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PREDICTION OF SOIL BEHAVIOR AND FACE STABILITY IN DEEP EXCAVATIONS SUPPORTED BY SOLDIER PILE WALLS

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SUMMARY

Deep excavations utilizing soildier pile lagging system for support are apt to experience various forms of instability and difficulties during excavation that are related to soil behavior at and in the immediate vicinity of the face of the excavation. Good prediction of the soil behavior is most essential to devise effective and practical combative measures in the design and execution of the excavation in such instances. This paper reviews factors that are inducive to these instability problems and current methods used for prediction of soil behavior in deep excavations in general. A procedure is developed for prediction of the soil behavior as related to the face instability problems in the case of using soldier pile-lagging system. This procedure uses the stability number in terms of remoulded undrained shear strength determined from index properties of the soil, and the pertinent information is presented in a graph form. In connection with the above treatises, a rational geotechnical basis is established for determination of lagging designs.

INTRODUCTION

Ground movements inevitably occur as a result of performing a deep excavation and are responsible for settlement of the adjacent ground behind the excavation. In most urban settings ground movement control, therefore, becomes the most important consideration in deep excavations. In deep excavations, ground movements are generally considered to result from movements of the support system itself, heaving of the base of excavation, settlement due to lowering of ground water level, and ground losses resulting from construction-related activities and other sources [1, 2, 3].

Soldier pile walls consist essentially of a series of steel wide flange beams supported by struts or tie-backs and installed as vertical piles in a single row, typically spaced in the range of 1.5 to 3 meters on centers. The cut face which is exposed between the piles is crossed by laggings installed at various stages as the excavation advances in depth. Because the excavation must be done in an unsupported condition until the lagging is put in place, excavations using soldier piles for lateral support are apt to experience various forms of instability problems during excavation at and in the immediate vicinity of the excavation face, and the nature and extent of these instability problems depend largely on the behavior of the soil. Ground loss due to what happens at and in the immediate vicinity of the cut face can account for a significant part of the overall ground movements responsible for settlement of the ground surface behind the excavation. Recent case studies [4, 5] reveal instances where such local instability was responsible for triggering a global instability of excavations supported by soldier pile walls. Therefore, the local instability problems at the excavation face become a critical concern in excavations supported by soldier pile walls.

This paper offers an approach for prediction of soil behavior which can be useful for assessing the potential instability problems at and in the immediate vicinity of the face of excavations supported by soldier pile walls. In connection with development of this approach, lagging is treated as an essential element to maintain stability of the excavation face and a rational geotechnical basis is developed for design of the lagging system.

SOURCES OF INSTABILITY AT AND NEAR THE CUT FACE OF EXCAVATIONS

Various ground condition-related problems are commonly experienced in soft ground tunneling, and the literature citing these problems over the years is extensive. Ground is usually classified as firm, raveling, running, flowing, squeezing or swelling to describe and predict the ground behavior in tunneling [6, 7, 8]. There are certain similarities in ground instability problems between soft ground tunneling and deep excavations utilizing soldier pile wall support, because both cases must deal with unsupported face of cuts for various durations.

Piles are installed either by driving or drilling. Ground disturbance from pile driving and drilling has been well recognized [9, 10, 11]. Clough and Chameau report of a significant level of strain being developed by sheet pile driving over a distance of several meters from the pile. Collapse and swelling of the side of drilled holes are common sources of ground loss and disturbance. The inevitable stress reorientation resulting from creation of the cavity by drilling of holes in the ground is another form of ground disturbance [12]. In clayey soils strength recovery upon disturbance of this type is generally time dependent (thixotropy, consolidation), and the prevailing strength at the time of excavation can be significantly lower than that was found during the initial investigation stage.

The face of the excavation is exposed for various durations until lagging has been installed. There are troublesome soils that hinder the excavation in such a setting. These may include collapsible soils (loose sands and gravels weakly bonded by silts), saturated soft and weak silts or dispersive soils. A case study has been reported [13], where the presence of dispersive soil was responsible for development of a flow type failure of excavation shoring using soldier piles. Soft and weak clays pose a different kind of problems involving creeping and squeezing. Swelling ground is also troublesome in excavation, and this type of occurrence is common in overconsolidated soils and highly plastic soils. Complete filling of the space between the excavation line and the lagging is not always attained, and the open space not only promote the ground settlement behind the wall but also cause more disturbance to the ground immediately behind the wall by allowing the ground to move until the support system fully takes over the support function.

Support systems for deep excavations normally require lateral supports at several elevations, and the lateral supports are preloaded to various levels of the computed support loads before the excavation continues into the subsequent stages. Finno et al. [13] report of case studies in which they determined from field measurement data the developed strain fields in the ground behind the wall at each stage of the excavation, and show shifting of the strain fields and changes in magnitudes of strains resulting from application of lateral support loads at various levels. Fig. 1 depicts these results.

