DISCUSSIONS AND CLOSURES

Discussion of "Bearing Capacity of Shallow Foundations in Anisotropic Non-Hoek-Brown Rock Masses" by Mahendra Singh and K. Seshagiri Rao

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The discusser, having a long and abiding interest in the engineering of jointed rock masses and a particular interest in shallow foundations on rock, welcomes the paper by Singh and Rao for its explicit recognition of rock mass as a discontinuum that requires treatment as such. There are many appealing aspects of the authors' bearing capacity analysis compared to more conventional approaches. However, the discusser is not able to accept the fourfold failure mode hypothesis, which is fundamental to their concept of bearing capacity for jointed masses.

The authors have described failure modes associated with splitting, shearing, sliding, and rotation based on the results of Singh's testing of a jointed block mass in uniaxial compression and published literature. The discusser does not have access to Singh's data (Singh 1997) but is familiar with Brown's triaxial tests on block jointed models (Brown 1970) and accepts the four failure modes under those test conditions. These failure modes would have wide acceptance throughout rock mechanics circles under general conditions. The point of difference here is that shallow foundations represent particular boundary conditions associated with a half space and, as a consequence, certain failure modes are inhibited. Just as with jointed rock slopes the more likely failure modes are slip by sliding along joints, shearing and toppling by rotation and failure by splitting is less likely; then the failure of shallow foundations is very unlikely to occur either by slipping or by rotation. The absence of a free face removes the possibility of kinematic mechanisms associated with those modes from developing along critical joint configurations, other than in special circumstances. The only practical failure modes for shallow foundations are splitting and/or shearing of intact material in general, and splitting alone in conditions of continuous jointing, as adopted by the authors.

The discusser acknowledges that slip and rotation are basic deformation mechanisms within a block jointed mass but has concluded that, for conditions of shallow foundations, their effect, separately and in combination, is to bring about nonuniformity in load transmission between individual blocks within the jointed mass (Burman and Hammett 1975). The extent to which slip and rotational deformations between blocks can develop in a half space is limited, as compared to a slope configuration, by the absence of a free face. Yet even very restricted slip and rotational movements of blocks within a jointed mass will lead to edge-to-face and edge-to-edge contacts and will result in applied loads being transmitted though the mass by concentrated contact force trajectories.

The general loading condition between blocks in a jointed mass is one of nonuniformity in contact stresses across joints, as a result of the mobility of individual blocks and their ability to slip and to rotate relative to one another. Nonuniformity is exacerbated by joint roughness and the other irregularities that occur with real joints. Geotechnical researchers have for years devoted significant resources and effort in attempts to measure rock strength under uniform compressive loadings. In the discusser's view this is counterproductive because uniformity is essentially an academic construct: there is little of it in nature.

When block jointed models are tested in uniform compression, in experiments such as those of Brown (1970) and Singh (1997), failure in many instances occurs by splitting as a result of indirect tension generated in an overall compressive stress environment due to nonuniform contact between individual blocks. The discusser sees virtue in recognizing the propensity for failure under indirect tension and has proposed the concept of Brazilian compressive strength as the minimum compressive strength for rock material. The Brazilian test, with its diametral line loading of a horizontal cylindrical sample, represents an extreme nonuniformity in compressive loading and can be considered as the minimum strength under compressive loading. On the other hand, unconfined compression, with its pseudouniform loading of a vertical cylindrical sample with specially prepared ends, represents an upper limit to rock strength under the most favorable loading conditions. It is reasonable to expect that the strength of a jointed mass will lie between the limits determined by Brazilian and unconfined compression test results

$$\text{Brazilian tensile strength } (\sigma_t) = 2 * P/\pi D \tag{1}$$

$$= P/1.57 * D \tag{2}$$

where P=failure load; and D=diameter of the cylindrical test sample

$$\text{Brazilian compressive strength } (\sigma_B) = P/D \tag{3}$$

$$\text{Thus} \quad \sigma_B = 1.57 * (\sigma_t) \tag{4}$$

As indirect tensile strength is commonly in the range of 10–20% of the unconfined compressive strength of intact rock we have

$$\sigma_B = 15\,\% \text{ to } 30\,\% \text{ of } (\sigma_{ci}) \tag{5}$$

The discusser had proposed that the allowable bearing capacity of a shallow foundation on a jointed rock mass could be conservatively taken as the allowable bearing capacity on the basis that there would be no fracture of intact rock up to that loading and it would represent the onset irrecoverable deformations of the rock mass (Burman and Hammett 1975).

