piles cast using any required reinforcement. After hardening, the secant (or male) piles (of the same or smaller diameter) are drilled; during this process the drilling removes segments of the primary piles so an interlock is obtained as shown (Fig. 14-3b). The secant piles may also be cascd, but here the casing does not have to be removed. They also may be reinforced—either with reinforcing bar cages or W, H, or I sections placed in the cavity before the concrete is placed. This pile configuration is possible because of the more recent development of high-torque drilling equipment capable of cutting hard materials such as rock and concrete with great efficiency.

Secant pile walls can also be constructed using a cement slurry for the primary piles so that the cutting for the secant piles is not quite so difficult.

Tiebacks may be used with the pile walls. If the piles are in fairly close lateral contact, the tiebacks will require wales. For the secant-type piles, the tiebacks are simply drilled through the pile (although if this is known in advance it might be practical to preset the top one or two anchor holes in place using large-diameter pieces of plastic tubing cut to size and inserted into the hole and held in place by some means).

Slurry walls will be considered in Sec. 14-9.

### 14-1.2 Drilled-in-Place Piles

Where pile-driving vibrations using either pile hammers or vibratory drivers may cause damage to adjacent structures or the noise is objectionable, some type of drilled-in-place piles are required.

Where the soil to be retained contains some cohesion and water is not a factor, the soldier beam or drilled-in-place pile spacing may be such that lagging or other wall supplement is not required, because arching, or bridging action of the soil from the lateral pressure developed by the pile, will retain the soil across the open space. This zone width may be estimated roughly as the intersection of 45° lines as shown in Fig. 14-3c, d. The piles will, of course, have to be adequately braced to provide the necessary lateral soil resistance. This kind of construction can only be used for a very short time period, because soil chunks will slough off from gravity and/or local vibrations as drying of the exposed surfaces takes place.

Where sufficient anchorage is available at the pile base (perhaps socketed into rock) and with an adequate diameter, one method is to design the pile as a prestressed beam (see Fig. 14-3e). After installation the tendon, cast in a conduit, is tensioned to a preset load and anchored at the top. The prestress load produces a qualitative stress as shown at various sections along the pile depending on the eccentricity. The pile tends to deflect toward the backfill/original ground with the tendon installed as shown, but this deflection is resisted by the soil so that the final result is a nearly vertical pile and (one hopes) no loss of ground from any deflection toward the excavation side.
Placing the prestress tendon with $e$ on the right side of the vertical pile axis would tend to deflect the pile away from the backfill. Although this deflection would more efficiently utilize the concrete strength $f'_{c}$ in bending, the lateral displacement into the excavation would encourage additional ground loss.

Where both the earth and water must be retained, the system will have to be reasonably watertight below the water table and be capable of resisting both soil and hydrostatic pressures. Lowering the water table is seldom practical for environmental reasons but, additionally, it will produce settlement of the soil (and of any structures on that soil). If there is a high differential water head (the construction area must be kept dry), sheetpiling joints cannot be relied on to retain water without adequate sealing and/or pumping the infiltration so the retaining wall solutions may become limited to the secant or slurry wall. It is evident that uplift or buoyancy will be a factor for those structures whose basements are below the water table. If uplift is approximately equal to the weight of the structure, or larger, it will be necessary to anchor the building to the soil. This can be done using anchor piles to bedrock. Two other alternatives are belled piles (tip enlarged) or vertical “tiebacks.”

When making excavations where adjacent property damage can occur from pile driving or excavation vibrations, one should take enough photographs of the surrounding structures to establish their initial condition so that future claims can be settled in a reasonable manner. A select number of ground elevation control stations should be established around the perimeter of the excavation to detect whether ground loss damage claims are real or imagined.

Ground loss is a very serious problem around excavations in built-up areas. It has not been solved so far with any reliability; where the ground loss has been negligible, it has been more a combination of overdesign and luck rather than rational analysis.

### 14-2 SOIL PRESSURES ON BRACED EXCAVATION WALLS

The braced or tieback wall is subjected to earth-pressure forces, as are other retaining structures, but with the bracing and/or tieback limiting lateral wall movement the soil behind the wall is not very likely to be in the active state. The pressure is more likely to be something between the active and at-rest state.

With tiebacks (and bracing) the wall is pressed against the retained earth, meaning the lateral pressure profile behind the wall is more trapezoidal than triangular. Figure 14-4 idealizes the development of wall pressures behind a braced wall.

In stage 1 of Fig. 14-4 the wall is subjected to an active earth pressure, and wall displacement takes place. The lateral deformation depends on cantilever soil-wall interaction as would be obtained by the finite-element program FADSPABW (B-9) of Chap. 13. Next a strut force is applied to obtain stage 2. No matter how large the strut force (within practical limitations), the wall and earth are not pushed back to their original position, but the strut force, being larger than the active pressure, causes an increase in the wall pressure.

The integration of the pressure diagram at the end of stage 2 would be approximately the strut force. It is not exactly that amount of force since inevitably there is soil and anchor creep and much uncertainty in earth-pressure distribution. As shown for the end of stage 2 the excavation causes a new lateral displacement between $b$ and $c$ and probably some loss of

---

1For convenience the term *strut force* will be used for any kind of restraint—from struts, tiebacks, or whatever.
Figure 14-4  Qualitative staged development of earth pressure behind an excavation. The strut force produces lateral pressures that generally are larger than the active values. The strut force generally changes with time and installation method.

Peck (1943) [using measurements taken from open cuts in clay during construction of the Chicago, IL, subway system (ca. 1939–41)] and later in the Terzaghi and Peck (1967) textbook, proposed apparent pressure diagrams for wall and strut design using measured soil pressures obtained as from the preceding paragraph. The apparent sand pressures of Fig. 14-5 were based primarily on their interpretation of those reported by Krey (in the early 1930s) from measurements taken in sand cuts for the Berlin (Germany) subway system.

These apparent pressure diagrams were obtained as the envelope of the maximum pressures that were found and plotted for the several projects. The pressure envelope was given a maximum ordinate based on a portion of the active earth pressure using the Coulomb (or Rankine) pressure coefficient.

The Peck pressure profiles were based on total pressure using $\gamma_{\text{sat}}$ (and not $\gamma' = \gamma_{\text{sat}} - \gamma_w$), and it was never clearly explained how to treat the case of both $\gamma_s$ and $\gamma_{\text{sat}}$ being retained.

These diagrams have been modified several times, with the latest modifications [Peck (1969)] as shown in Fig. 14-5. When the Peck pressure diagrams were initially published, Tschebotarioff and coworkers [see Tschebotarioff (1973)] noted that Peck's initially proposed clay profiles could produce $K_a = 0.0$ for certain combinations of $s_w/\gamma H$, so a first modification was made to ensure that this did not occur.

Tschebotarioff observed that for most cohesionless soils $0.65K_a \approx 0.25$ for all practical purposes, since $\phi$ is usually approximated. On this basis he drew some slightly different suggested pressure profiles that have received some use.
The figure and table shown in Fig. 14-5 allow use of either the Peck or Tschebotarioff apparent (total) pressures or any others by suitable choice of the $z_i$ values.

If one designs a strut force based on the apparent pressure diagram and uses simply supported beams for the sheeting as proposed by Terzaghi and Peck, the strut force will produce not more than the contributory area of that part of the apparent pressure diagram. The sheeting may be somewhat overdesigned, because it is continuous and because simple beam analysis always gives larger bending moments; however, this overdesign was part of the intent of using these apparent pressure diagrams.

That these apparent pressure diagrams produce an overdesign in normally consolidated soils was somewhat verified by Lambe et al. (1970) and by Golder et al. (1970), who predicted loads up to 50 percent smaller than measured strut loads. This difference is not always the case, however, and if ground conditions are not exactly like those used by Peck in developing his apparent pressure profiles, the error can sometimes be on the unsafe side.

For example, Swatek et al. (1972) found better agreement using the Tschebotarioff apparent pressures for clayey soils in designing the bracing system for a 21.3-m deep excavation in Chicago, IL. Swatek, however, used a “stage-construction” concept similar to Fig. 14-4 along with the Tschebotarioff pressure diagram. In general, the Tschebotarioff method may be more nearly correct in mixed deposits when the excavation depth exceeds about 16 m.
A major shortcoming of all these apparent pressure diagrams is what to do when the retained soil is stratified. In this case it would be reasonable [see also suggestions by Liao and Neff (1990)] to do the following:

1. Compute two Rankine-type pressure diagrams using the Rankine $K_a$ and $K_o (= 1 - \sin \phi)$ and using effective unit weights. Make a second pressure diagram for the GWT if applicable.
2. Plot the two pressure diagrams [use 0 for any (-) pressure zones] on the same plot.
3. Compute the resultant $R_a$ and $R_o$ for the two pressure plots.
4. Average the two $R$ values, and from this compute an apparent pressure diagram. Take a rectangle ($\sigma = R/H$) or a trapezoid. For example if you use $z_1 = z_3 = 0.25H$, the average pressure $\sigma$ is

$$R_{av} = \frac{H + 0.5H}{2} \sigma \rightarrow \sigma = \frac{2R_{av}}{1.5H}$$

5. Include the water pressure as a separate profile that is added to the preceding soil pressures below the GWT depending on the inside water level.
6. Instead of using an average of the two $R$ values from step 3, some persons simply multiply the active pressure resultant $R_a$ by some factor (1.1, 1.2, 1.3) and use that to produce the apparent soil pressure diagram. It may be preferable to factor $R_a$ and compare this diagram to the “average” pressure diagram (using unfactored $R_a$ and $R_o$) and use the larger (or more conservative) value.

14-2.1 Soil Properties

The soil properties to use for design will depend on whether the wall is temporary or permanent and on the location of the GWT behind the wall.

If the ground is reasonably protected and above the water table, drained soil parameters would be appropriate (or at least parameters determined from consolidated undrained tests at the in situ water content). If the retained soil is partly above and partly submerged, the drained parameters would apply to the region above the water table.

For retained soil below the water table, consolidated-undrained tests would be appropriate. The lateral pressure from the tieback or bracing would tend to put the soil below the GWT into a consolidated-undrained condition, but this state would depend on how long the wall is in place and the permeability of the retained soil. If the wall is in place only a week or so, undrained strength parameters should be used. Keep in mind that pore water drainage in cohesionless soils is rapid enough that the drained $\phi$ angle can be used.

The interior zone of the wall is in a plane strain condition whereas the ends or corners are in more of a triaxial state. When the angle of internal friction $\phi$ is not measured or is taken (estimated) as less than about 35°, it is not necessary to adjust for plane strain conditions.

14-2.2 Strength Loss with Elapsed Time

Bjerrum and Kirkedam (1958) measured strut forces in an excavation from September through November that indicated the lateral earth pressure increased from 20 to 63 kPa owing to an apparent loss of cohesion. This observation was based on back-computing using consolidated-undrained strength values of both $\phi$ and $c$ and later assuming only a drained
\( \phi \) angle. Ulrich (1989) observed that tieback and/or strut loads increased with time in over-consolidated clays. Others have also reported that tieback or strut loads increase with time but not in a quantitative manner. It appears, however, that 20 to 30 percent increases are not uncommon. These increases seldom result in failure but substantially reduce the SF.