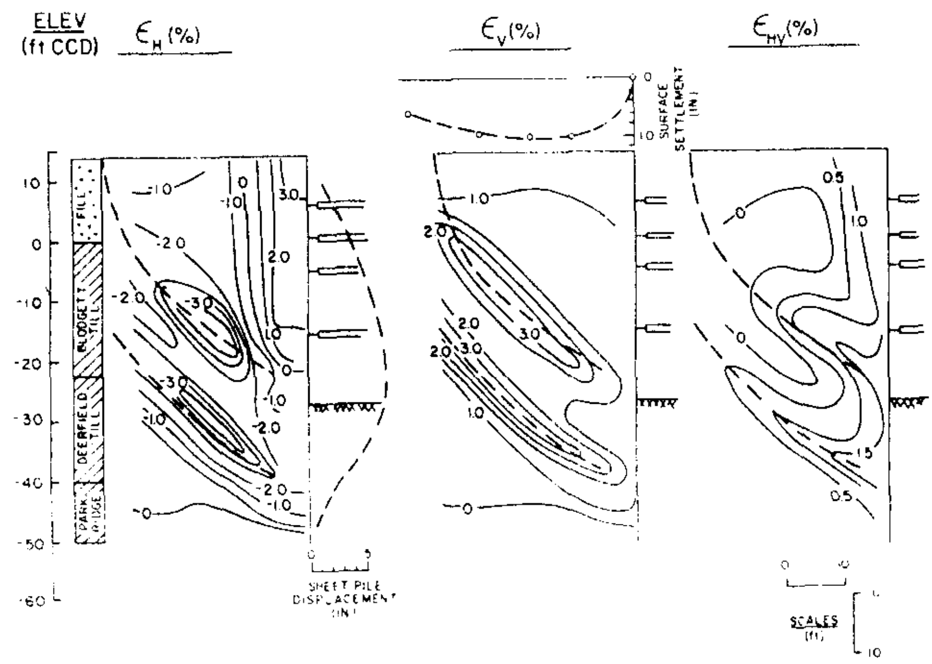


Fig. 1 Strain Contours behind Braced Excavation (after Finno et al.)

The evidence of large horizontal compressive strains immediately behind the wall and tensile strains at a short distance away from the wall in Fig. 1 suggests that the ground immediately behind the wall undergoes extensive disturbance associated with the shifting of the strain fields and changes in strain magnitudes in a complex manner during the course of the excavation as the support loads are applied. The large horizontal compressive strains immediately behind the wall will tend to squeeze soft and weak soils through the gap between the lagging or into any open space between the pile and lagging. Swanson and Larson [4] report of a failure of soldier pile-lagging system due to soil squeezing, which occurred in 1988 with one of the Washington Metro projects. In this reported case, lagging members were dislocated and pushed to various orientations by soil squeezing

Finno et al. [14] also obtained field measurements of the direction of the soil displacements behind the wall during excavation, and found that the horizontal movement of the ground is not in a direction perpendicular to the wall alignment but is inclined 15 degrees or so toward the opposite direction of the excavation. The divergence of the horizontal ground movement behind the wall in direction from normal to the wall alignment could be related to the unbalanced and nonuniform stress fields resulting from directivity in execution of the excavation. The tendency of the horizontal soil displacement behind the wall to divert from the normal direction to the wall alignment, coupled with the possible erratic pattern of the soil movements behind the wall, can cause the lagging to shift in position. Lagging construction, which utilizes timbers or steel members of small crossway dimension, would lack torsional rigidity to resist this type of soil movements.

All discussions presented above point to the conclusion that the soil immediately behind the soldier pile wall in deep excavations exists in a much more extensively disturbed condition than we might have conceived in the past. Recent case studies on shoring failures [4, 5] conclude that the main cause of the failures is in the lack of correct characterization of the soil conditions and poor prediction, or lack of understanding of the soil behavior as affected by the excavation.

SOIL BEHAVIOR IN EXCAVATION SUPPORTED BY SOLDIER PILES

Predictions are made routinely in various forms in geotechnical engineering. They range from simple predictions based on prior experiences or simple physical or index properties of the soil to predictions based on highly sophisticated analyses. Accuracy of a geotechnical prediction is not always proportional to the degree of sophistication introduced in the process for making the prediction. Sometimes simple but systematic correlating of various soil parameters can produce quite adequate and useful informations for use in many types of geotechnical predictions. This section devotes to this attempt for the case of soil behavior associated with excavations supported by soldier pile walls.

Review of Previous Work on Prediction of Soil Behavior in Deep Excavations

The practice in deep excavations in early days was concerned mainly with stability of the support system. However, the practice has been evloved, due to the mounting liabilities with damages to the adjacent structures, to recognize the importance of the ground movement control. The ground movement control requires an estimate of the ground settlement profile behind the excavation. The first practical approach for estimating the ground settlement profile was proposed by Peck [7], and incorporates the stability number as related to basal heaving in the case of soft clays. Over the years the soil-structure interaction effect of the support system has been widely recognized as an important factor determining the ground behavior in a global scale [3, 15, 16], as well as the basal heave. In this regard, an approach was proposed by Clough et al. [15] for prediction of the settlement profile behind the excavation in terms of the Terzaghi factor of safety against the basal failure and a system stiffness for the support system for soft to medium clays. Most of the previous approaches in prediction of soil behavior as related to the basal heave problems and stability of the unsupported face of excavations in clays involve a parameter which may be termed as the stability number in the form as used by Peck [7].

