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A numerical study of factors governing the performance of stone columns supporting rigid footings on soft clay

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ABSTRACT: The Vibro Replacement technique is now frequently used as a means of improving the bearing capacity and settlement performance of soft cohesive soils. In this paper, a parametric study using the finite element method is presented which examines the influence of some key variables on the behaviour of small groups of stone columns supporting rigid footings. There is great potential to use the finite element method in an applied sense, as analytical approaches have many shortcomings and high quality field data is scarce.

1 INTRODUCTION

The optimum design of stone columns supporting small loaded areas (such as pad and strip footings) is arguably the most challenging aspect of stone column design in soft soils. Analytical theories developed to date assume an infinite grid of stone columns subjected to wide-area loading which is implemented mathematically using the *unit cell* approach (for example, Priebe's 1995 method for settlement design). Therefore they do not directly capture the behaviour of those columns under footings that are not equally confined on all sides; correction factors are applied within the design for this purpose. In addition, the vertical stress beneath footings decays much more sharply with depth than the stress beneath loaded wide areas, allowing partial-depth treatment to be used. Current design practice relies heavily on empirical methods for partial-depth treatment as analytical theory is much less well developed in this area.

Equally, a database of measured field settlement improvement factors in fine soils compiled by McCabe *et al.* (2009) highlights a dearth of data for strip and pad footings. High quality physical models of footings on soft clay supported by stone columns (i.e. McKelvey *et al.*, 2004, Black *et al.*, 2010) have been informative, although there are obvious difficulties in extrapolating model test findings to field scale, and the proportion of area under each pad that has been replaced with stone in these tests has tended to lie at the high end of what might commonly be used in practice.

Publications in which the finite element method has been used to model ground improved with stone columns mostly relate to wide-area loading, using either a unit cell (i.e. Domingues *et al.,* 2007) or 2-D axisymmetric (i.e. Elshazly *et al*., 2008) approximation. Some 3-D modelling of wide-area loading has also been carried out (i.e. Gab *et al*., 2008); however, hardly any 3-D modelling of footings has been published. In this paper PLAXIS 3-D Foundation (Version 2.2) is used to model the behaviour of rigid square pad footings supported by stone columns. The soil profile at the former geotechnical test site at Bothkennar, Scotland, is used as it is representative of many soft soil profiles in Ireland and the UK for which the applicability of stone columns is of growing interest. This paper begins to identify some of the key factors relevant to the design of small groups of stone columns, such as column arrangement, spacing, length, and Young's modulus of the column material.

2 MODEL OF BOTHKENNAR SOIL PROFILE

Located on the Firth of Forth estuary near Grangemouth, in Scotland, Bothkennar is the former UK test site for soft soil engineering research and as a result is extensively characterised. A weathered crust extends to a depth of 1.5 m and is underlain by 13.0 m of soft uniform Carse clay, deposited under shallow marine or estuarine conditions.

2.1 *General soil parameters*

The clay properties used in the soil model are presented in Table 1 and separated into crust, upper Carse clay and lower Carse clay. A high critical state friction angle (ϕ') of 34° (attributable to a high proportion of angular silt particles, Allman & Atkinson, 1992) is used for the Carse clay, and a nominal cohesion value of 1 kPa is used for numerical stability. A slightly higher cohesion value of 3 kPa was used for the weathered crust layers. Nash *et al.* (1992a) report the variation of yield stress ratio, which is equivalent to the overconsolidation ratio (OCR) measured in an oedometer, and *in situ* lateral earth pressure coefficient (K_0) with depth, suggesting that the stress state of the Carse clay may have been influenced by erosion of material, a relative drop in sea

Table 1. Parameters for Bothkennar soil model.

		Crust	Upper Carse clay	Lower Carse clay	Stone backfill
Depth	(m)	0.0–1.5	1.5–2.5	2.5–14.0	–
γ	(kN/m^3)	18.0	16.5	16.5	19.0
ϕ'	(°)	34	34	34	45
ψ	(°)	0	0	0	15
c'	(kPa)	3	1	1	1
OCR	(−)	1.0	1.0	1.5	–
POP	(kPa)	15	15	0	–
K_0	(−)	1.5	1.0	0.75	0.3
[1]E_{50}^{ref}	(kPa)	1068	506	231	70000
E_{ur}^{ref}	(kPa)	5382	3036	1164	210000
p_{ref}	(kPa)	13	20	30	100
m	(−)	1.0	1.0	1.0	0.3

[1]E_{50}^{ref} assumed equal to E_{oed}^{ref} (i.e. Elshazly *et al.*, 2009).

level and fluctuating groundwater levels (summing to a 15 kPa drop in vertical effective stress).