Interestingly, the authors and the discusser would arrive at similar conclusions for the example cited by the former. The authors estimated an ultimate bearing capacity between 26 and 52 MPa, which with a conventional factor of safety of 3 would give allowable bearing capacity between 8.7 and 17.3 MPa. The discusser would estimate an allowable bearing capacity between 15% and 30% of the intact unconfined strength of 50 MPa, or 7.5 to 15 MPa. Perhaps this level of agreement is not wholly unexpected as both the authors and the discusser started from similar points; the reduction in strength of a jointed mass as compared to the strength of its elements. However, the journeys were quite different, the authors' being almost entirely experimental while the discusser's was more conceptual, being based an understanding of the mechanics of discontinua.

By way of constructive comment the discusser would make the following points:

- The discusser would have expected the n-parameter in Table 2 and the $J_{f(1\ \mathrm{cm})}$ factor in Fig. 1 to have had similar values for extreme orientations of 0° and 90° as found by Brown (1970) in his tests on block jointed plaster models. Is it possible that the difference determined by the authors arises from differences in the experimental arrangements for the two extremes?
- The discusser would appreciate an explanation as to how the $J_{f(1\ \mathrm{cm})}$ factor can vary by at least an order of magnitude (1,000%) between 0° and 90° orientations when the n-factor varies by about 20% between the two limits given that, for the same number of the same type of joints, J_f is inversely proportional to n from Eq. (13).
- As discussed above the discusser does not accept that sliding and rotation modes will lead to foundation failure and considers that Fig. 2 should provide for only the shearing/splitting mode.
- It is noted that for low values of J_f, less than, say, 10, the ratio of jointed to intact strengths is close to unity. It is presumed that low values of J_f represent blocks that are large relative to the footing dimension and jointed strength might be thought to approach intact strength in that case. It is the discusser's expectation that there would be significant strength reduction even for the condition of large blocks, because of inescapable nonuniformity effects on mass strength, and hence the discusser is concerned that, for low to moderate values of J_f, Fig. 2 may result in strength ratios that are unrealistic and bearing capacities that are nonconservative.
- In most practical circumstances bearing capacity is determined by foundation movement criteria and neither approach deals with that issue directly. The discusser, in attempting to bypass the settlement issue by means of a no-fracture concept, acknowledges that it is considerably more intractable than strength based approaches.

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Discussion of "Bearing Capacity of Shallow Foundations in Anisotropic Non-Hoek–Brown Rock Masses" by Mahendra Singh and K. Seshagiri Rao

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The authors have presented the results of uniaxial and triaxial shear tests on intact and jointed rocks to evaluate bearing capacity of shallow foundations on the jointed rock masses. The discussers, having conducted numerous similar tests on jointed rocks samples, with and without gouge (Arora 1987; Trivedi 1990; Arora and Trivedi 1992), find the experimental work to be of keen interest and importance and are particularly impressed with the considerable efforts made to present actual material parameters and shear strength data based on direct measurements of deformation fields by different investigators including that by one of the discusser (Arora 1987; Ramamurthy and Arora 1994). With the help of a few common parameters such as intact rock mass strength (σ_{ci}), joint roughness parameter (r), joint inclination parameter (n), joint number parameter (J_n), joint thickness parameter (J_t), joint depth parameter (J_{dj}), joint factor (J_f), and strength ratio (σ_{cr}), the discussers shall present some results and reasons why they feel "uniqueness of the joint factor criteria" should still be considered valid and reasonable over the interpretation of bearing capacity. This discussion is planned with the help of following arguments.