The following excerpts from Peck [7] well describe the role of the stability number in assessing soil behavior in deep excavations in general:

"The information now at our disposal appears to justify the following conclusions. The behavior of the soil and the bracing system depends on the stability number." and "The distinction in behavior for cuts in clays cannot be made, therefore, solely with respect to the softness or stiffness of the clay. It must be drawn on the basis of the behavior of the cut which, for the time being, will be considered to be reflected by relative values of the stability number N."

The stability number approach as used by Peck is concerned primarily with whether the soil would behave elastically or plastically when subjected to shear deformation. This is an important factor to consider in assessing the global stability of the support system, as this affects the mobilization of the earth pressure and the deformational characteristics of the soil mass influenced by the excavation. However, the range of soil behavior that controls the stability at and in the immediate vicinity of the face of deep excavation supported by soldier piles is much wider. In this context, the evolution of the parameter, expressed in the form of Eq. 1 and commonly known as the stability number is reviewed.

$$N = (\gamma H)/S_u \,. \tag{1}$$

where S_u : undrained shear strength in the zone where the stability is sought,

γ : unit weight of soil, and

H : depth of excavation.

Terzaghi [17] treats the base stability of an open cut in clay as a bearing capacity problem, and shows that N in an excavation problem is equivalent to the bearing capacity factor, N_c, whose values depend only on the friction angle of the foundation soil. On this basis, Peck [7] maintains that the soil behavior in the base of an infinitely long excavation is elastic when N is less than 3.14, and a plastic failure commences when N becomes 5.14.

In 1956, Bjerrum and Eide [18] treated the base stability problem also as a bearing capacity problem, using a solution derived by Gibson for N, which is given by:

$$N = 3/4\ (\mathrm{Log}\ E/S_u + 1\ ) + 1 \tag{2}$$

where E is the elastic modulus of the clay in undrained deformation. For values of E/S_u expected from ordinary clays, Bjerrum and Eide indicate that the values for N should vary from 7.6 to 9.4 at a plastic failure state in the base of an excavation.

Deformational characteristics of unsupported tunnels in clays are commonly assessed in terms of a parameter similar in the form to the stability number [6]. According to Deere et al., tangential stresses along the perimeter of a circular tunnel in undrained deformation are given by:

$$\sigma_t = p_z\ (3K_o\text{-}1) \text{ at crown and invert,}$$

$$\sigma_t = p_z\ (3 - K_o) \text{ at the springline.} \tag{3}$$

where p_z : overburden pressure at depth z,

K_o : the ratio of horizontal pressure to the vertical pressure for undrained condition.

For $K_o = 1$, tangential stress becomes $2p_z$ at the crown and the springline. The ratio of the tangential stress to unconfined compressive strength at $K_o = 1$ is defined as Simple Overload Factor (OFS) [6], and is equal to the ratio of the overburden pressure to undrained shear strength, the stability number. In this approach, perfectly elastic behavior prevails for N values of less than

1, and then, through elasto-plastic state with increasing values of N, perfectly plastic behavior prevails at much higher values of N. On this basis, Deere et al. indicate that the movement of unsupported tunnel face in clayey soil is primarily elastic for N values of less than 2 to 3.

Basic Concept of Proposed Procedure for Prediction of Soil Behavior in Cuts Supported by Soldier Pile Walls

Various sources of soil disturbance in excavations supported by soldier pile walls were discussed earlier. Because of the disturbance effects, the soil strength pre g during excavation should be closer to remoulded strength behind the wall, especially near the excavation face. It is possible to relate remoulded strength to index properties of the soil. Index properties are often used in geotechnical engineering to describe behavioral aspects of clayey soils. The stability number is expressed in terms of undrained shear strength of the soil. Therefore, when the remoulded strength as determined from index properties of the soil is used for the stability number, it will be possible to correlate the stability number and index properties in a graph form. This type of presentation will enhance the utility of the stability number and index properties of the soil in assessing the soil behavior.

Index Properties of Soils

Atterberg Limits have been widely used to classify soils, to present behavioral aspects of the soil, and to deduce other engineering parameters of soils.

The research work carried out by Seed et al. [19, 20] has presented a clear insight of physical characteristics of both liquid limit (w_L) and plastic limit (w_P) and the relationships between liquid limit and plasticity index (I_P). For inorganic soils, Seed et al. suggest:

$w_P = 20\%$ for clay content of less than 40%, and
$w_P = 0.5C\%$ for clay content (C in %) of over 40%.

Correspondingly plasticity index becomes:

$I_P = w_L - 20$ for clay content of less than 40%, and
$I_P = w_L - 0.5C$ for clay content of over 40%.

