Palmer, A. C. *et al.* (2005). *Géotechnique* **55**, No. 4, 338–340

DISCUSSION

Uplift resistance of buried submarine pipelines: comparison between centrifuge modelling and full-scale tests

A. C. PALMER, D. J. WHITE, A. J. BAUMGARD, M. D. BOLTON, A. J. BAREFOOT, M. FINCH, T. POWELL, A. S. FARANSKI and J. A. S. BALDRY (2003). *Géotechnique* **53**, No. 10, 877–883

K. J. L. Stone, *University of Brighton, UK*
T. A. Newson and M. F. Bransby, *University of Dundee, UK*
The authors' contribution is most timely, owing to the paucity of comparisons between centrifuge model data and their corresponding prototypes. The problem addressed is the uplift resistance of buried pipelines in granular soils. Owing to the strong similarities between the uplift resistance of pipelines, buried plates and the classic trap-door problem, a number of aspects of this problem have been addressed previously (e.g. Vardoulakis *et al.*, 1981; Cole & Lade, 1984; Lade *et al.*, 1984; Stone, 1988). This discussion concerns the scale effects reported by the authors, and is intended to confirm the hypothesis that any inconsistency in the conventional understanding of centrifuge tests is the result of the dependence of the mobilised shear stress across localisations (or shear bands) on the relative displacement across the localisation.

It has been previously proposed (Scarpelli & Wood, 1982; Muhlhaus & Vardoulakis, 1987; Stone & Wood, 1992) that this relative displacement is primarily a function of the soil grading (characterised by the mean particle size, d_{50}), and thus for small-scale modelling of problems involving discontinuous soil mechanics, it may be necessary to scale the particle size (or, more correctly, the grading curve) to ensure that: (a) the kinematics of the reduced scale model are compatible with those of the corresponding prototype; (b) the scale of the localisations themselves does not adversely affect the model response; and (c) the mobilised strength on localisations in the model is compatible with that which would be present in the corresponding prototype.

Consider a localisation or shear band propagating through a dense cohesionless soil from an induced boundary displacement, say a trapdoor (passive case) or uplift of an inclusion such as a buried plate or pipeline, as illustrated in Fig. 9(a).

The kinematics of the problem constrain the initial localisation to be inclined at the angle of mobilised dilation (initially a maximum, ν_{max}) to the major principal stress (assumed to be vertical). A section of the localisation can be idealised as a condition of direct shear, as is rather crudely replicated in a direct shear box test, as indicated in Fig. 9(b).

Typical shear box tests on dense sands of different uniform gradings (characterised by the d_{50} value) but at the same initial void ratio and normal stress are shown in Fig. 10 (Stone & Wood, 1992). It is apparent from these tests that the relative displacement across the shear box (and hence across the shear band), which coincides with the peak and critical state friction angles, appears to be a function of the grain size. This is intuitively likely, as it would be expected that, for geometrically similar 50 mm and 5 mm diameter particles, the larger particles would require significantly more relative displacement to achieve corresponding peak critical state conditions.

Furthermore, as the ratio of the shear box dimensions to grain size increases, the measured peak strength reduces. This is consistent with the observations reported by Garnier (2002).

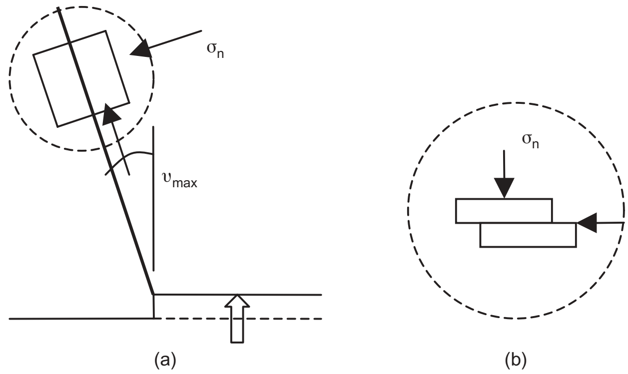

Fig. 9. (a) Schematic representation of shear band propagating through dense cohesionless soil from induced boundary displacement; (b) direct shear analogue of shear band

For the purposes of this discussion, it has been assumed that the degree of restraint provided by the shear box (i.e. the ratio of particle size to shear box dimensions) is such that only one shear band forms, aligned with the zero extension (horizontal direction). When less restraint is provided such that shear bands can form freely (i.e. with large particle size to shear box dimension ratios), complex shear band patterns can develop (e.g. Scarpelli & Wood, 1982).