In choosing the friction angle of the stone backfill, reference was made to McCabe *et al.* (2009) who used measured settlement improvement data from the field to suggest that the conventionally- used value of $\phi' = 40°$ may be conservative for columns in soft cohesive soils constructed using the bottom feed system. Subject to adequate workmanship, the value of $\phi' = 45°$ shown in Table 1 should be readily achievable. The angle of dilatancy (ψ) was calculated based on the relationship $\psi = \phi' - 30°$.

2.2 *Hardening soil parameters*

The advanced elastic-plastic Hardening Soil (HS) model in PLAXIS 3-D Foundation was chosen to simulate the behaviour of the weathered crust, Carse clay and stone backfill. The HS model is an extension of the hyperbolic model developed by Duncan and Chang (1970). Creep behaviour is not considered in this model.

Nash *et al.* (1992b) report the variation with depth of the initial voids ratio (e_0), compression index (C_c) and swelling index (C_s) of the Carse clay. These parameters were entered into PLAXIS, which uses standard relationships to convert these one-dimensional parameters to three-dimensional quantities for the HS model; Young's modulus at half the maximum deviator stress (E_{50}) and the oedometric modulus (E_{oed}) are derived from C_c while the unload-reload Young's modulus (E_{ur}) is derived from C_s. The stiffness parameters adopted for the stone backfill are less certain, and thus are subject to parametric study in section 4.3. The reference stiffness (E^{ref}) is the stiffness corresponding to the confining pressure (p_{ref}); p_{ref} is the horizontal effective stress in the case of E_{50}^{ref} and E_{ur}^{ref}, and is the vertical effective stress in the case of E_{oed}^{ref}.

An important feature of the HS model is its ability to capture the stress dependence of soil stiffness. The parameter 'm' is used to control the relationship
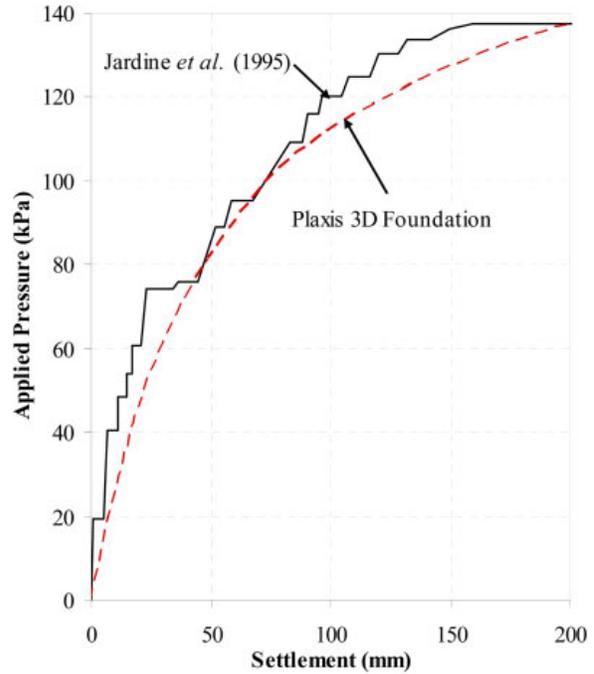

Figure 1. Validation of soil profile and parameters for a footing on untreated Carse clay.

between soil stiffness (E) and the corresponding confining stresses (p) according to eqn (1).

$$\frac{E}{E^{ref}} = \left(\frac{c' \cos \phi' + p \sin \phi'}{c' \cos \phi' + p^{ref} \sin \phi'} \right)^m \quad (1)$$

The reference stiffness moduli were chosen from the aforementioned Nash *et al.* (1992b) tests at the values of p_{ref} quoted in Table 1. This data also indicates that m = 1 is appropriate for the Carse clay. The value of m = 0.3 for the stone backfill is assumed based upon Gab *et al.* (2008) and others.