The above finding suggests that a useful relationship could be developed between liquid limit and plastic limit for inorganic soils. Authors found that a good trend is discernable when liquid limits are normalized by plastic limits and plotted against liquid limits, as shown in Fig. 2. Much of the data used in Fig. 2 were taken from the published literatures, and the data cover soils from various parts of the world, including Japan. Plotted also in Fig. 2 is the Casagrande "A" Line, which represents the boundary between inorganic and organic soils. Data points in Fig. 2 indicate that the soils are almost exclusively inorganic. The average trend of the data points may be represented by:

$$w_P = 40w_L / (30+w_L), \ \% \tag{4}$$

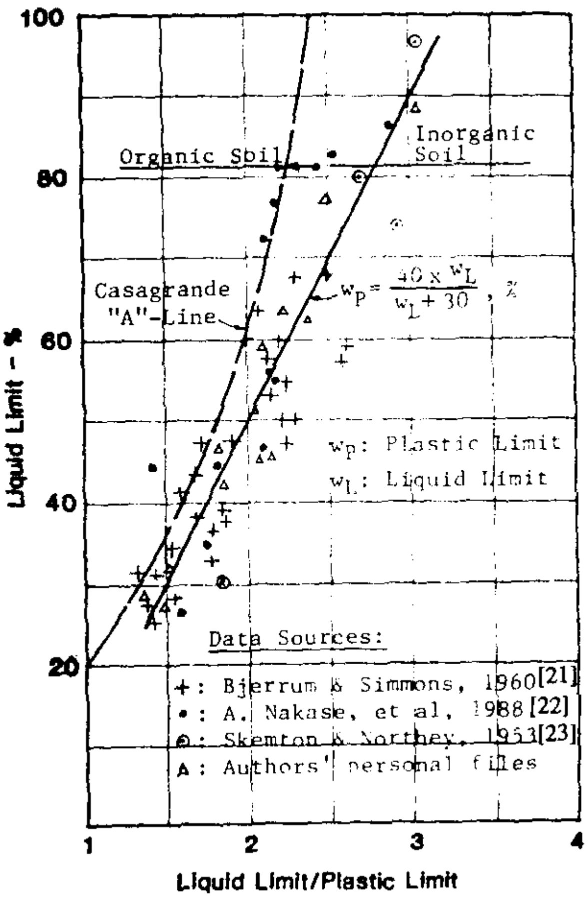


Fig. 2 Relationship between Liquid Limit (w_L) and Plastic Limit (w_P)

Plastic limit of most ordinary inorganic soils is in the neighborhood of 20% and increases slowly with increase in clay content, as indicated by Seed et al. The empirical relationship between liquid limit and plastic limit shown by Eq. 4 provides a trend that generally agrees well with the trend suggested by Seed et al.

The most significant advantage of this type of relationship between liquid limit and plastic limit is in elimination of the need for performing tests to determine plastic limit. Plastic limit determination is highly subjective to judgement of the person performing the tests, thus is susceptible to human error, whereas human error is minimized in liquid limit testing, because it is done with a mechanical device.

Relationship between Index Properties and Remoulded Undrained Shear Strength

Many attempts have been made by others [21, 23, 24, 25] to correlate index properties with important engineering parameters of soils, including strength.

Wroth and Wood [24] present a finding from analysis of test data on remoulded soils produced by Skempton and Northey [23] that undrained shear strength at plastic limit is approximately 100 times of the strength at liquid limit. Using the average strength of 1.7 kPa at liquid limit from the work of Youssef et al. and the finding from the work of Skempton and Northey, Wroth and Wood have established an equation showing the relationship between remoulded undrained shear strength and liquidity index in the following form:

$$Su = 170 \exp(-4.6 I_L), \text{ kPa} \tag{5}$$

Once liquid limit and the natural moisture content of the soil are known, we can determine the liquidity index by obtaining the plastic limit from Eq. 4, and the shear strength can then be obtained from Eq. 5. However, the approximation introduced in the Wroth-Wood equation by taking the average strength at liquid limits can be avoided by establishing an equation that represents the trend of changes in strength with changing values of liquid limit. Youssef et al. [25] indicate that shear strength decreases with increasing value of the liquid limit, and this trend, shown in Fig. 3, may be represented by the following equation:

$$S_{uL} = 7.2 (w_L)^{-0.323}, \text{ kPa} \tag{6}$$

where S_{uL} is the shear strength at liquid limit.

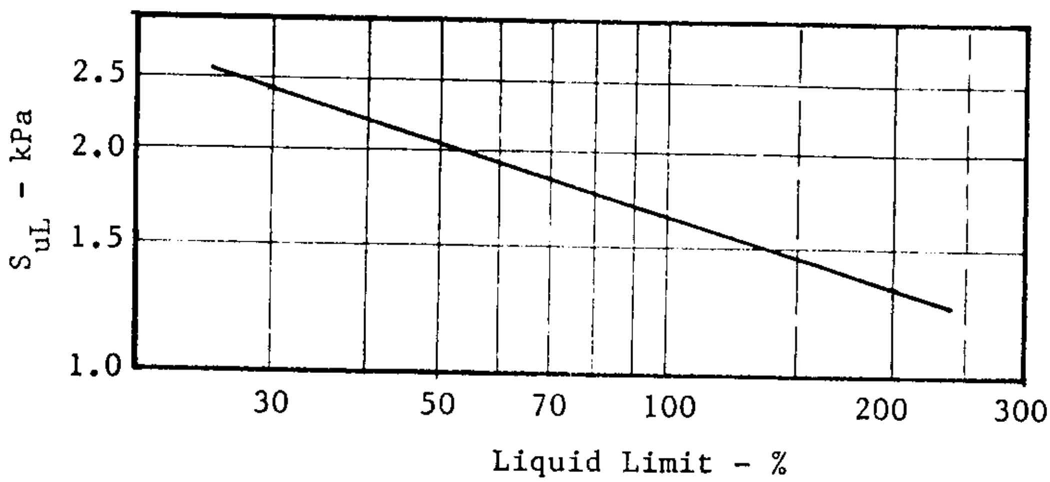


Fig. 3 Relationship between Liquid Limits and Remoulded Strength (after Youssef et al.)

