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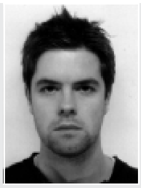

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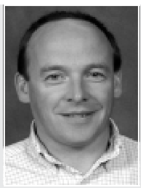

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Uplift of shallow foundations with cement-stabilised backfill

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This paper describes the results of a series of centrifuge model tests undertaken to investigate the effects of modifying a loose granular backfill using cement on the uplift performance of shallow anchors. These model tests, which involved a range of cement contents, are supported using a series of laboratory element tests and finite-element analyses. The study indicates that significant increases in uplift stiffness and peak capacity can be achieved by the addition of relatively small quantities of cement. Such increases are, however, limited to relatively low uplift displacements because of the brittle nature of the improved backfill shear strength characteristics.

NOTATION

B	Anchor width
c'	Shear strength at zero effective stress
D_r	Relative density
E'	Young's modulus
F_u	Uplift resistance
G_0	Elastic shear modulus
H	Anchor embedment depth
L	Distance between bender elements
n	Centrifuge scale factor
N_{us}	Uplift coefficient
q_c	CPT end resistance
q_{fu}	Normalised anchor capacity
q_{uc}	Unconfined compressive strength
t	Time
V_s	Shear wave velocity
w_p	Displacement at peak load
γ'	Effective unit weight
ρ	Soil density
σ'_v	Vertical effective stress
ϕ'_p	Peak friction angle

1. INTRODUCTION

Foundation systems for electricity transmission lattice tower structures are required to resist both uplift and compressive loading, and generally comprise shallow spread footings constructed using reinforced concrete, steel grillages or pressed plates. These foundations may require (relatively costly) piles or ground anchors to provide the required uplift stiffness and capacity when loose or unstable soils exist near the surface. This paper examines a potential soil backfill modification process to improve footing uplift performance that involves the addition of cement to excavated in situ material for subsequent use as backfill. Such potential is currently the subject of some debate for transmission tower foundations in Brazil (where the cement is mixed with in situ residual soil), but clearly has a wider international relevance.

Soil stabilisation is an established practice for road construction where the control of settlement is required. Similarly, soil stabilisation techniques have been developed both to improve the stability of marginal slopes and to limit deformations associated with tunnelling operations. These techniques typically involve the mixing of a hardening agent such as cement or lime with the soil to create a bond between the soil and stabiliser that enhances its mechanical properties, and can be applied either in situ or ex situ depending upon the application. The use of Portland cement as a soil stabilising agent was trialled in Japan[1] and applied in situ using a slurry to distribute the cement within the soil matrix. Subsequent investigations indicated that significantly greater increases in soil strength following curing may be obtained by distributing the cement using a dry-mixing process rather than by using a cement slurry. Field studies reported by Stefanoff *et al.*,[2] Consoli *et al.*[3] and Thomé *et al.*[4] have shown that the compressive bearing capacity of spread footings founded on soft soils can be enhanced appreciably by the addition of cementing agents to the backfill placed above the footings.

The benefits of backfill stabilisation for the uplift performance of shallow anchors are investigated here in a series of physical model tests conducted in a geotechnical drum centrifuge. These tests are examined in finite-element back-analyses using backfill properties determined in a complementary laboratory element testing programme. The backfill employed in the centrifuge (and laboratory tests) contained a range of cement contents to assist assessment of the optimum degree of backfill treatment.

2. BACKFILL PROPERTIES

2.1. Materials

The backfill used was a non-plastic uniform fine quartz sand with a mean effective particle size (D_{50}) of 0·19 mm and minimum and maximum void ratios of 0·52 and 0·81 respectively. The sand backfill was modified by the addition of 1%, 3% and 5% (by dry weight) early strength Portland cement (Type III). This cement is more finely ground and includes a higher proportion of blast-furnace slag than ordinary Portland cement, and has a setting time of approximately 3 h. The cement was selected as the majority of its strength gain takes place within 20 h, and hence the curing time required in the centrifuge tests was not excessive.

The soil specimens for centrifuge and element tests were prepared by first hand-mixing the dry sand and cement and then adding water to a moisture content of 12%. The void ratio of uncemented sand was 0·74, and that of the treated sand was in the range 0·73–0·68 for cement contents of 1–5% respectively.

2.2. Laboratory testing programme

The strength and stiffness of the treated sand were measured in direct shear and unconfined compression tests; direct shear tests were also performed on the untreated sand. Isotropic compression tests with shear wave velocity measurements using bender elements allowed measurement of the very small strain shear modulus ($\boldsymbol{G}_0$). Moulded specimens for the $\boldsymbol{G}_0$ determinations and the unconfined compression tests were prepared by placing the sand–cement samples into a split mould, 80 mm in diameter by 160 mm high, in three equal layers. Specimens employed for the direct shear tests were prepared by forming a single layer in a shear box 71 mm in diameter by 35 mm high. All tests were conducted after leaving the samples to cure for 20 h—that is, the same curing period as adopted in the centrifuge tests.

