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STABILITY OF TUNNEL WORKING FACE IN SQUEEZING GROUND

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SYSNOSIS : Practical solutions that are available today for assessing stability of tunnel working face are largely based on the concept of critical stability ratio. The accuracy of a prediction of the soil behavior in the working face, thus, depends on the ability of the solution to completely and accurately describe the stress fields or kinematics generated by the excavation and the accuracy of the undrained shear strength of the soil introduced in the computation.

This paper reviews the selected solutions describing stability of the tunnel heading in squeezing ground, and suggests a reference solution which is estabiished based on comparison of the solutions and field data on stability of tunnel headings in clays. Although dealing with the shear strength determination is an important companion part of the geotechnical prediction for stability of the tunnel heading in clays, this part is beyond the scope of this paper at this time.

1. INTRODUCTION

Tunnelling inevitably disturbs the in-place stress field of the ground. The resulting response of the ground is manifested in various forms, depending on the initial condition of the in-place stress field, characteristics of the formation and tunnelling method. Predictions are made of the likely behavior of the ground in response to the intended tunnelling, and the results form the important basis for determination of planning and design of the tunnel construction.

Among the common geotechnical occurrences experienced in soft ground tunnelling, squeezing ground poses probably the most difficult problems to cope with during and subsequent to erection of the tunnel. Squeezing ground condition develops most commonly in soft to medium clays. One of the major problems associated with squeezing ground condition in soft ground tunnelling is the instability of the working face.

Modern tunnelling technology has introduced various types of shield machines designed to cope with the instability of the working face in soft ground tunnelling (Fujita, 1990). However, besides their relatively high cost, these machines are not uniformly adaptable to the variable and mixed ground conditions often encountered in practice. Thus, instances can arise in practice whereby it is more practical and economical to use the conventional open type shield and then devise methods to combat the face instability problems during erection of the tunnel.

Prediction of the ground behavior in response to tunnelling requires solutions describing the stress fields or kinematics generated by the excavation in the affected ground mass and soil parameters to enter in the computation, as in many other types of geotechnical predictions. The behavior of the squeezing condition is such that the most important soil parameter for the stability prediction is the undrained strength of the soil. Therefore, the accuracy of the prediction depends on the accuracy of the solution itself and of the undrained shear strength of the soil.

Numerous solutions have been proposed in this subject. Because of the complex nature of the soil behavior in the tunnel heading and its dependency on other factors, simplified mechanisms and stress fields have been assumed in the solutions, and these assumptions affect the solutions in various ways. This paper reviews the selected solutions describing stability of the tunnel heading in squeezing ground condition, and suggests a reference solution which is established based on comparison of the solutions and field data on stability of tunnel headings in clays.

There is no other area in geotechnical practice where the accuracy of the soil strength can be more directly and dramatically tested than in tunnelling. This is so because the loads are almost instantaneously imposed on the soil mass and the soil is stressed commonly much beyond the yield limit. Contrary to tunnelling, in problems dealing with foundations, slopes and deep excavations, the mobilized stress level in the soil mass is low because a factored strength is used in the design, and the loads are gradually applied. Therefore, a significant portion of the error in prediction of the tunnel heading stability can result from inaccurate determination of the shear strength of the soil. Dealing with the shear strength determination is thus an important companion part of the geotechnical prediction for stability of the tunnel heading in clays. However, this part is beyond the scope of this paper.

2. SQUEEZING GROUND

The term "squeeze" is used in tunnelling to describe the phenomenon of slow but continual advances of the exposed surfaces towards the tunnel opening. Indiscriminate usage of this term without adhering to the specific causes of the ground convergence toward the tunnel can result in misunderstanding the mechanism of squeezing ground. Therefore, this section clarifies the phenomenon of squeezing ground as used for the purpose of this paper.