The work of Youssef et al. further shows that the relationship between the water content and remoulded shear strength of a given clay is represented by straight line when plotted with logarithmic scales.

Parametric Relationships among Index Properties, Remoulded Strength and the Stability Number

Using the relationships discussed in the previous subsection, we can plot a series of straight lines, each of which representing a given soil, as shown in Fig. 4. The sequence of determining these lines is summarized as follows:

1. For a given value of liquid limit, find the plastic limit from Eq. 4.
2. Determine shear strength at this liquid limit from Eq. 6. This provides one point (I_L=1.0) for the straight line representing the soil.
3. Obtain shear strength at the computed plastic limit by taking one hundred times of the strength at liquid limit. This provides another point (I_L=0) to establish the straight line representing that given soil.

Also plotted on Fig. 4 are a series of parallel lines representing different values of the stability number using the remoulded strength determined from the above procedure.

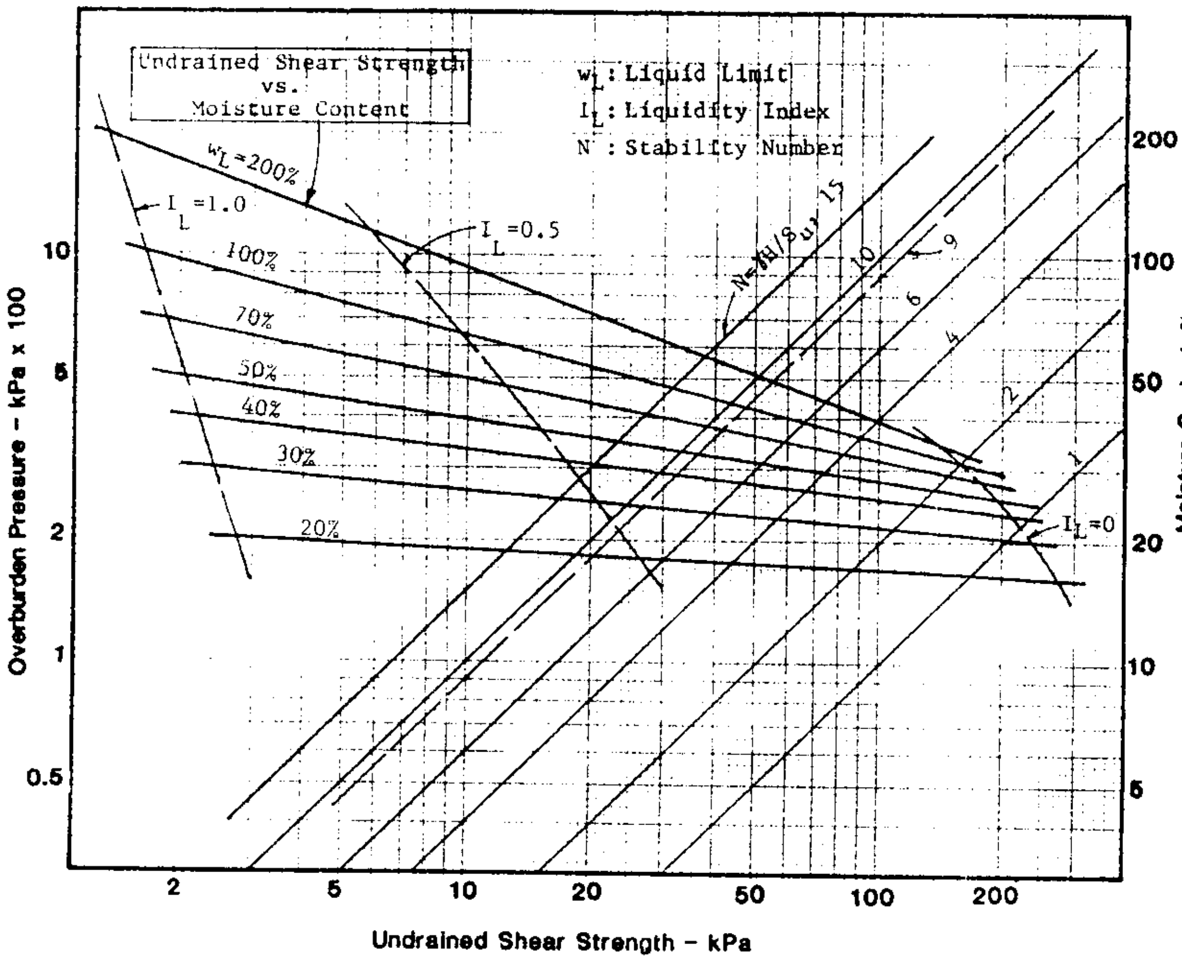


Fig. 4 Graphical Derivation of Stability Number

Fig. 4 consists of two parts; the relationships between liquidity index and remoulded shear strength and the stability number part. The use of Fig. 4 is illustrated in the following examples:

