Effect of Fines and Confining Stress on Undrained Shear Strength of Silty Sands[a]

Discussion by Robert W. Day,[2] Fellow, ASCE

The author has prepared a paper on the effect of fines on the shear strength of silty sand. As indicated in Table 4 (USCS), silty sands (SM) are classified as those soils with more than 12% fines and that have Atterberg Limits data (LL, PI) that plot below the A-line on the Plasticity Chart.

The discusser believes that the main criterion governing the shear strength of silty sands is whether or not the fines are plastic. For nonplastic fines, the shear strength of the silty sand will be governed by the frictional and interlocking resistance between individual soil particles. But if the fines are plastic, then the shear strength will be governed by the plasticity characteristics (LL and PI). This is one of the reasons that the discusser believes that soils should be classified as either nonplastic (cohesionless) or plastic (cohesive) such as shown in Table 5 (Day 1994, 1999).

APPENDIX. REFERENCES

Day, R. W. (1994). ''Inorganic Soil Classification System Based on Plasticity.'' *Bull. Assn. of Engrg. Geologists*, 31(1), 21–24.

Day, R. W. (1999). *Geotechnical and foundation engineering: Design and construction*. McGraw-Hill, New York, 808 pp.

[a]June 1998, Vol. 124, No. 6, by S. Thevanayagam (Paper 14559).

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Closure by S. Thevanayagam,[3] Member, ASCE

The writer thanks the discusser for his interest in this paper. The first question raised for clarification is whether the silty sand involved in the study contained plastic or nonplastic fines. The answer is nonplastic. The second question pertains to how one should classify soils, and the discusser offers a chart (Table 5) for classification as an alternate to the ASTM Unified Soil Classification System (Table 4).

The latter question is better answered by starting to ask what we want the classification to reveal about the anticipated behavior of the soil. It is an acceptable notion that if the fines in a soil mix are plastic then the physicochemical interactions among finer grains may influence the soil behavior whereas if they are nonplastic then intergrain friction may influence the behavior. The traditional thinking has been rather conveniently laid that if the fines content exceeds 50% of the ''soil mix'' then the anticipated behavior resembles that of the finer grain soil, in general, and vice versa if it is less than 50% as shown in both Tables 4 and 5. Table 5 is not too different from the USCS chart (Table 4) except for differentiating the influence of plastic and nonplastic fines.

The paper under discussion addresses this question differently by asking further: (1) What relative roles do the finer and coarser grains play in the load-deformation-resistance characteristics of a soil mix? (2) When does the behavior resemble that of the host coarser grain soil or the host finer grain soil? and (3) What index parameter does better characterize the anticipated behavior either in terms of the behavior of the

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TABLE 4. Unified Soil Classification System (USCS)