Joint Orientation

Hoek and Bray (1981) considered the ratio of base width to depth of the joint to be greater than the tangent of base friction angle for stable blocks. The load bearing block may slide upon the condition of base angle greater than friction angle. These eventualities do not call for bearing capacity evaluation; rather, a slip alone. Hence the consideration of sliding and rotation plots for strength

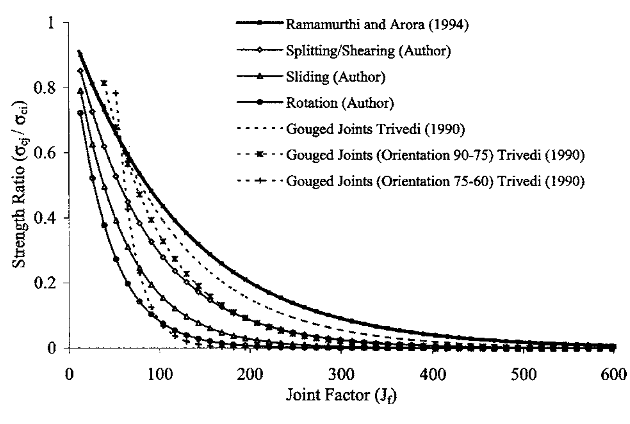

Fig. 1. Variation of strength ratio with joint factor

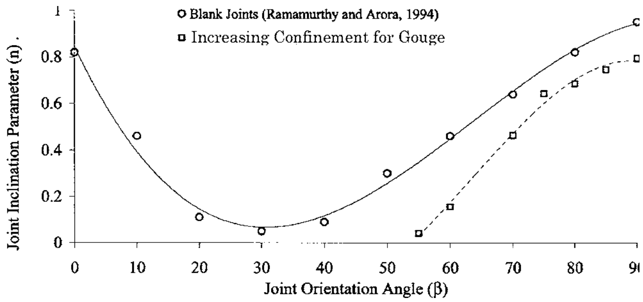

Fig. 2. Variation of inclination parameter with joint orientation angle and confinement

ratio (Fig. 1) may not be necessary. The consideration of shearing and splitting failure by the authors for the estimates of strength ratio fall closely within the standard deviation from the Eq. (1) if equivalent value of joint inclination parameter (n) for appropriate joint orientation angle (β) and confinement be selected (Fig. 2). It may be noted that increase in confinement tends to diminish the effect of β. Therefore, Eq. (1) in the present form (Ramamurthy and Arora 1994) is an acceptable guideline for simplistic estimate of strength ratio as also admitted by the authors late in the text

$$\sigma_{cr} = \exp(-0.008 J_f) \tag{1}$$

where σ_{cr}=strength ratio of jointed and intact rock; and J_f=joint factor=joint number (J_n)/inclination parameter $(n)^*$ roughness parameter (r).

Effect of Gouge

One of the discussers (Trivedi 1990) evaluated the strength ratio of jointed Kota sandstone (σ_{ci}=80.4 MPa) by uniaxial compression tests with artificially induced joints varying in number, inclination, depth, and thickness (J_t) of the fill material (gouge). Having gradual reduction in joint orientation angle (β) in combination with joint number, the plot for strength ratio was observed to depart progressively (Fig. 1) from the best fit shown by Eq. (1). This departure was more prominent for larger inclinations and thickness of the gouge (Arora and Trivedi 1992). Incidentally, the strength ratio observed for larger inclination (β=60 to 75°) for gouged joints match significantly with the predictor for sliding

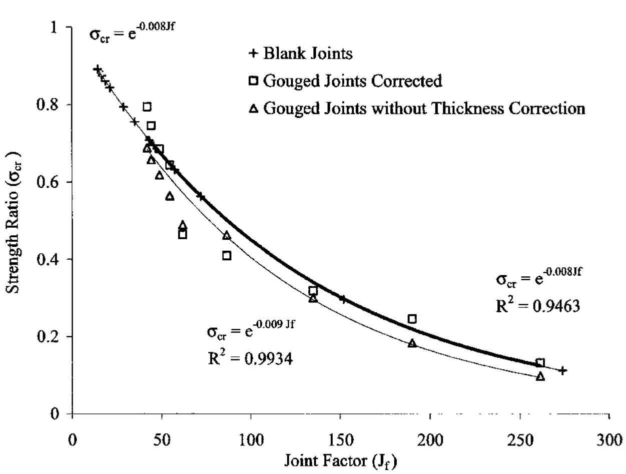

Fig. 3. Variation of strength ratio with joint factor (without sliding points)

