Finite- element analysis for Braced excavations

Omid Ghiasi Tabrizi

Department of Civil Engineering, Sofian Branch, Islamic Azad University, Sofian, Iran
* Corresponding Author's: omid ghiasi tabrizi
Ghiasi_Omid1367yahoo.com

Abstract: The finite-element (FE) method can be used to analyze a braced excavation. Both the finite element of the elastic continuum and the method of the sheet pile/beam on elastic foundation computer program in the Appendix can be used. Both these methods will be briefly discussed, with some of the limitations and disadvantages of each presented. Either method can be used for stage construction and work best in an interactive computer environment. The methods can be used for either braced (struts and/or rakers) or tieback construction. Both methods have best application for making rough predictions of expected field performance in terms of wall movements and ground loss.

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Introduction:

Neither of the FE methods is likely to predict wall movements accurately except as a happy coincidence for several reasons:

1. The forces are distributed through a flexible system; the earth quantities involved are large.
2. Methods of wall and bracing installation tend to make the wall pressure model indeterminate; soil creep tends to produce transitory lateral pressures.
3. Soil properties are not accurately known and response to this type of loading is uncertain.
4. Accuracy and care in wall construction varies widely. 5. Perimeter loads are often unknown.[1]
6. Installation and measurement accuracy of monitoring system.

The finite element can model the assumed conditions rather well if enough time is taken and the computer program has sufficient options. The disadvantages include:

1. Requires rather accurate assessment of soil parameters E_s, μ, and a program option to modify E_s where tensile stresses develop. An option may be used to adjust the shear modulus G as different from that obtained from E_S where there are saturated cohesive soil strata. Since output directly depends on E_s best answers are obtained when measured data are available to back-compute input. [See Mana and Clough (1981).][2]
2. Requires some means (or interpretation of output) to be able to incorporate soil weight needed to produce the wall movements and ground loss; self-weight tends to compute vertical settlements which have already occurred during deposition.
3. Requires a massive amount of input data on elements and for each stage.
4. Careful coding is required to enable partial reuse of input data for succeeding stages.
5. Essential output is voluminous.
6. May compute ground loss in reverse of what is usually observed.
7. Requires care in locating the model boundaries; if too near the boundary will attract force; if too far may increase bandwidth of matrix and/or greatly increase the computing time/run.
8. Computation time/stage is relatively large-even for modest model dimensions.
9. Requires a program option to incorporate bending-translational FE of wall with translation FEs of soil. [3]

Important points:

The finite-element method using the beam on elastic foundation/sheet-pile program requires making some assumptions but the soil-pressure diagram is that normally computed using K_a (or other K as deemed desirable). The brace forces can be estimated from the force necessary to contain the lateral earth pressure with a suitable factor of safety. This is obtained from the computer output of the preceding stage. Alternatively the brace force is computed from some type of pressure envelope. The computer output is not lengthy and a stage run should take from a few seconds to not over 3 to 4 min depending on the computer system. Similarly the input data are not considerable and about one-half is directly reusable from stage to stage. These several factors make this method particularly attractive[4]. The disadvantages include:

1. The method works best in an interactive environment.
2. Several trials may be required to obtain what may be the best output for any stage.

3. Certain assumptions may be debatable.
4. The method works best when one is back-computing to measured data (also for the alternative FEM).

The latter FEM has more application for the smaller offices and computers since a wall can be typically analyzed in about two days for a number of stages (and trials within a stage). Taking this into consideration the method will be outlined in the following steps:

1. Draw the wall configuration and soil profile to a reasonable scale and showing the soil parameters γ, c and ϕ; compute and show the pressure coefficients K_a, Ko and K_p. Tentatively locate nodes and brace points.
2. Select the structural member for the wall. If a rolled section is used, it can be readily changed since a single line entry does this. The member may be a steel sheet pile, concrete wall (as from slurry construction), or a system of soldier beams and wood lagging. The bracing may be either struts or tiebacks. For the tiebacks the horizontal force component is the input parameter. Including the vertical force component in a P-Δ effect is probably not warranted, since wall friction from the horizontal component is neglected.