The samples tested in unconfined compression (UC) were cured for 16 h following mixing, and then submerged in a water tank (maintained at 23 $\pm$ 3°C) for a further 4 h for saturation to minimise suction effects. Excess surface water was removed using an absorbent cloth prior to testing. At least three specimens at each cement content were tested, and the average unconfined compressive strengths ($\boldsymbol{q}_{uc}$) obtained are summarised in Table 1. These strengths are indicative of a weakly to moderately cemented soil, depending on the classification considered.[5–7]

A similar curing and saturation regime to that of the UC tests was adopted for the samples subjected to direct shear. These samples were fully immersed throughout the test duration and sheared at a constant displacement rate of 0·5 mm/min. The peak strength envelopes inferred from tests with normal stresses of 50 kPa, 150 kPa and 300 kPa are shown in Fig. 1(a), and the corresponding Mohr–Coulomb $\boldsymbol{c}'$ and ϕ'_p are listed in Table 1. It is apparent that, as expected, the cement content has a marked effect on the value of $\boldsymbol{c}'$, increasing from zero for uncemented sand to 57 kPa for sand with a cement content of 5%; Fig. 1(c) indicates that this relationship is almost linear. The cement content also has a modest effect on ϕ'_p owing to the higher densities of the cemented sand and/or modest differences between respective samples.

Typical shear stress–displacement curves measured in the direct shear tests are shown in Fig. 1(b) for a normal stress of 50 kPa. These highlight the brittle nature of the cemented sand and the tendency, after a large relative displacement, for the shear strength to reduce to close to that of the uncemented sand. It should be noted that the reducing area of contact between the sample in the top and bottom halves of the shear box as the relative displacement increases leads to progressively less reliable data as the relative displacement increases above about 3 mm.

The bender element (BE) test procedure was introduced by Shirley and Hampton,[8] and is now a standard technique for deriving the elastic shear modulus $\boldsymbol{G}_0$ of a soil. The shear wave velocity ($\boldsymbol{V}_s$) propagating across the specimen and $\boldsymbol{G}_0$ may be determined from

$$\boldsymbol{G}_0 = \rho \boldsymbol{V}_s^2 = \rho\left(\frac{L^2}{t^2}\right) \tag{1}$$

where ρ is the total mass density of the soil, L is the tip-to-tip length between the bender elements, and t is the travel time of the shear wave through the sample. Each sample was allowed to cure for 20 h in a stress path cell under an effective stress of 20 kPa. The BE tests involved transmission of a single-shot sine wave from a BE at one end of the sample and measurement of its first arrival by the BE at the opposite end. Following the recommendations of Jovicic *et al.*,[9] high frequencies were employed to avoid near-field effects, and a range of frequencies were investigated to ensure that the measured arrival time was not frequency dependent.

The measured $\boldsymbol{G}_0$ values are listed in Table 1, which indicates that the cemented sand specimens were about 5, 11 and 20 times stiffer than the uncemented sand for 1%, 3% and 5% cement contents respectively. As for the relationship between $\boldsymbol{c}'$ and cement content (Fig. 1(c)), there is also a near-proportional relationship between $\boldsymbol{G}_0$ and cement content.

3. CENTRIFUGE TESTS

The use of an appropriately scaled model in a geotechnical centrifuge is a well-established and convenient physical

Specimen	Unconfined compressive strength, $\boldsymbol{q}_{uc}$: kPa	Peak friction angle, ϕ'_p: degrees	Shear strength at zero effective stress, $\boldsymbol{c}'$: kPa	Elastic shear modulus, $\boldsymbol{G}_0$:* MPa
Uncemented	–	34·7	0	50
1% cement	25	35·3	17·7	249
3% cement	87	39·8	28·2	566
5% cement	365	41·5	57·4	973

* At a mean effective stress of 20 kPa.