(a) Example A

Test borings, soil sampling and laboratory soil tests established the following information:

Liquid limit : 70%
In-place water content : 40%
Overburden pressure : 100 kPa

Directly from Fig. 2 or from Eq. 4, we find the plastic limit of the soil to be approximately 30%. The plasticity index is then 40%. This soil then belongs to the CH group of the Unified Soil Classification System. We now can deduce from the index pro' that the soil is a highly plastic and potentially expansive clay. Directly from the graph, we obtain a value of 30 kPa for the remoulded undrained strength and obtain an approximate value of 4.0 for the stability number. According to Peck [7], this cut is on the borderline between elastic and plastic behavior. The stability number approach does not consider the effect of the in-place water content on the shear deformational behavior of the soil. It is reasonable to anticipate that this clay at the in-place water content of 40% is likely to exhibit large strains even at low levels of stress, thus it would be more prudent to anticipate the cut to behave plastically. The stability number determined in this way at a given depth is used for construction of a stability number profile for the entire depth of the excavation, which may be used for design of the lagging as illustrated in the subsequent section.

(b) Example B

We now consider a case where the liquid limit is 70% as before, but the in-place water content of the soil is 30%. The lower in-place water content gives a remoulded strength of 130 kPa, putting the soil in the very stiff category. In this case, it would suggest that the soil might have been heavily overconsolidated. We already know from the above example this soil is potentially expansive with increase in water content. Using a unit weight of the soil of 1.8 t/m^3, we find from Fig. 4 that the excavation can reach approximately 30 meters in depth to attain the stability number close to 4.0. The stiffness of the soil and the stability number would tend to maintain the excavation in an elastic behavior. However, the anticipated high swelling potential and the effect of the overconsolidation at such great depth would require a special consideration in selection of the earth pressure for design [15, 26].

APPLICATION OF PROPOSED PROCEDURE TO LAGGING DESIGN

The opinion as to whether the lagging should be designed as a structural element or not is diverse. One opinion asserts that lagging is provided as needed without designing it as a structural element because the soldier piles suppopt the earth which arches between the piles, and the lagging prevents the earth from falling out and destroying the arch. In practice, it is not so uncommon to realize a successful excavation with soldier piles alone, without the lagging. Also there are some computational proofs [27, 28], based on a soil arching concept and certain assumed conditions, that the side stability in excavation is attainable only with soldier piles.

However, the majority cases in practice tend to treat the lagging as a structural element and design it conservatively for the safety side, as reflected in the survey summary of Korean practices with lagging design and construction presented in Table 1. It is noted that the detailed identification of the projects included in the survey is not given with the view that the information might have been privileged.

Table 1 Lagging Design and Construction Practice

Site location	Soil description	depth	Pile spacing	Lagging (txL)
Chongro, Gongpyong-dong	sandy clay	7m	1.8m	130x1750mm
〃	weathered rock	32	1.8	110x1750
Chongro, Yonji-dong	sandy clay	18	1.9	110x1850
〃	wheathered rock	25	1.9	100x1850
Kangnam, Yoksam-dong	wheathered rock	10	2.0	90x1950
〃	hard rock	14	2.0	none
Ansan, Seongpo-dong	silty clay	9	1.8	70x1750

Note: The common width of the lagging member is 150mm.

In order to design the lagging as a structural member, it is necessary to establish the anticipated loading on the lagging. Various assumptions and possibilities have been conceived for the magnitude and distribution of the earth pressure acting on the lagging. Armento arbitrarily chooses 50% of the apparent earth pressure from the Terzaghi-Peck method for strut loads for design of the lagging on the basis of an assumed soil arching effect. It is not uncommon to assume the Rankine active pressure for design of the lagging. However, the various sources of the ground disturbance in the vicinity of the face of the excavation that were discussed earlier seem to invalidate the fundamental conditions on which the assumption mentioned above are based.

Lagging is obviously required in order to maintain stability of the side of the excavation until the final facing (permanent wall) is put in place. The opinion discussed earlier that the earth is supported by the soldier piles by arching has a validity. However, not treating the lagging as a structural element can be grossly wrong in certain instances, depending on the characteristics of the soil. Therefore, the lagging design must consider the soil behavior vulnerable to falls and other forms of instability on the side of the excavation and local shear failures in the immediate vicinity of the side of the excavation.

Solutions have been obained [6, 29, 30] for openings and horizontal slits through vertical walls supporting clays. These situations are to some extent analogous to the side of excavation exposed during installation of lagging.