Cohesion is often reduced in cuts because of changes in moisture content, oxidation, tension cracks, and possibly other factors, so that on a long-term basis it may not be safe to rely on large values of cohesion to reduce the lateral pressure. Temporary strut load increases may also result from construction materials and/or equipment stored on the excavation perimeter.

Where the cut is open only 2 to 5 days, soil cohesion is relied upon extensively to maintain the excavation sides.

### 14-3 CONVENTIONAL DESIGN OF BRACED EXCAVATION WALLS

The conventional method of designing walls (but not pile walls) for excavations consists in the following steps:

1. Sketch given conditions and indicate all known soil data, stratification, water level, etc.
2. Compute the lateral pressure diagram using Peck's method, Tschebotarioff's method, or the procedure outlined in the preceding section, depending on the quality (and quantity) of soil data and what is to be retained. In the case of a cofferdam in water for a bridge pier or the like, the lateral pressure is only hydrostatic pressure.
3. Design the sheeting, wales, and struts or tiebacks; in the case of a bridge pier cofferdam, the compression ring.

The sheeting making up the wall can be designed either as a beam continuous over the several strut/tieback points or (conservatively) as a series of pinned beams as in Fig. 14-6. For continuous sheeting a computer program\(^2\) is the most efficient means to obtain bending moments.

The *wales* can be designed similarly to those for anchored sheetpile walls. They may be conservatively taken as pin-ended; however, where a computer program is available, they can be taken as continuous across the anchor points. Alternatively, we can estimate the fixed-end moments (fem) conservatively as \(wL^2/10\) (true fem are \(wL^2/12\)) as was done in Example 13-5. The wale system for a braced cofferdam for a bridge pier and the like, where the plan area is small, may be designed primarily for compression with the wales across the ends accurately fitted (or wedged) to those along the sides so that the effect is a compression ring (even though the plan is rectangular). In this case there may be some struts across the width, but the end wale loads will be carried into the side wales as an axial compression force.

If tiebacks cannot be used and piles or a slurry wall would be too costly, the only recourse is to use wales with either struts or rakers as shown in Fig. 14-1b and c.

Struts and rakers are actually beam-columns subjected to an axial force such as \(R_n\) of Fig. 14-6 and bending from member self-weight. Since the strut is a column, the carrying capacity

---

\(^2\)You can use your program B-5 as follows: JTSoIL = node where soil starts, \(k_r = \)?, NZx = no. of brace points if \(x = 0\) m; convert pressure diagram to node forces and input NNZP values. Input \(E\) and \(I\) for a unit width (1 m or 1 ft) of sheeting. Make similar adjustments for wales.
Figure 14-6  Simplified method of analyzing the sheeting and computing the strut forces. This method of using a *simple beam* for strut forces is specifically required if you use the Peck apparent pressure profiles.

is inversely proportional to the ratio \((L/r)^2\). The only means to reduce the \(L/r\) ratio is to use intermediate bracing. These might be struts used for the end walls; if so, they will greatly increase the construction area obstructions and will require design of the framing.

Usually vertical supports will be required for horizontal struts unless the unsupported span is relatively short.

The intended purpose of the struts and rakers (and tiebacks) is to restrain the wall against lateral movement into the excavation. Any inward movement that takes place must be tolerated, for forcing the wall back to the original position is impossible.

Because lateral movement of the wall is associated with a vertical ground settlement in a perimeter zone outside the excavation (termed *ground loss*), the following are essential:

1. The wall must fit snugly against the sides of the excavation. This criterion is critical with soldier beam and lagging or when the wall is placed against the earth face after some depth of excavation.
2. The struts, rakers, or tiebacks must allow a very limited amount of lateral displacement. These are all elastic members with an \(AE/L\), so some movement toward the excavation always occurs as the equivalent "spring" stretches or compresses under the wall load.
3. The wales must be sufficiently rigid that displacements interior from the anchor points are not over 1 to 3 mm more than at the anchors. This criterion assumes the wales are in close contact with the wall sheeting, so the assumption of a uniform wall pressure computed as \(w = \frac{F_{at}}{s}\) is valid.
4. The bracing must be located vertically so that large amounts of wall bulging into the excavation do not occur between brace points. This restriction either puts minimum limits on the stiffness of the wall facing (or sheeting) or limits the vertical spacing of the wales—or both.
5. The struts or rakers are slightly prestressed by constructing the brace point so that a hydraulic jack and/or wedges can be driven between the wale and strut both to force the wales against the wall and to compress the strut or raker. The system of jacking and/or wedges usually requires periodic adjustments during construction to maintain the necessary strut prestress.

The location of the first wale can be estimated by making a cantilever wall analysis using program FADSPABW (B-9) and several trials for the dredge line location and by inspecting the output for lateral movement into the excavation. This approach is applicable for all soils; however, in cohesive soils, the depth should not exceed the depth of the potential tension crack $h_t$ (see Fig. 14-7a) obtained from using a suitable SF.

The formation of this tension crack will increase the lateral pressure against the lower wall (it now acts as a surcharge), and if the crack fills with water the lateral pressure increases considerably. Also, this water will tend to soften the clay in the vicinity for a reduction in shear strength $s_u$.

The choice of the first wale location should also consider the effect of the location of successive Rankine active earth wedges as in Fig. 14-7b, since they will develop at approximate zero moment points from the wall slightly below the excavation line. Note, however, the wedge angle $\rho$ is not always $\rho = (45^\circ + \phi/2)$—it depends on the cohesion, wall adhesion, and backfill surcharges. Program SMTWEDGE or WEDGE may be used to approximately locate the wedge angle $\rho$.

Where lateral movement and resulting ground subsidence can be tolerated, the depth to the first strut in sandy soils may be where the allowable bending stress in the sheeting is reached from a cantilever wall analysis as in Fig. 14-7c.

**Example 14-1.** Make a partial design for the braced sheeting system shown in Figs. E14-1a, b using PZ footprint 27 sheet-pile sections for the wall. Use either a pair of channels back to back or a pair of I sections for wales and W sections for struts. The struts will use lateral bracing at midspan for the weak axis of the struts (giving 2.5 m of unbraced length) as shown by the dotted lines in the plan view of Fig. E14-1a.

Horizontal and vertical construction clearances require the strut spacing shown. The water level near the bottom of the excavation will be controlled by pumping so that there is no water head to consider. We will make only a preliminary design at this point (design should be cycled in a computer program to see if lateral movements are satisfactory for controlling ground loss outside the perimeter). Use the apparent lateral pressure diagrams of Fig. 14-5 and check using a $K_o$ pressure.
Required. Draw pressure diagram, code the problem, create a data set, and use computer program FADSPABW (B-9) to analyze strut forces and bending; check bending in sheeting and axial force in the critical strut.

Solution.

Step 1. Obtain the pressure diagram using Fig. 14-5. For loose sand we have $z_1 = z_3 = 0$ and $z_2 = H = 9.3$ m. The lateral pressure for the resulting rectangle shown dashed in Fig. E14-1b is

$$
\sigma_h = 0.65 \gamma H K_a = 0.65 \times 16.5 \times 9.3 \times 0.333 = 33.2 \text{ kPa}
$$

This value will be increased 15 percent to allow for water or other uncertainties, giving for design

$$
\sigma_{des} = 1.15 \times 33.2 = 38.2 \text{ kPa} \rightarrow \text{ use 38 kPa (Fig. E14-1b)}
$$

What would be the design pressure if we used $k_o$ for the pressure coefficient? $K_o = 1 - \sin 30^\circ = 0.50$ and the total wall force is

$$
R_o = \frac{1}{2} \times 16.5 \times 9.3^2 \times 0.5 = 357 \text{ kN}
$$

Dividing by wall height, we obtain

$$
\sigma_o = \frac{357}{9.3} = 38.38 \text{ kPa} \quad \text{(very close, so use 38 kPa)}
$$
Step 2. Code wall as shown in Fig. E14-1b and set up data file EX141.DTA (on your diskette) for using computer program FADSPABW. Use the following input control parameters:

- \( NP = 28 \)
- \( NM = 13 \)
- \( NNZP = 0 \) (no node forces)
- \( NLC = 1 \) (1 load case)
- \( ITYPE = 0 \) for sheet pile wall
- \( LISTB = 0 \) (no band matrix list)
- \( NCYC = 1 \) (embedment depth fixed)
- \( NRC = 0 \) (input equation for \( k_s \))
- \( JTSoil = 10 \)
- \( NONLIN = 0 \) (no check)
- \( IAR = 3 \) (the struts)
- \( NZX = 0 \) (no B.C.)
- \( IPRESS = 11 \) (11 node pressures input)
- \( IMET = 1 \) (SI units)

Step 3. Estimate the modulus of subgrade reaction \( k_s \) for the base 2 m of embedment depth using Eq. (9-10) with the bearing capacity equation as previously used in Chap. 13:

\[
k_s = 40(\gamma N_q Z + 0.5\gamma BN_r)
\]

From Table 4-4 obtain \( N_q = 18.4 \) and \( N_r = 15.1 \) (Hansen values), which give

\[
k_s = 4983 + 12144Z^{1}
\]

(use AS = 5000; BS = 12000; EXPO = 1)

To keep \( k_s \) from increasing significantly in the 2-m depth we will use an EXPO value of 0.5 instead of 1.0. The value of NRC initially input (and on the data file) informs the program of the type of equation that will be used. During program execution a "beep," followed by a screen request to input EXPO, alerts the user to input the value. The EXPO value is output with the equation so you can check that the correct value was input.

Since the sheeting is continuous, we can use any value for moment of inertia \( I \); however, we will make a side run (not shown) and try a PZ22, which is the smallest Z section in Table A-3 (Appendix A). Compute for the PZ22 section the following:

\[
I/m = 64.39/0.560 = 114.98 \times 10^{-6} \text{ m}^4/\text{m}
\]

\[
S/m = 0.542/0.560 = 0.9679 \times 10^{-3} \text{ m}^3/\text{m}
\]

We must estimate a W section for the strut so that we can compute the spring \( AE/L \) (struts are horizontal). From previous runs (not shown) we will try to use a

\[
W200 \times 52 \quad f_y = 250 \text{ MPa} \quad (\text{Fps: W8} \times 35 \quad f_y = 36 \text{ ksi})
\]

\[
A = 6.65 \times 10^{-3} \text{ m}^2 \quad \text{and} \quad L = 5.5 \text{ m about } x \text{ axis}
\]

Since the strut spans the excavation and is compressed from both ends we will use half the spring for each wall, and for spacing \( s = 3 \text{ m} \) the input spring \( K \) is

\[
K_{strut} = \frac{AE}{L} = \frac{6.65 \times 200 \times 10^3}{3 \times 2 \times 5.5} = 40300 \text{ kN/m}
\]

The spring is slightly rounded,\(^3\) consistent with the accuracy of the other data.