Removal of the lateral support from the ground mass by tunnelling allows the ground to deform toward the opening. This action causes the stress field to change, including development of shear stresses. Contrary to most rock formations, the ratio of the in-place horizontal stresses to vertical stresses is typically less than 1.0 in soft to medium clays that are susceptible to squeezing. Therefore, the change of the stress field associated with tunnelling through such soils is more critically influenced by the vertical pressure. Weight of the overburden above the opening is partially transferred to the less displaced portion of the ground through shearing action (half dome action), and this process results in formation of more intensely developed shear zones which extend roughly upwards above the sides of the opening. The ground deformation inside the tunnel opening is small enough to be not readily perceptible at low levels of shear stresses with respect to shear strength of the ground, but the movement accelerates and becomes large as the shear stresses approach the strength level. Rate of the movement can vary with time during exposure of the tunnel surfaces, but the movement becomes more or less continual when the shear stress to strength ratio exceeds a certain limit. This type of ground movement is defined as "squeeze" (Proctor and White,1977; Deere, et al, 1969; Peck, 1969; Varsheney, 1988). It is important to note that the ground movement in squeezing condition does not cause volume change of the material involved in the movement. The squeeze involves the materials outside the tunnel opening and the volume of the material involved in the movement can be very large, depending on the soil conditions and the cover-to-tunnel diameter ratio. The squeezing phenomenon considered in this paper, therefore, involves the characteristics of a plastic flow. Swelling ground is similar to squeezing ground in the sense that both involve movement of the ground advancing towards the tunnel opening. However, swelling is the consequence of expansion of clay minerals associated with increase in moisture content of the ground, and its effect is generally limited to the immediate vicinity of the opening. Therefore, swelling and squeezing involve different mechanisms.

The mechanism of squeezing ground in rock tunnelling is sometimes explained not entirely in the same context as in soft ground tunnelling in the literature. Singh (1988) explains that the rock mass around and in the vicinity of the tunnel opening fails when the induced tangential stress exceeds the unconfined compressive strength of the rock mass, and the resulting volume expansion of the failed rock mass causes convergence of the tunnel opening. He defines the above processes as "squeezing". Lo, et al (1987) use the term "rock squeeze" to describe time-dependent deformations in the form of invert heave or lateral inward deflection of tunnel openings in swelling rocks. They attribute this phenomenon in the Southern Ontario region of Canada to the high residual horizontal stresses in the rock mass, where the ratio of the in-situ horizontal stresses to vertical stresses is extremely high, reaching up to 30.

The squeeze condition developing in rock masses as outlined above is most critically influenced by the initial in-place stress conditions and initiated by volume expansion of the rock mass around and in the vicinity of the tunnel opening. Therefore, the consequence is generally more of a tunnel opening convergence problem, which in turn results in a large time-dependent increase in the liner support loads. However, the type of squeeze behavior discussed above for soils can also occur in rocks, depending on the intensity of the overburden and the softness of the rock mass (Varsheney, 1988).

3. STABILITY SOLUTIONS OF TUNNEL HEADING

3.1 Definition of Stability

The definition of stability of unsupported tunnel faces should be considered on the basis of the stand-up time, because the length of time available before the state of excessive displacements or collapse is reached determines the construction scheme and the combative measures necessary to maintain the tunnel construction within the acceptable tolerance. In this approach, one must be able to first predict the stress-strain-time behavior of the soil and then to determine the influence of other factors, such as the tunnel size and the rate of excavation, among others, on the soil behavior during and subsequent to excavation of the tunnel opening. In general, numerous rheological models have been suggested to describe the dependency of the stress-strain relationship on time (Deere, et al, 1969). Kavazanjian and Mitchell (1977) have proposed a stress-strain-time behavior model incorporating the creep model of Singh and Mitchell (1969) which describes the delayed deviatoric deformation under a constant stress. This model requires seven parameters and one graphical relationship obtained from laboratory soil test data. Myer, et al (1977) investigated through a series of model tests the relationship between the tunnel size, the rate of excavation, and the stand-up time. Although the above approach in the stress-strain-time behavior study of soft clays is useful for understanding of the fundamental aspects of the behavior, it has not found much application in practice, primarily due to the difficulty of producing reliable soil parameters, the complexity of its application, and the question as to the adequacy of the models representing the complex conditions developed during the excavation. An alternative cause of the time-dependent behavior has been also suggested as the dissipation of excess negative pore water pressures induced by the excavation at the tunnel face (De Moore and Taylor, 1989).