Subdivisions (1)	USCS symbol (2)	Typical names (3)	Laboratory classification criteria (4)
(*a*) Coarse-grained soils (more than 50% retained on No. 200 sieve)			
Gravels (more than 50% of coarse fraction retained on No. 4 sieve)	GW	Well graded gravels or gravel-sand mixtures, little or no fines	Less than 5% fines[a]; $C_u \geq 4$ and $1 \leq C_c \leq 3$
	GP	Poorly graded gravels or gravelly sands, little or no fines	Less than 5% fines[a]; $C_u < 4$ and/or $1 > C_c > 3$
	GM	Silty gravels, gravel-sand-silt mixtures	More than 12% fines[a]; minus No. 40 soil plots below A-line
	GC	Clayey gravels, gravel-sand-clay mixtures	More than 12% fines[a]; minus No. 40 soil plots on or above A-line
Sands (50% or more of coarse fraction passes No. 4 sieve)	SW	Well-graded sands or gravelly sands, little or no fines	Less than 5% fines[a]; $C_u \geq 6$ and $1 \leq C_c \leq 3$
	SP	Poorly graded sands or gravelly sands, little or no fines	Less than 5% fines[a]; $C_u < 6$ and/or $1 > C_c > 3$
	SM	Silty sands, sand-silt mixtures	More than 12% fines[a]; minus No. 40 soil plots below A-line
	SC	Clayey sands, sand-clay mixtures	More than 12% fines[a]; minus No. 40 soil plots on or above A-line
(*b*) Fine-grained soils (50% or more passes the No. 200 sieve)			
Silts and clays (liquid limit less than 50)	ML	Inorganic silts, rock flour, silts of low plasticity	Inorganic soil; PI < 4 or plots below A-line[b]
	CL	Inorganic clays of low plasticity, gravelly clays, sandy clays, etc.	Inorganic soil; PI > 7 and plots on or above A-line[b]
	OL	Organic silts and organic clays of low plasticity	Organic soil; LL (oven dried)/LL (not dried) < 0.75
Silts and clays (liquid limit 50 or more)	MH	Inorganic silts, micaceous silts, silts of high plasticity	Inorganic soil; plots below A-line
	CH	Inorganic highly plastic clays, fat clays, silty clays, etc.	Inorganic soil; plots on or above A-line
	OH	Organic silts and organic clays of high plasticity	Organic soil; LL (oven dried)/LL (not dried) < 0.75
(*c*) Peat			
Highly organic	PT	Peat and other highly organic soils	Primarily organic matter, dark in color, and organic odor

[a]''Fines'' are those soil particles that pass the No. 200 sieve. For gravels and sands with between 5 and 12% fines, use of dual symbols required (i.e., GW-GM, GW-GC, GP-GM, or GP-GC).

[b]If $4 \leq PI \leq 7$, then dual symbol (i.e., CL-ML) is required.

TABLE 5. Inorganic Soil Classification Based on Plasticity (ISBP)

Subdivisions (1)	ISBP symbol (2)	Typical names (3)	Laboratory classification criteria (4)
		(*a*) Nonplastic soils	
Gravels (greater fraction of total sample is retained on No. 4 sieve)	GW	Well graded gravels, sandy gravels, silty-sandy gravels	$C_u \geq 4$ and $1 \leq C_c \leq 3$
	GP	Poorly graded gravels, gravel-sand-silt mixtures	$C_u < 4$ and/or $1 > C_c > 3$ and percent passing No. 200 sieve is less than 15%
	GM	Poorly graded, nonplastic silty gravels, gravel-silt mixtures	$C_u < 4$ and/or $1 > C_c > 3$ and percent passing No. 200 sieve is greater than or equal to 15%
Sands (greater fraction of total sample is between No. 4 and No. 200 sieves)	SW	Well graded sands and gravelly sands	$C_u \geq 6$ and $1 \leq C_c \leq 3$
	SP	Poorly graded sands or sand-gravel-silt mixtures	$C_u < 6$ and/or $1 > C_c > 3$ and percent passing No. 200 sieve is less than 15%
	SM	Poorly graded, nonplastic silty sands, sand-silt mixtures	$C_u < 6$ and/or $1 > C_c > 3$ and percent passing No. 200 sieve is greater than or equal to 15%
NP silt	MN	Nonplastic silts, rock flour. Gravelly silts and sandy nonplastic silts	Greater fraction of total sample passes No. 200 sieve. Silts are nonplastic
		(*b*) Plastic soils	
Plastic silts (minus No. 40 fraction plots below A-line)	GM[a]	Plastic silty gravels, gravel-silt mixtures	50% or more particles retained on No. 200 sieve with greater fraction of gravel size
	SM[a]	Plastic silty sands, sand-silt mixtures	50% or more particles retained on No. 200 sieve with greater fraction of sand size
	ML, MI, MH	Plastic silts, sandy silts, and clayey silts	For silt of low plasticity (ML), $PI \leq 10$; for silt of intermediate plasticity (MI), $10 < PI \leq 30$; for silt of high plasticity (MH), $PI > 30$
Clays (minus No. 40 fraction plots on or above A-line)	GC[a]	Clayey gravels, gravel-clay mixtures	50% or more particles retained on No. 200 sieve with greater fraction of gravel size
	SC[a]	Clayey sands, sand-clay mixtures	50% or more particles retained on No. 200 sieve with greater fraction of sand size
	CL, CI, CH	Clay, sandy clays, and silty clays	For clay of low plasticity (CL), $PI \leq 10$; for clay of intermediate plasticity (CI), $10 < PI \leq 30$; for clay of high plasticity (CH), $PI > 30$

