(ii) proposed buildings are close to slope shoulders, and (iii) groundwater level changes occur to depths greater than twice the depth of climatic influence. Table 3 uses the damage risk classification from Table 2 to outline construction considerations in regions of expansive soils.

Successful examples of industrial and urban structures on expansive soils in China

1. New city development

One-third of the buildings in Yunxian County, Hubei Province has suffered damage from expansive soil, and it was decided to relocate the city near the Danjiang River Reservoir. Soil deformations were accentuated by a complex, undulating topography.

Site investigations examined the distribution, expansion–shrinkage characteristics, and physicochemical properties of the soils. As a result, topographical and subsoil features could be taken into account in siting new buildings using the criteria outlined in Tables 2 and 3. Site preparation included cutting steps in slopes and construction of cuts and fills for highway access. Multifloor buildings were placed on carefully engineered foundations in areas of high expansion–shrinkage potential, while areas of low potential were reserved for smaller, nonengineered apartments. Careful attention was paid to construction on slopes. Piers, short piles, retaining walls, and apron slopes were used to control down-slope movements. Construction has now been in progress for some years and is successfully providing attractive accommodation in mixed units of high-rise and low-rise buildings.

2. Large chemical fertilizer plant

The modern fertilizer plant at Hubei produces 300 000 t of synthetic ammonia and 450 000 t of urea annually. Strict limits—5 mm, and in places less—were placed on absolute settlements. Site investigation and testing showed the foundation soil to be Quaternary alluvial–diluvial expansive soil with expansion pressure 30–80 kPa, expansion rate 4–8%, and shrinkage ratio 20–30%.

Construction methods included deepening the footings, increasing the contact pressures, soil replacement, sand cushions, piers, and large diameter bored piles. Careful attention was paid to controlling surface water, soil moisture conservation, superstructure stiffness, and construction management. Deformations of the main building after many years of operation are 1–5 mm, and have met all operational requirements.

HOU, S., AND WANG, S. 1980. Primary consideration of principles for classifying areas of expansive–shrinkable soils according to engineering geology. Engineering Investigation and Exploration (Internal Journal), No. 5, Institute of Geodetic, Geotechnical and Hydrogeological Investigation, Hubei, China.

HOU, S. 1980. Types of expansive–shrinkable soils in China and their engineering geological characteristics. Bulletin of the International Association of Engineering Geology, No. 21, pp. 5–10.

Heave of coal shale fill

JACK A. CALDWELL AND ADRIAN SMITH

Steffen Robertson and Kirsten, 801-1030 W. Georgia Street, Vancouver, B.C., Canada V6E 3J7

AND

JOHAN WAGNER

Rand Mines Limited, Rand Mines Building, Simmonds Street, Johannesburg, South Africa

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Washing plant discard, predominantly laminated and micaceous carbonaceous shale, was used as fill beneath a thick concrete floor slab. This paper discusses the cause of heave of the fill and the severe cracking of the floor slab—up to 70 mm—about a year after construction.

Observation of the fill and the nature of the fill indicate that the heave is caused by absorption of water (but not clay mineral swelling) and predominantly by alteration of the sulphides in the fill to sulphates.

These observations are confirmed by measurement of the chemistry of fresh washing plant discard, weathered fill, and fresh discard subjected to accelerated weathering. Oedometer tests confirm the swelling of the rocks as a result of water absorption. As these processes were still occurring, further heave was anticipated; accordingly, the fill was removed.

Keywords: heave, shale, sulphide alteration, weathering, clay mineral swelling.

Can. Geotech. J. 21, 379–383 (1984)

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Des résidus d'usine de lavage, comportant essentiellement des schistes carbonatés et micacés laminés, ont été utilisés comme remblai sous une épaisse dalle de béton. Cet article discute des causes du gonflement du remblai et de la fissuration importante de la dalle (jusqu'à 70 mm) environ un an après la construction.

L'examen du remblai et la nature du remblai indiquent que le gonflement est causé par l'absorption d'eau (mais pas par le gonflement de minéraux argileux) et, de façon prédominante, par l'altération des sulfures en sulfates dans le remblai.