The \( y \) axis has lateral bracing to give an unbraced length \( L_u \) of 2.75 m; also

\[
r_x = 89 \text{ mm} \quad r_y = 52 \text{ mm}
\]

\(^3\)Note that in SI when \( 10^{-3} \) and \( 10^3 \) are used and cancel they are not shown—this is one of the major advantages of using SI.
giving \( r_x / r_y = 1.71 < 2 \) so the \( x \) axis controls the column stress. Thus,

\[ S_x = 0.51 \times 10^{-3} \text{ m}^3 \]

From these data, using the rectangular pressure diagram of Fig. E14-1b we create a data file \( \text{EX141.DTA} \) and use it to produce the output sheets shown as Fig. E14-1c.

**Step 4.** Make an output check and design the members.

a. First check \( \sum F_h = 0 \). Output is 362.9 kN. Using the formula for the area of a trapezoid (and noting the bottom triangle with a length of 0.5 m), we find the pressure diagram gives

\[ R = \frac{9.8 + 9.3}{2} \times 38 = 362.9 \quad (\text{O.K.}) \]

b. A visual examination of the near-end and far-end moments in the output tables shows \( \sum M_{\text{nodes}} = 0 \).

c. Check if the strut is adequate. The largest strut force is at \( S_1 = 120.4 \) kN. The self-weight of the strut over a 5.5-m span is \( 52 \text{ kg} \times 9.807 \text{ N/kg} \times 0.001 \text{ kN/N} = 0.51 \text{ kN/m} \). The resulting maximum bending moment is

\[ M_{\text{max}} = \frac{wL^2}{8} = \frac{0.51 \times 5.5^2}{8} = 1.93 \text{ kN} \cdot \text{m} \]

The stress is

\[ f_s = \frac{M}{S} = \frac{1.93}{0.51} = 3.8 \text{ MPa} \quad (\text{insignificant}) \]

The allowable axial load for a \( W200 \times 52 \) section (in column load tables provided by AISC (1989) or elsewhere) is

\[ P_{\text{allow}} = 553 \text{ kN} \gg 361.2 \quad [3 \times 120.4 \quad (\text{may be overdesigned})] \]

\[ f_s = \frac{P_{\text{allow}}}{A} = \frac{553}{6.65} = 83.2 \text{ MPa} \quad (\text{bending can be neglected}) \]

Now the question is whether we should use this section or one much smaller. This is answered by looking at the displacements at the strut nodes. We find these values:

<table>
<thead>
<tr>
<th>Node</th>
<th>Displacement, mm</th>
<th>Strut force, kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>2.987</td>
<td>120.4</td>
</tr>
<tr>
<td>5</td>
<td>2.712</td>
<td>109.3</td>
</tr>
<tr>
<td>8</td>
<td>2.703</td>
<td>108.9</td>
</tr>
</tbody>
</table>

Consider the following:

1. These are theoretical displacements, and the actual displacements will probably be larger.
2. When jacking or wedging the struts against the wales, axial loads that are greater than the computed strut loads might be developed.
3. The strut forces are nearly equal; the strut displacements are nearly equal, which is ideal.

Considering these several factors, we find the struts appear satisfactory. Keep in mind this is not a very large rolled \( W \) section.
## Solution for Sheet Pile Wall—Cantilever or Anchored

**Input Parameters:**

- **No of Members:** 13
- **No of Loads:** 1
- **Max No of Iterations:** 1
- **Nonlinear Check:** 0
- **Node Soil Starts:** JTSOIL = 10
- **No of Anchor Rods:** IAR = 3
- **No of Non-Zero P-Matrix Entries:** 0
- **Imet (SI > 0):** 1
- **List Band Matrix:** LISTB (IF > 0) = 0
- **Input Node Pressures:** !PRESS = 1
- **Input Node Pressures:** 2
- **Modal of Elasticity:** 200000.0 MPA
- **Soil Modulus:** 5000.00 + 12000.00*Z**0.50 KN/M**3
- **Node Ks Reduction Factors:** JTSOIL = 0.70, JTSOIL+1 = 0.90
- **Sheet Pile and Control Data:**
  - **Width:** 1.000 M
  - **Initial Embed Depth:** DEMB = 2.000 M
  - **Depth Incr Factor:** DEPINC = 0.500 M
  - **Dredge Line Convergence:** CONV = 0.050 M

**Anchor Rods Located at Node Nos:** 2, 5, 8

**Member and Node Data for Wall Width:** 1.000 M

<table>
<thead>
<tr>
<th>MemNo</th>
<th>NP1</th>
<th>NP2</th>
<th>NP3</th>
<th>NP4</th>
<th>Length (M)</th>
<th>Inertia (M^4)</th>
<th>Node</th>
<th>Ks (KN/M^3)</th>
<th>Springs</th>
<th>Xmax (M)</th>
<th>Node Q (KPa)</th>
<th>Node P (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>1.5000</td>
<td>.0001150</td>
<td>1</td>
<td>.000</td>
<td>0.00</td>
<td>.000</td>
<td>38.0000</td>
<td>28.5000</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>1.0000</td>
<td>.0001150</td>
<td>2</td>
<td>.000</td>
<td>0.00</td>
<td>.000</td>
<td>38.0000</td>
<td>28.5000</td>
</tr>
<tr>
<td>3</td>
<td>5</td>
<td>6</td>
<td>7</td>
<td>8</td>
<td>1.0000</td>
<td>.0001150</td>
<td>3</td>
<td>.000</td>
<td>0.00</td>
<td>.000</td>
<td>38.0000</td>
<td>28.5000</td>
</tr>
<tr>
<td>4</td>
<td>7</td>
<td>8</td>
<td>9</td>
<td>10</td>
<td>1.0000</td>
<td>.0001150</td>
<td>4</td>
<td>.000</td>
<td>0.00</td>
<td>.000</td>
<td>38.0000</td>
<td>28.5000</td>
</tr>
<tr>
<td>5</td>
<td>9</td>
<td>10</td>
<td>11</td>
<td>12</td>
<td>1.0000</td>
<td>.0001150</td>
<td>5</td>
<td>.000</td>
<td>0.00</td>
<td>.000</td>
<td>38.0000</td>
<td>28.5000</td>
</tr>
<tr>
<td>6</td>
<td>11</td>
<td>12</td>
<td>13</td>
<td>14</td>
<td>1.0000</td>
<td>.0001150</td>
<td>6</td>
<td>.000</td>
<td>0.00</td>
<td>.000</td>
<td>38.0000</td>
<td>28.5000</td>
</tr>
<tr>
<td>7</td>
<td>13</td>
<td>14</td>
<td>15</td>
<td>16</td>
<td>1.0000</td>
<td>.0001150</td>
<td>7</td>
<td>.000</td>
<td>0.00</td>
<td>.000</td>
<td>38.0000</td>
<td>28.5000</td>
</tr>
<tr>
<td>8</td>
<td>15</td>
<td>16</td>
<td>17</td>
<td>18</td>
<td>.9000</td>
<td>.0001150</td>
<td>8</td>
<td>.000</td>
<td>0.00</td>
<td>.000</td>
<td>38.0000</td>
<td>28.5000</td>
</tr>
<tr>
<td>9</td>
<td>17</td>
<td>18</td>
<td>19</td>
<td>20</td>
<td>.9000</td>
<td>.0001150</td>
<td>9</td>
<td>.000</td>
<td>0.00</td>
<td>.000</td>
<td>38.0000</td>
<td>28.5000</td>
</tr>
<tr>
<td>10</td>
<td>19</td>
<td>20</td>
<td>21</td>
<td>22</td>
<td>.5000</td>
<td>.0001150</td>
<td>10*</td>
<td>3500.000</td>
<td>1594.729</td>
<td>.0100</td>
<td>38.0000</td>
<td>23.4333</td>
</tr>
<tr>
<td>11</td>
<td>21</td>
<td>22</td>
<td>23</td>
<td>24</td>
<td>.5000</td>
<td>.0001150</td>
<td>11*</td>
<td>12136.750</td>
<td>5753.917</td>
<td>.0150</td>
<td>.0000</td>
<td>3.1667</td>
</tr>
<tr>
<td>12</td>
<td>23</td>
<td>24</td>
<td>25</td>
<td>26</td>
<td>.5000</td>
<td>.0001150</td>
<td>12</td>
<td>17000.000</td>
<td>8319.475</td>
<td>.0200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>25</td>
<td>26</td>
<td>27</td>
<td>28</td>
<td>.5000</td>
<td>.0001150</td>
<td>13</td>
<td>19696.940</td>
<td>9813.193</td>
<td>.0250</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* = Ks Reduced by FAC1 or FAC2

---

**Figure E14-lc**
MEMBER MOMENTS, NODE REACTIONS, DEFLECTIONS, SOIL PRESSURE, AND LAST USED P-MATRIX FOR LC = 1

<table>
<thead>
<tr>
<th>MEMNO</th>
<th>MOMENTS - NEAR END 1ST, KN-M</th>
<th>NODE</th>
<th>SPG FORCE, KN</th>
<th>ROT, RADS</th>
<th>DEFL, M</th>
<th>SOIL Q, KPA</th>
<th>P-, KN-M</th>
<th>P-, KN</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00</td>
<td>1</td>
<td>0.000</td>
<td>-0.0221</td>
<td>0.0560</td>
<td>0.00</td>
<td>0.00</td>
<td>28.500</td>
</tr>
<tr>
<td>2</td>
<td>-42.750</td>
<td>2</td>
<td>120.3660</td>
<td>-0.0081</td>
<td>0.0299</td>
<td>0.00</td>
<td>0.00</td>
<td>47.500</td>
</tr>
<tr>
<td>3</td>
<td>1.616</td>
<td>3</td>
<td>0.000</td>
<td>-0.0013</td>
<td>0.0278</td>
<td>0.00</td>
<td>0.00</td>
<td>38.000</td>
</tr>
<tr>
<td>4</td>
<td>7.982</td>
<td>4</td>
<td>0.000</td>
<td>-0.00035</td>
<td>0.0317</td>
<td>0.00</td>
<td>0.00</td>
<td>38.000</td>
</tr>
<tr>
<td>5</td>
<td>-23.652</td>
<td>5</td>
<td>109.2735</td>
<td>0.00021</td>
<td>0.0271</td>
<td>0.00</td>
<td>0.00</td>
<td>38.000</td>
</tr>
<tr>
<td>6</td>
<td>15.988</td>
<td>6</td>
<td>0.000</td>
<td>0.00038</td>
<td>0.0315</td>
<td>0.00</td>
<td>0.00</td>
<td>38.000</td>
</tr>
<tr>
<td>7</td>
<td>17.627</td>
<td>7</td>
<td>0.000</td>
<td>0.00035</td>
<td>0.0317</td>
<td>0.00</td>
<td>0.00</td>
<td>38.000</td>
</tr>
<tr>
<td>8</td>
<td>-18.733</td>
<td>8</td>
<td>108.9314</td>
<td>-0.00033</td>
<td>0.0270</td>
<td>0.00</td>
<td>0.00</td>
<td>36.100</td>
</tr>
<tr>
<td>9</td>
<td>14.091</td>
<td>9</td>
<td>0.000</td>
<td>0.00024</td>
<td>0.0255</td>
<td>0.00</td>
<td>0.00</td>
<td>34.200</td>
</tr>
<tr>
<td>10</td>
<td>16.134</td>
<td>10</td>
<td>3.3081</td>
<td>-0.00083</td>
<td>0.0207</td>
<td>7.260</td>
<td>0.00</td>
<td>23.433</td>
</tr>
<tr>
<td>11</td>
<td>7.207</td>
<td>11</td>
<td>9.1457</td>
<td>-0.00108</td>
<td>0.0159</td>
<td>19.291</td>
<td>0.00</td>
<td>3.167</td>
</tr>
<tr>
<td>12</td>
<td>1.270</td>
<td>12</td>
<td>8.4922</td>
<td>-0.00117</td>
<td>0.0102</td>
<td>17.353</td>
<td>0.00</td>
<td>0.000</td>
</tr>
<tr>
<td>13</td>
<td>-0.421</td>
<td>13</td>
<td>4.2252</td>
<td>-0.00118</td>
<td>0.0043</td>
<td>8.481</td>
<td>0.00</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>14</td>
<td>-0.8426</td>
<td>-0.00118</td>
<td>-0.0016</td>
<td>-3.491</td>
<td>0.00</td>
<td>0.000</td>
</tr>
</tbody>
</table>