Another approach in assessment of the stability of tunnel headings in soils susceptible to squeezing has been based on the concept of the critical stability ratio. This concept, as applied to assessment of the tunnel face stability, was first suggested by Broms and Bennermark (1967), based on the results of their analysis of stability of a cohesive soil located behind a vertical hole through a sheet pile wall. They indicated that there is a vertical pressure at which the soil fails through the hole, and defined the critical stability ratio as the ratio of this vertical pressure to the undrained shear strength of the soil. Therefore, the assessment of the tunnel face stability in this approach entails comparing the actual stability ratio against the critical stability ratio. Solutions are required defining the critical stability ratio. The practical advantage of this approach over the approach utilizing rheological models is obvious, as finding the actual stability ratio of a given case is a simple matter.

In verification of the above concept with laboratory experiments, Broms and Bennermark chose the critical vertical pressure as the yield stress on a stress/deformation curve from a soil extrusion test. The yield stress involves no time dependent nature of the stress-strain behavior of the soil. In an actual tunnel construction, a full loading condition develops in the working face when the excavation has advanced to a point and stopped for erection of a temporary support system. The soil would exhibit a time- dependent behavior from the time of the excavation stoppage, until the movement is restrained by installing braces. The time-dependent behavior of the soil during the face exposure period may be represented by one of the forms shown in Fig. 1. What Fig. 1 signifies is that a tunnel face which is stable based on the critical stability ratio as defined by Broms and Bennermark can become unstable in tunnelling, depending on the characteristics of the deformation-time history of the soil.

The important role of the stability ratio in assessment of the behavior of soft to medium clays in excavations was emphasized and clarified by Peck (1969). Peck indicated that whether the behavior of clayey soils subject to shear deformation in excavation faces is elastic or plastic depends on relative values of the stability ratio. Therefore, we can anticipate a range of the soil behavior based on the relative values of the stability ratio, even including approximately the time-dependency. The role of the stability ratio and its physical significance in deep excavation problems have been extensively treated by the authors (Sohn and Sohn, 1990).

3.2 Solutions for Critical Stability Ratio, Nc

Since the introduction of the critical stability ratio concept

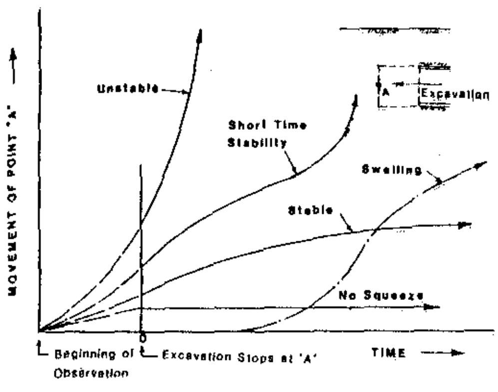

FIG. 1. SQUEEZE-TIME HISTORY CURVES

by Broms and Bennermark, numerous solutions have been proposed in this direction for assessment of the heading stability in clay tunnelling.

a) Broms-Bennermark Solution and Its Extension by Deere, et al

The original Broms-Bennermark solution dealt with a horizontal slit and assumed the failure surface to be cylindrical in a medium of isotropic stress field. Broms and Bennermark obtained an Nc value of 6.28 for horizontal slits and 7,5 for circular openings, irrespective of the size of the openings. Deere, et al (1969) extended the Broms-Bennermark solution and obtained a more general solution by a simple work calculation for the assumed mechanisms as shown in Fig. 2. The solutions of Deere, et al are given in terms of the vertical pressure at the axis level of the tunnel, pz, in Eqs. 1a and 1b. These equations are also expressed in terms of the tunnel cover-to-diameter ratio (C/D), to facilitate comparison of the solutions with others.