Ces observations sont confirmées par les mesures de la composition chimique des résidus frais de l'usine, du remblai altéré et de résidus frais soumis à une altération accélérée. Des essais oedométriques confirment que le gonflement du remblai résulte de l'absorption d'eau. Comme ces processus étaient toujours en cours, une continuation du gonflement était attendue; en conséquence le remblai a été enlevé.

Mots-clés: soulèvement, schiste, altération des sulfures, altération, gonflement des minéraux argileux.

[Traduit par la revue]

Introduction

The fill used at a new kitchen and dining room complex in Utrecht, Natal, South Africa, was the discard from the mine's coal washing plant. Approximately a year after being placed, the fill had heaved up to 70 mm, and in so doing, had caused extensive cracking of a thick floor slab and outward movement of walls.

This paper describes the nature of the fill and the possible causes of heaving of the fill. Heave is shown to be caused by (1) absorption of water (but not clay mineral swelling) and (2) alteration of sulphides to sulphates in the fill.

Description of the problem

The Utrecht operations mine coal from the Vryheid sequence of the Karoo Series. Underground mining of the coal seam inevitably involves some removal of the overlying and underlying shales and mudstones. Material removed from the mine is crushed to less than 75 mm and passed through a washing plant. The light coal is floated out and the heavier coals and rocks discarded on surface waste dumps.

Washing plant discard was used as fill beneath the floor slab of the new complex. Between 400 and 1500 mm of washing plant discard were compacted into place to bring the floor to grade.

Figure 1 shows the pattern of floor heave observed in the dining room approximately 2 years after placing the fill. The greatest rate of change of the heave profile appears to be within approximately 2 m of the outer wall. The concrete is continuously but irregularly cracked around the dining room approximately 2 m from the wall.

The north end wall of the dining room has moved approximately 25–35 mm outwards at the top. Approximately 10–15 mm of movement of the brick wall beneath the damp-proof course is observed.

The fill in the kitchen is approximately 400 mm thick. Floor profiles were not measured in the kitchen, but extensive cracking of the floor slab has occurred. Single-brick division walls approximately 1.5 m high have cracked and opened at the top approximately 15–25 mm.

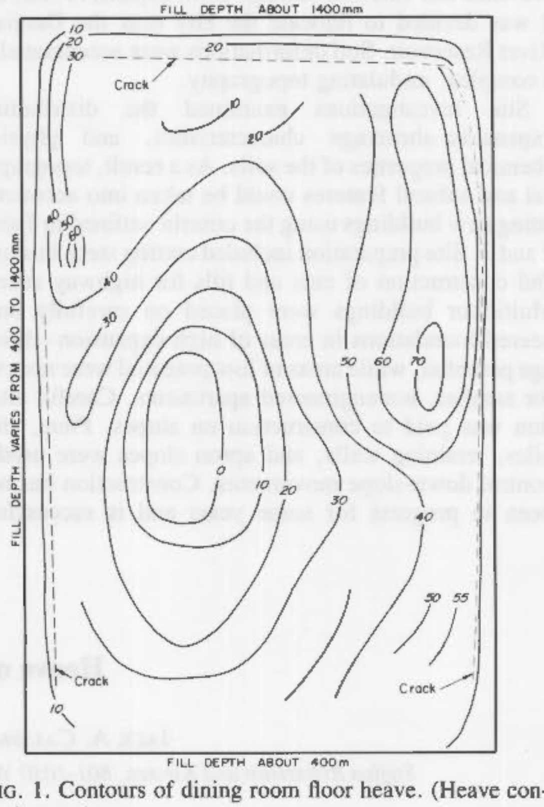

FIG. 1. Contours of dining room floor heave. (Heave contours in mm.)

Description of the fill

The fill, as observed in test holes in the dining room, is slightly moist, very dark gray, mottled yellow, and ash gray, generally medium dense to dense, medium coarse (up to 250 mm), and composed of angular to platy pieces of laminated and micaceous carbonaceous shale mixed with occasional sandstone and low-grade coal. By comparison, the fresh material from the washing plant is black and gray and has no yellow mottle.

