Computer Modeling of Excavation in Cohesive Soils

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ABSTRACT: Most soils have stiffness properties characterized by nonlinearity and inelasticity. Soils are often heterogeneous and may be subject to complicated stratigraphy. The presence of ground water and the localized pressures of that ground water often have a major effect on the design of support systems for deep excavations in predominantly fine-grained soils. Because of the interaction of nonlinear soil response with the behavior of structural entities and components, the designer's ingenuity lies in the choice of a computational method that most reasonably approximates a real life problem but still permits a solution to be obtained. In the absence of a convenient source of codification of detailed hand or elementary computer calculation routines, the teaching methods in excavation design currently include procedures that could be carried through by hand or with a calculator. Hand calculations with the design of support systems for some excavations may require prodigious amounts of labor and are usually accomplished by dividing the overall task into smaller segments, each requiring considerations of only a few design parameters. Since computational capabilities of digital computers have, for all practical purposes, surpassed engineers' ability to model materials with which they have to deal, the key to improving educational effectiveness with courses dealing with excavation design certainly rests on multiplying the time the student actually spends learning basic design skills through working out a comprehensive range of problems.

The basic objective of the paper is to illustrate with the software "CLAYEX" to both practitioners and students the advantage of using a computer to perform a design process that is otherwise lengthy and tedious. The "CLAYEX" software package is an interactive program developed for the complete design of support systems for excavations made in fine-grained soils. The program uses recent design practices, computerizes manual methods of design to achieve improved speed as well as reliability of calculations, and runs on IBM-PC or other compatible microcomputer hardware using DOS operating system. The program follows a minimum learning curve strategy and achieves a high degree of user friendliness through an easily followed format and highly explicit data prompts.

Primary emphasis in developing the software is placed on assessment of the effect of complex as well as nonuniform soil conditions and such random factors as the force with which the structural members are wedged home and time elapsed between excavation and installation of support members in order to achieve the safest and most economical design. The apparent lateral pressure distributions, used in design of support systems, are taken as the semi-empirical envelopes covering all the random distributions obtained from the field measurements under varying soil conditions. The software considers effects of heaving of bottom of excavation, depth of penetration of soldier piles or sheetpiles below the bottom of excavation, and settlement as well as lateral movement of the adjacent ground. For each support system, soldier piles and laggings or

sheetpiles, the software provides several choices of steel members (struts, piles, and wales), based on the requirements of the American Institute of Steel Construction (AISC) ASD Manual.

INTRODUCTION: The design of support systems for excavations in soft ground is perhaps the most critical operation from a technical and cost standpoint. Rolled steel soldier piles and timber laggings, as well as steel sheetpiles with continuous interlocks on the sides are extensively and typically used[7]. Internal bracing systems consist of multiple levels of longitudinal horizontal beams or wales placed adjacent to the wall and transverse horizontal compression members or struts. These bracing systems are arranged and designed to carry the ground loads imposed on wall supports across the excavation. The design of internally braced excavation is still essentially empirical. Design methods were described by Terzaghi and Peck[12] and were modified and expanded by Peck[10]. Recent practice is based on a method outlined by Peck and on modifications to this method suggested by Peck, et al.[11], Brahma and Biddlecome[2] and Brahma[3,4]. Considerations during design are given to the lateral pressure distribution on the wall, local as well as overall stability, base failure, loadings (bending moments, shear and axial forces) on each structural members, and anticipated displacements adjacent to and below excavation. Adequate initial drainage measures are, more often than not, undertaken towards the removal of the water pressure from free-draining elements, such as sand layers carrying hydrostatic pressure or fissures carrying free water, in the soil behind the cut.

BASE FAILURE: Excavation removes a mass of soil and water producing a reduction of total stress at the bottom of the cut. The soil outside the walls acts like a surcharge and causes the soft to medium soil below the bottom of excavation to move upward, unless a relatively stiff soil stratum lies near the base of the cut. Such movements associated with a deep cut made in soft soils are particularly large. The rise of the base is invariably accompanied by an inward movement of the soil below the excavation. The base failure of an open excavation is ordinarily analyzed using procedures for foundation stability[1,5,6]. Base failure occurs when the surcharge pressure ($\gamma H + q$), due to the uniform surface loading (q) and the weight of the column of soil (unit weight γ) of height H adjacent to the excavation, becomes equal to the net bearing capacity of the soil underlying the bottom of excavation (Figure 1). Accordingly, the factor of safety against base failure (FS_b) is given by

$$FS_b = N_{cb}\, S_u / (\gamma H + q)$$

where, N_{cb} = bearing capacity factor
S_u = undrained shear strength of the soil near the base of the cut, using the extension strength test
H = height of the excavation

For excavations made in stratified soil deposits, an weighted average undrained shear strength, based on results of extension strength tests conducted on soils from the zone from 2.5B above the base of the cut to 0.707B below the base of the cut, is substituted for S_u in the above equation.