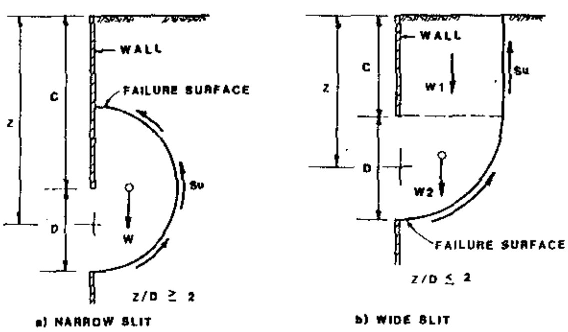

FIG. 2. STABILITY OF HORIZONTAL SLIT

For $z/D > 2$ or $C/D > 1.5$:

$$Nc = 2\pi[1+(1/6)(D/z)] = \frac{2\pi}{1+(1/6)[1/(0.5+C/D)]} \qquad (1a)$$

For $z/D < 2$ or $C/D < 1.5$:

$$Nc = [2(z/D)+\pi-1]/[1+(1/6)(D/z)] = \frac{[2(0.5+C/D)+\pi-1]}{1+(1/6)[1/(0.5+C/D)]} \qquad (1b)$$

b) Proctor-White Solution and Its Extension

Proctor and White (1977) considered that if squeeze is to occur, the movement of the triangular wedge, bec in Fig. 3, at the tunnel face opening must be initiated.

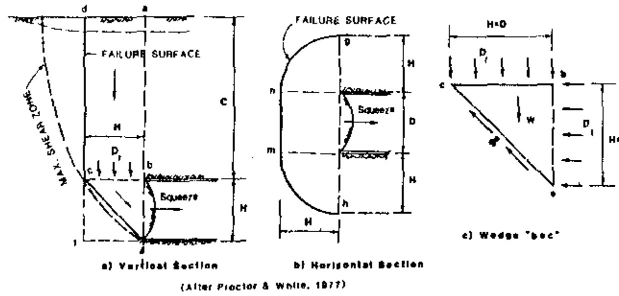

FIG. 3. STABILITY OF TUNNEL FACE CLAY

Based on the assumed failure mechanism for a rectangular tunnel of D x H, as shown in Fig. 3, Proctor and White derived Eq. 2 for determination of a net driving force acting on the wedge, bec, by considering the force equilibrium in the vertical direction.

$$pr = C\,[w - (Su/D)\,\{(\pi + D/H)/(1 + \pi(H/2D2)\} \qquad (2)$$

where, pr = restrained overburden pressure, and
w = total unit weight of the soil.

Eq. 2 may be reduced to Eq. 2a for a square tunnel of D x D.

$$pr = C\,[w - (1.61/D)\,Su] \qquad (2a)$$

Proctor and White derived pr for the purpose of obtaining the load ratio which is the ratio of pr to the unconfined compressive strength of the soil, and they used the load ratio as a measure of the rate of squeeze.

By considering the force equilibrium of the triangular wedge, bec, in Fig. 3c, the critical stability ratio can be simply derived by using pr from Eq. 2a for a plane strain heading condition, as in Eq. 2b.

$$Nc = (pz - pi)/Su = 2 + 1.61\,(C/D) \qquad (2b)$$

where, pi = internal pressure (air pressure or other support loads, if any)

c) Solutions of Davis, et al

Davis, et al (1980) have obtained upper bound and lower bound solutions for a plane strain heading and for a circular tunnel heading. For the upper bound solutions, arbitrary sliding block mechanisms were assumed and the solutions were obtained by work calculations on the force system. Isotropic initial stress field and a weightless medium were assumed in the lower bound solutions which describe the stress field in equilibrium everywhere in the medium without yield being exceeded under the imposed loading.

c-i) Plane strain heading

The plane strain heading condition is essentially similar to the horizontal slit problems solved originally by Broms and Bennermark, and then extended by Deere et al, as in Eqs. 1a and 1b. A lower bound solution was obtained by treating the heading as a bearing capacity problem. The sliding block mechanism as shown in Fig. 4c and 4i was assumed for the upper bound solution and the three variable angles, α, β and θ, were optimized. These equations are presented as Eqs. 3a and 3b.