The yellow mottle is observed not only on the surfaces of the individual particles but also on the bedding planes of the shale, which may be exposed by breaking. The flat stone and shale particles in the fill tend

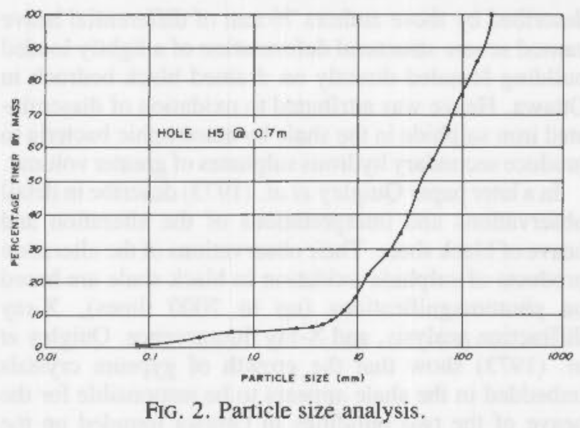

FIG. 2. Particle size analysis.

to lie in the same orientation, and the general impression is that the fill was adequately compacted.

The temperature of the fill when examined at the end of summer was 26°C. The room temperature was 22°C and the outside temperature in the shade was 24°C. The fill beneath the kitchen floor is saturated, mainly as a result of water leaking from broken pipes below the floor slab.

Figure 2 is the grading curve of the fill. Table 1 gives the moisture content of various fractions of a sample from beneath the dining room floor.

The table shows that the average moisture content of the fill is approximately 4–5%. The moisture content of the fines (arbitrarily defined as the 10% and less fraction) is 7–8%, while the individual, larger particles have a moisture content of 3–4%.

The major minerals of the fill are organic material, quartz, feldspar, and mica. Minor constituents are pyrite, gypsum, water soluble salts, hydrated iron oxides, and possibly sulphur. X-ray diffraction indicates the clay minerals illite and kaolinite only; the clays are nonswelling. Pyrite is randomly distributed through the samples; some are devoid of pyrite, while others contain concentrations of up to 10%. Gypsum rosettes and other secondary products are observed on the weakly cohesive bedding planes.

TABLE 1. Dining room fill moisture content

Sample	Moisture content Average	Specific density
Total sample	4.47	0.45
Fines only (i.e., particles smaller than 4.75 mm)	7.55	0.75
Particles larger than 4.75 mm only	4.57	0.40
Selected coarse particles (uncrushed)	3.88	0.39
Selected coarse particles (crushed)	3.69	0.38

The fresh material from the washing plant does not have gypsum rosettes or other oxidation products along the bedding planes. Fresh particles of material from the washing plant readily adsorbed water and tended to break up and disintegrate.

Possible causes of heave of the fill

Possible causes of heave of the fill or structural distress of the floor considered but discounted are the following:

(1) Formation of gases in the fill: the open nature of the fill and the opportunities for escape of gas makes this most unlikely.

(2) Heating of the fill and concrete slab with subsequent expansion: the temperature increase of the fill is insufficient to account for the volume change observed.

(3) Swelling of a clay fraction: as noted above there are no swelling minerals in the fill.

(4) Stress relief of the particles: stress relief is likely to occur soon after mining and thus is not a likely explanation.

(5) Settlement of the foundations: the foundations are on competent sandstone; also, the horizontal movement of the brickwork discounts foundation settlement.

There are, however, two apparently viable causes of heave which might be considered: (1) absorption of water by the shales with a subsequent increase in volume and (2) alteration of the sulphides in the fresh washing plant discard to sulphates and secondary oxidation products.

As noted above, the fill beneath the kitchen floor and the south end of the dining room is only approximately 400 mm thick. The fill in these areas is saturated. Both water absorption and sulphide alteration may account for the increase of volume of the fill. Conversely, the fill beneath most of the dining room floor is only partially saturated. The predominant cause of heave in this area is believed to be sulphide alteration.

The following sections discuss the nature of the two phenomena and the testing done to confirm that heave is due to water absorption and sulphide alteration.

Sulphide alteration

The pH, sulphide, and sulphate content of both the discard from the washing plant and the fill from beneath the floor were measured. In addition, these parameters were measured on samples of the washing plant discard after different degrees of simulated accelerated weathering in the laboratory, using methods described by Smith *et al.* (1982). This essentially involves repeated wetting and drying at elevated temperatures. Figure 3 summarizes the results.