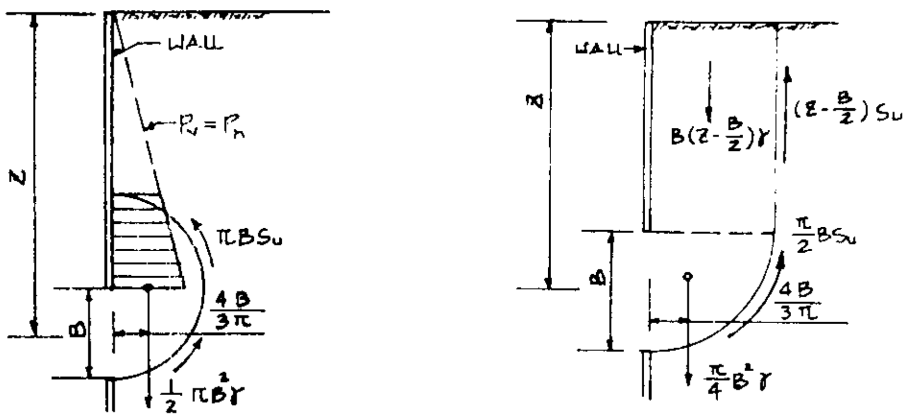
Broms and Bennermark [30] obtained a solution for a narrow horizontal slit in a vertical wall supporting clay for $K_0=1$ condition, and showed that the critical overburden pressure to cause failure of the clay through the slit is 6.28 S_u. Deere et al. [6] extended the Broms-Bennermark solution to provide more general solutions for a wider range of boundary conditions. They considered two different cases: narrow and wide slits in comparison with the depth as shown in Fig. 5. And they found the critical ratio of the overburden pressure at the slit and the undrained shear strength as in the following two equations:

For the narrow slit (z/B > 2),

$$(p_z/S_u)_{crit.} = 2\pi/[1+(1/6)(B/z)] \tag{7}$$

For the wide slit ($z/B \leq 2$),

$$(p_z/S_u)_{crit.} = [2(z/B) + \pi - 1]/[1 + (1/6)(B/z)] \tag{8}$$



(a) z/B > 2

(b) z/B ≤ 2

Fig. 5 Stability of Horizontal Slits (after Deere et al.)

Attewell and Boden [29] performed laboratory extrusion tests on clays to verify the analytical solution and experimental results obtained by Broms and Bennermark [30], and suggest that a maximum acceleration of the extrusive movement occurs at a stability number of 4.5 and indicate that this corresponds more closely to a failure state of tunnel face in clay.

The restraining pressure which must be provided by lagging to avoid failure would be:

$$p_a = p_v - N_c \cdot S_u \tag{9}$$

where p_v is the overburden pressure and N_c is the critical value of the stability number.

Without adhering to a rigorous solution, a reasonable value of N_c may be deduced for a lagging design from the analytical solutions and experimental results discussed above. Boundary conditions of the given stage are obviously not that of a horizontal slit in a vertical wall, and it could be intuitively suggested that N_c would be larger for the unsupported face of the excavation because of the restraints by the piles in three dimensional nature. Boundaries defining B in Eqs. 7 and 8 are fixed for the case of a horizontal slit, while the location of a fixed boundary to define B on the side of the base of the excavation is uncertain but should be much larger than the vertical opening over the unsupported surface because of the yielding of the soil at or below the base of the excavation. These boundary effects are compensatory. In light of this, in addition to the experimental results of Attewell and Boden from which N_c was suggested in the range of 4.5 to 6.28, it appears reasonable to assume N_c to range from 5 to 6 with the lower limit assigned to soils potentially more susceptible to local failures.

On the basis of above discussions, it can be concluded that lagging should be treated as a structural element when N from Fig. 4 exceeds N_c. When N is less than N_c, minimal lagging should suffice. Once unit weights, in-place water contents and liquid limits of the soils are known for all the strata forming the soil profile, the stability number profile can be determined simply from Fig. 4. A vertical line representing the chosen value of N_c and the stability number profile determine the zones over which the lagging should be designed as a structural element. This is illustrated in Fig. 6.