[a]Must state of high, intermediate, or low plasticity.

coarser or finer grain soil? It underscores the fact that a classification based on 50% fines content or less is insufficient. What matters is not whether more or less than 50% fines is there, but how they are arranged in the soil matrix. It does also matter, as the discusser indicates, whether or not the fines are physicochemically active.

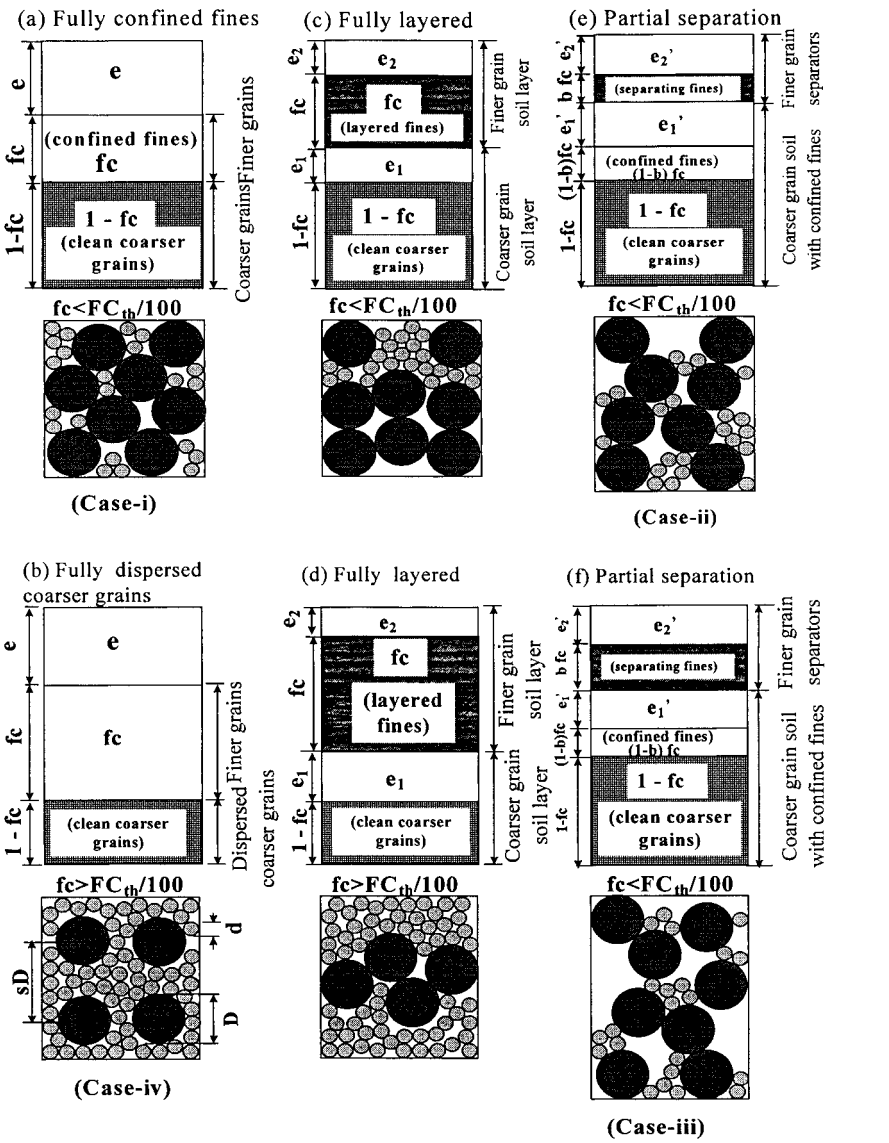
Figs. 15(a–e) show a limited number of cases of particle arrangements, schematically, for a two-sized particle system to illustrate this view point. The first category [Fig. 15(a)] is obtained when the finer grains are fully confined within the void spaces between the coarser grains with no contribution whatsoever in supporting the coarser grain skeleton. The second category [Fig. 15(b)] is applicable when the coarser grains are fully dispersed in the finer grain matrix. The third category [Figs. 15(c and d)] is possible when the coarser and finer grains constitute a fully layered system where the coarser grain layers have no fines contained in them and vice versa. A fourth category [Figs. 15(e and f)] is obtained when partial separation of coarser grains by the finer grains is present. Figs. 15(a, c, e, and f) are relevant at low finer grains content (FC). Figs. 15(b and d) are relevant at high FC. Fig. 16 summarizes the relative role of and interaction between the coarser and finer grains for each case shown in Figs. 15(a, b, e, and f). Fig. 17 identifies the corresponding regions in a void ratio versus finer grains content plot using e_s, e_f, and simple characteristics such as the respective maximum and minimum void ratios achievable for the host coarser ($e_{max,HS}$, $e_{min,HS}$) and finer grain ($e_{max,HF}$, $e_{min,HF}$) soils.