Table 1. Summary of laboratory testing for uncemented and cemented sand

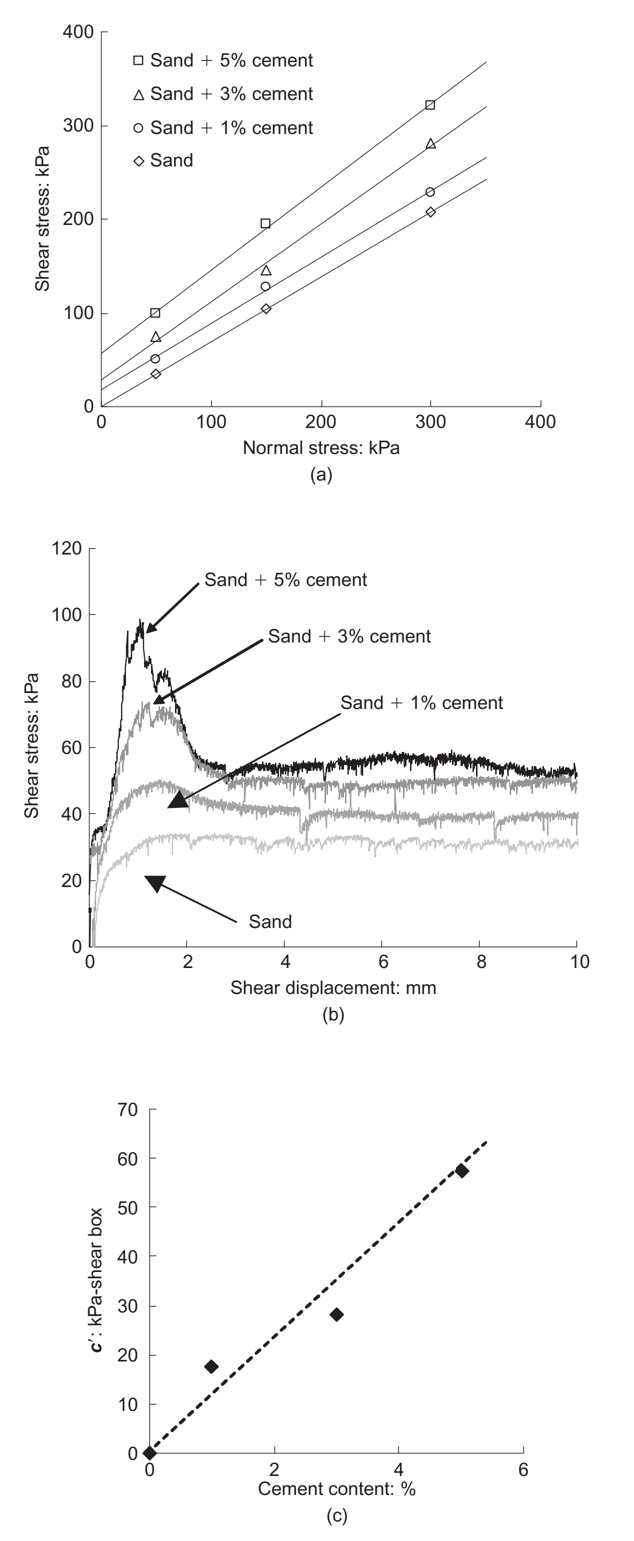

Fig. 1. Shear box test results: (a) strength envelope; (b) shear stress displacement variation at $\sigma'_n = 50$ kPa; (c) relationship between c' and cement content

modelling technique that overcomes problems encountered in small-scale laboratory tests conducted at low stress levels (because of the stress level dependence of the soil's mechanical properties) by imposing an elevated gravitational field on the model. The model is rotated at a constant angular velocity to impose an acceleration of n times gravity (g). In a 1:n scale centrifuge model, linear dimensions are reduced by n, while stresses and pore water pressures are the same at corresponding depths in the model (at ng) and in the field.

Uplift tests were conducted on four 1:50 scale plate anchors in the geotechnical drum centrifuge at the University of Western Australia (UWA); see Fig. 2(a). A complete description of this facility is provided by Stewart *et al.*[10] The initial phase of the experiment involved placement and consolidation of a kaolin sample in the drum channel. This sample was consolidated at 250g for four days prior to halting of the centrifuge, when excavations were made to facilitate location of the anchors and subsequent backfilling. The anchors comprised a 5 mm thick square aluminium base with a width (B) of 30 mm and a central stem fabricated from a 7 mm square steel section. As shown in Fig. 3(a), the anchors were placed at a depth of 45 mm directly on top of a free-draining sand that had been deposited at the base of the excavation; this sand ensured that no suctions could be generated at the anchor base during uplift.[11] Sand or cement-treated sand backfill was then placed manually within the (45 mm deep) excavation up to the sample

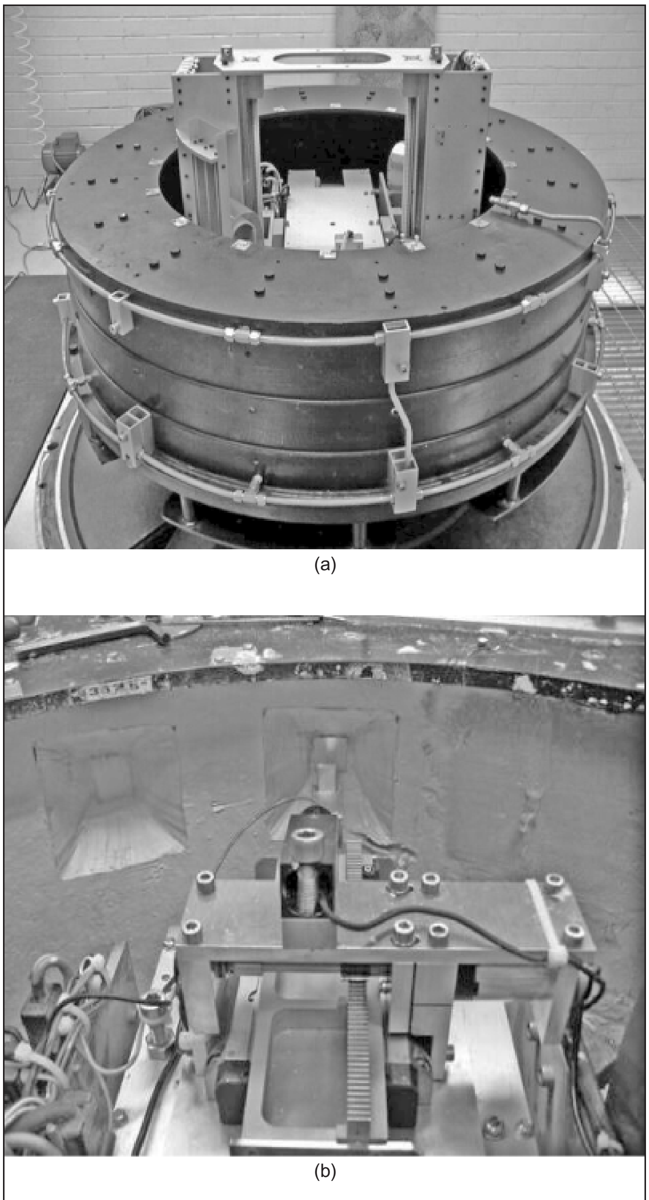

Fig. 2. (a) UWA drum centrifuge; (b) plate footing at base of excavation prior to backfilling