SUM SPRING FORCES = 362.90 VS SUM APPLIED FORCES = 362.90 KN

(*) = SOIL DISPLACEMENT > XMAX(I) SO SPRING FORCE AND Q = XMAX*VALUE

NOTE THAT P-MATRIX ABOVE INCLUDES ANY EFFECTS FROM X > XMAX ON LAST CYCLE

DATA FOR PLOTTING IS SAVED TO DATA FILE: wall.pit

AND LISTED FOLLOWING FOR HAND PLOTTING

<table>
<thead>
<tr>
<th>NODE</th>
<th>DEPTH</th>
<th>KS</th>
<th>COMP X, MM</th>
<th>XMAX</th>
<th>SHEAR V(I,1), V(I,2)</th>
<th>MOMENT MOM(I,1), MOM(I,2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>.000</td>
<td>.0</td>
<td>5.599</td>
<td>.000</td>
<td>LT OR T</td>
<td>LT OR TOP</td>
</tr>
<tr>
<td>2</td>
<td>1.500</td>
<td>.0</td>
<td>2.987</td>
<td>.000</td>
<td>28.50</td>
<td>-1.12</td>
</tr>
<tr>
<td>3</td>
<td>2.500</td>
<td>.0</td>
<td>2.782</td>
<td>.000</td>
<td>-44.37</td>
<td>-1.62</td>
</tr>
<tr>
<td>4</td>
<td>3.500</td>
<td>.0</td>
<td>2.783</td>
<td>.000</td>
<td>-6.37</td>
<td>31.63</td>
</tr>
<tr>
<td>5</td>
<td>4.500</td>
<td>.0</td>
<td>2.712</td>
<td>.000</td>
<td>31.63</td>
<td>-39.64</td>
</tr>
<tr>
<td>6</td>
<td>5.500</td>
<td>.0</td>
<td>3.152</td>
<td>.000</td>
<td>-39.64</td>
<td>-1.64</td>
</tr>
<tr>
<td>7</td>
<td>6.500</td>
<td>.0</td>
<td>3.173</td>
<td>.000</td>
<td>-1.64</td>
<td>36.36</td>
</tr>
<tr>
<td>8</td>
<td>7.500</td>
<td>.0</td>
<td>2.703</td>
<td>.000</td>
<td>-36.36</td>
<td>-36.47</td>
</tr>
<tr>
<td>9</td>
<td>8.400</td>
<td>.0</td>
<td>2.546</td>
<td>.000</td>
<td>-36.47</td>
<td>-2.27</td>
</tr>
<tr>
<td>10</td>
<td>9.300</td>
<td>3500.0</td>
<td>2.074</td>
<td>10.000</td>
<td>-2.27</td>
<td>17.85</td>
</tr>
<tr>
<td>11</td>
<td>9.800</td>
<td>12136.8</td>
<td>1.589</td>
<td>15.000</td>
<td>17.85</td>
<td>11.87</td>
</tr>
<tr>
<td>12</td>
<td>10.300</td>
<td>17000.0</td>
<td>1.021</td>
<td>20.000</td>
<td>11.87</td>
<td>3.38</td>
</tr>
<tr>
<td>13</td>
<td>10.800</td>
<td>19696.9</td>
<td>.431</td>
<td>25.000</td>
<td>3.38</td>
<td>-0.84</td>
</tr>
<tr>
<td>14</td>
<td>11.300</td>
<td>21970.6</td>
<td>-.159</td>
<td>25.000</td>
<td>-.84</td>
<td>.00</td>
</tr>
</tbody>
</table>
Step 5. Check the sheet-pile bending stresses. From the output sheet the largest bending moment is 42.75 kN · m and occurs at node 2:

\[ f_s = \frac{M}{S} = \frac{42.75}{0.9679} = 44.2 \text{ MPa} \] (well under 0.6 or 0.65 \( f_y \))

In summary, it appears this is a solution. It may not be the absolute minimum cost, but it is both economical and somewhat (but not overly) conservative. Remember: Before the wales and struts are installed, excavation takes place to a depth that allows adequate workspace for the installation. That is, already some lateral displacement has not been taken into account here (we will make an estimate in Example 14-3).

Also, although it is self-evident that we could use two lines of struts (instead of the three shown), the vertical spacing would be such that the lateral movement in the region between struts could represent unacceptable perimeter ground loss.

14-4 ESTIMATION OF GROUND LOSS AROUND EXCAVATIONS

The estimation of ground loss around excavations is a considerable exercise in engineering judgment. Peck (1969) gave a set of nondimensional curves (Fig. 14-8) that can be used to obtain the order of magnitude. Caspe (1966, but see discussion in November 1966 critical of the method) presented a method of analysis that requires an estimate of the bulkhead deflection and Poisson’s ratio. Using these values, Caspe back-computed one of the excavations in Chicago reported by Peck (1943) and obtained reasonable results. A calculation by the author indicates, however, that one could carry out the following steps and obtain results about equally good:

1. Obtain the estimated lateral wall deflection profile.

Figure 14-8 Curves for predicting ground loss. [After Peck (1969).]
2. Numerically integrate the wall deflections to obtain the volume of soil in the displacement zone \( V_s \). Use average end areas, the trapezoidal formula, or Simpson’s one-third rule.

3. Compute or estimate the lateral distance of the settlement influence. The method proposed by Caspe for the case of the base soil being clay is as follows:
   a. Compute wall height to dredge line as \( H_w \).
   b. Compute a distance below the dredge line

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Use ( H_p ) ( = )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi = 0 )</td>
<td>( B )</td>
</tr>
<tr>
<td>( \phi = \frac{\pi}{2} )</td>
<td>( 0.5B \tan(45 + \frac{\phi}{2}) )</td>
</tr>
</tbody>
</table>

where \( B = \) width of excavation, m or ft. From steps (a) and (b) we have

\[
H_t = H_w + H_p
\]

c. Compute the approximate distance \( D \) from the excavation over which ground loss occurs as

\[
D = H_t \tan\left(45^\circ - \frac{\phi}{2}\right)
\]

4. Compute the surface settlement at the edge of the excavation wall as

\[
s_w = \frac{2V_s}{D}
\]

5. Compute remaining ground loss settlements assuming a parabolic variation of \( s_i \) from \( D \) toward the wall as

\[
s_i = s_w \left(\frac{x}{D}\right)^2
\]

Example 14-2. Using the values provided by Caspe, verify the method just given. Figure E14-2 displays data from Caspe and as plotted on Peck’s settlement curve. The excavation was 15.85 m (52 ft) wide and 11.58 m (38 ft) deep. The upper 4.25 m was sand backfill with the remaining depth being a soft to stiff clay with an undrained \( \phi = 0^\circ \). Displacements were taken on 1.2-m (4-ft) distances down the wall to the dredge line, and Caspe estimated the remaining values as shown on the displacement profile.

Solution. Caspe started by computing the total settlement depth based on \( H_w = 11.58 \) m + \( H_p = B = 15.85 \) m (\( \phi = 0^\circ \)) = 27.43 m = \( D \). Integrating the wall profile from 0.6 m to \(-26.83 \) m (27.43 - 0.6) using the average end area formula, we obtain

\[
V_s = \left(\frac{30.5 + 5.0}{2} + 33.0 + 35.6 + 49.6 + 45.7 + \cdots + 18.0 + 12.7\right) \times 1200
\]

\[
= 807\,900 \text{ mm}^3 \rightarrow 0.8079 \text{ m}^3 \quad \text{(per meter of wall width)}
\]

At the wall face the vertical displacement is

\[
s_w = \frac{2 \times 0.8079}{26.23} = 0.0616 \text{ m} \rightarrow 62 \text{ mm} \quad \text{(Peck \approx 50 mm)}
\]
Figure E14-2a

Distance from excavation, m

Figure E14-2b
At distances from the wall of 6.1, 12.2, and 18.3 m the distances from $D$ are 21.3, 15.24, and 9.1 m, giving a parabolic variation of

\[
\sigma_{6.1} = 62 \left( \frac{21.3}{27.4} \right)^2 = 37.5 \text{ mm} \quad (\text{Peck} \approx 33.0 \text{ mm})
\]

\[
\sigma_{12.2} = 62 \left( \frac{15.24}{27.4} \right)^2 = 19.2 \text{ mm} \quad (\text{Peck} \approx 18.0 \text{ mm})
\]

\[
\sigma_{18.3} = 62 \left( \frac{9.1}{27.4} \right)^2 = 6.9 \text{ mm} \quad (\text{Peck} \approx 7.6 \text{ mm})
\]

These displacements are shown on the settlement versus excavation distance plot on Fig. E14-2.

Several factors complicate the foregoing calculations. One is the estimation of displacements below the excavation line. However, satisfactory results would probably be obtained by integrating the soil volume in the lateral displacements to the dredge line. The displacements shown here below the dredge line are an attempt to account somewhat for soil heave (which also contributes to ground loss) as well as lateral wall movement.

**14-5 FINITE-ELEMENT ANALYSIS FOR BRACED EXCAVATIONS**

The finite-element method (FEM) can be used to analyze a braced excavation. Either the finite element of the elastic continuum (Fig. 14-9) using a program such as FEM2D (noted in the list of programs in your README.DOC file) or the sheet-pile program FADSPABW (B-9) can be used.

**14-5.1 Finite-Element Method for the Elastic Continuum**

The FEM2D program (or similar) uses two-dimensional solid finite elements (dimensions of $a \times b \times$ thickness) of the elastic continuum. These programs usually allow either a plane-stress ($\sigma_x, \sigma_y > 0; \sigma_z = 0$) or plane-strain ($\varepsilon_x, \varepsilon_y > 0; \varepsilon_z = 0$) analysis based on an input control parameter. They usually allow several soils with different stress-strain moduli ($E_s$) and $\mu$ values for Poisson’s ratio.

For us to use these programs, it is helpful if they contain element libraries (subroutines) that can compute stiffness matrix values for solids, beam-column elements (element axial forces and bending moments), and ordinary column ($AE/L$) elements. Some programs allow additional elements, but for two-dimensional analyses of both walls and tunnel liners, these are usually sufficient and are a reasonable balance between program complexity and practical use.