Transition from Fig. 15(a) to Fig. 15(b) occurs naturally with an increase in FC beyond a threshold value (FC_{th}, Fig. 16, Thevanayagam 1999a,b). This transition fines content can be much less than 50%. The category shown in Fig. 15(a) is possible only if (1) the size of the finer grains is much smaller than the possible minimum pore opening size in the coarser grain skeleton—for spherical particles this implies that $D/d > 6.5$ where d and D = sizes of finer and coarser grains, respectively; and (2) the intergranular voids are not completely filled with the fines. From a conceptual standpoint FC_{th} is expected to occur when the interfine void ratio e_f decreases below $e_{max,HF}$ where $e_{max,HF}$ = maximum void ratio of the pure finer grain soil beyond which it has no appreciable strength. The rationale is that as e_f decreases below $e_{max,HF}$, the finer grains are packed close enough so that direct finer-grain-to-finer-grain interaction becomes active. At $FC < FC_{th}$, primarily the intergranular contacts between the coarser grains affect the mechanical response. The intergranular void ratio e_s may be considered as an index of active contacts.

When $FC > FC_{th}$ the finer grains begin to play a rather important role. The coarser grains begin to disperse [Fig. 15(b)] and provide a sort of reinforcement effect until they are separated sufficiently apart when FC exceeds a limiting fines content FC_1 (Fig. 16). The limiting fines content can be much higher than 50%. At $FC_{th} < FC < FC_1$, neither e_s nor e_f can sufficiently represent the active contacts. Beyond FC_1, the soil behavior may be characterized using e_f as an index. Whether the fines are plastic or not does have an influence on the location the transition regions in Fig. 17.

Fig. 17 shows the regions corresponding to the different cases schematically with gradual transition between them. Based on Fig. 17, using e_s and e_f as indices, for example, one may deduce the relative trend in the behavior of different soil mixes prepared (using the same fine and coarse grains) at the same global void ratio but different fines contents (Chang 1990).

This is a rather simplified portrayal of real soils containing a mix of particles of different sizes and fines of different activity, yet allows a rational way to think of the anticipated behavior beyond what could be judged based on the classifi-



Notes: e=Global Void Ratio; fc=FC/100; FC_{th}=threshold fines content (%)