Figure 3*a* indicates that the fresh washing plant discard has a high pH and a low sulphate content. Those samples subjected to accelerated weathering display a

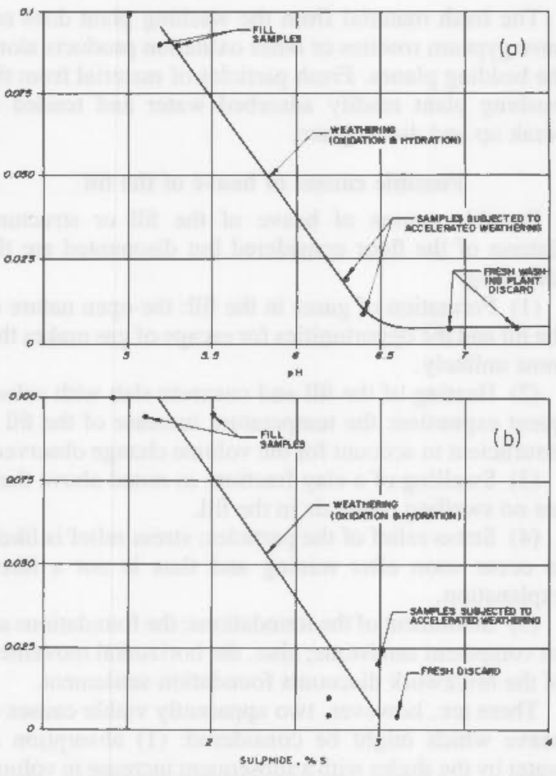

FIG. 3. Accelerated weathering test results.

reduced pH and a slight increase of sulphate content. The samples from the fill have a considerably lower pH and a considerably greater sulphate content than the fresh material. Figure 3*b* shows that the sulphide content of the fresh material is greater than that of the weathered material. The sulphate content of the fresh material is, however, lower than that of the weathered fill.

The observation of gypsum rosettes, calcium sulphate, in oxidised fill implies a source of calcium. Although no calcium was identified as a mineral phase of the shale, this does not mean that there is no available calcium in the shale. Analysis of such shales normally shows a significant proportion of calcium that can be mobilized in the saturated fill.

Thus, it is apparent that, with weathering, the sulphides alter to sulphates. As described by Coveney and Parizek (1977), this leads to an increase in the volume of shales. Depending on the sulphide oxidation process perceived and on simplifying assumptions on the nature of the oxidation product, a volume increase in excess of 60% can be postulated. In the case described by Coveney and Parizek (1977), a floor cast on a shale, with a pyrite content up to 3%, heaved between 100 and 200 mm.

Heave of black shale has also been reported in Canada by Quigley and Vogan (1970). In the case history described by those authors 76 mm of differential heave caused severe structural deformation of a lightly loaded building founded directly on drained black bedrock in Ottawa. Heave was attributed to oxidation of disseminated iron sulphide in the shale by autotrophic bacteria to produce secondary hydrous sulphates of greater volume.

In a later paper Quigley *et al.* (1973) describe in detail observations and interpretations of the alteration and heave of black shale. Their observations of the alteration products of sulphide oxidation in black shale are based on photomagnifications (up to 7000 times), X-ray diffraction analysis, and X-ray fluorescence. Quigley *et al.* (1973) show that the growth of gypsum crystals imbedded in the shale appears to be responsible for the heave of the two buildings in Ottawa founded on the shale. Jarosite and natrojarosite alteration products, probably produced by bacterial action, filled open cavities and cracks. The authors concluded that while the jarosite is probably critical to the overall alteration process it is probably of secondary importance in generating heave pressures.

Gillott *et al.* (1974) describe a microstructural analysis of altered and unaltered black shale from a site where foundation heave occurred. They examined a range of samples from relatively fresh to severely rotted specimens that were coated with secondary alteration products. In unaltered specimens examined by them the principal minerals are illite, chlorite, quartz, calcite, and pyrite. In altered specimens, gypsum and jarosite are the most abundant new minerals. Pyritic sulphur is present in both altered and unaltered shale. Sulphate sulphur is present in the altered but not the unaltered shale and there is a marked decrease in carbonate content between the unaltered and the altered shale. Gillot *et al.* (1974) present a detailed discussion based on their observations, as summarized above, of whether it is the gypsum or the jarosite that is responsible for the heave of the black shales, and on the basis of their discussion conclude that: heave of black shales results from the formation of secondary minerals of increased volume formed in association with the bacterial oxidation of pyrite and the production of sulphuric acid; the main reaction products are gypsum and jarosite; and it is not certain which of these minerals generates heave pressure.