In an analysis for an excavation with a wall one would develop a model somewhat as shown in Fig. 14-9. Initially it would be rectangular, but one should try to take advantage of symmetry so that only the excavation half shown is analyzed to reduce input and computational time (and round-off errors). The cross section represents a unit thickness, although FEM2D allows a thickness to be input such that shear walls, which are often one concrete block thick, can be analyzed.

It would be necessary to estimate the lateral and vertical dimensions of the model. Lateral fixity is assumed along the vertical line of symmetry (the C.L.). It is convenient to model the other two cut boundaries with horizontal and vertical columns or struts as shown.
The use of strut-supported boundaries allows a quick statics check, since the sum of the axial forces in the bottom vertical struts is the weight of the block at any stage. The horizontal struts on the right side provide a structurally stable soil block. If the right-side nodes are fixed, they tend to attract stresses unless a very large model is used. By using struts, their $EI/L$ can be varied so that they provide structural stability without attracting much stress. If there are large axial loads in the horizontal struts, for any $EI/L$, the model is too small. If large axial loads occur, either additional elements must be added to the right or the element $x$ coordinates of several of the right-side nodes must be increased to produce wider elements and a larger model.

Wall bracing (or struts) can be modeled by inputting either lateral node forces or springs; tiebacks can be modeled by inputting $AE/L$-type elements defined by end coordinates. The node spacing should be such that any interior tiebacks lie along corner nodes, or so the horizontal component of $AE/L$ is colinear with a node line. This methodology also allows modeling the vertical force component if desired.

Node spacing should be closer (as shown in Fig. 14-9) in critical regions, with larger element spacing away from the critical zone. Node spacing can be somewhat variable using more recent FEM that employ isoparametric elements; there is less flexibility with the older FEM programs, which used triangular nodes (rectangles are subdivided into triangles, manipulated, and then converted back). The trade-off, since either model computes about the same results, is to use the method with which you are most familiar.

Model soil excavation in stages as:

1. Vertical—nodal concentration of removed soil weight overlying a node as $\uparrow$ force.
2. Horizontal—nodal concentration of $K \times$ removed soil weight overlying the node as a horizontal node force toward the excavation.
The finite-element analysis for an excavation involves several steps [see also Chang and Duncan (1970)], as follows:

1. Grid and code a block of the elastic continuum, taking into account excavation depth and any slopes. It is necessary that the several excavation stages coincide with horizontal grid lines.
2. Make an analysis of the unexcavated block of step 1 so that you can obtain the node stresses for the elements in the excavation zone.
3. Along the first excavation line of the finite-element model, obtain the stresses from step 2 and convert them to nodal forces of opposite sign as input for the next analysis, which will be the excavation of stage 1. Remove all the elements above the excavation outline.
4. Execute the program with the new input of forces and the model with the elements that were removed in step 3. From this output, obtain the node stresses along the next excavation line. Also, remove all the elements above the current excavation line.
5. Repeat steps 3 through 4 as necessary.

It requires clever node coding to produce an initial data set that can be reused in the several excavation stages by removing a block of elements for that stage. It may be preferable to use some kind of element data generator for each stage; some programs have this program built in and call it a “preprocessor.”

There are major problems with using the FEM of the elastic continuum for excavations, including at least the following:

1. A massive amount of input data is required. Several hundred elements may be required for each stage plus control parameters and other data.
2. Obtaining soil parameters $E_s$ and $\mu$ for the several strata that may be in the model is very difficult.
3. Most critical is the change in the elastic parameters $E_s$ and Poisson’s ratio $\mu$ when the soil expands laterally toward the excavation or against the excavation wall and vertically upward (heaves) from loss of overburden pressure. If these values are not reasonably correct, one does a massive amount of computation to obtain an estimate that may be as much as 100 percent in error.

Clough and coworkers at Virginia Polytechnic Institute claim modest success using this procedure and have published several papers in support of these claims—the latest is Clough and O’Rourke (1990), but there were several earlier ones [Clough and Tsui (1974); Clough et al. (1972)]. Others have used this method in wall analysis, including Lambe (1970), but with questionable success.

14-5.2 The Sheet-Pile Program to Estimate Lateral Wall Movements

The sheet-pile program FADSPABW can be used to make a wall movement estimate as follows:

1. Locate the nodes at convenient spacings. You will want to locate nodes at all tiebacks or struts. Also locate nodes about 0.5 m below where any tiebacks or struts are to be installed so there is room for their installation.
2. Code the full wall depth including to the dredge line and the depth of embedment for stage 1. You do this so that most of the element and other data can be reused in later stages by editing copies of the initial data file. Use NCYC = 1 and probably NONLIN = 0 to avoid excessive refinement.

3. Referring to Fig. 14-4, make a number of trials using the conventional lateral pressure profile and including any surcharge. Do not use a pressure diagram such as in Fig. 14-5 at this point. These several trials are done to find a reasonable depth of excavation so that the first strut can be installed without excessive lateral deflection of the wall top. Depending on the situation, this displacement probably should be kept to about 25–30 mm.

4. Copy the foregoing data set and edit it to install the first strut and excavate to the next depth. You can now continue using the conventional lateral pressure profile or some kind of diagram of Fig. 14-5. The following program parameters are changed: JTSOIL; IAR = 1 (first strut); and IPRESS [to add some PRESS(I) values]; and reduce XMAX(I) entries.

5. Now copy this data set at a second strut; change JTSOIL; IAR = 2; IPRESS [and PRESS(I)]; and again reduce the number of XMAX(I) entries.

6. Make a copy of this data set, add the next strut, and so forth.

The displacement profile is the sum of the displacements with the sign from the preceding sequence of steps.

**Example 14-3.** Make an initial estimate of the lateral movements of the braced excavation for which the sheeting and struts were designed in Example 14-2. The data sets for this problem are on your program diskette as EX143A, EX143B, EX143C, and EX143D.DTA, so you can rapidly reproduce the output.

**Solution.**

**Stage 1.** Draw the full wall height of 11.3 m as shown in Fig. E14-3a and locate nodes at strut points and other critical locations. For struts S1, S2, additional nodes of 0.5 m are added below the strut to give enough room for its installation. The 1.8 m depth below strut S3 is only divided into two 0.9-m elements. From this information the initial input is

\[
\text{NP} = 34 \quad (17 \text{ nodes—we are using more than in Example 14-2})
\]

\[
\text{NM} = 16
\]

\[
\text{IAR} = 0
\]

\[
\text{JTSOIL} = 4
\]

The soil pressure diagram used is shown in Fig. E14-3b, and the last (4th node value) nonzero entry = 16.5 kPa = \(16.5 \times K_0 = 16.5(2.0)(0.5)\). Note use of \(K_0\) and not \(K_a\) to give a somewhat more realistic model.

Refer to data set EX143A.DTA for the rest of the input and use it to make an execution for a set of output.

**Stage 2.** Make a copy of EX143A.DTA as EX143B.DTA (on your diskette) with the strut installed IAR = 1, JTSOIL, IPRESS, PRESS(I), and XMAX(I) reduced. Refer to the pressure profile for the additional PRESS(I) entries.

Make an execution of this data set to obtain a second set of output. You have at this point excavation to node 8 with strut S1 installed at node 3.

**Stage 3.** Make a copy of EX143B.DTA as EX143C.DTA (on your diskette). Now install strut S2 using IAR = 2. Use JTSOIL = 12 and adjust IPRESS, PRESS(I), and XMAX(I).

Make an execution of this data set to obtain a third set of output.
Soil:
Silty Loose Sand
\( \phi = 30^\circ \)
\( \gamma = 16.50 \text{ kN/m}^3 \)
Use \( K_o = 0.5 \)

(a) \( P-X \& \) nodes.

(b) Pressure profile.

Figure E14-3

Stage 4. Make a copy of EX143C.DTA as EX143D.DTA (on your diskette). Now install strut S3 using IAR = 3. Use JTSOIL = 13 (will excavate to bottom of excavation) and adjust IPRESS, PRESS(I), and XMAX(I).

Make an execution of this data set to obtain a fourth set of output, etc. (takes 8 data sets).

A partial output summary follows:

<table>
<thead>
<tr>
<th>Stage 1</th>
<th>Stage 2</th>
<th>Stage 3</th>
<th>Stage 4</th>
<th>( \sum \delta_h )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Node</td>
<td>2.0 m</td>
<td>5.0 m</td>
<td>8.4 m</td>
<td>9.3 m</td>
</tr>
<tr>
<td>1</td>
<td>7.0 mm</td>
<td>-3.1</td>
<td>-0.5</td>
<td>-0.6</td>
</tr>
<tr>
<td>S1 3</td>
<td>3.7</td>
<td>1.4</td>
<td>0.4</td>
<td>0.8</td>
</tr>
<tr>
<td>5</td>
<td>1.7</td>
<td>4.2</td>
<td>1.2</td>
<td>1.8</td>
</tr>
<tr>
<td>S2 7</td>
<td>—</td>
<td>5.3</td>
<td>4.8</td>
<td>3.0</td>
</tr>
<tr>
<td>9</td>
<td>—</td>
<td>3.3</td>
<td>9.5</td>
<td>4.0</td>
</tr>
<tr>
<td>S3 11</td>
<td>—</td>
<td>0.2</td>
<td>13.5</td>
<td>4.3</td>
</tr>
<tr>
<td>B.E. 13</td>
<td>—</td>
<td>-0.2</td>
<td>6.1</td>
<td>3.9</td>
</tr>
</tbody>
</table>
The total node displacements $\sum \delta_i$ look reasonable for this type of excavation. It is not unreasonable that there could be 18.0 mm of displacement at strut S3. If the value is deemed high, one can do some adjusting, but basically this procedure gives you an estimate of what the lateral movements might be. They could be less than this but are not likely to be more unless there is extremely poor workmanship.

The strut forces and wales were designed in Example 14-2. This example is only to give an estimate of lateral displacements. If you work with copies of the data sets you might be able to improve the displacements, but keep in mind that no matter how you manipulate the numbers the actual measured values are what count.

Example 14-3 gives a fairly simple means to make an estimate of lateral wall movement into an excavation. Notice that the first set of data (EX143A.DTA) is the most difficult. Beyond that only a few values are changed. Actually, to avoid confusing the user the data sets have generally been edited more than actually required for all but the last one. Note also that a backfill surcharge or some earth pressure factor other than $K_0$ can be used to produce a number of different earth pressure and displacement profiles.

In any case this procedure is about as accurate (in advance of construction) as any other procedure and far simpler than the FEM of the elastic continuum.

14-6 INSTABILITY DUE TO HEAVE OF BOTTOM OF EXCAVATION

When a braced excavation (sometimes called a cofferdam) is located either over or in a soft clay stratum as in Fig. 14-10a, the clay may flow beneath the wall and into the excavation, producing heave if sufficient soil is removed that the resisting overburden pressure is too small.

The pressure loss from excavation results in a base instability, with the soil flow producing a rise in the base elevation commonly termed heave, which can range from a few millimeters to perhaps 300 mm. This case can be analyzed from Mohr's circle using Eqs. (2-54) and (2-55) as done in Fig. 14-2 or as the bearing failure of Fig. 4-1.