FIG. 15. Particle Arrangement and Intergranular Phase Diagram

Case	FC	e_s	e_f	Roles of coarser-grains and finer-grains	Fig.
i	$FC<FC_{th}$	$e_s<e_{max,HS}$	$e_f>e_{max,HF}$	Finer grains are inactive (or secondary) in the transfer of inter particle forces. They may largely play the role of "filler" of intergranular voids. The mechanical behavior is affected primarily by the coarser grain contacts.	15(a)
ii		e_s *near* $e_{max,HS}$		Finer grains may be supporting the coarser-grain skeleton that is otherwise unstable. They act as a load transfer vehicle between "some" of the coarse-grain particles in the soil-matrix while the remainder of the fines play the role of "filler" of voids.	15(e)
iii		$e_s>e_{max,HS}$		Finer grains may play an active role of "separator" between a significant number of coarse-grain contacts and therefore begin to dominate the strength characteristics.	15(f)
iv-2	$FC_{th}<FC<FC_l$		$e_f<e_{max,HF}$	The fines may carry the contact and shear forces while the coarser grains may act as reinforcing elements embedded within the finer grain matrix.	15(b)
iv-1	$FC>FC_l$	$e_s>>e_{max,HS}$	$e_f<e_{max,HF}$	The fines may carry the contact and shear forces while the coarser grains are fully dispersed.	15(b)

Notes: $e_{max,HS}$, $e_{max,HF}$ = maximum void ratio of the coarser grains and finer grain media, respectively.

$$FC_{th} \le \frac{100e_s}{1+e_s+e_{max,HF}}\% = \frac{100e}{e_{max,HF}}\%; e_f > e_{max,HF}\ ;\quad FC_l \ge 100\left[1-\frac{\pi(1+e)}{6s^3}\right]\% \ge FC_{th}; e_f \le e_{max,HF}\ ;\quad s=1+ad/D=1+a/R_d,$$

a=10 (After Thevanayagam 1999).

FIG. 16. Granular Mix Classification

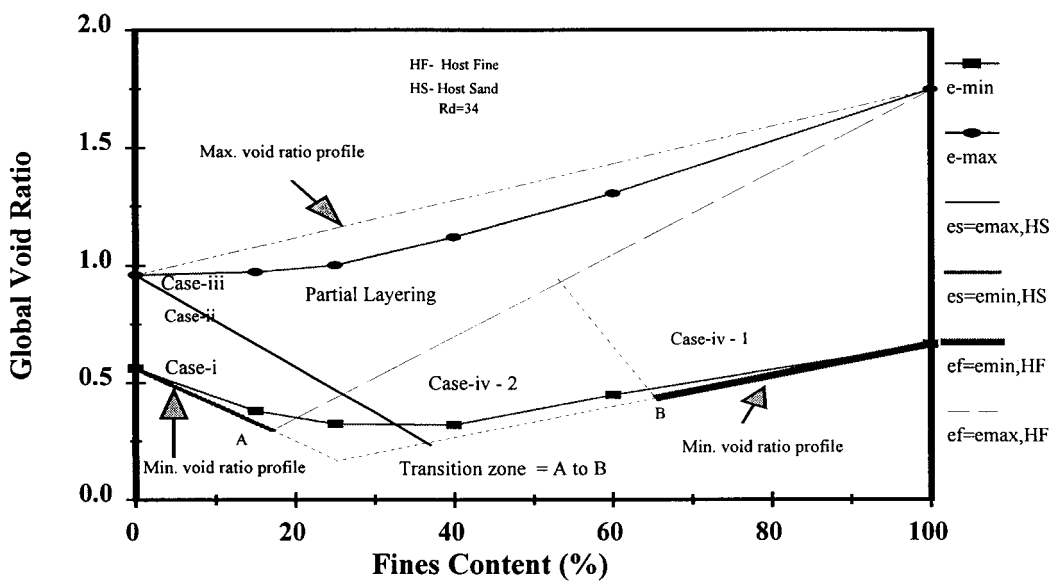


FIG. 17. Intergranular and Interfine Matrix Phase Diagram

cations of the kind shown in Tables 4 and 5. Such a rationale is not limited to sands and fines but also to gravel soils containing sand. That was the underlying concept behind the paper under discussion. It requires further development.

The experimental study in the paper was limited to nonplastic fines as clearly indicated in the experimental program.