The orientation of the localisation as shown in Fig. 9(a) is kinematically compatible only if the shear band is dilating at the maximum rate (ν_{max}). At some relative displacement across the localisation the dilation reduces to zero, and kinematics dictate that a new vertical shear band should form associated with zero dilation. However, formation of a new shear band results in fresh dilation, and the (new) vertical shear band itself is now kinematically inadmissible, possibly resulting in curvature of the propagating shear band. The above illustrates the potential complexity of the kinematics of shear band formation for problems of this type, especially where multiple shear bands can develop (e.g. Cole & Lade, 1984; Lade *et al.*, 1984).

From the above it is apparent that, to achieve geometric similarity in patterns of shear band formation in a reduced scale model, the particle size should also be factored, strictly speaking by the model scale factor, but in practice this is not always necessary or realistic.

As the relative displacements (across a propagating shear band) required to mobilise peak and critical state, at model scales, are primarily a function of the grain size used in the model, then for the pipeline tests reported by the authors it would be expected that the mobilisation distance would be primarily a function of the grain size. This appears to be in disagreement with the plot of mobilisation distance against

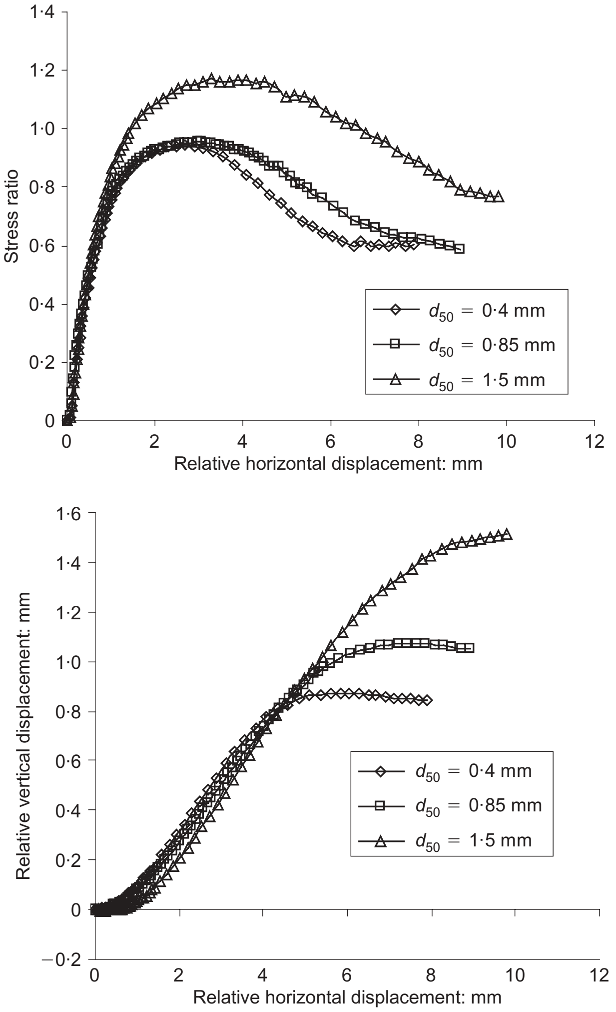

Fig. 10. Typical shear box tests on dense sands of different uniform gradings (characterised by d_{50} value) but at same initial void ratio and normal stress (3·2 kPa) (from Stone, 1988)

grain size shown by the authors (see Fig. 4). For the sand tests the ratio of mobilisation distance to particle size is reasonably constant, implying that for these gradings the mobilisation distance is a function of the grain size, and therefore would not be expected to scale unless the grain size was also scaled. It seems reasonable to assume that, as the cover depth is relatively shallow, then for the dense sand tests the peak uplift is coincident with the development of an initial shear band associated with a maximum rate of dilation; it is noted here that the mechanism proposed by the authors in Fig. 7 would be kinematically associated with loose sand (zero dilation). For the tests conducted in the silt and coarse gravel materials any correlation between the mobilisation distance and grain size appears to break down. It is suggested that this is due to the following.

For the 'gravel-sized' material, for the postulated mechanism involving the formation of twin shear bands, assuming that each shear band is 10–20 × d_{50} wide (e.g. Roscoe, 1970; Muhlhaus & Vardoulakis, 1987; Oda & Kazama, 1998), the total thickness of the shearing material would be approximately 500–1000 mm, which is obviously not possible. The actual mechanism is more likely to be that the overlying gravel particles are displaced around the pipe as it is pulled towards the soil surface: that is, the mechanism is not comparable to that of the sand tests and does not involve the formation of distinct shear bands. This is an illustration of the fact that the dimensions of a localisation are dictated predominantly by the grain size of the soil. Failure to scale the grain size means that the localisation dimensions remain at prototype scale, and may have a detrimental effect on the observed model response.