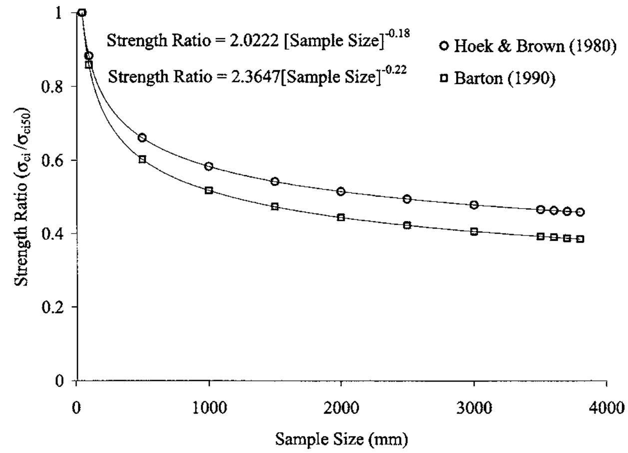

Fig. 4. Variation of strength ratio ($\sigma_{ci}/\sigma_{ci50}$) with sample size

failure by the authors (Fig. 1). Ramamurthy (1992) suggested that the consideration of joint factor must be based upon the strength equivalent of a number of parallel joints at (β=90°). Incorporating thickness and joint depth correction (Fig. 3), consequent fitting tends to converge with Eq. (1) without the data points of sliding failure (β=60 to 75°) which still falls apart

$$\sigma_{cr} = \exp(-0.008 J_{ftd}) \quad (R^2 = 0.9463) \tag{2}$$

where J_{ftd} (J_f corrected)=$J_n^* J_{dj}^* J_t / n^* r$; J_{dj}=correction for the depth of joint; and J_t=correction for the thickness of gouge in joint.

Scale Effects

Regarding the scale effect based upon the size of sample, the authors have interpretation by Medhurst and Brown (1998), which considers coal samples of a lone compressive strength (σ_{ci}=32.7 MPa). While Barton (1990) criteria for the scale effect considers samples in a large range of compressive strength (350–150 MPa). Further, this criterion (Fig. 4) covers the sizes in the range of laboratory scale to large field sample. In order to consider the footing size effect, the strength changes consequent to actual size of the footing should be evaluated.

Bearing Capacity

The modern-day problem of bearing capacity is essentially considered as a function of progressive failure. The failure is initiated at contact points by stress concentration and its progression through the joints. The progression of failure through the joints causes dilation of the joints and hence changes in the stress pattern. The adjustments in joint angle of friction are also required for the plane-strain case since the strength ratio is derived from triaxial tests. As footing load is gradually increased, shear strength is not simultaneously mobilized at all joints on the slip surface. Shear strength is first mobilized at points where shear strains are greatest, with strength mobilization progressing to other regions as shear strains develop and advance through the joints. The consequence of this phenomenon is that when the peak load-carrying capacity of the foundation is reached, only the material along the slip plane may be contributing a shear strength that is dependent on the joint's peak friction angle, whereas the remaining regions contribute a shear strength dependent on a residual, constant volume, or critical state friction angle. This phenomenon becomes more prominent with increasing footing width, and for a massively jointed rock.

Further, the observation from triaxial shear tests on jointed rocks that oriented jointed rocks subjected to low levels of confinement have a more marked difference between peak and minimum strength as compared to the rock subjected to a higher level of confinement (Fig. 2) suggests that strength variation at shallow depths is more acute due to low confinement conditions or for smaller footing widths. The discussers take a conscious note of difficulty in assessment of rock mass variants as once reiterated by Karl Terzaghi to R. L. Loofbourow (in 1953), "how are you going to describe what you know about the physical properties of rocks and how can you correlate your experiences with those of others if you have no adequate language in common?" While a common language for the strength ratio seems visible, at the same time, without the concerns of progressive failure in modern day bearing capacity evaluation, an approach considering the stress condition of only two elements may be termed archaic.

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Closure to "Bearing Capacity of Shallow Foundations in Anisotropic Non-Hoek–Brown Rock Masses" by Mahendra Singh and K. Seshagiri Rao

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The writers are thankful to the discussers for their keen interest in the paper. The discussions presented by these discussers are more in the form of contributions and add to the understanding of the complex problem of assessing bearing capacity of shallow foundations in jointed rock masses. Nevertheless, there are certain points raised and clarification sought. Point-to-point response is presented in the following paragraphs.