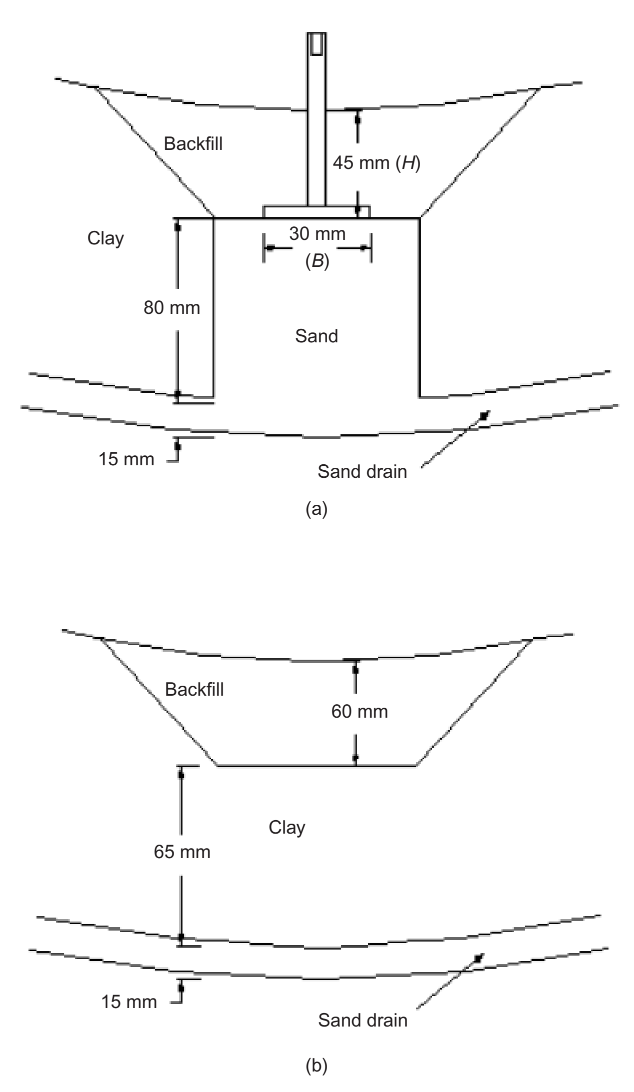

Fig. 3. Configuration in drum centrifuge at: (a) location of uplift tests; (b) location of CPTs

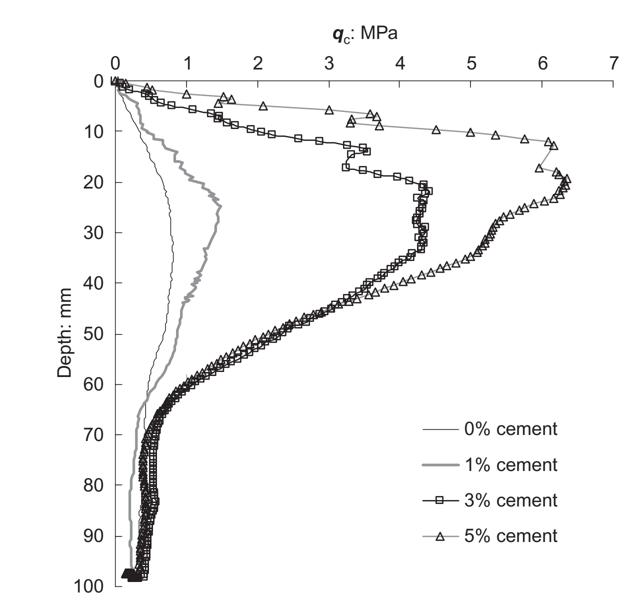

Fig. 4. CPT q_c profiles in backfill with various cement contents

surface, while each anchor was held in place at the head of the stem using the centrifuge actuator as shown in Fig. 2(b).

After placement of the anchors and backfill in the excavations, the centrifuge acceleration was increased back up to 50g and anchors were loaded to failure at a constant uplift rate of 0·1 mm/s when the curing period of cemented backfill had reached 20 h. Subsequent to the uplift tests, backfill was placed in additional excavations made in the clay sample to facilitate cone penetrometer test (CPT) characterisation of the various backfill materials used in the tests. The geometry of these excavations and the configuration adopted for each footing test is shown in Fig. 3. The water level was maintained at about 3 mm above the top surface of the centrifuge sample throughout the testing period.

The CPT end resistance q_c, measured at 50g in excavations backfilled with uncemented and cemented backfill after a curing period of 20 h, is plotted in Fig. 4. The q_c values reach a maximum at depths between 15 mm and 40 mm and then reduce owing to the presence of relatively soft kaolin located at a depth of 60 mm (see Fig. 3). The relative density (D_r) of an uncemented sand in the UWA drum centrifuge is related to the CPT q_c value by[12]

2	$$D_r = \left(\frac{q_c}{250\,\sigma'_v}\right)^{0.5}$$

Equation (2) indicates that D_r for the uncemented sand is approximately 30% in the upper 40 mm of the sample; this relative density is broadly in line with the target void ratio of 0·74 employed in the laboratory tests.

The ratio of q_c values measured in the backfill with a cement content of 3% to that at 1% cement content is similar to the ratio of the respective unconfined compressive strengths (q_{uc}); see Table 2. However, q_c measured in the sand with a cement content of 5% is relatively low compared with its q_{uc} value, and it would appear on inspection of Table 2 that there is no simple general correlation between q_c and either the effective stress strength parameters (c' and ϕ'_p) or q_{uc}.