Lower bound solution: $N_c = 2 + 2\ \ln[(C/D) + 1]$ (3a)

Upper bound solution: $N_c = 4\ \sqrt{[(C/D) + 1/4]}$ (3b)

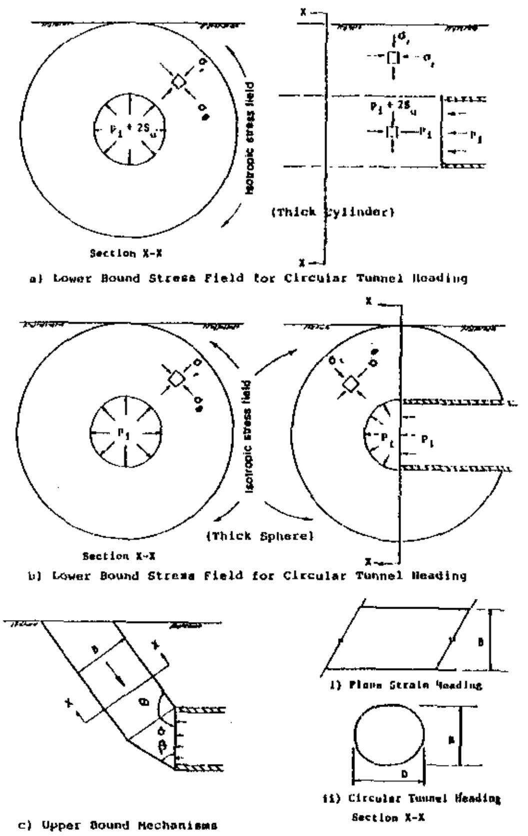

FIG. 4. CLAY TUNNEL FACE MECHANISMS OF DAVIS, etal(1980)

c-ii) Circular tunnel heading

Davis, et al assumed two conditions for the stress field for lower bound solutions; one corresponding to a thick cylinder and the other corresponding to a thick sphere, as shown by Figs. 4a and 4b. For the upper bound solution, they assumed the sliding blocks as shown in Fig. 4c, similar to the case of a plane strain heading, but the cross sections of the sliding blocks were assumed to have an elliptical shape as shown by Fig. 4ii.

Lower bound solutions:

Cylinder: $N_c = 2 + 2\ \ln[(2C/D) + 1]$ (3c)

Sphere: $N_c = 4\ \ln[(2C/D) + 1]$ (3d)

No closed form solution was provided for the upper bound mechanism described above, but results of the solution are available in a graph form.

4. DISCUSSION OF THE SOLUTIONS

Results of the solutions presented above are plotted on Fig. 5 for comparison purposes. Also, plotted on Fig. 5 are the field data on stability of tunnels in saturated clays from Table I. It is noted that the field data in Table I was categorized in terms of the squeeze potential based on the description given in the