Discussers: A. Trivedi and V. K. Arora

Uniqueness of Joint Factor Criteria and Failure Mode

It is stated by the discussers that consideration of sliding and rotation plots for strength ratio may not be necessary as foundations are unlikely to fail either by slipping or rotation. Similar observation is made by the other discusser also. Bell's approach of computing bearing capacity as advocated by Wyllie (1992) has been used in this paper. The approach considers two elements, one just outside and the other just below the footing. These elements have been termed as elements I and II in the paper. The argument given by the discussers may be valid for element II (just below the footing); however, element I acts under very low confining stress. For a shallow footing the jointed mass above element I will be free to dilate and blocks may rotate or slip along the joint plane depending on the joint geometry. So as long as Bell's approach is used, any failure mode that is possible under uniaxial loading condition may occur. It is important to mention here that as per the present approach, if rotation or sliding is inhibited, say for example by bolting, it may result in considerable enhancement in the ultimate bearing capacity.

Effect of Gouge

The effect of gouge was beyond the scope of the present paper. The information presented by the discussers is quite interesting and a full-fledge publication on this aspect related to bearing capacity of foundation by the discussers may be of great importance.

Scale Effects

As stated in assumptions for non-Hoek–Brown jointed rock mass, the foundation is assumed to be sufficiently large compared to the block size so that the scale effects are minimized. There are no fixed rules but the mass may be expected to behave free of scale effects if foundation width is more than about 5–6 times the spacing of the joints.

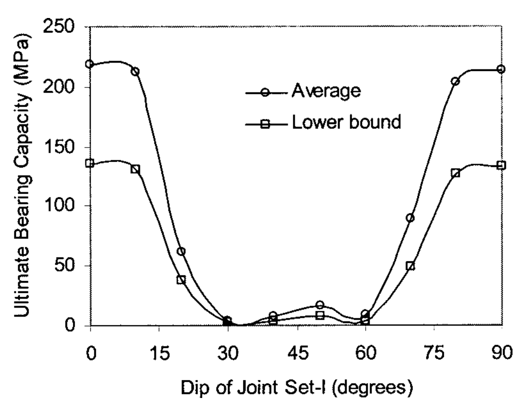

Fig. 1. Anisotropy in ultimate bearing capacity

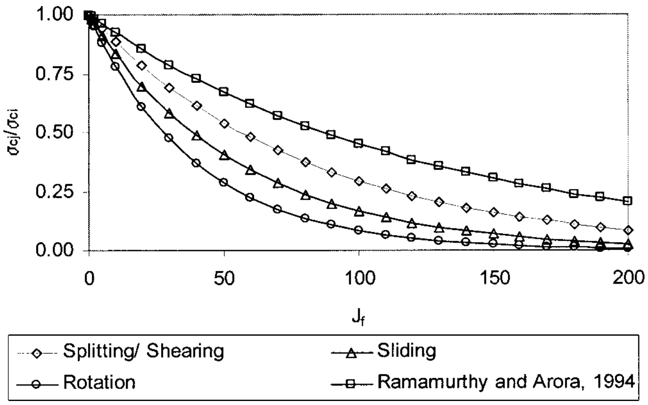

Fig. 2. Variation of strength with J_f

Nonuniformity in Stresses

There is always nonuniformity in contact stress across joints. By considering the footing to be sufficiently large compared to the individual blocks, the consideration of average stress on individual blocks is not likely to accrue appreciable error. However, for smaller foundations, appreciable error may be introduced due to this aspect. The present approach is applicable only for those situations where footing size is large enough compared to the size of the block, as discussed in the assumptions.

Bearing Capacity vis-a-vis Progressive Failure

The discussers have pointed out that the bearing capacity is essentially a function of progressive failure. It is also argued that the shear strength is mobilized at points where shear stresses are maximum. They have also termed the present approach "archaic," as the approach considers only two elements. The writers would like to emphasize that the present approach, and, for that matter, any empirical or semiempirical approach, is generally derived from experience in the field or laboratory. These approaches, in general, do not go into the mechanics in much detail. The present paper aims at presenting a simple and practical approach of making estimates of bearing capacity that could be easily understood and used by the practicing engineers in the field. Highly sophisticated numerical techniques in 2D and 3D are available in literature for both in the continuum and discontinuum mechanics. These techniques could look into the details elaborated by the discussers. The techniques were obviously not the aim of the present research.