LATERAL EARTH PRESSURE: The magnitude and distribution of lateral earth pressures, that act on a lightly prestressed braced wall, depend on soil conditions, stiffness of the support system, lateral movements, and construction sequence as well as construction procedures, such as preloading of struts. For a well constructed strutted excavation, the maximum movement usually occurs at the current excavation level or just below as the excavation proceeds. A properly placed

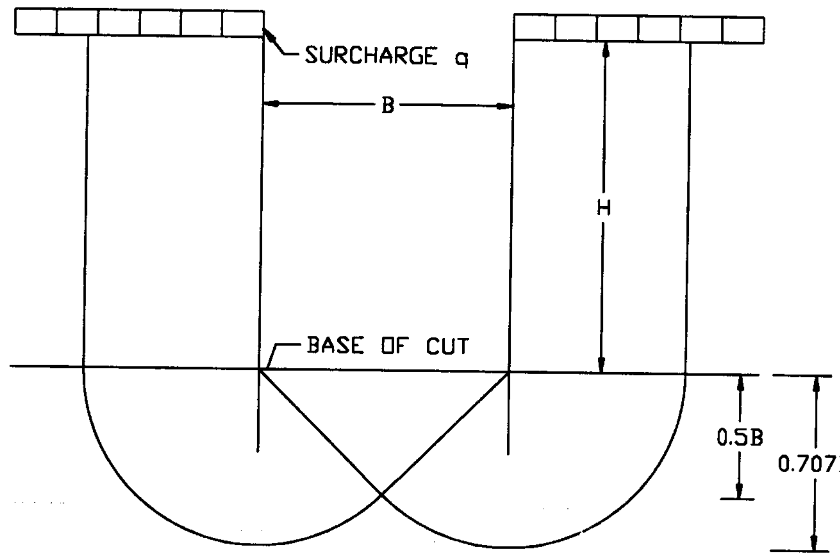


Figure 1: Base Failure

strut fixes the movement at the excavation level so that further excavation causes little, if any, additional movement at the level of the strut. The wall movement corresponds roughly to translation or rotation about a point near the top of the wall causing an arching active condition. The conventional approach for development of design loadings for internally braced temporary support systems is to use the apparent earth pressure diagrams proposed by Terzaghi and Peck[12] as shown in Figure 2.

The behavior of an excavation made in cohesive soils depends to a large extent on a stability number N, where, $N = \gamma H/S_u$ and S_u is the average undrained shear strength. Strains associated with excavations having a stability number less than or equal to 4 are essentially elastic and the apparent earth pressure envelope shown in Figure 2c is utilized to estimate loads on support members. While the width of the envelope varies between P2 (equal to $0.2\gamma H$) and P3 (equal to $0.4\gamma H$), the software "CLAYEX" uses an average width of $0.3\gamma H$. If the stability number is greater than 4 and the lateral force per unit length of excavation is greater than $0.3\gamma H^2$, the apparent earth pressure envelope having a maximum width of magnitude P1 equal to $\gamma H(1-4mc_u/\gamma H)$ as shown in Figure 2b is used for estimating member loads. If this is not the case, the apparent pressure envelope shown in Figure 2c is used even though the stability number exceeds 4. If the cohesive soil is normally consolidated, extends to a great depth below the excavation, and has a high potential for bottom heave, the wall deflection beneath the excavation undergoes arching with accompanied transfer of earth pressure to the stiffer part of the support system. Consequently, the width of apparent earth pressure diagram in Figure 2b increases considerably and a value of m, which is generally assumed to be 1.0, as low as 0.4 is used in ascertaining the magnitude of P1.

LOCAL AND DEEP SEATED FAILURES: Local failures occur below the excavation level immediately adjacent to the wall. The lateral pressures on the wall together with the reduction on