APPENDIX. REFERENCES

Chang, N. Y. (1990). "Influence of fines content and plasticity on earthquake-induced soil liquefaction." *Contract No. DACW3988-C-0078*, U.S. Army Waterways Experiment Station, Vicksburg, Miss.

Thevanayagam, S. (1999a). "Intergrain contacts and shear modulus of nonplastic granular mixes." *Proc., 13th ASCE Conf. Engrg. Mech.*, N. Jones and R. Ghanem, eds., ASCE, Reston, Va. (available on CD-ROM).

Thevanayagam, S. (1999b). "Role of intergrain contacts, friction, and interactions on undrained response of granular mixes." *Proc., Int. Workshop on Phys. and Mech. of Soil Liquefaction*, P. V. Lade and J. A. Yamamuro, eds., Balkema, Rotterdam, The Netherlands, 67–78.

Effects of Time on Capacity of Pipe Piles in Dense Marine Sand[a]

Discussion by John A. Focht Jr.,[5] Fellow, ASCE

The authors of this paper and the several preceeding papers published since about 1990 by one or more of that group are to be commended for their improved procedures for predicting the capacity of pipe piles in sand, particularly in reference to their increase in capacity with time. The data presented in Table 6 of this paper and in Table 1 of the paper by Jardine, et al. (1998) for API RP2A procedures encouraged the discusser to look back at his work in the mid-1950s and early 1960s. During that period, he, Bramlette McClelland, and William J. Emrich at McClelland Engineers, Inc. evolved and tried to improve standardized procedures for predicting the ''ultimate'' frictional capacity of pipe piles driven into sands. Those procedures that appeared in the McClelland Engineers ''Appendix C'' from 1953 through at least 1974, modified some with time, were later adopted for API RP2A, which underwent further slight modifications. A brief historical summary of capacity criteria for piles in both sand and clay was presented by Pelletier et al. (1993).

In reference to ''ultimate'' capacity, I am reasonably sure that we thought then that this capacity would be reached in a relatively short period of time—two to four weeks. To my knowledge there were no data from tests with longer setup times. From the available data (almost all from tests on relatively short piles conducted as verification tests, not research) we knew that actual capacities ranged widely coordinating poorly with our simple procedures. I do not remember making statistical analyses but would expect that the COV values would have been generally similar to the 0.86 in Table 1 of the Jardine et al. (1998) for shaft friction. We also knew at the time that the simple procedure would usually grossly underestimate unit skin friction at shallow penetrations, and thereby the total capacity of short piles. But we had the feeling that it might overestimate unit skin friction at considerable depths.

Out intent was to produce conservative but reasonable predictions for piles with penetrations of 100 to 200 ft anticipating considerable scatter of results. I am pleased that the mean *Qc*/*Qm* has been shown to be about 0.8, for that tends to validate what we thought we were doing. Our intuitive judgmental selection of criteria based on meager and widely scattered data was pretty good. Now, predictions can be made with less average conservatism and much greater reliability.

APPENDIX. REFERENCES

Jardine, R. J., Overy, R. F., and Chow, F. C. (1998). ''Axial capacity of offshore piles in dense North Sea sands.'' *J. Geotech. and Geoenvir. Engrg.*, ASCE, 124(2), 171–178.

McClelland Engineers Inc. (1967). ''Appendix C, criteria for predetermining pile capacity.'' *Rep. Section Included for Clients of Offshore Geotech. Investigations*.

Pelletier, J. H., Murff, J. D., and Young, A. C. (1993). ''Historical development and assessment of current API design methods for axially loaded piles.'' *Proc., 25th Offshore Technol. Conf., OTC 7157*, 253–282.

[a]June 1998, Vol. 124, No. 6, by F. C. Chow, R. J. Jardine, F. Brucy, and J. F. Nauroy (Paper 15534).