Discusser: Brian C. Burman

The discusser also points out that the only possible failure modes for semi-infinite medium beneath a foundation are splitting or shearing. Also, matter of nonuniformity of stresses is indicated. Both these aspects have been clarified above. The discusser has also noted an interesting finding of his work, which concludes that the strength of a jointed mass will lie between the limits determined by Brazilian and unconfined compression test results. It is interesting that for the example worked out in the paper to demonstrate the methodology, the allowable bearing capacity closely matches with that obtained by the discusser. The writers would like to term it only a pleasant coincidence. The approach suggested by the discusser is dependent only on the UCS of the intact rock. The joint frequency, their shear strength, and orientation (anisotropy) are not considered in this approach. To emphasize the difference, the ultimate bearing capacity of the foundation considered in the paper was worked out by changing the orientation of the joints. Orthogonal sets of joints were considered with orientations 0/90, 10/80, 20/70, ..., 90/0°, respectively. The variation of the ultimate bearing capacity is given in Fig. 1. It may be seen that the bearing capacity obtained from the suggested approach by the writers is highly anisotropic and the results are different from that suggested by the discusser. The following clarifications are offered for the points raised by the discusser.

Parameter *n*

The parameter n was derived by Arora (1987) by performing UCS tests on specimens with different frequency and orientation of joints. An equivalent criterion was obtained between the effect of orientation and joint frequency on the strength of the specimen. Based on this equivalence the joint inclination parameter was derived. The low value of this parameter for $\theta=90°$ indicates that the joints oriented parallel to the loading direction have more influence on strength than the joints oriented perpendicular to the loading direction.

Values of $J_{f(1\ \mathrm{cm})}$

The charts have been prepared to simplify the computational procedure to obtain J_f. From the chart one can read corresponding J_f for joints spaced at 1 cm normal to their joint plane. The parameter n varies from 1.00 to 0.046 and has great influence on J_f values. The parameter J_n will also vary as it represents the number of joints per meter in the direction of loading. Some typical computations are given below to make computation procedure clear.

Say

$$\phi_j = 30° \Rightarrow \mathrm{r} = \tan(\phi_j) = 0.578$$

For $\theta=0°$; where J_n=number of joint per meter in loading direction=100; and n=1.00 (Table 2 of the paper)

$$J_{f(1\ \mathrm{cm})} = \frac{J_n}{nr} = \frac{100}{1 \times 0.578} = 173$$

For $\theta=60°$

$$J_n = \frac{100}{(1/\cos\theta)} = 50$$

$$n = 0.046$$

$$J_{f(1\ \mathrm{cm})} = \frac{J_n}{nr} = \frac{50}{0.046 \times 0.578} = 1{,}880$$

Similarly for $\phi j=10°$, r=0.176

For $\theta=0°$

$$J_n = 100; \quad n = 1.00; \quad J_{f(1\ \mathrm{cm})} = \frac{100}{1 \times 0.176} = 568$$

For $\theta=60°$

$$J_n = \frac{100}{(1/\cos\theta)} = 50; \quad n = 0.046$$

$$J_{f(1\ \mathrm{cm})} = \frac{50}{0.046 \times 0.176} = 6{,}176$$

Sliding and Rotation Modes of Failure

The clarification is the same as given above for the other discussers Trivedi and Arora.

Low Values of J_f

The discusser presumes that low value of J_f will represent rock blocks that are large relative to the footing dimension. It is emphasized as discussed earlier that the present approach is applicable only when the footing size is sufficiently large. In case the block size is large this approach should not be used. Low values of J_f will be obtained when the spacing between the joints approaches 1 m (i.e., the rock mass is massive and joints are not critically oriented). Fig. 6 of the paper indicates that the applicability of the J_f concept has been verified for a large range of J_f. The minimum value of J_f in this figure is very close to 10. Fig. 2 of this closure presents variation of σ_{cj} with J_f for all modes of failure. The variation as per Ramamurthy and Arora (1994) (Fig. 6 of the paper) is also shown in this figure. For $J_f = 10$, σ_{cj} will be 88, 84, and 78% of σ_{ci} for splitting, shearing, and rotation modes, respectively. As per Ramamurthy and Arora (1994) this value will be 92% of σ_{ci}. It is expected that for $J_f < 10$, the strength should follow the same trend as given in the figure. It is however to be reported that in the analysis, the data were available for the specimens with small joint spacing, and no experimental data for a mass with joint spacing as large as 1 m were available. To get a true picture, validation in the field or laboratory is needed.