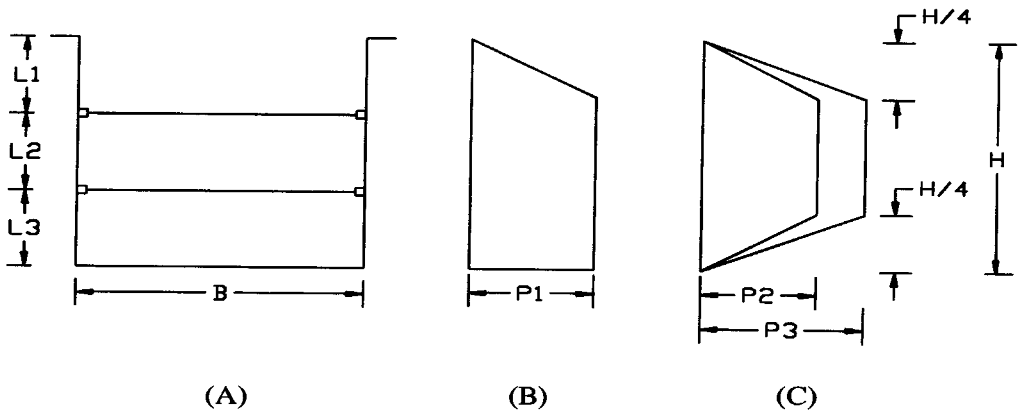


Figure 2: A) Excavation Support System; B) Apparent Pressure Diagram for N ⟩ 4; C) Apparent Pressure Diagram for N ⟨ 4

total stress during excavation may cause local yielding of the soil, resulting in excessive inward deflections of the wall and partial loss of lateral ground support. The depth of embedment of the wall below the base of the excavation should be sufficient to prevent such failures of the system of bracing with accompanied excessive movement. If the materials below the bottom of excavation are very soft to soft cohesive soils and are unable to develop required passive resistance, the wall, penetrating to a minimal depth of 20 percent of the excavation depth, is designed as a cantilever below the level of the last strut above the base.

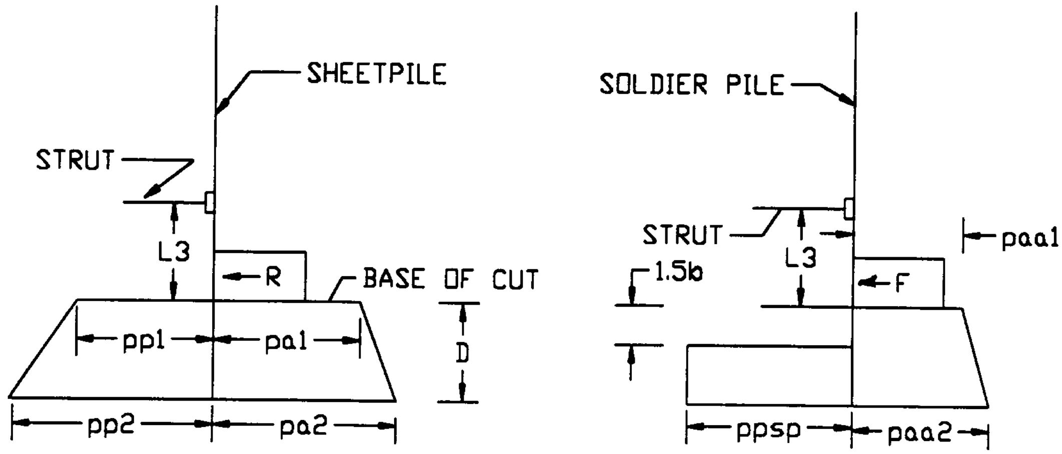
For sheeted excavation in cohesive soils (Figure 3A), the active earth pressures pa1 at the base of the excavation and pa2 at the tip of the sheetpile are [γH -2C] and [γ (H + D) - 2C] respectively. The passive earth resistance in front of the wall increases from pp1 equal to 2C at the base to a maximum pp2 of [γD + 2C] at the tip. In competent soils, the minimum depth of penetration is determined by satisfying equilibrium of all lateral forces acting against the wall, including the necessary reaction R to be equal to 0.5 L_3 P1 to be developed at the base of the excavation. Accordingly,

$$R + 0.5\, D\, (pa1 + pa2) = 0.5\, D\, (pp1 + pp2)$$

Soldier pile walls are not continuous below excavation level. Utilizing the procedures outlined by Brooms[8], which consider the effect of arching in soils, the uniform passive resistance of 9S_u is neglected to a depth of 1.5 pile diameter (width) b. Figure 3B shows the distribution of both active and passive earth pressures acting on the embedded portion of the soldier pile. The passive resistance ppsp remains constant from a depth of 1.5b below the base of the cut to the tip of the pile. The active earth pressures paa1 at the base of the excavation and paa2 at the tip of the sheetpile are [γH - 2C] and [γ(H + D) - 2C] respectively. If the reaction F to be equal to 0.5 L_3 P1 S is to be developed at the base for a horizontal pile spacing of S and for a pile penetration of D below the cut, then it follows from the equilibrium of all lateral forces:

$$F + 0.5\, (paa1 + paa2)\, D\, S = ppsp\, (D - 1.5b)\, b$$

Should the ground adjacent to the excavation slope upward or the excavation be underlain by soft soils, the overall stability of the excavation is analysed using the classical circular arc method and analytical method of wedge stability.