4. UPLIFT TEST RESULTS

Table 2 summarises the measurements obtained at the peak uplift load (F_u) in the centrifuge uplift tests, and the associated variations of uplift resistance with displacement are provided in Fig. 5. The value of F_u is normalised in this paper by the anchor base area ($B \times B$) to allow direct comparison of the capacities determined using the following established expression for coarse-grained backfill

3	$$q_{fu} = N_{us}\sigma'_v$$

where $q_{fu} = F_u/B^2$ is the ultimate uplift stress, σ'_v is the vertical effective stress at the level of the anchor, and N_{us} is the uplift coefficient, which is a function of the anchor embedment and the sand's peak friction angle (ϕ'_p). A review of solutions presented by Murray and Geddes,[13] Merifield *et al.*[14] and others suggests that N_{us} in loose sand is approximately 4 for

Backfill material	Footing width, B: mm	Embedment ratio, H/B	Peak resistance, F_u/B^2: kPa	Displacement at peak load, w_p: mm	Normalised displacement to peak, w_p/B: %
Uncemented	30	1·5	55	0·53	1·75
1% cement	30	1·5	100	0·39	1·30
3% cement	30	1·5	164	0·42	1·40
5% cement	30	1·5	252	0·29	1·00

Table 2. Summary of centrifuge pull-out tests

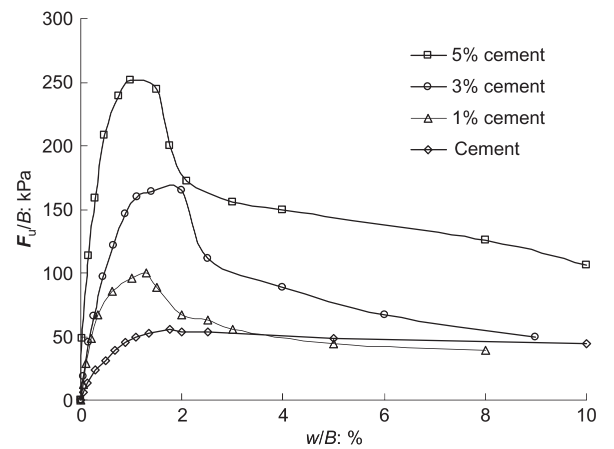

Fig. 5. Uplift resistance variation with normalised displacement

anchors. The addition of cement does, however, lead to a brittle response, and uplift resistance reduces sharply after the peak resistance is mobilised at a displacement of 1·2 ± 0·2% of the footing width. These reductions are considered to be related to the destruction of the cement bonds between the sand grains with continued shearing (i.e. reduction in the c' component of strength), and are analogous to the shear stress–displacement response shown in Fig. 1(b) for the cemented sand. Such brittleness may not be a significant design consideration for transmission tower foundations, which often limit uplift to 20 mm under ultimate loads; this uplift equates to a w/B value of about 1% for typical foundation widths of 2 m.

The relationship between uplift capacity (q_{fu}) and properties of the backfill is explored in Fig. 6, which plots measured q_{fu} values against (a) the shear box c' values, (b) the G0 values at mean effective stress of 20 kPa, the CPT q_c recorded at a depth of 22·5 mm in the centrifuge (= half the depth of anchor embedment) and (d) the unconfined compressive strengths quc. A proportionate increase in q_{fu} with both c' and G_0 (and also cement content) is apparent, which suggests that the relative change in the value of any of these parameters provides a

the ratio of the anchor embedment to width employed (= 1·5; see Fig. 3). This value of 4 is a little higher than the back-figured N_{us} value of 3·1 for tests employing the uncemented backfill, assuming a buoyant sand unit weight (γ') of 8 kN/m^3.

As seen in Table 2, the value of q_{fu} increases strongly with the quantity of cement in the backfill, reaching a value five times greater than the uncemented case for a cement content of 5%. Even with the relatively modest addition of 1% cement to the backfill, the capacity is twice that of the uncemented case. As seen in Fig. 5, the initial pre-peak anchor stiffness also benefits strongly from the addition of cement. These findings alone highlight the significant potential of backfill treatment for the improvement of the performance of shallow plate