Settlement Issue

The writers agree with the discusser that settlement is an important issue and in many cases governs the allowable pressure. With more understanding now on strength aspect, the research should also focus on assessing the settlement of foundations.

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Discussion of "Observed Performance of a Deep Multistrutted Excavation in Shanghai Soft Clays" by G. B. Liu, Charles W. W. Ng, and Z. W. Wang

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The authors provide an interesting observational record on a deep excavation in soft clay in Shanghai, China. The discusser has similar research interests and had been involved in several excavation cases in Taipei, Taiwan, which has a similar ground condition. In the paper the authors describe how bulge-shaped lateral wall deflections were observed during the excavation. This behavior is similar to the discusser's observations in Taiwan. However, double layers of grouting increase the difficulty during the construction and the discusser is interested in knowing the reason of using such treatment in this case. Also, if the generated behaviors of diaphragm wall and ground caused by the compaction grouting (before any main excavation) could be provided by the authors, it would be helpful for this discusser to compare as well as to carry out a further study regarding the influence of grouting in the excavation.

Figs. 1 and 2 show the observed behavior of diaphragm walls and the cross section of the excavation at City Hall Station and Yung-Tsung Station in Taipei metro. At these sites, 3–4 m-thick high-pressure jet grouting had been carried out to fully replace the soil beneath the final excavation level. Details of the grouting have been shown in Hsiung et al. (2001). Excavation depth of the sites in Taipei is 1.2–3.3 m deeper than that of the case the authors study, but the magnitude of the maximum lateral movement of the wall is similar. Also, some outward movements of the diaphragm wall were generally seen in the excavation with grouting (Wong and Poh 2001; Hsiung et al. 2001), but it seems such movements have not occurred in the case in Shanghai. The bulge-shaped lateral wall deflection was observed at sites in both Taipei and Shanghai. Slightly greater movements were found at top of the wall at the cases in Taipei and the longer construction period of shallow excavation and prop installation should be the reason for that.

The ratio of maximum wall displacement ($\delta_{h\max}$) to maximum excavation depth (H) varies from 0.12% to 0.25%, which was interpreted from the data presented by the authors. Wu et al. (1997) reported that 0.07–0.2% of the same ratio based on the observations from excavations in Taipei metro. The thicker walls (1.0–1.2 m) were used in Taipei and this is expected to be the key factor to induce smaller wall displacements. Some data from Taipei have been plotted to discuss the relationship between maximum ground settlement ($\delta_{v\max}$) and $\delta_{h\max}$, as shown in Fig. 3. Among them, excavations at Taipei metro have short ex-