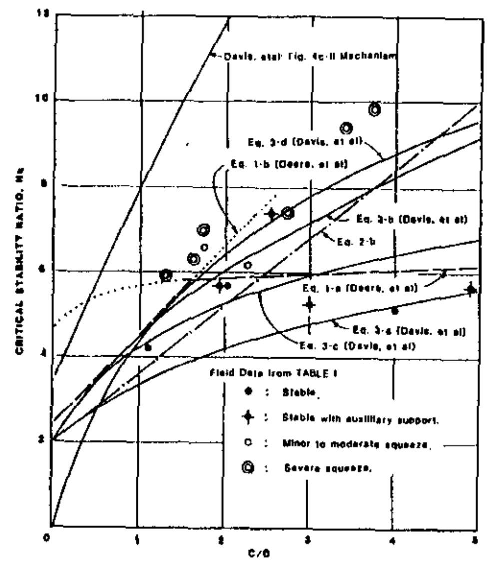

FIG. 5. STABILITY SOLUTIONS FOR TUNNEL HEADING IN CLAY

TABLE I

FIELD DATA ON STABILITY OF TUNNELS IN SATURATED PLASTIC CLAYS

No.	Case	z ft.	D ft.	C/D	S_u ksf	P_z ksf	P_i ksf	N
1	Detroit, water	68	15.0	4.03	0.8	8.0	3.9	5.1
2	Toronto, subway	43	17.0	2.03	0.7	5.5	1.4	5.7
3	Chicago, subway	36	20.0	1.30	0.44	4.3	1.7	5.9
4	Tokyo, subway	74	23.0	2.72	0.76	5.6	1.2	7.4
5	Osaka, municipal railway	51	23.0	1.72	0.60	5.0	1.0	6.6
6	San Francisco BART	59	18.0	2.63	0.80	5.9*	0	7.4*
7	Edsådalen	28	6.6	3.74	0.31	3.0	0	9.8
8	Gothenburg	14	4.0	3.0	0.33	1.7	0	5.2
9	Mårten	14	2.6	4.9	0.27	1.5	0	5.6
10	Spånga	10	4.0	2.0	0.19	1.1	0	5.7
11	Bromma	30	7.7	3.4	0.34	3.2	0	9.4
12	Tyholt, Sta. 199	56	26.0	1.65	0.72	7.8	3.3	6.3
13	Tyholt, Sta. 194+5	72	26.0	2.27	0.61	6.2	2.5	6.1
14	Chicago	40	25.0	1.10	0.60	4.4	1.9	4.2

REMARKS:

No. 1): Plastic clay; hand mined, concreted daily; some squeeze.
2): 4 ft. of clay cover under dense sand; stable.
3): Plastic clay; hand mined; moderate squeeze at N=5.9, excessive squeeze at N=7.2 (due to drop in air pressure).
4): Sensitive clay; difficult to shove shield while face supported.
5): Sensitive clay; uncontollable squeeze at N=7 (p_i=0.8 ksf).
6): Moderately sensitive clay; shield with breasted face.
7): Sensitive varved silty clay; failure after 1½ hours.
8): Stable with jacked pipe.
9): Stable with jacked pipe.
10): Jacked pipe.
11): Jacked pipe, clay flow into pipe.
12): Sensitive clay with silt layers; face propped; squeeze in.
13): (same as above)
14): Partly closed shield; opening stable.

NOTES:
a) No. 1 through 5 were taken from Table I of R. Peck (1969).
b) No. 6 was taken from Table V of R. Peck (1969).
c) No. 7 through 14 were taken from Table IV-I of Deere, et al (1969).
d) * Assumed.

sources from which the data was taken. In doing so, some arbitrary judgement was exercised when the data was not sufficiently descriptive for the condition of squeeze.

Fig. 5 indicates large variations between most of the solutions. This should come as no surprise when viewed in light of the assumptions introduced in the mechanisms supporting the solutions. It has been well recognized that the geometry of the maximum shear zone resulting from a tunnel excavation is significantly influenced by the C/D ratio. Other than Eq. 3b, the mechanisms assumed for the upper bound solutions presented previously did not consider the influence of the C/D ratio. An example of the influence of the C/D ratio on the shape of the maximum shear zone may be examined from Fig. 6, which was presented by Hansmire and Cording (1975). Fig. 6 shows field measurements of maximum shear strains developing above the tunnel cross sections during the course of shoving a shield through clayey soils. Fig. 6a shows the strain contours directly above the center of the shield at a section approximately 3 meters away from Sta. C shown in Fig. 6b. The C/D ratio for this tunnel is approximately 1.8. The boundary of the maximum shear zone attains a semi-elliptical shape, with the maximum horizontal axis being not much greater than the diameter of the tunnel. However, if the C/D ratio was near 1.0, then the shear zone boundary would become more or less similar to the failure surface as assumed in Fig. 2b by Deere, et al for a shallow tunnel. It is reasonable to expect that the maximum shear zone shape in the tunnel heading should be approximately similar to that depicted by the Hansmire-Cording field records for tunnel cross sections.