Grattan-Bellew and Eden (1975) describe an investigation of the heave of the basement floor of a church in the New Edinburgh area of Ottawa. They discovered that the concrete under the floor had been reduced to a mushy consistency due to attack by sulphate solution formed by the oxidation of pyrite in the underlying shale. Gypsum was the main product in the weathering zone of the shale, although small amounts of jarosite were observed. They showed that heave resulted from the growth of gypsum crystals between the lamellae in the shale.

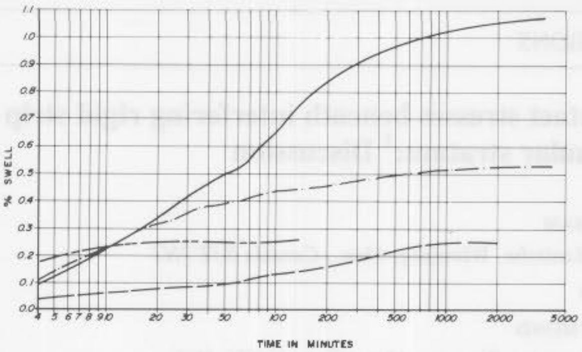

FIG. 4. Results of swell tests on carbonaceous shale.

Water absorption

The finer fraction of the fill was placed in an oedometer and saturated. The change in volume with time is shown in Fig. 4. This confirms that water adsorption can account for a volume change of the fill of the order of magnitude observed.

The phenomenon of swelling of rocks by water absorption can account for a volume change of the fill of (1968). They performed an extensive series of tests on various rock types and showed that the greatest swelling due to water absorption occurred for clays, marls, shales, and mudstones. None of the materials tested by them contained clay minerals with an expanding clay lattice structure.

Conclusions

On the basis of limited physical, mineralogical, and chemical testing, the cause of heave of a floor slab in a new dining room and kitchen complex is shown to be (1) absorption of water and (2) alteration by weathering and oxidation of sulphides in the fill to sulphates.

As the majority of the fill was still only partially saturated and as there is still abundant unweathered pyrite in the fill, continued heave is possible. Accordingly, the fill was removed and replaced with an inert material.

COVENEY, R. M., and PARIZEK, E. J. 1977. Deformation of mine floors by sulphide alteration. Bulletin of the Association of Engineering Geologists, **14**(3), pp. 131–156.

DUNCAN, N., DANNE, M., and PETTY, S. 1968. Swelling characteristics of rocks. Water Power, May, 1968, pp. 185–192.

GILLOTT, J. E., PENNER, E., and EDEN, W. J. 1974. Microstructure of Billings shale and biochemical alteration products, Ottawa, Canada. Canadian Geotechnical Journal, **11**(4), pp. 482–489.

GRATTAN-BELLEW, P. E., and EDEN, W. J. 1975. Concrete deterioration and floor heave due to biogeochemical weathering of underlying shale. Canadian Geotechnical Journal, **12**(3), pp. 372–378.

QUIGLEY, R. M., and VOGAN, R. W. 1970. Black shale heaving at Ottawa, Canada. Canadian Geotechnical Journal, **7**, pp. 106–112.

QUIGLEY, R. M., ZAJIC, J. E., MCKYES, E., and YONG, R. N. 1973. Biochemical alteration and heave of black shale; detailed observations and interpretations. Canadian Journal of Earth Sciences, **10**, 1005–1015.

SMITH, A., DEHRMANN, A. A., and HARPLEY, D. P. 1982. Laboratory procedures manual. Steffen Robertson and Kirsten internal report, Johannesburg, South Africa.