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Closure by F. C. Chow,[6] R. J. Jardine,[7] F. Brucy,[8] and J. F. Nauroy[9]

The writers thank the discusser for his contribution and for his reminder of the uncertainties faced by the early offshore pile designers in the 1950s and 1960s. The interpretation of noninstrumented pile tests is difficult, relying on several simplifying assumptions that can mask many important aspects of pile behavior. The recent advances have been achieved through the development and use of reliable on-pile instrumentation, capable of measuring the effective stresses at the pile-soil interface throughout pile installation and testing.

Reliability studies have shown that each reduction of 0.1 in the COV associated with a design method can lead to an order of magnitude improvement in foundation reliability. This highlights the importance of employing design methods with low COV values, particularly when low design factors of safety are adopted, as in the case of offshore pile foundations where factors of 1.5 are often used for the extreme design case.

The writers are pleased to see that new cases demonstrating the effects of time on the capacity of piles are continuing to appear and that pile designers are now able to take account of these effects in the planning and back-analysis of pile tests.

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Undrained Limit Analyses for Combined Loading of Strip Footings on Clay[a]

Discussion by Jean Salençon,[4] Member, ASCE, and Alain Pecker[5]

In their paper the authors occasionally and briefly refer to the contributions by Salençon and Pecker (1995a,b) on the evaluation of the bearing capacity of a strip footing under combined loading. A comparison between the results obtained by the authors and those obtained by the discussers (apparently the best results available previously to the paper) is worth making, and may help point out the breakthrough of the authors' very valuable contribution from the theoretical point of view of the application of limit analysis to the study of the bearing capacity of strip footings.

With the paper's notations the problem studied by the discussers corresponds to Mode II combined loading. Lower-

[a]March 1998, Vol. 124, No. 3, by Boonchai Ukritchon, Andrew J. Whittle, and Scott W. Sloan (Paper 15586).

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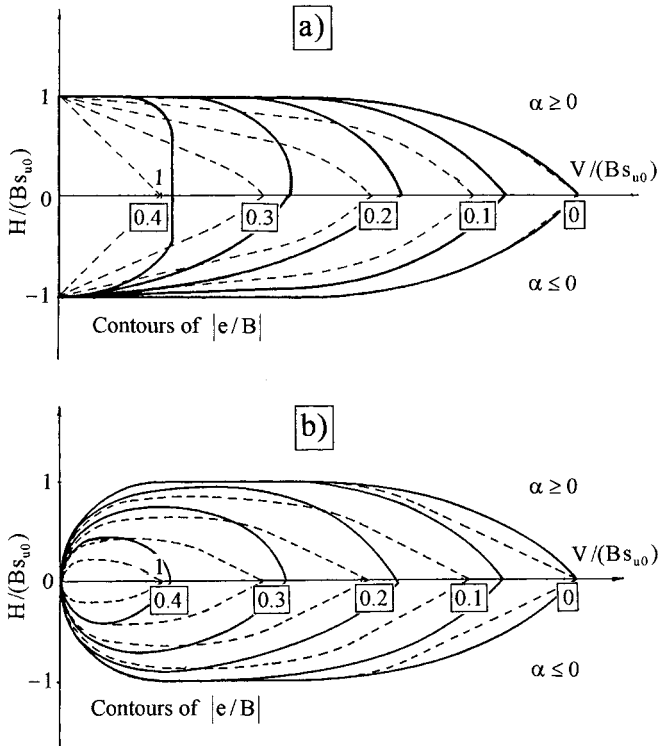

FIG. 21. Projections of Failure Surface for Combined Loading of Footing: (a) on Homogeneous Clay; (b) on Homogeneous Clay with Zero Tensile Strength

bound and upper-bound estimates have been produced through analytical procedures associated with numerical minimization on a few parameters as regards the kinematic approach; exact solutions have been obtained in some cases. With the notations of the paper, Fig. 21 shows the results obtained by the discussers to be compared with those presented in Fig. 12 (where the charts seem to be mislabeled).

The upper-bound estimates obtained in both papers are very close to each other when they don't coincide. It is noticeable that the dissymmetry of the failure surface predicted by the discussers' upper-bound results is now confirmed.