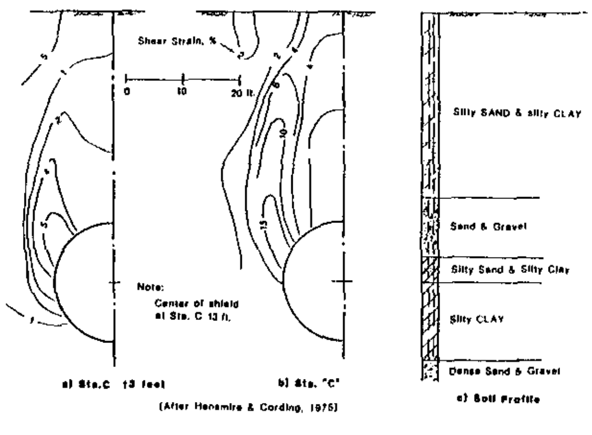

FIG. 6. MAXIMUM SHEAR ZONE

From the example cited above on the shape of the maximum shear zone and comparison of the mechanisms assumed in the solutions, the following comments may be made for the upper bound solutions:

a) Eqs. 1a, 1b and 3b deal with the similar boundary condition of the heading (plane strain condition). The difference between the solutions are due to the difference in the mechanisms assumed. The semi-circular failure surface assumed for Eq. 1a is unrealistic, in light of the example in Fig. 6. Davis, et al show that the three variable angles in Fig. 4c are optimized at $\tan \alpha = \tan \beta = 2\,\gamma/D + 1/4$ and $\theta = \pi 2$ in derivation of Eq. 3b. This would give a 45° failure wedge at C/D = 0, and B = 1.3D and the sliding block containing the angle θ to rise at 21° to the vertical at C/D = 1.5. There is an approximate similarity of the failure surface geometry between Eq. 1b and Eq. 3b for C/D less than 1.5, and thus the two equations agree well in this range of the C/D ratio. Although the optimization of the angles in the mechanism for Eq. 3b adjusts the inclination and size of the sliding block for different values of C/D, the mechanism fails to attain the boundary of the maximum shear zone that resembles the shape as shown in Fig. 6, and the width of the sliding block becomes unreasonably large at higher values of C/D. This would result in underestimating Nc values at higher values of C/D, but the fact that still the solution plots near the failure zone indicated by the field data in Fig. 5 raises a question as to the reasonableness of the assumed mechanism. This plus the elliptical cross section mechanism assumed for the sliding block cross sections may explain why the solution of Davis, et al for a circular tunnel heading attains excessively high values of Nc in Fig. 5.

b) The mechanism assumed for Eq. 2b accounts partially for the three-dimensional effect of the heading. The extensive zone of the ground movement assumed in the mechanism would result in underestimating the restrained overburden pressure, pr, for the lower range of C/D, and thus the Nc values. Also, the square cross section assumption for the tunnel would be unreasonable for a circular tunnel.

c) Upper bound solutions are reasonable for C/D values of less than 1.5. This is so because the potential maximum shear zones are relatively easy to describe in a simple mathematical form, while attaining a reasonably good match between the assumed and actual shear zones in this range of C/D (shallow tunnel).

d) The mechanisms for the upper bound solutions considered in this paper appear to be unreasonable based on the example of Fig. 6 for C/D values over 2.0 (deep tunnel).

The following aspects may be pointed out in connection with the lower bound solutions:

a) The plane strain heading assumption is unreasonable for a shield driven tunnel, and underestimates the Nc values (Eq. 3a).

b) The remaining lower bound solutions are Eqs. 3c and 3d. These equations are based on idealization of the tunnel, for simplicity, either as a long cylindrical cavity or as a spherical cavity. Further, Eq. 3c assumes that the cylindrical cavity continues into the unexcavated soil mass. Therefore, this equation may be valid only at or in the immediate vicinity of the tunnel face. Still there is another important question as to how good the cylindrical and spherical cavity assumptions are, and which assumption is more accurate for the tunnel heading. In view of the fact that the rebounding of the ground at the tunnel face upon excavation and the associated build-up and dissipation of negative pore water pressures can significantly weaken the soil mass and subject it to slaking, it would be reasonable to assume that the spherical cavity assumption is the better representation of the heading. However, it would be more compromising to take the average of the two equations.

c) Eqs. 3c and 3d treat the medium as weightless and neglect the influence of the boundary at the ground surface. However, Mindlin (1940) shows that neglecting the boundary effect is sufficiently accurate when C/D is approximately 3.5 or greater. On this basis, as well as the good agreement with the field data as shown in Fig. 5, the lower bound solutions are deemed more accurate for higher values of C/D.