DISCUSSIONS

Some experimental studies concerning the contact stresses beneath interfering rigid strip foundations resting on a granular stratum:[1] Discussion

J. GRAHAM

Department of Civil Engineering, University of Manitoba, Winnipeg, Man., Canada R3T 2N2

AND

G. P. RAYMOND

Department of Civil Engineering, Queen's University, Kingston, Ont., Canada K7L 3N6

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The paper is a welcome addition to the number of experimental projects in sand reported from Canada in recent years. The authors are to be complimented on the fundamental nature of the question they have chosen to examine, on the elegance of the design of their testing program, and on the quality of their results for two interfering footings.

Consideration of a closely similar interference problem between three parallel footings (Graham *et al.* 1984) permits comments to be made on several points raised by the authors. In these three-footing tests, the stress fields associated with the outer footings caused the stresses under the central footing to be symmetrical, as they would be, for example, in a line of parallel interfering footings. This contrasts with the asymmetric stresses under the authors' footings.

Separate series of tests in the writers' laboratories have examined how loads are shared between neighbouring footings through superstructures of different stiffnesses. Prefailure and failure conditions have both been studied. In one series (Fig. 1*a*), the footings were forced downwards together by a rigid loading system, and the distribution of loads between them was measured by separate load cells. The ratio λ of the load on the outer footing to the load on the central footing did not vary greatly between the beginning and end of the tests. In the second series (Fig. 1*b*), the central footing was independent of the outer footings, and different values of λ were held constant during the tests. The results provide general support to the authors' view that interference effects are largely restricted to centre line spacing $S/B < 4$. However, some evidence of interaction with the test container is seen in both of our test series, especially when the sand has a higher coefficient of uniformity than in the authors' tests. It is perhaps more common to expect interference to cease at centre line spacings of six to eight footing widths.

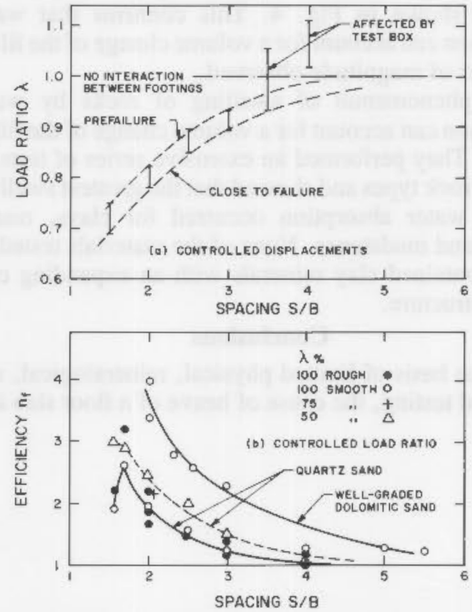

FIG. 1. Load sharing and efficiencies between symmetrically loaded interfering footings.

The authors tested "smooth" footings ($\delta < 7°$), partly to correspond with assumptions of the physical modelling in their testing equipment. At first sight this may represent a limitation on the usefulness of their results since most real footings are rough. However, the difference in bearing capacity between smooth and rough footings is small (Vesic 1973). Recent tests (Graham *et al.* 1984) on 38 mm wide × 305 mm long isolated footings in crushed quartz sand suggest that the differences may be only 5–10%.

The paper directs attention to the important question of the contact stress distribution beneath footings. The

[1]Paper by A. P. S. Selvadurai and S. A. A. Rabbaa. 1983. Canadian Geotechnical Journal, **20**, pp. 406–415.

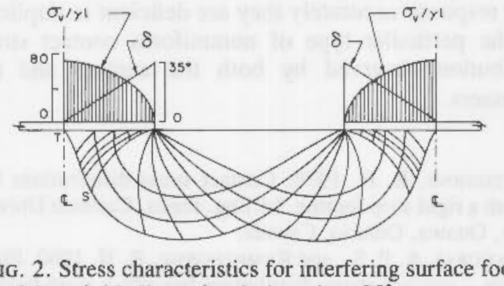

FIG. 2. Stress characteristics for interfering surface footing with $S/B = 2.02$, linear δ-solution, $\phi = 35°$.

authors show stresses that differ markedly from the uniform distribution commonly assumed for design, or the triangular distributions that result from theoretical solutions using complete contact friction mobilization ($\delta = \phi$) across the full width of the footing base. Friction mobilization is a strain-controlled response and not a fundamental soil property (Graham and Stuart 1971).