3 FINITE ELEMENT ANALYSIS

3.1 *Validation of soil model*

A well-documented field test on a pad footing at the Bothkennar site (no stone column support), see Jardine *et al.* (1995), was simulated using PLAXIS 3-D Foundation in order to substantiate the use of the parameters in Table 1. The footing, which was 2.2 m square and 0.8 m thick, was loaded to failure over 3 days using kentledge blocks, with loading pauses overnight and whenever settlement rates exceeded 8 mm/h. The Carse clay is modelled as undrained due to the short duration of the load test; concrete is modelled as a linear elastic material (Young's Modulus $E_{conc} = 30$ GPa; Poisson's ratio $\nu_{conc} = 0.15$). The load-settlement response of the footing recorded by Jardine *et al.* (1995) is shown in Figure 1 together with the PLAXIS 3-D prediction. It is clear that both curves are in good agreement, affirming the selection of the adopted soil profile and material parameters.
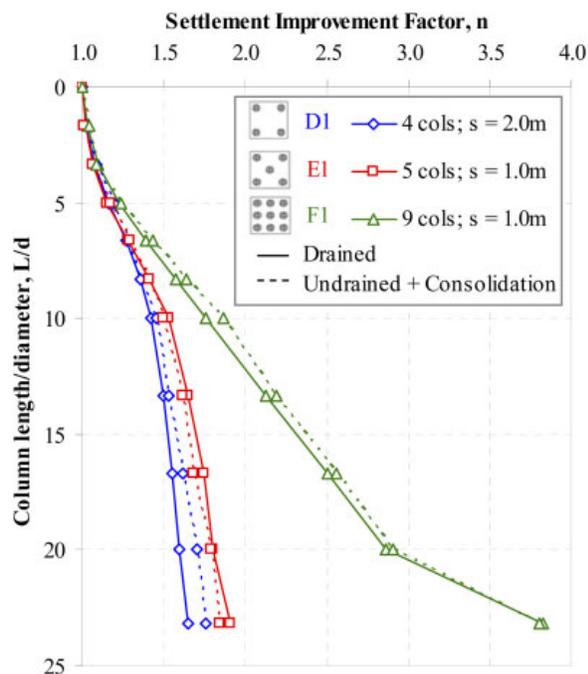
Figure 2. Influence of column length, arrangement and drainage analysis type upon settlement performance of stone columns.

3.2 Modelling of drainage

PLAXIS 3-D enables the long-term behaviour to be modelled in two ways: (i) drained analysis, using effective stress parameters and (ii) undrained analysis, using effective stress parameters, followed by consolidation. Figure 2 shows that for a 3 m × 3 m footing, loaded to 50 kPa and supported by various configurations of stone columns (depicted in the inset, refer also to Table 2), approaches (i) and (ii) produce quite similar (if not perfectly consistent) results. On this basis, the parametric study in Section 4 is based on a comparison of type (i) analyses only. Type (ii) analyses take longer to perform but these analyses are underway and will be published at a later date.

3.3 Other modelling issues

The stone columns in this study have been wished-in-place, i.e. the ground properties have not been modified to reflect changes induced by installation of the columns. Some authors (i.e. Watts et al., 2000, Kirsch, 2008) have reported increases in total stress after column installation, but as noted by McCabe et al. (2009), it is the equalized effective stresses around columns (once pore pressures have dissipated) which influence column performance under load, and these have not been measured.

The interaction between the stone column and the surrounding soil is simulated using elastic-plastic interface elements. Owing to the process of column construction, the stone is tightly interlocked with the surrounding soil and it is assumed that a perfect bond (total adhesion) occurs along this interface.

Table 2. Details of parametric study.

Test name	Ftg size (m)	k (−)	s (m)	F (−)	$E_{50,col}$ (MPa)
A	2 × 2	4	1.0	3.5	70
B	3 × 3	4	1.0	8.0	70
C	3 × 3	4	1.5	8.0	70
D1	3 × 3	4	2.0	8.0	70
D2	3 × 3	4	2.0	8.0	50
D3	3 × 3	4	2.0	8.0	30
E1	3 × 3	5	1.0	6.4	70
E2	3 × 3	5	1.0	6.4	50
E3	3 × 3	5	1.0	6.4	30
F1	3 × 3	9	1.0	3.5	70
F2	3 × 3	9	1.0	3.5	50
F3	3 × 3	9	1.0	3.5	30
G	4 × 4	16	1.0	3.5	70

This approach was also adopted by Guetif et al. (2004) and others.