The discussions presented above provide some bases that may be used for construction of a "reference solution". These include:

a) The reference solution should agree closely with the upper bound solutions, Eqs. 1b and 3b for C/D values of up to 1.5.

b) The reference solution may be approximated as the average of Eqs. 3c and 3d for C/D values of over 3.0. This, then, will provide the following equation:

$$Nc = 1 + 3\ln[(2C/D) + 1] \text{ for } C/D > 3.0 \qquad (4)$$

c) The reference solution for C/D values in between the above ranges of C/D, may be obtained by connecting the curves with the trend of the curves and the field data honored.

The reference solution meeting the above conditions is shown in Fig. 7, with the field data from Fig. 5 shown again for comparison with the reference solution curve.

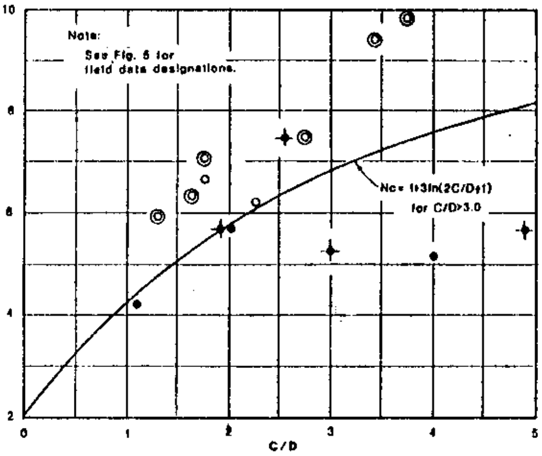

FIG. 7. CRITICAL STABILITY CURVE FOR TUNNEL HEADING IN CLAY

5. SUMMARY AND CONCLUSIONS

Several selected solutions available for assessment of stability of tunnel headings in soft to medium clays have been reviewed and the results from each of the solutions have been compared. The solution results were also compared with field data on stability of tunnels in clays. An ultimate purpose of the above undertakings was to establish a critical stability ratio curve that honors the reasonable parts of the solutions and the field data points.
It has become clear from the above work that any one particular solution cannot be generalized to adequately handle the complex and variable conditions created by tunnelling, and each of the solutions is good only as far as the assumptions supporting the solution are reasonable.

The stability ratio concept is a convenient practical tool for assessment of stability of the tunnel heading in soft to medium clays. Although the initial usage of the critical stability ratio ignored its dependency on C/D (Broms and Bennermark, 1967; Peck, 1969), the solutions and the field data indicate (Figs. 5 and 7) that Nc depends critically on C/D. This requires a careful consideration of the variation of the undrained shear strength of the soil with depth.

The time-dependent nature of the ground behavior in the tunnel heading must be considered in assessing the stability. The critical stability ratio concept does not directly account for the time-dependent ground behavior. However, it is possible to indirectly account for this effect by maintaining the field stability ratio significantly lower than Nc. In this regard, the curve in Fig. 7 could be considered as a reference defining the mid-range of moderately squeezing ground. Then, the zone above the curve should represent high potential of squeezing or severe squeezing.

As a concluding remark, it is noted that a solution alone is not complete and sufficient for a good geotechnical prediction. Often, the accuracy of the prediction is more significantly dictated by the soil data, and the tunnel heading case is deemed to be more so. The determination of the undrained shear strength of the soil must consider at least the effect of stress path caused by the tunnel excavation, the sample disturbance effect, the strain hardening behavior in testing, and the depth effect as related to the C/D effect on Nc, in order to be commensurate with the ability of the solution itself for the prediction.

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