An advantage of the stress-characteristic method for calculating bearing capacities is that contact stress distributions are developed directly from the analysis once reasonable assumptions have been made regarding the δ-distribution. Figure 2 shows the stress distribution calculated for two symmetrically loaded interfering footings at a spacing of about two footing widths. The calculations assumed that no shear stresses would occur at the centres of the footings and that δ varied linearly from ϕ at the edge to zero at the centre. This assumption was first suggested intuitively by Graham and Stuart (1971), and has since been supported by the experimental results of Bauer *et al.* (1979). The shapes of the theoretical and experimental vertical contact pressure distributions are approximately parabolic, and are similar to those suggested by the authors.

Finally, it is perhaps worth noting that the increased flexural moments identified by the authors are only experieneed when the interference is asymmetric.

BAUER, G. E., SHIELDS, D. H., SCOTT, J. D., and NWABUOKEI, S. O. 1979. Normal and shear stress measurements on a strip footing. Canadian Geotechnical Journal, **16**, pp. 177–189.

GRAHAM, J., and STUART, J. G. 1971. Scale and boundary effects in foundation analysis. ASCE Journal of the Soil Mechanics and Foundations Division, **97**(SM11), pp. 1533–1548.

GRAHAM, J. RAYMOND, G. P., and SUPPIAH, A. 1984. Bearing capacity of three closely-spaced footings on sand. Geotechnique, **34**, in press.

VESIC, A. S. 1973. Analysis of ultimate loads of shallow foundations. ASCE Journal of the Soil Mechanics and Foundations Division, **99**(SM1), pp. 45–73.

Some experimental studies concerning the contact stresses beneath interfering rigid strip foundations resting on a granular stratum:[1] Reply

A. P. S. SELVADURAI

Department of Civil Engineering, Carleton University, Ottawa, Ont., Canada K1S 5B6

AND

S. A. A. RABBAA

Department of Civil Engineering, Al-Azhar University, Madinet Nasr, Cairo, Egypt

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The authors would like to thank Professors J. Graham and G. P. Raymond for their interest in the paper and for the stimulating discussion. The problems cited in the discussion also clearly illustrate the necessity for considering interference effects that can exist between closely spaced foundations. In the three-footing problem, where the plane of symmetry exists at the centre line of the central footing, the contact stresses at the central footing should, of course, exhibit a symmetrical pattern. The outer two footings, however, will exhibit an asymmetric contact stress distribution. Even in the central footing the contact stress distribution will be far removed from the uniform pattern assumed in conventional structural design calculations. The main purpose of the experimental investigation was to ascertain the nature of the contact stress distributions that can exist at the base of shallow footings resting on granular media.

[1]Discussion by J. Graham and G. P. Raymond. 1984. Canadian Geotechnical Journal, **21**, this issue.

The experimental procedure used determines, relatively accurately, such contact stress distributions (see, e.g., Kempthorne 1978; Selvadurai and Kempthorne 1980; Rabbaa 1981). The nonuniform contact stress distributions, whether symmetrical or asymmetrical, are typical of contact stress distributions that can occur at the base of shallow foundations resting on loose to medium dense granular soils. It must also be noted that any theoretical assessment of this class of nonlinear soil–foundation interaction problem should be capable of accurately predicting both the load displacement response of the footings and the contact stress generated at the interface for a complete range of footing settlements. Current numerical calculations indicate that while certain numerical stress analysis codes duplicate the load displacement response accurately they are deficient in duplicating the particular type of nonuniform contact stress distribution observed by both the authors and the discussers.

KEMPTHORNE, R. H. 1978. Contact stress distributions beneath a rigid strip footing. M.Eng. thesis, Carleton University, Ottawa, Ontario, Canada.

SELVADURAI, A. P. S., and KEMPTHORNE, R. H. 1980. Plane strain – contact stress distribution beneath a rigid footing resting on a soft cohesive soil. Canadian Geotechnical Journal, **17**, pp. 114–122.

RABBAA, S. A. A. 1981. Interference between rigid strip foundations resting on cohesionless soils. Ph.D. thesis, Carleton University, Ottawa, Ontario, Canada.