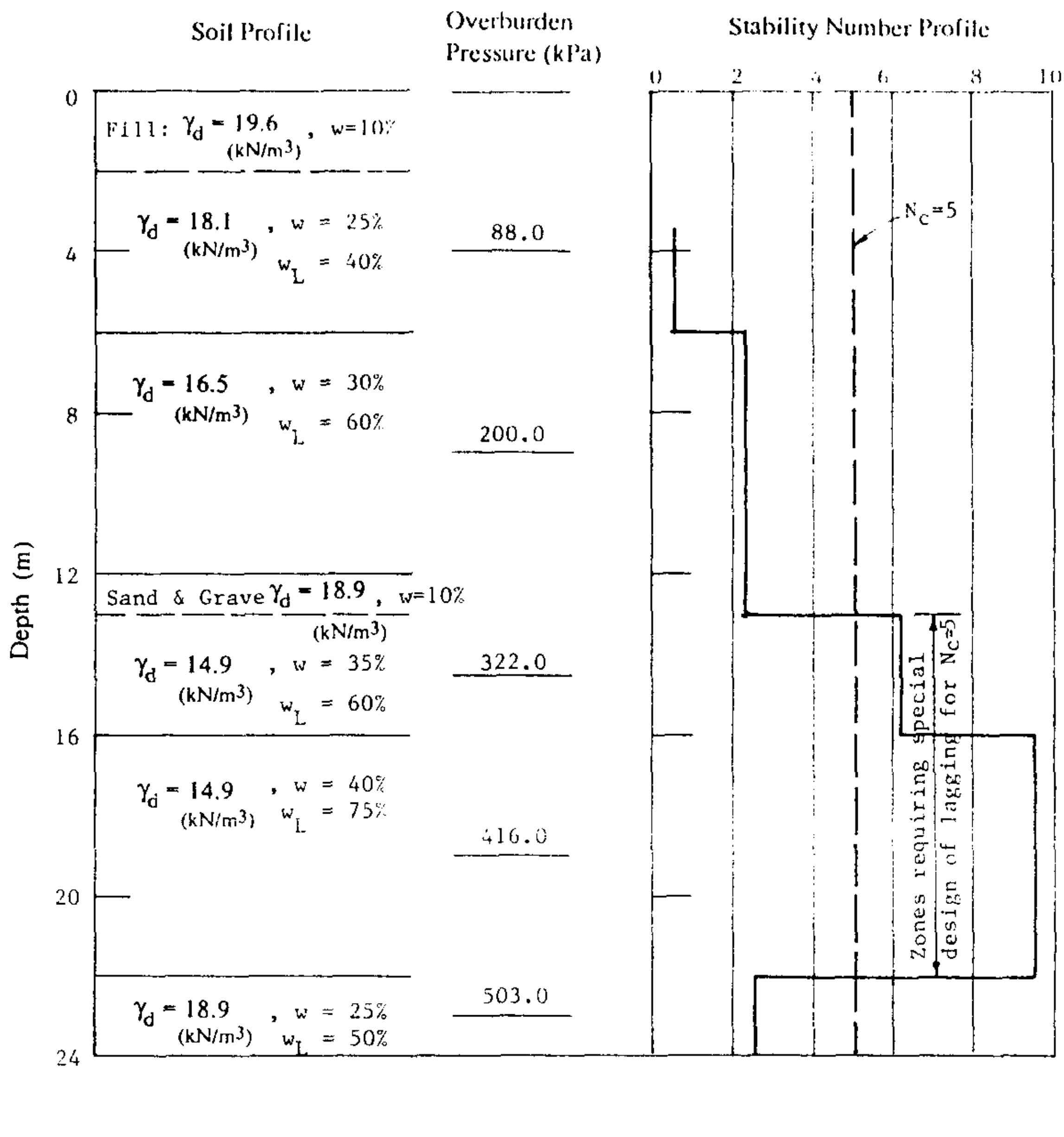


Fig. 6 Construction of Stability Number Profile

In Fig. 6, a value of 5.0 was chosen for N_C. It should be noted that the soil strength was assumed constant with depth for each stratum, and the overburden pressure was computed at midpoint of each stratum in construction of the stability number profile in Fig. 6. Also the stability number is shown in Fig. 6 to be constant with depth for each stratum, although in reality it should vary with depth. The purpose of Fig. 6 is to illustrate the methodology and to define the zones requiring special design of lagging, treating it as a structural element. More correct construction of the stability number profile consistent with the pertinent theoretical basis should be utilized in actual problems.

SUMMARY AND CONCLUSIONS

Ground instability problems of soldier pile-lagging system are often experienced on and in the immediate vicinity of the cut face of deep excavations. These instability problems occur in a variety of forms and are related to local shear failures and disturbance of the soil. Good prediction of soil behavior related to these occurrences is most essential to devise adequate combative measures dealing with these instability problems.

The stability number approach for prediction of soil behavior in deep excavation in a conventional sense is more concerned with whether the soil subjected to shear deformation would behave more elastically or plastically in a global scale, and thus it lacks the ability for predicting what can happen on a local scale on and in the immediate vicinity of the face of the excavation. Soil behavior in general is critically affected by the in-place water content of the soil, and is more so in situations where the soil gets disturbed. However, the stability number approach does not directly reflect the effect of the in-place water content.

The proposed method presented in this paper for prediction of soil behavior in deep excavations supported by soldier pile-lagging system combines the ability of the stability number with that of index properties of soils for describing a variety of physical behavioral aspects. Therefore, the proposed method allows assessment of the soil behavior, thus the anticipated instability problems, more completely than stability number approach alone can.

Whether the lagging should be designed as a structural element or not depends on the characteristics of the local soil conditions on and in the immediate vicinity of the side of the excavation. The lagging should be treated as a structural member when N exceeds N_C.

Finite element methods and the technological advances in recent years have improved our ability for better control of the ground movements associated with deep excavations in soils. Although the finite element method is a powerful tool, various difficulties in its application to practical problems can often result in erroneous predictions, and a simplified empirical method, such as the one treated in this study, is a valuable reference guide with which the result from the FEM may be compared.

While the empirical relationships and methodology presented in this paper for predicting the soil behavior in deep excavations can be a practically useful tool or guide in preliminary design studies and execution of the excavation, their usage has obvious limitations, the same as any other empirical approaches in geotechnical engineering practice, and should not be considered as a substitute for other detailed geotechnical analyses.

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