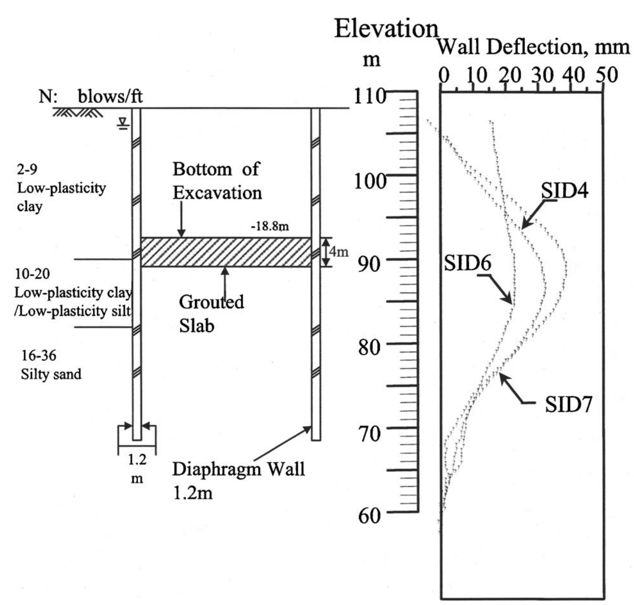

Fig. 1. City Hall Station

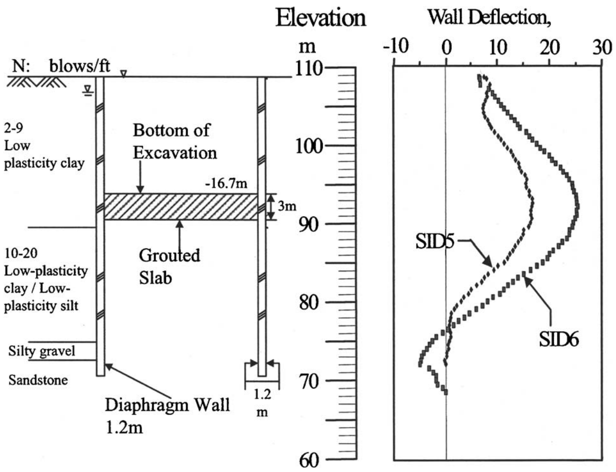

Fig. 2. Yung-Tsung Station

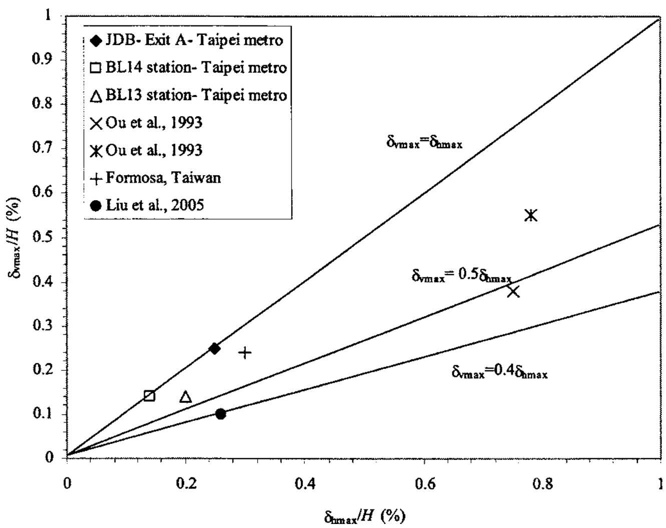

Fig. 3. Relationship between maximum ground settlement and maximum wall deflection

cavation sections rather than cases which Ou et al. (1993) have and excavations at Taipei metro use prestressed steel props for the excavation. It is interesting to see that cases from Taipei metro have the similar ratio of $\delta_{v\max}$ to $\delta_{h\max}$ with cases that Ou et al. (1993) referred. Therefore, for the outcome of the relationship between $\delta_{v\max}$ and $\delta_{h\max}$ that the authors suggested, it may be connected to ground conditions in Shanghai, not the use of a short excavation section and active prestressed steel struts.

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Discussion of "Estimating Consolidation Coefficient and Final Settlement: Triangular Excess Pore-Water Pressure" by Sushil K. Singh

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The author has, in this paper, developed methods for the evaluation of c_v and final settlement for cases of triangular excess pore pressure dissipation in clays. This is a matter not previously pursued, essentially because such situations are not normally used for laboratory investigation of consolidation properties.

However, the "diagnostic curve" method proposed here for the evaluation of c_v is not new, as claimed by the author, but was first developed by the discusser nearly thirty years ago and published in *Geotechnique* as the "velocity method" (Parkin 1978). This is hardly an obscure journal and highlights the importance of a diligent literature survey. Not mentioned here is the particular merit of this method in that it is completely independent of any initial elastic compression or subsequent secondary effects, which require assumptions and special procedures in the case of the t_{90} and $\log t$ methods still in standard use.

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Closure to "Estimating Consolidation Coefficient and Final Settlement: Triangular Excess Pore-Water Pressure" by Sushil K. Singh

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The proposed "diagnostic curve method" is for cases of triangular excess pore-water pressure. The "velocity method" proposed by the discusser (Parkin 1978) is for a rectangular excess pore-water pressure. However, the diagnostic curve method is different from the velocity method in respect to the following aspects:

1. The velocity method requires drawing a straight line through the initial points, while the diagnostic curve method does not.
2. If the initial points are absent or not measured, the velocity method cannot be applied for estimating the consolidation coefficient, while the diagnostic curve method can be applied.

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