There are two general cases to consider:

Case 1. In this instance the goal is to provide sufficient depth of the piling of Fig. 14-10 to prevent the soft underlying clay from squeezing into the excavation. For this case and

![Figure 14-10](image-url)  
*(a) Cofferdam on soft clay; (b) theoretical solution.*
referring to Fig. 14-10 for identification of terms we have (noting $K_a = \sqrt{K_a} = 1$) for element $A$

$$\sigma_3 = \gamma D - 2s_u$$  \hspace{1cm} (a)

and for element $B$ we have

$$\sigma'_1 = \gamma h + 2s_u$$  \hspace{1cm} (b)

since $\sigma'_1 = \sigma_3$ and $\sigma'_3 = \gamma h = \sigma'_1 - 2s_u$. Substituting values, we find that

$$\gamma h = \gamma D - 2s_u - 2s_u$$

Solving for the critical depth $D = D_c$ and inserting an SF we obtain the desired equation:

$$D_c = \frac{\gamma h + 4s_u}{\gamma(SF)} \quad (\phi = 0^\circ)$$  \hspace{1cm} (14-1)

**Case 2.** This is a general analysis for excavation depth where the depth of excavation is limited such that the effective bearing capacity of the base soil can be utilized.

This more general analysis is as follows (refer to Fig. 14-11). Block $OCBA$ produces a net vertical pressure $\sigma_v$ on $OA$ of

$$\sigma_v = \gamma D + q_s - \frac{F_f - Dc_a}{r}$$  \hspace{1cm} (a)

where terms not shown on Fig. 14-11 are

- $F_f = \frac{1}{2}\gamma D^2 K_a \tan \phi$
- $\phi =$ friction angle of soil above dredge line
- $c =$ cohesion of soil above dredge line
- $c_a =$ wall adhesion as fraction of $c$
- $c' =$ base soil cohesion
- $q_s =$ any surcharge
- $r = 0.707B$

---

**Figure 14-11**  Stability of excavation against bottom heave using bearing capacity fundamentals.
Substitution for $F_f$ into Eq. (a) and equating $\sigma_v = q_{ul}$ (at the same depth on either side of the wall) we obtain

$$\frac{\gamma D r + q_s r - \left(\frac{1}{2} \gamma D^2 K_a \tan \phi + c_a D\right)}{r} = q_{ul}$$

where $q_{ul} = c'N'_c + \gammaNhq$.

**TABLE 14-1**

**Bearing capacity factor $N'_c$ for square and circular bases and for strip bases**

Interpolate table or plot $N'_c$ (ordinate) versus $D/B$ (abscissa) for intermediate values. Tabulated values are similar to those given by Skempton (1951) [see also Meyerhof (1972)] and later on the Bjerrum and Eide (1956) curves. Values for $N'_c$ can be obtained from Hansen's bearing capacity equation of $N'_c = 5.14(1 + \phi' + d'_c)$ shown in Table 4-1 and Table 4-5a. The Hansen values are compared to Skempton's, which are given in parentheses. In general, $N_c$ for a rectangle is computed as

$$N'_{c, rect} = N_c(0.84 + 0.16B/L);$$

For a strip, $B/L \to 0$.

<table>
<thead>
<tr>
<th>$D/B$</th>
<th>$(1 + \phi' + d'_c)$</th>
<th>$N'_c$</th>
<th>$N'_{c, strip}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>5.14(1 + 0.2 + 0) = 6.2 (6.2)</td>
<td>$\pi + 2 = 5.14$</td>
<td></td>
</tr>
<tr>
<td>0.25</td>
<td>(1 + 0.2 + 0.1) = 6.7 (6.7)</td>
<td>$\times 0.84 = 5.6$</td>
<td></td>
</tr>
<tr>
<td>0.50</td>
<td>(1 + 0.2 + 0.2) = 7.2 (7.1)</td>
<td>$\times 0.84 = 6.0$</td>
<td></td>
</tr>
<tr>
<td>0.75</td>
<td>(1.2 + 0.4 $\times$ 0.75) = 7.7 (7.4)</td>
<td>$= 6.5$</td>
<td></td>
</tr>
<tr>
<td>1.0*</td>
<td>(1.2 + 0.4 $\tan^{-1}$ 1) = 7.8 (7.7)</td>
<td>$= 6.6$</td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>(1.2 + 0.4 $\tan^{-1}$ 1.5) = 8.2 (8.1)</td>
<td>$= 6.9$</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>(1.2 + 0.443) = 8.4 (8.4)</td>
<td>$= 7.1$</td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td>(1.2 + 0.476) = 8.6 (8.5)</td>
<td>$= 7.2$</td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td>(1.2 + 0.500) = 8.7 (8.8)</td>
<td>$= 7.3$</td>
<td></td>
</tr>
<tr>
<td>4.0</td>
<td>(1.2 + 0.530) = 8.9 (9.0)**</td>
<td>$= 7.5$</td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td>(1.2 + 0.549) = 9.0 (9.0)</td>
<td>$= 7.5$</td>
<td></td>
</tr>
</tbody>
</table>

*Discontinuous at $D/B = 1$ (from 0.4$D/B$ to 0.4$\tan^{-1} D/B$).

**Examples**

**Given.** Square footing on soft clay with a $D/B = 2$. Obtain $N_c$.

**Solution.** At $D/B$ above obtain directly $N'_c = 8.4$.

**Given.** Rectangular footing on soft clay. $B = 2$ m, $L = 4$ m and embedment depth $D = 1$ m. Obtain $N_c$.

**Solution.** Compute

$$B/L = 2/4 = 0.5; \quad D/B = 1/2 = 0.5$$

At $D/B = 0.5$ obtain

$$N'_c = 7.2$$

and

$$N'_{c,rect} = N_c(0.84 + 0.16B/L) = 7.2(0.84 + 0.16 \times 0.5) = 6.6$$
Substituting and simplifying, we obtain the maximum depth of wall $D'$ (including any freeboard depth $F_b$) as

$$D' = \frac{r(c'N_c' + \gamma hN_q - q_s)}{\gamma r - \frac{1}{2}rDK \tan \phi - c_a} + F_b$$  \hspace{1cm} (14-2)

For the case of $\phi = 0$ above the base of the wall, Eq. (14-2) reduces to

$$D' = \frac{c'N_c' + \gamma hN_q - q_s}{\gamma - c_a/r} + F_b$$ \hspace{1cm} (14-2a)

In these equations use an SF on the order of 1.2 to 1.5 (i.e., $d_{des}' = D'/SF$); use the upper range of around 1.4 to 1.5 for anisotropic soils [see Mana and Clough (1981)]. Carefully note that the inside depth $h$ above the wall base is a factor, and if the upper soil has a $\phi$ angle, the critical depth is found by trial using Eq. (14-2). Equation (14-2a) contains a bearing capacity factor $N_c'$. This value is obtained from Table 14-1, and one uses either $N_c'$ or $N_{c,\text{rect}}'$ depending on whether the excavation is square or has $B/L < 1$. The values in Table 14-1 were given as curves by Skempton (1951), who plotted them from work by Meyerhof in the late 1940s. Bjerrum and Eide (1956) are usually incorrectly credited with the curves. The author has elected to provide tabulated values so that users can either compute values or draw curves to a useful scale.

Bjerrum and Eide (1956) used the $N_c'$ bearing-capacity factors from Table 14-1 to analyze the base stability of 14 deep excavations and found a very reasonable correlation of ±16 percent. Later Schwab and Broms (1976) reanalyzed the Bjerrum and Eide excavations plus two others and concluded that the correlation might have been improved if anisotropy had been considered.

**Example 14-4.** Refer to Fig. E14-4. Can an excavation be made to 18 m, and if so what depth of sheeting is required to avoid a bottom heave (or soil flow into the excavation) based on using an SF of at least 1.25? Note that this problem formulation is the usual situation, and Eqs. (14-1) and (14-2) are of little value, but the derivation procedure is valuable since we have to use some of the parts.

**Solution.** Let us use these estimates:

$$\gamma_{\text{sand}} = 17.00 \text{ kN/m}^3$$
$$\gamma_{\text{clay}} = 18.00 \text{ kN/m}^3$$

Consider the 3 m of sand as a surcharge, giving

$$q_s = 3 \times 17.00 = 51 \text{ kPa}$$

Take an unweighted average of the undrained shear strength of the clay for design, so that

$$s_{u,av} = \frac{40 + 60}{2} = 50 \text{ kPa}$$

---

4 An unweighted average is acceptable if no strength value in the region from $B$ above to $2B$ below the base depth is smaller than 50 percent of the average strength. If any strength values are smaller than 50 percent of the average, then you should weight the strength as $s_{u,av} = \frac{\sum s_{u,j} \times H_j}{\sum H_j}$. 
Assume that there will be wall adhesion of 0.8c. We will try an initial depth of $D = 18 + 1 = 19$ m. This gives the depth of interest as

$$D' = 19 - 3 \text{ m (of sand)} = 16 \text{ m}$$

The dimension $r = 0.707B = 0.707 \times 12 = 8.5 \text{ m}$. For this state we have

$$\sigma_v = -D'c/r + \gamma D' + q_s$$

and substitution gives

$$\sigma_v = -16(0.8 \times 50/8.5) + 16(18.0) + 51 = -75.3 + 288.0 + 51 = 263.7 \text{ kPa}$$

The bearing-pressure resistance $q_{ult} = c N'_c + \gamma h$ ($h = 1 \text{ m}$). Thus,

$$N'_c = (5.14 \times 1.2)(1 + 0.4 \cdot \tan^{-1} 1.5)(0.84 + 0.16 \times 0.50) = 6.17 \times 1.39 \times 0.92 = 7.89$$

This result gives $q_{ult} = 50(7.89) + 18(1) = 412.5$. The resulting safety factor is

$$SF = \frac{412.5}{263.7} = 1.56 \text{ (O.K.)}$$

Note: The wall sheeting would have to be at least 18 m, but 19 m gives some base restraint as well as a slight increase in the SF.

14-7 OTHER CAUSES OF COFFERDAM INSTABILITY

A bottom failure in cohesionless soils may occur because of a piping, or quick, condition if the hydraulic gradient $h/L$ is too large. A flow net analysis may be used as illustrated in Fig. 14-12a or to reduce the hydraulic head $h$ by less pumping from the cell. In a few cases it may be possible to use a surcharge inside the cell.
Flow path with length $L$ may be scaled from a flow net construction.

Any soil

Impervious stratum

Sand

Figure 14-12 (a) Conditions for piping, or quick, conditions; (b) conditions for a blow-in (see also Figs. 2-10, 2-11, and 2-12).

In Fig. 14-12b, the bottom of the excavation may blow in if the pressure head $h_w$ indicated by the piezometer is too great, as follows ($SF = 1.0$):

$$\gamma_w h_w = \gamma_s h_s$$

This equation is slightly conservative, since the shear, or wall adhesion, on the walls of the cofferdam is neglected. On the other hand, if there are soil defects in the impervious layer, the blow-in may be local; therefore, in the absence of better data, the equality as given should be used. The safety factor is defined as

$$SF = \frac{\gamma_s h_s}{\gamma_w h_w} > 1.25$$

14-8 CONSTRUCTION DEWATERING

Figure 14-12 indicates that water inflow into an excavation can cause a bottom failure. Where it is impractical or impossible to lower the water table, because of possible damage claims or environmental concerns, it is necessary to create a nearly impervious water barrier around the excavation. Because no barrier is 100 percent impervious, it is also necessary to provide drainage wells below the bottom of the excavation, called sump pits, that are pumped as necessary to maintain a reasonably dry work space.