For zero eccentricity ($e/B = 0$) the lower bound in Fig. 12 is not as good as the one shown in Fig. 21. As a matter of fact it was proven that the lower bound in Fig. 21 matches the upper bound and yields the exact failure surface for $90° > |\alpha| > 7°$ and for $\alpha = 0°$; for $0° < |\alpha| < 7°$ the gap between the upper- and lower-bound estimates is very narrow. For increasing values of the eccentricity it had been observed by the discussers that the lower bound in Fig. 21 was getting poorer and the conjecture had been put forward that the exact failure surface was most likely in the form of the upper-bound estimate. It is a major contribution of the paper to produce reliable statically admissible stress fields for the lower-bound approach, thanks to which, in the considered case, the validity of the conjecture is now definitely proven; all the comments issued by the discussers about the assessment of the classical correction factors are corroborated and reformulated in the paper.

Salençon and Pecker (1995b) also drew attention to the importance of the tensile strength assumed for the clay layer and produced the corresponding results in the case of a zero tensile strength cohesive soil [Fig. 21(b)]. It is likely that the application of the very efficient numerical approaches of the authors would help fill the gap between the obtained lower- and upper-bound estimates of the failure surfaces.

It must be emphasized that the numerical procedures presented in the paper are of the highest interest in the production of good and reliable upper and lower bounds in limit analysis. One may even venture to say that the lower-bound aspect is the most important as the construction of ''efficient'' statically admissible stress fields is always a challenge.

Closure by Boonchai Ukritchon,[6] Andrew J. Whittle,[7] and Scott W. Sloan[8]

The writers would like to thank the discussers for their most generous recognition of our paper. The writers agree with their observation that the key to the success of the proposed numerical limit analyses lies in the calculation of high-quality lower-bound solutions, which are not easily found by other methods (as shown in Fig. 21). Although the numerical solutions are able to give very tight bounds on the true collapse loads (all of the cited examples are within ±5%), they are clearly no substitute for exact solutions as derived by the discussers for cases with $|e/B| = 0$.

Erratum. The following correction should be made to the original paper: in Fig. 12(b), the sequence of contour labels $|e/B| = 0.0–0.35$ should be reversed such that Figs. 12(a) and 12(b) are compatible. A corrected Fig. 12 is shown here.

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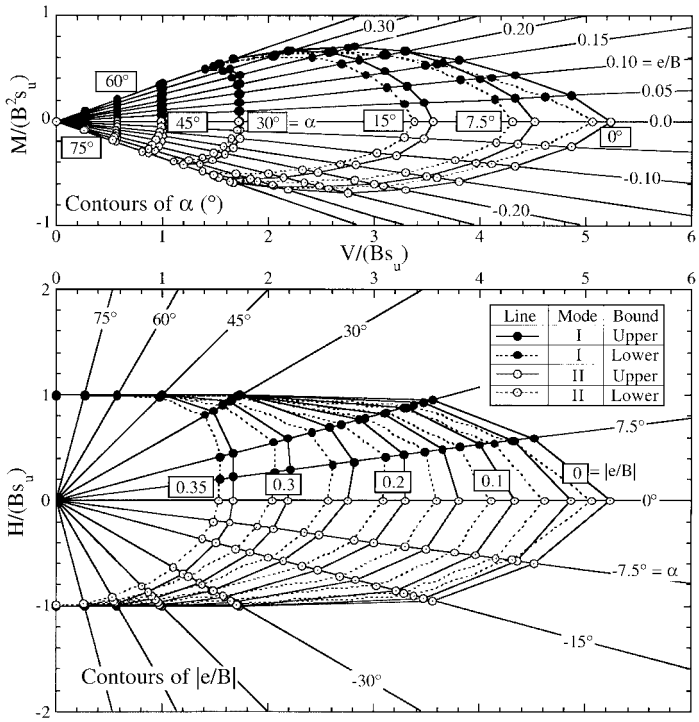

FIG. 12. Projections of Failure Surface for Combined Loading of Footing on Homogeneous Clay