Estimate the modulus of subgrade reaction for the stages from the dredge line down and for the retained earth for the several stages.

The stage analysis proceeds essentially as follows:

Stage 1. Treat wall as a cantilever sheet pile for some depth of excavation. At this point analyze for the optimum depth by observation of the top movements of the pile. Use the active earth (or other) pressure to the dredge line. Estimate k, for the soil below dredge line for passive wall resistance.

Estimate k_s Apply the first strut or tieback. The force should be larger than the driving force $\times$ a suitable safety factor. Apply a modulus of subgrade reaction behind the wall to restrict the backward lateral movement to a "reasonable" value. This modulus should extend to the dredge line. Solve this wall-soil system. The sum of the nodal movements is the current lateral position of the nodes with reference to the initial wall position. A lateral modulus behind the wall is necessary to adsorb the unbalanced strut force, else the program would produce the necessary stability resistance from the soil below the dredge line but the deflections would be enormous.

Stage 3. Excavate the next depth. Input the strut force and restraining sub-grade modulus above the previously existing dredge line; input the lateral soil pressures from the ground surface to the new dredge line; input a value of k, for dredge-line soil. At this stage we may wish to reduce the strut force by 10 to 20 percent for soil creep. Inspect the output for forward deflection. If the net movement is back, the model is not good and should be revised. Recycle as necessary so that some reasonable movement is obtained toward the exca-vation.

Stage 4. Input the next strut force with additional subgrade modulus behind the wall to the dredge line. At this point consider if the overburden confinement is sufficient to produce a vertical force in the confined soil from the upper strut load of

$V=P \times K$

and then an outward push at the nearest adjacent nodes of

$P_h=V \times K$

Initially both K values might be K_o but one could use something else including K_a or K_p; note $\sum P_h \leq P$, however. A force at further nodes might be $P_h/2$. this P_h would be expected to dissipate from creep to a lesser value (perhaps $\frac{1}{2}$) at subsequent stages.

An alternative to this compulation is to use a larger earth pressure to allow for the wall pressing back from the strut forces and developing a lateral pressure that is larger than the active value. This pressure diagram might be obtained stage to include stage N.

Final wall deflections are the cumulative sums of the deflections from each stage. Instant deflections are the cumulative deflections up through that stage. These deflections can be used with the procedure of Sec. 14-4 to obtain an estimate of the ground loss.

Step 1 Code the wall. Use Rankine K_a and K_p and K_o= 1+- sin ϕ. Also:

(a) Obtain the strut forces (both design and final) from paper as follows:

Strut	Design (initial). kips	Measured (final from plot),
1	170	90
2	493	215
3	605	300
4	566	230
5	615	270

Lateral strut spacing s= 12.0 ft

Note that here we will use the "design" P rather than obtaining the lateral earth-pressure force from computer output and applying a safety factor.

(b) Soil springs behind wall are taken as

Element 1: 2.3 and 2.5 to stage 4 then 5.0 for stiffening effect

Element 2: 2.5 and 5 to stage 4 then 5.0 kcf

Element 3: 5.0 and 7.5 Stage 6 and later elements 1 to 3: 7.5 kcf

Elements 4 and 5: 10 kcf Element 6: 15

Elements 7: 20 Element 8: 30 and 40 (each end)

Elements 9 to 14: 50 kcf Element 15: 5000 kcf (rock)

(c) Soil springs in front of wall:

$$k_s = 12(\bar{q}N_q + 0.5\gamma BN_\gamma)$$

for $\gamma = 35^\circ$ $N_q = 33$ $N = 40.7$

$\gamma' = 0.036 \text{kof}$

abtain $k_s \cong 9 + 14Z^t$

(d) Compute soil pressures using K_a = 0.27 for 8 ft (obtain 0.212 ksf at D.L. as shown on Fig. E13-4 stage 1).