The groundwater level outside the excavation will require monitoring wells to avoid real (or imagined) claims for damages from any lowering of the original groundwater level.

Where it is allowed to depress the water table in the vicinity of the excavation, a system of perimeter wells is installed. This system may consist of a single row of closely spaced wellpoints around the site. A wellpoint is simply a section of small-diameter pipe with perforations (or screen) on one end that is inserted in the ground. If the soil is pervious in the area of the pipe screen, the application of a vacuum from a water pump to the top of the pipe will pull water in the vicinity of the pipe into the system. A vacuum system will be limited in the height of water raised to about 6 m. Theoretically water can be raised higher, but this type of system is less than theoretical. More than one set of perimeter wells can be installed as illustrated in Fig. 14-13. This type of system is seldom "designed"; it is contracted by
companies that specialize in this work. Although rough computations can be made, the field performance determines the number of wellpoints and amount of pumping required.

Where wellpoints are not satisfactory or practical, one may resort to a system of perimeter wells that either fully or partially penetrate the water-bearing stratum (aquifer) depending on site conditions and amount of pumpdown. Again, only estimates of the quantity of water can be made, as follows.

One may use a plan flow net as in Fig. 14-14 to obtain the seepage quantity. A plan flow net is similar to a section flow net as in Chap. 2. The equipotential drops are now contour lines of equal elevation intersecting the flow paths at the same angle. Sufficient contour lines must be established to represent the required amount of drawdown to provide a dry work area.

Some approximation is required, since it is not likely that the piezometric head is constant for a large distance around an excavation. Furthermore, approximation is necessary because a system of wells located around the excavation will not draw down the water to a constant contour elevation within the excavation. The water elevation will be a minimum at—and higher away from—any well. From a plan flow net the quantity of water can be estimated as

\[ Q = \alpha k(\Delta H) \frac{N_f}{N_e} L \]  

where  

\[ N_f = \text{number of flow paths (integer or decimal)} \]  

\[ N_e = \text{number of equipotential drops (always integer)} \]
Figure 14-14  Plan flow net. Note it is only necessary to draw enough flow and equipotential lines to obtain $N_f, N_e$.

$$\Delta H = H^2 - h^2 \text{ for gravity flow (see Fig. 14-15)}$$
$$= H - h_w \text{ for artesian flow}$$

$L = 1.0$ for gravity flow

$= \text{thickness of aquifer for artesian flow}$

$k = \text{coefficient of permeability in units consistent with } H \text{ and } L$

$\alpha = 0.5$ for gravity flow

$= 1.0$ for artesian flow

An estimate of the number of wells and flow per well is obtained by placing one well in the center of each flow path. The resulting flow per well is then

$$\text{Number of wells} = N_f$$

$$\text{Flow per well} = Q/N_f$$

An estimate of the quantity of water that must be pumped to dewater an excavation can also be obtained by treating the excavation as a large well (Fig. 14-15) and using the equation for a gravity flow well,

$$q = \frac{\pi k(H^2 - h_w^2)}{\ln(R/r_w)} \quad (14-4)$$

where terms not previously defined are

$H = \text{surface elevation of water at the maximum drawdown influence a distance } R \text{ from well center}$

$h_w = \text{surface elevation of water in well}$

$r_w = \text{well radius (use consistent units of m or ft)}$

This equation is for gravity wells; that is, the piezometric head and static water level are coincident, which is the likely case for pumping down the water table for a large excavation.
Excavation analyzed as large well of radius, $r_w$

\[
A = BL = \pi r_w^2
\]
\[
r_w = \sqrt{\frac{BL}{\pi}}
\]

The maximum radius of drawdown influence $R$ is not likely to be known; however, one may estimate several values of $R/r_w$ and obtain the corresponding probable pumping quantities $Q$. The value of the static groundwater level $H$ is likely to be known, and $h_w$ would normally be estimated at 1 to 2 m below the bottom of the excavation.

This estimate of well pumping to dewater an excavation should be satisfactory for most applications. It is not likely to be correct, primarily because the coefficient of permeability $k$ will be very difficult to evaluate unless field pumping tests are performed. It is usually sufficient to obtain the order of magnitude of the amount of water to be pumped. This is used for estimating purposes, and the contract is written to pay for the actual quantity pumped.

**Example 14-5.** Estimate the flow quantity to dewater the excavation shown in Fig. 14-14. Other data are as follows:

\[
H = 50 \text{ m} \quad a = 60 \text{ m}
\]
\[
\Delta H = 15 \text{ m} \quad b = 100 \text{ m}
\]
\[
k = 0.2 \text{ m/day} \quad D = 100 \text{ m}
\]

The soil profile is as shown in Fig. E14-5.

**Solution.** We will use a plan flow net (Fig. 14-14 was originally drawn to scale) and compute the quantity using Eq. (14-3) and check the results using Eq. (14-4).

**Step 1.** Compute $Q$ for the plan flow net (assume gravity flow after drawdown is stabilized). From Fig. 14-14, $N_f = 10$; $N_e = 2.1$; and from Fig. E14-5 we obtain

\[
H = 50 \text{ m} \quad H^2 = 2500 \text{ m}^2
\]
\[
h_w = 34 \text{ m} \quad h_w^2 = 1156 \text{ m}^2
\]
Substitution of these values into Eq. (14-3) with \( \alpha = 0.5 \) yields
\[
Q = \alpha k (\Delta H) \frac{N_f}{N_e} L = 0.5 \times 0.2 \times (2500 - 1156)^{1.10} = 640 \text{ m}^3/\text{day}
\]
and since \( N_f = 10 \) the number of wells = 10.

**Step 2.** Check results using Eq. (14-4):
\[
Q = \frac{\pi k (H^2 - h_w^2)}{\ln(R/r_w)} \quad \text{(may be O.K. when drawdown is stabilized)}
\]
\[
R = 100 \text{ m} \quad \text{unless we draw down the river} + r_w
\]
\[
r_w = \frac{A}{\pi} = \sqrt{\frac{60 \times 100}{\pi}} = 43.7 \rightarrow \text{use 44 m}
\]
Substitution gives
\[
Q = \frac{\pi \times 0.2(2500 - 1156)}{\ln(144/44)} = 712 \text{ m}^2/\text{day}
\]
This flow quantity compares quite well with the flow net construction, and the actual flow quantity may be on the order of 675 to 750 m\(^3\)/day.

---

### 14-9 SLURRY-WALL (OR -TRENCH) CONSTRUCTION

The placement of a viscous fluid, termed a slurry, in a narrow trench-type excavation to keep the ground from caving is a method in use since the early 1960s. The basic method had been (and is) used for oil well and soil exploration drilling to maintain boreholes in caving soils without casing. The large hydrostatic pressure resulting from several hundred meters of slurry allowed retention of oil or gas in oil wells until they could be capped with valving to control the fluid flow rate. The slurry used for these procedures is generally a mix of bentonite (a montmorillonitic clay-mineral-based product), water, and suitable additives.

Walls constructed in excavations where a slurry is used to maintain the excavation are termed *slurry, diaphragm-slurry*, or simply *diaphragm walls*. Figure 14-16 illustrates a
Drive piles 1, 2, possibly 3 and 4

Excavate 1

Diam = 0 to wall

0.5 - 2 m = 1

-2 - 4 m - 2 - 6 m

Diaphragm or wall

Diaphragm

Fill with slurry and later set reinforcing and tremie concrete

Excavation depth

Wall depth for lateral stability (use bracing or tiebacks if required)

Figure 14-16 Slurry method for diaphragm wall. Drive piles for tying wall sections together. Excavate as zones 1 and 2, perhaps 3 using slurry to keep excavation open. Set rebar cages and tremie concrete for wall 1, 2, perhaps 3. Excavate zone 4, perhaps 5 using slurry, set rebar cages and tremie concrete to complete a wall section.
method of constructing a diaphragm wall. Here piles are driven on some spacing, and alternate sections are excavated, with slurry added to keep the cavity full as excavation proceeds to the desired depth. It is necessary to maintain the cavity full of slurry—and sufficiently agitated to maintain a uniform density—to keep the sides of the excavation from caving. Reinforcing bar cages are then put in place, and concrete is placed by a tremie (a pipe from the surface to carry the concrete to the bottom of the excavation) to fill the trench from the bottom up.

The slurry displaced by the concrete is saved in a slurry pit for use in the next section of wall, etc. The pipe piles shown (not absolutely necessary for all walls) can be pulled after the first wall sections are formed and partially cured, or they can be left in place. The purpose of the piles is to provide a watertight seal and continuity between sections. Although the piles shown are the full wall width, they can be some fraction of the width and serve equally well.

In cases where the wall depth is too great for piles to self-support the lateral pressure from outside the excavation, the walls can be braced or tiebacks used. Tiebacks require drilling through the concrete, but this is not a major task with modern equipment so long as the reinforcement cages are designed so that the drill does not intersect rebars. These types of walls are usually left in place as part of the permanent construction.

This method and similar wall construction methods are under continuous development—primarily outside the United States. Figure 14-17 illustrates one of the more recent proprietary procedures for producing a slurry-type wall, which consists of three drills aligned with mixing paddles as shown. Here a soil-cement slurry (with various additives depending on wall purpose) is used, producing what is called a soil-cement mixed wall (SMW). Wide-flange beams can be inserted into the freshly constructed SMW section for reinforcing if necessary. Wall sections can range in width from about 1.8 to 6 m and up to 61 m in depth. Taki and Yang (1991) give some additional details of installation and use. A major advantage of this type of construction is that there is very little slurry to dispose of at the end of the project.

Open trenches that are later backfilled or filled with clay, clay-soil, or lean concrete to act as cutoff walls (as for dams) and to confine hazardous wastes are termed slurry trenches. These are widely used with bentonite as the principal slurry additive.

---

**Figure 14-17** New method for constructing a soil-cement mixed wall (SMW). (Courtesy Osamu Taki, SCC Technology, Box 1297, Belmont, CA 94002.)

(a) SMW machine. (b) SMW installation procedure.
Concrete walls constructed using the slurry method can use wale and strut or tiebacks for additional support against lateral movement. Walls have been built with lateral displacements as low as 6 mm (¼ in.); however, excessive lateral wall displacements can occur if the site and soil conditions are not correctly assessed.

Slurry walls are about two times as expensive (per m² or ft²) as walls of sheetpiling or soldier beams and lagging. For this reason, they are used when it is essential that ground loss be kept to near zero and when the walls can be used as part of the permanent construction. They are generally more impermeable than sheetpiling when used as water barriers; however, geotextiles can be competitive for this type of construction.