For the 'silt-sized' material, several factors may contribute to the differences observed. First, there is the possibility that the soil response during the pipe uplift test was not fully drained, and even that suctions could have developed behind the pipe. Pore pressure data from a transducer located in the pipe would be very useful to investigate this possibility. Second, if fully drained conditions did occur, then the potential for multiple shear bands to occur exists, and direct comparison with the sand tests becomes difficult.

Consideration of the model test kinematics should be included in the interpretation of any reduced-scale test where discontinuities form (both at 1*g* and in the centrifuge), and in particular it may be necessary to scale the particle size (or grading curve), if not by the actual model scale factor, then at least sufficiently enough to allow the full development of localisation patterns and the attainment of the correct values of mobilised friction at corresponding points on the model and prototype localisations. This confirms the authors' hypothesis that mobilised shear stress across localisations depends on the absolute relative displacement across the localisation.

Authors' reply

The authors thank the discussers for their interesting contribution.

We agree that it seems reasonable to suppose that, if the particles can be treated as rigid and inert, then in geometrically similar systems with localised deformation the displacements would have to scale with the particle size. That could be tested experimentally by comparing a conventional shear box test on sand with a test on gravel with a particle size 10 times larger (and as far as practicable the same mineralogy and particle shape), in a shear box itself scaled up 10 times. The same argument would in turn indicate that in a centrifuge test the soil ought itself to be scaled, so that in a 100*g* test at 1/100 scale the particle size distribution ought to be the prototype particle size distribution scaled downwards 100 times. However, obvious difficulties are that a sand or silt prototype would then be represented by a clay model, that clay has qualitatively different behaviour, and that clay particles interact with pore water in a more complex way than do particles of sand.

The opposite view, that the shape and mineralogy of a particular grading of material have a stronger influence on the response than the particle size, has been historically favoured by small-scale modellers, who have chosen to use the 'prototype' material. In element testing this view is rarely stated, but is always assumed. When stiffness measurements are reported from continuum deformation of high-quality intact triaxial samples, one never asks 'Should the grains have been scaled down by the ratio of the sample height to the prototype zone of soil that the sample represents?' When modelling continuum deformation such scaling is not considered necessary.

This contrast between the modelling of events that are governed by stress–strain and stress–displacement laws is not widely appreciated. We welcome increased interest in the formation and detailed structure and mechanics of shear bands, and further investigations following the pioneering work of Scarpelli, Wood and Garnier.

REFERENCES

Cole, D. A. & Lade, P. V. (1984). Influence zones in alluvium over dip-slip faults. *J. Geotech. Engng, ASCE* **110**, No. 5, 599–615.

Lade, P. V., Cole, D. A. & Cummings, D. (1984). Multiple failure surfaces over dip-slip faults. *J. Geotech. Engng, ASCE* **110**, No. 5, 616–627.

Muhlhaus, H. B. & Vardoulakis, L. (1987). The thickness of shear bands in granular materials. *Géotechnique* **37**, No. 3, 271–283.

Oda, M. & Kazama, H. (1998). Microstructure of shear bands and its relation to the mechanisms of dilatancy and failure of dense granular soils. *Géotechnique* **48**, No. 4, 465–481.

Roscoe, K. H. (1970). The influence of strains in soil mechanics. *Géotechnique* **20**, No. 2, 127–170.

Scarpelli, G. & Wood. D. M. (1982). Experimental observations of shear band patterns in direct shear. *Proceedings of the IUTAM symposium on deformation and failure of granular materials* (eds P. A. Vermeer and H. J. Luger), pp. 473–484. Rotterdam: Balkema.

Stone, K. J. L. (1988). *Modelling of rupture development in soils.* PhD thesis, University of Cambridge.

Stone, K. J. L. & Wood, D. M. (1992). Effects of dilatancy and particle size observed in model tests on sand. *Soils Found.* **32**, No. 4, 43–57.

Vardoulakis, I., Graf, B. & Gudehus, G. (1981). Trap-door problem with dry sand: a statical approach based on model test kinematics. *Int. J. Numer. Anal. Methods Geomech.* **5**, 58–78.