(A) (B)

Figure 3: Depth of Penetration; A) Sheetpile Wall; B) Soldier Pile Wall

BRACING SYSTEM: With the exception of excavations where the embedded portion of the wall undergoes excessive inward movements leading ultimately to the collapse of the system of bracing, the failure of most excavations is due to the initial failure of one of the struts, resulting in a progressive failure of the whole system. The load in any strut is designed to be equal to the earth pressures acting on the wall over a rectangular area extending halfway to the adjacent struts both vertically and horizontally. Deflection of relatively flexible members, such as sheetpiles, soldier piles, wales, and laggings, results in a reduction of earth pressure near the center of the span and a concentration of earth pressure at the supports due to arching of soil. Accordingly, moments, axial forces, shear forces in soldier as well as sheet piles and wales are computed using 80 percent of the apparent lateral earth pressures and the full hydrostatic pressures[9]. The apparent lateral earth pressures used for the design of highly flexible timber laggings are reduced by 50 percent to account for the transfer of loads by soil arching, or shear, or both. The laggings, soldier piles and sheetpiles are designed for the maximum bending moments. The struts are designed as compression members and the wales are designed for the maximum bending moments and axial forces. The design of all steel members follows the provisions of the AISC specifications.

DESIGN EXAMPLE: For the subsurface condition consisting predominantly of stiff clay (Figure 1), details of a particular sheetpile bracing system are as follows:

DESIGN INPUT: Support system: Steel sheetpiles, struts and wales; L1 = 3.048m (10.0 ft), L3 = 1.524m (5 ft), and L2 = 4.572m (15 ft); Longitudinal spacing of struts = 3.66m (12ft); H = 9.144m (30 ft); B = 4.572m (15 ft); Length of excavation = 121.9m (400.0 ft); Subsoil: stiff clay, γ_m = 18.86 KN/m^3 (120 pcf), ϕ_u = 0; S_u = 47.88 KN/m^2 (1.0 ksf); Number of struts and wales = 3; Struts and wales are placed vertically at depths of 0m (0 ft), 3.048m (10 ft) and 7.62

(25 ft) below the surface; and Structural steel: A36 for struts and wales.

DESIGN OUTPUT:

Depth of embedment of sheetpile = 0.41m (1.35 ft)

Maximum strut load = 697.40 KN (156.80 Kips)

Maximum sheetpile moment = 107.48 KN-m (24.15 K-ft)

Maximum wale moment = 380.2 KN-m (280.41 K-ft)

Maximum axial force on wale = 332.7 KN (74.78 kips)

Factor of Safety against Bottom Heave =2.00

Steel sheetpile section: SZ18

steel strut sections:

W 205 mm X 60 kg/m (W 8 in X 40 lbs/ft)
W 255 mm X 58 kg/m (W 10 in X 39 lbs/ft)
W 305 mm X 60 kg/m (W 12 in X 40 lbs/ft)
W 355 mm X 72 kg/m (W 14 in X 48 lbs/ft)
W 410 mm X 85 kg/m (W 16 in X 57 lbs/ft)
W 460 mm X 97 kg/m (W 18 in X 65 lbs/ft)
W 535 mm X 109 kg/m (W 21 in X 73 lbs/ft)
W 610 mm X 92 kg/m (W 24 in X 62 lbs/ft)
W 690 mm X 125 kg/m (W 27 in X 84 lbs/ft)
W 765 mm X 148 kg/m (W 30 in X 99 lbs/ft)
W 840 mm X 176 kg/m (W 33 in X 118 lbs/ft)
W 915 mm X 201 kg/m (W 36 in X 135 lbs/ft)

Steel wale sections:

W 305 mm X 203 kg/m (W 12 in X 136 lbs/ft)
W 355 mm X 162 kg/m (W 14 in X 109 lbs/ft)
W 460 mm X 158 kg/m (W 18 in X 106 lbs/ft)
W 535 mm X 150 kg/m (W 21 in X 101 lbs/ft)
W 610 mm X 155 kg/m (W 24 in X 104 lbs/ft)
W 690 mm X 170 kg/m (W 27 in X 114 lbs/ft)
W 765 mm X 173 kg/m (W 30 in X 116 lbs/ft)
W 840 mm X 176 kg/m (W 33 in X 118 lbs/ft)
W 915 mm X 201 kg/m (W 36 in X 135 lbs/ft)

CONCLUSIONS: The software "CLAYEX", if properly used, provides several alternative solutions of temporary excavation support systems consistent with the specified range of soil response and structural constraints more quickly and at lower cost. Teaching methods utilizing the software would develop students' integrative, analytical, innovative, synthesizing and contextual capabilities.

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BIOGRAPHY

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