(e) Obtain sheet-pile data:

E = 30×10^3 × 144 = 4320 000 ksf

(f) Data input for first stage:

Line		Item							
1-2		Title and units							
3	32	15	0	1	1	0	0	0	
4	4	0	0	0	5	0			
5	4320000	0.0135 1.0		1.0		0		0.8	(dredge-line reduction (actor)
6	9	14		1	(k,)				
7	2.0	2.0	4.0	4.0	5.0	5.0	5.0	3.0	5.0
8	5.0	2.0 3.0		3.0	5.0	3.0	3.0	5.0	[H(I)]
9	0.0	0.053 0.106		0.106	0.212	0.0	[PRESS(I)]		

These are data for stage 1.

Step 2 Stage 2: Apply strut 1

(a) Inspect output to see if deflections are realistic (in practice change dredge-line location and redo if not satisfactory); here we will continue since problem is more fixed in scope.

(b) Duplicate lines 1 to 8 with NNZP = 1 and IBRAC = 3

Line Item

9 2.5 2.5 2.5 5.0 5.0 7.5 (IBRAC w/3 sets of entries)

10 0.0 0.053 0.106 0.212 0.212 (IPRESS(I))

11 6 1 -14.2 (Strut force based on design of 170/12 = 14.2)

This is input of stage 2.

Step 3 Stage 3: Excavate to node 8 (see Fig. E14-3)

(a) Compute additional soil pressures including hydrostatic effect:

Q_a = 0.212 + 0.0356 x 0.27y + 0.0624y

obtain 1.508 ksf as shown on figure.

(b) Obtain NNZP entries of 4 as follows:

P(6) = 12.0 kips (85 percent of 14.2 as a small loss)

P(8) = 12.0×Ko×Ka = 1.39 kips

$$P(10) = P(12) = \frac{P(8)}{2} = \frac{1.39}{2} = 0.7 \text{ kip}$$

(c) Data input

Line Item

1-9 same except NNZP = 4, JTSOIL = 8 and IPRESS = 9

change line 6 to 4.5 7.0 1.0 (ks for silt seam)

10 0.0 0.053 0.106 0.212 0.572 0.932 1.292 1.508

11 0.0 (9 IPRESS entries)

12 6 1 -12.0

13 8 1 1.39

14 10 1 0.7

This is input of stage 3.

Step 4 Stage 4: Apply strut 2 (P = 493/12 = 41 kips)

(a) Nonzero P-matrix entries as follows:

P(6) = 0.9×12 = 10.8 kips

P(8) = 1.39

P(14)= -41

P(12) = P(16) = 41×Ko ×Ko = 7.58 (at this depth Ko not Ka)

$$P(10) = 0.7 + \frac{7.58}{2} = 4.49 \text{ kips}$$

(b) Adjust line 3 for NNZP = 6 and IBRAC = 7

Duplicate and change lines as required and add additional lines for additional P-matrix entries. The remaining stages through stage 9 are handled somewhat similarly with creep adjustments in strut forces made (but not to less than measured values). A very stiff spring is placed behind element 15 in stage 9 to account for the sheeting being in rock. The final outcome for the author's runs are as follows (units of feet):

Node	Stage	1	2	3	4	5	6	7	R	9	Sum
1	X(2)	0.016	-0.166	-0.153	-0.087	-0.112	-0.072	-0.064	-0.063	-0.055	-0.756
8	X(16)	0.0	0.002	0.063	-0.022	0.070	0.041	0.047	0.054	0.040	0.295
9	X(I8)	0.0	0.001	0.037	-0.012	0.087	0.043	0.081	0.073	0.046	0.356
13	X(26)	0.0	0.0	-0.002	0.001	0.016	0.003	0.141	0.032	0.014	0.205

The top moves back into soil 0.756 ft (also in the Lambe paper, but not this much). The rest of wall moves into excavation (measured values were some larger than these, but these results are not off by an order of magnitude-more like 50 percent). The methodology is reasonable and the wall moves in the correct directions. The outcome might be less certain if there were not measured data for comparison).

Conclusion

1. When a cofferdam is located either over, or in, a soft clay stratum the clay may flow beneath the sheeting into the excavation if sufficient soil is removed.
2. The critical excavation depth $H = H_c$ including a safety factor F is obtained by inserting this value of Q_3 and Q_3 into Eq. (2-46) and rearranging to obtain

$$H_c = \frac{\gamma h + 2s_u}{\gamma \times F}$$

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