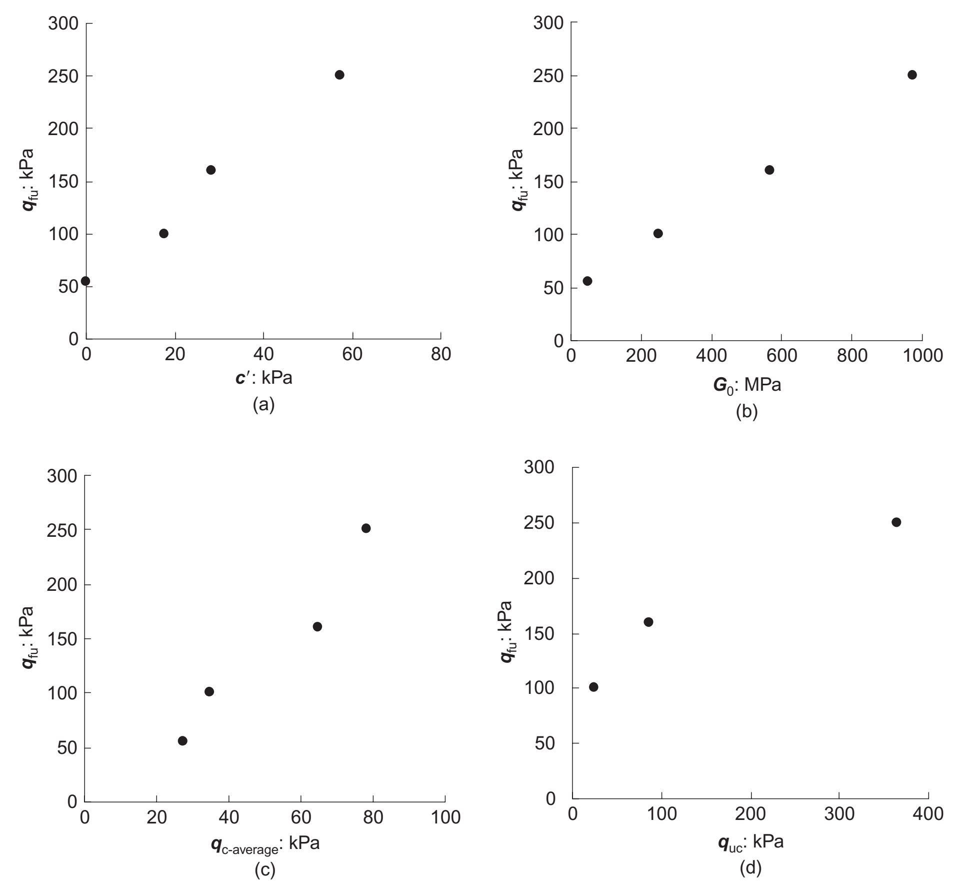

Fig. 6. Peak uplift capacity plotted as function of: (a) c'; (b) G_0; (c) q_c; (d) q_{uc}

direction indication of the corresponding change to q_{fu}. There is not a linear relationship between q_{fu} and q_{uc}, and q_{fu} may be shown to vary approximately with $(q_{uc})^{0.5}$; Rowe and Armitage,[15] and others, have also found that the shear strength of a cemented material was better correlated with $(q_{uc})^{0.5}$. Although the evidence is limited, it would also appear that q_{fu} varies roughly with $q_c^{0.5}$ rather than directly with q_c.

Finally, it should be pointed out that q_{fu} cannot be expected to increase indefinitely as c' increases. Ultimately, as discussed later, the capacity is limited by the weight of the cemented block in the excavation, when the c' value is sufficient to allow the material to behave as a unit.

5. NUMERICAL ANALYSES

The centrifuge uplift tests provide a clear indication of the benefits of the addition of cement to sand backfill for the anchor type under consideration. To facilitate generalisation of these findings, finite-element (FE) analyses of the centrifuge tests were performed to investigate whether the observed response could be replicated. Initially, and for simplicity, all soils in the analyses were assumed to behave as isotropic linear elastic-perfectly plastic materials, with the Mohr–Coulomb strength parameters inferred from the direct shear box tests (see Table 2). For each anchor test, one analysis was performed using the peak strength parameters (c' and ϕ'_p) to predict the peak capacity, and a second analysis was conducted using the same value of ϕ'_p but with c' set to zero to model ultimate uplift conditions (i.e. when the cement bonds had been completely broken).

The FE analyses were performed using the SAFE finite-element program.[16] The analyses adopted an axisymmetric mode of deformation, and therefore the square anchors were represented by equivalent circular anchors with the same area. Fully rough interfaces were assumed between the anchor and surrounding