Basically, slurry construction consists in making an evaluation of the required density and properties of the slurry based on the site soil profile; providing a means to develop large quantities of the water admixture; and, as the excavation proceeds, keeping the ground cavity filled to the necessary depth with the slurry. When excavation is complete, the slurry-filled cavity is periodically agitated to keep the admixture in suspension. Obviously the agitation must be carefully done to avoid wall caving. Next the cavity is filled using a tremie so that the wall is cast from the bottom up. This action ensures a solid wall and, in the case of concrete, a minimum exposure (for both strength and bonding quality) to slurry. The slurry is displaced from the top and saved for use in the next trench section if stage construction is employed. Disposal of slurry (a slime) is the greatest disadvantage of this type of construction.

Slurry construction depends upon two factors for successful performance:

1. Formation of a filter skin or "cake" about 3 mm thick at the interface of the slurry and excavation via gel action and particulate precipitation—the primary purpose of select additives.
2. Stabilization of lateral pressure owing to the dense slurry pushing against the filter skin and sidewalls of the excavation. Slurry density is adjusted by using select additives as well.

Since field performance indicates that walls are usually (but not always) stable with a slurry pressure 65 to 80 percent of the active soil pressure, the filter cake must provide considerable stability [Gill (1980)].

The slurry must be of sufficient viscosity that it does not easily drain out through the sides of the excavation and the filter skin coat. If the filter skin forms reasonably well, exfiltration loss will likely be minimal and the filter skin penetration into the sides of the excavation may be on the order of only a few millimeters where fine-grained soils are supported. A slurry excavation in gravel was reported by La Russo (1963) to have penetrated some 16 m into the surrounding soil, but this may be considered exceptional.

Slurry construction can be used for both caving and cohesive soils and has been used for drilled piers as well as wall and trench construction [O’Neill and Reese (1972), Lorenz (1963)]. Slurry densities up to $\rho = 1.92 \text{ g/cm}^3$ can be obtained using a mixture of barium sulfate (barite of specific gravity $G = 4.3$ to 4.5) and bentonite (for gel action with $G = 2.13$ to 2.18). Other materials, including silt, clay, and fine sand from the excavation, may be included in the slurry mix to reduce the quantity of commercial admixture.

Where the soil is loose, subject to caving, or gravelly, it may first be grouted to obtain some stability before constructing the slurry wall. In some cases, the grout alone may be sufficient to allow the excavation to stand long enough to place wall sections. This may be possible...
owing to the strength gain from the grout and arching action of the soil. It should be evident that when this is done the wall segments must be fairly short.

Cement and finely ground slag have been used in slurry as admixtures to increase $\rho$. At present there are polymers based on carboxymethyl cellulose, xanthan gum, and several polyacrylates that can be used for special site conditions. Generally their costs are six to eight times that of the more common bentonite-based slurries, but if they can be reused sufficiently, their cost becomes competitive.

Commonly, slurry densities of $\rho = 1.15$ to $1.25 \text{ g/cm}^3$ are employed using a mixture of bentonite, barite, and a dispersing agent to reduce the tendency of the clay to floc. The gel is a natural by-product of the admixture, and the basic design element consists in determining the required density of the slurry.

The slurry mixture is a trial process in the laboratory, where water, clay, and any other admixture(s) are mixed by trial until a slurry with the desired density $\rho$ (and gel properties) is obtained. In use it will be necessary to check the slurry density on a regular basis and either agitate or revise the basic formula as required.

Referring to Fig. 14-18a, for a clay excavation without a slurry, the critical depth is as computed in Chap. 11-1

$$H = \frac{4c}{\gamma \sqrt{K_a}}$$

With slurry in the trench and the GWT at the ground surface (not the general case shown), a horizontal force summation for the usual case of undrained conditions (terms are identified on Fig. 14-18a) gives

$$\frac{1}{2} \gamma_{slur} H^2 - \frac{1}{2} (\gamma_s - \gamma_w) H^2 - 2s u H - \frac{1}{2} \gamma_w H^2 = 0$$

Solving for depth $H$, we obtain the following equation, which is usually used in clay:

$$H = \frac{4s u}{\gamma_s - \gamma_{slur}}$$

And with an SF we have the resulting design equation of

$$H = \frac{4s u}{\text{SF}(\gamma_s - \gamma_{slur})} \quad (14-5)$$

Either the safety factor or the excavation depth $H$ can be made larger by increasing $\gamma_{slur}$. This equation was first presented by Nash and Jones (1963) and later verified by Meyerhof (1972).

In cohesionless soils (Fig. 14-18b) the slurry density is obtained (with the GWT at the ground surface) as

$$\frac{1}{2} \gamma_{slur} H^2 - \frac{1}{2} \gamma_s' H^2 K_i - \frac{1}{2} \gamma_w H^2 = 0$$

from which we have the slurry unit weight (usually in g/cm$^3$) as

$$\gamma_{slur} \geq \gamma_s' K_i + \gamma_w(g/cm^3) \quad (14-6)$$

In this equation take $K_a \leq K_i \leq K_o$. 


Cohesionless soil.

Height as required

Slurry

Filter skin (few centimeters)

(a) In clay.

$P_{slur} = \frac{1}{2} \gamma_{slur} H^2$

$P_{slur} = \frac{1}{2} \gamma_{slur} H^2$

$P_w$

Filter skin (few centimeters)

(a) In clay.

(b) Cohesionless soil.

Figure 14-18  Slurry wall stability analysis.
The filter skin or cake that forms at the soil-slurry interface contributes stability to the trench; however, a reliable means of predicting its effect is not available. The beneficial effect can implicitly be allowed for by using a small factor of safety. To ensure skin formation, the slurry head should be 1 m above the water table in cohesive soils and about 1.5 m in granular soils [Gill (1980)]. Carefully note that although you can directly use Eqs. (14-5) and (14-6) for design, for the general case you must be sure the slurry is above the GWT as just noted. You may also, however, compute more accurate replacements for Eqs. (14-5) and (14-6) using a horizontal force summation that considers the actual groundwater location together with the wet \( \gamma_s \) and saturated \( \gamma'_s \) soil unit weights. This is a substantial amount of work for a doubtful increase in project confidence.

Another alternative to the use of Eqs. (14-5) and (14-6) is to use the trial wedge method to obtain a “wall force” \( P_w \), which is resisted by \( P_{slur} \) (of Fig. 14-18). Now equate the wall force to \( P_{slur} \) and obtain the required slurry density as

\[
\frac{1}{2} \gamma_{slur} H^2 \geq P_w
\]

Depending on site geometry, you may be able to use the computer programs SMTWEDGE or WEDGE to obtain \( P_w \).

Example 14-6. Show the effect of slurry density on excavation depth \( H \) in a cohesive soil and using an \( SF = 1.5 \). Other data for this problem are

\[
s_u = 35 \text{kPa} \quad \gamma_s = 18.2 \text{kN/m}^3
\]

Solution. Use Eq. (14-5) with several \( \gamma_{slur} \) values to make a short table. Setting up Eq. (14-5) for these problem parameters, obtain

\[
H = \frac{4s_u}{SF(\gamma_s - \gamma_{slur})} = \frac{4 \times 35}{1.5(18.2 - \gamma_{slur})}
\]

Using this equation now create the following table:

<table>
<thead>
<tr>
<th>( \rho_{slur}, \text{g/cm}^3 )</th>
<th>( \gamma_{slur}, \text{kN/m}^3 )</th>
<th>( H, \text{m} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.10</td>
<td>10.79</td>
<td>12.6</td>
</tr>
<tr>
<td>1.15</td>
<td>11.28</td>
<td>13.5</td>
</tr>
<tr>
<td>1.20</td>
<td>11.77</td>
<td>14.5</td>
</tr>
<tr>
<td>1.25</td>
<td>12.26</td>
<td>15.7</td>
</tr>
</tbody>
</table>

PROBLEMS

14-1. Reanalyze Example 14-1 for \( \phi = 28^\circ, 32^\circ, \) or \( 34^\circ \) as assigned.

14-2. Compute the strut forces of Example 14-1 using simple beam theory (refer to Fig. 14-6) and compare your answer to those output in Example 14-1 from computer analysis.

Ans.: Strut 1: 57 kN

14-3. What is the critical depth using an \( SF = 1.5 \) for the first excavation stage of Fig. 14-2 if \( \gamma_s = 15.72, \phi = 30^\circ \), and cohesion = 10 kPa?

Ans.: \( D = 5.88 \text{m} \)

14-4. Using a copy of data set EX141.DTA, revise and try a PZ27 sheet-pile section. Comment whether this section will be satisfactory to use.
14-5. Using the output of Example 14-1, redesign the wales using the lightest pair of I sections you can find.

14-6. Using the data sets provided for Example 14-3, run all the stages and obtain the node displacements for each stage and the final displacement profile. Numerically integrate the displacements using the average end area method and estimate the ground loss profile. Make your best comparison with the Peck method and comment on what you would do.

14-7. Repeat Example 14-4 but revise the sheet-pile section to the next larger section. Compare the node displacement to those given in the text. If assigned by the instructor, make an estimate of ground loss.

14-8. What is the $N_c$ factor for $D/B = 0.9$ and $B/L = 1.0$ and also for $D/B = 1.3$ and $B/L = 0.25$? What is the significance of $B/L = 0$?

14-9. What can you use for $D$ in Example 14-4 if the excavation width $B$ changes to 15 m? Note that $B/L$ will also change.

14-10. Resketch the plan flow net of Fig. 14-14 so that there is at least $N_f = 11$ and recompute the flow quantity $Q$. Is there a significant difference?

14-11. Redo Example 14-5 if $k = 2$ m/day.

14-12. Refer to Example 14-5 and Fig. 14-14 and estimate the flow quantity for the following as assigned (use the same $k$, $H$, and $h_w$ as in that example).

<table>
<thead>
<tr>
<th>$a \times b$, m</th>
<th>$D$, m</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) 75 \times 110</td>
<td>110</td>
</tr>
<tr>
<td>(b) 175 \times 295</td>
<td>300</td>
</tr>
<tr>
<td>(c) 65 \times 95</td>
<td>90</td>
</tr>
<tr>
<td>(d) 165 \times 180</td>
<td>240</td>
</tr>
</tbody>
</table>

14-13. Make a new table as in Example 14-6 if $\gamma_s = 19.25$ kN/m$^3$ instead of 18.2 in the example. Can you draw any conclusions?

14-14. Design the mix proportions to provide a slurry of $\rho = 1.25$ g/cm$^3$. Use a mixture of water, bentonite, and barite. Use 20 percent bentonite based on total mixture weight. Partial answer: percent barite = 12.1.

14-15. Design a slurry mixture for the wall of Fig. 14-18b if $h_w = 2$ m, $\gamma_s = 17.9$ kN/m$^3$, and the trench is 10.0 m deep. Hint: Assume a value of $G$ so as to compute the saturated unit weight of sand below water level or take $\gamma'_s = 9.5$ kN/m$^3$.

14-16. You are the project engineer on a slurry wall project. A wall segment is 3.2 m long $\times$ 1 m thick $\times$ 15 m deep, and the steel bar cage has a mass of 1508 kg. You observe that the concrete trucks deposit 47.1 m$^3$ into the cavity. Is this wall section satisfactory? Comment on the several factors that may account for any discrepancy so that you can justify your action either to remove or accept the wall section.