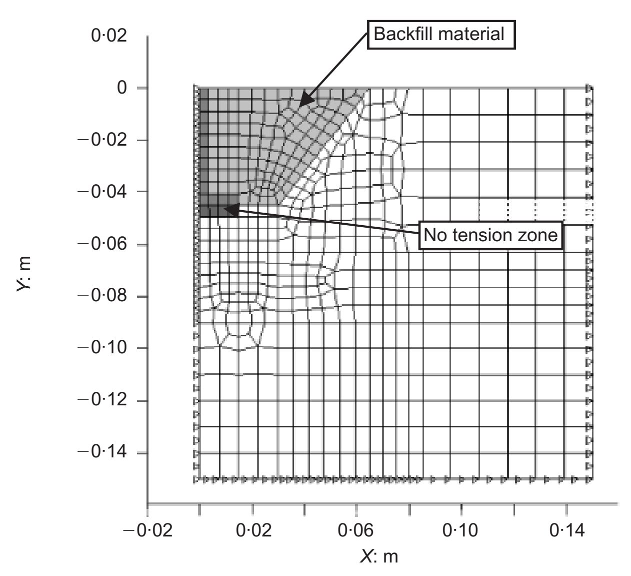

Fig. 7. Finite-element mesh

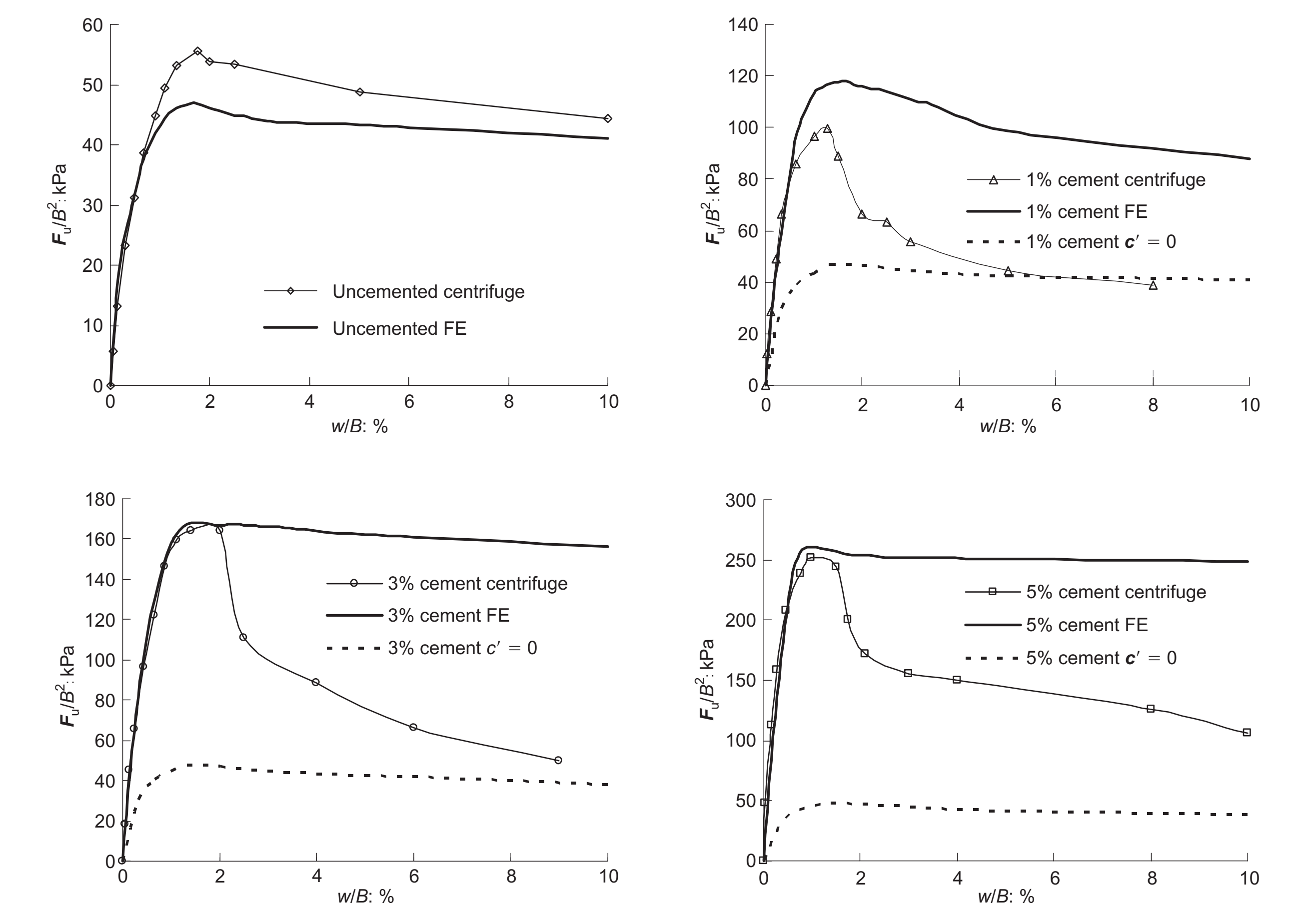

Fig. 8. Comparison of load–displacement response measured in centrifuge tests with FE predictions

soil, and tension was not permitted at the anchor base interface. The Young's modulus E' for each type of backfill was varied as a set multiple of the very small strain shear modulus (G_0) listed in Table 2; a best fit to the predictions discussed below was found by setting $E' = G_0/30$. A nominal E' value of 20 MPa was specified for the clay outside the excavated area (this value had no effect on the predicted anchor response), and E' values for the aluminium stem and steel base of 70 GPa and 200 GPa were employed.

The FE mesh is shown in Fig. 7; it consisted of 530 eight-noded quadrilateral elements, each with four Gauss points. The unit weights of soil and pore water input into the numerical model were factored by $n = 50$, consistent with the centrifugal acceleration applied in the centrifuge tests. The numerical analyses therefore directly modelled the centrifuge tests rather than their equivalent prototypes. The location of the lower horizontal boundary of the mesh was specified at the same depth as the base of the centrifuge channel, and the far vertical boundary was located at 10 times the equivalent footing radius from the axis of symmetry. Each (effective stress) analysis assumed fully drained conditions, and displacements at the top of the anchor stem were increased incrementally until failure occurred.

The curves of q_{fu} against w/B curves predicted by the FE analyses are compared in Fig. 8 with the measured response in the centrifuge. It is apparent that the predicted peak capacities are within 15% of the observed peak values for all cases, and almost perfectly match the capacities measured with backfill cement contents of 3% and 5%. The ultimate capacities at $w/B = 10\%$ are also well predicted by assuming $c' = 0$. Evidently the true ultimate capacity of the anchor with a backfill cement content of 5% is not reached at $w/B = 10\%$.

The predictions in Fig. 8 indicate that the use of a constant

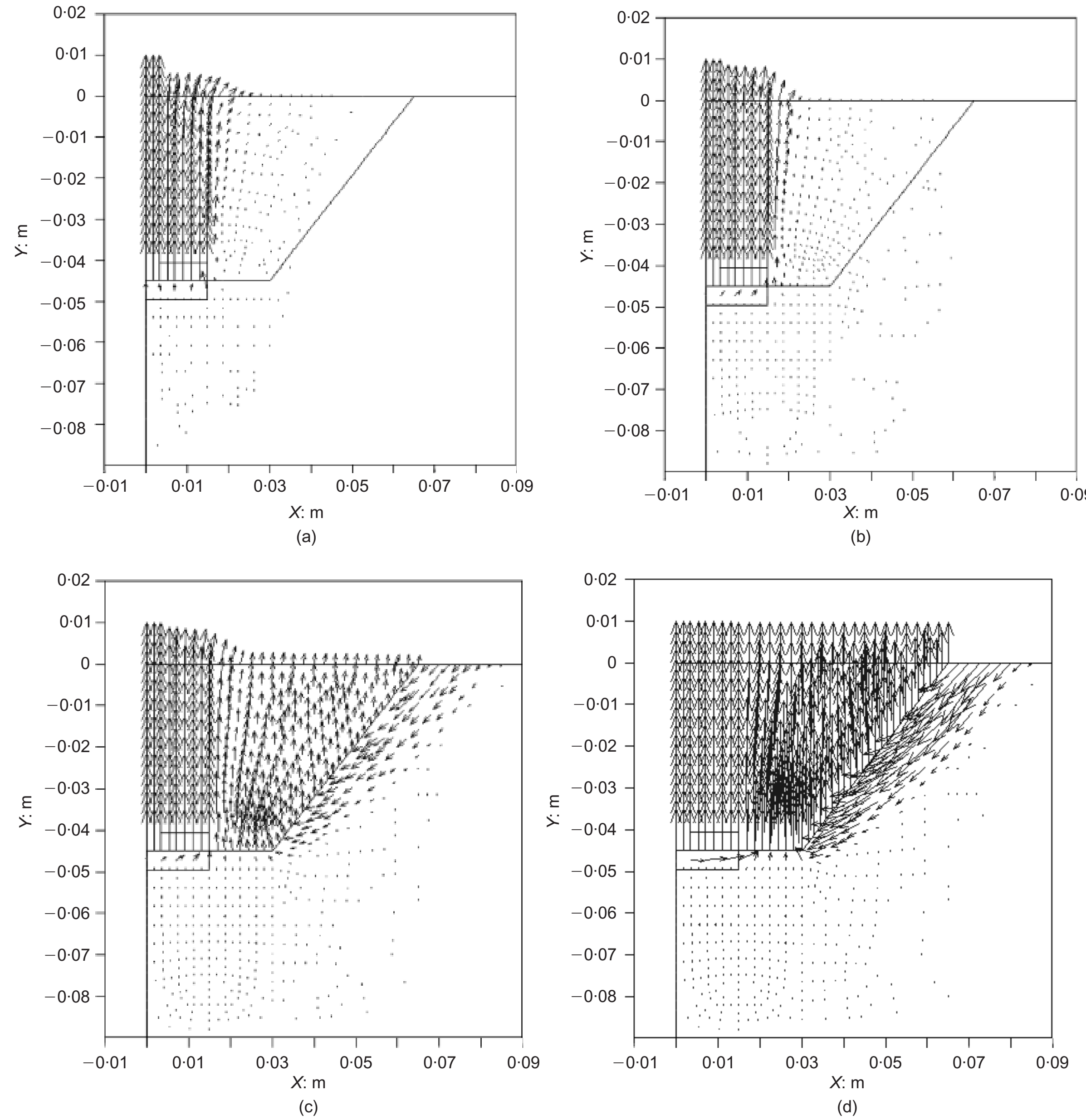

Fig. 9. Displacement vectors from FE analyses for uplift of footings in uncemented and cemented backfill: (a) 0% cement; (b) 1% cement; (c) 3% cement; (d) 5% cement

backfill stiffness of $\boldsymbol{E}' = \boldsymbol{G}_0/30$ allows the pre-peak stiffness and the displacement to peak capacity to be estimated with a good level of accuracy. It follows that the anchor stiffness increases in the same way with the backfill $\boldsymbol{c}'$ value (or cement content) that is apparent for anchor capacity in Fig. 6.

The computed displacement vectors at peak uplift load for the uncemented and cemented backfill are presented in Fig. 9. It is clear that the addition of cement to the backfill sand leads to a progressive outward shift of the failure mechanism, and it is this shift that provides the additional anchor stiffness and strength. For the case when the backfill cement content is 5%, it is apparent that the mechanism simply involves lifting of the entire block within the excavation. It follows that any further increase in the level of cementation will not lead to an increase in anchor capacity. This was verified in additional FE analyses, which predicted the same $\boldsymbol{q}_{fu}$ value for any value of $\boldsymbol{c}'$ above 60 kPa ($\equiv$ cement content just above 5%).

6. CONCLUSIONS

(*a*) Centrifuge tests and parallel numerical analyses have shown that very significant gains in stiffness and capacity may be obtained for shallow anchors subjected to uplift when relatively small amounts of cement are added to the (coarse-grained) backfill.

(*b*) The peak and ultimate capacities of anchors in cemented soil predicted using finite-element analysis with strength parameters obtained from direct shear tests were in good agreement with results from a series of physical modelling tests.

(*c*) The rate of gain in anchor stiffness and capacity varies directly with the backfill $\boldsymbol{c}'$, which in the experiments reported here varies approximately with the backfill cement content. No increase in capacity is possible above a cement content at which the backfill acts as an integral block.

(*d*) Anchors with cemented backfill exhibit a brittle response and a large reduction in available resistance after a normalised displacement (w/B) of $1{\cdot}2 \pm 0{\cdot}2\%$.

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