3.4 Parametric study details

Settlement rather than bearing capacity criteria generally govern the design of stone columns in soft soils. Key variables in the settlement design of stone columns to support footings include footing size (B), column length (L), column spacing (s), number of columns (k), column arrangement and stiffness of the column material (E_{50}). The various parametric combinations considered in this paper are labelled A-G in Table 2. The footing, which is 600 mm thick, is founded 600mm below ground level. Column diameter is not normally a significant variable in design as the poker is of fixed diameter and the final column size is a function of soil consistency. A diameter of 600 mm is assumed here for soft soils constructed using the instrumented bottom feed system.

In Table 2, the quantity $F = A_f/kA_c$ is referred to as the footprint replacement ratio, and is a measure of the extent to which the soil area under the footing is replaced by stone (A_f is the footing area, k is the number of supporting columns and A_c is the cross-sectional area of each column). Since each footing is square (of width B) and all columns are 600mm diameter, the expression for F can be simplified to:

$$F \approx 3.54 \left(\frac{B^2}{k} \right) \qquad (2)$$

F is an adaptation for footings of the area replacement ratio (A/A_c, A is the total loaded area) used by Priebe (1995) for unit cells / infinite grids. For each configuration A-G outlined in Table 2, at least 9 different lengths were investigated; covering various extents of partial depth treatment and full depth treatment to 14.5 m below ground level. The effect of varying the E_{50} value of the column material is assessed for cases D1-3, E1-3 and F1-3.

In subsequent plots, the extent of settlement improvement is quantified by means of the settlement

improvement factor (n), defined as the ratio of the settlement of the footing without treatment to the corresponding settlement with treatment (with the same stress applied to the footing in each case).

4 PARAMETRIC STUDY

4.1 Influence of column length and footprint replacement ratio

The type (i) data for the 3 m × 3 m footing shown in Figure 2 is revisited, which shows the influence of column length (L) and footprint replacement ratio (F) upon the predicted value of settlement improvement factor (n) for a column stiffness $E_{50,col} = 70$ MPa. In all cases, it is clear that n increases with L. Settlement improvement is not observed until columns are installed beyond $L/d \approx 3$, as the weathered crust, which extends to 1.5 m, is already competent.

The classical Boussinesq solution for vertical stress distribution under a footing would suggest that the stress increment applied to the footing is no longer perceived at $L/B = 2$, which is equivalent to $L/d = 10$ in this instance. The PLAXIS output indicates that improvement can still be achieved by constructing columns longer than $L = 10d$. McKelvey et al. (2004) suggest that while no benefit to bearing capacity was achieved by extending columns beyond $L/d = 6$, additional benefit to settlement was achieved up to $L/d = 10$, the maximum length of the partial depth columns.

The length effect in this study is most pronounced for the 9 column group (F = 3.5) which also appears to benefit greatly from end bearing (onto a boundary at 14.5 m, modelled as rigid), suggesting that the applied load in this case is being transmitted to great depth. This tendency is also noted in the model test data of Black et al. (2010) with similarly low F values. This would indicate that the benefit of lengthening columns is greatest when they are already closely spaced. However, the F values for the 4 column (F = 8.0) and 5 column (F = 6.4) groups are more representative of practice.

Muir Wood et al. (2000) conducted a series of laboratory scale model tests to investigate the influence of column diameter, length and spacing upon the mechanisms of stone column behaviour. The authors observed that as the replacement ratios increased, the columns bulged in the upper zones of the soil layers and transferred the load to greater depth.

4.2 Influence of column position beneath footing

The column spacing for a given footprint replacement ratio (or alternatively thought of as the position of the columns in relation to the edge of the footing) is seen in Figure 3 to have a minor influence on the behaviour of the footing. It appears that small benefits can be gained by keeping the columns closer to the footing edge, although these benefits become negligible beyond $L/d \approx 15$. It is well known that the
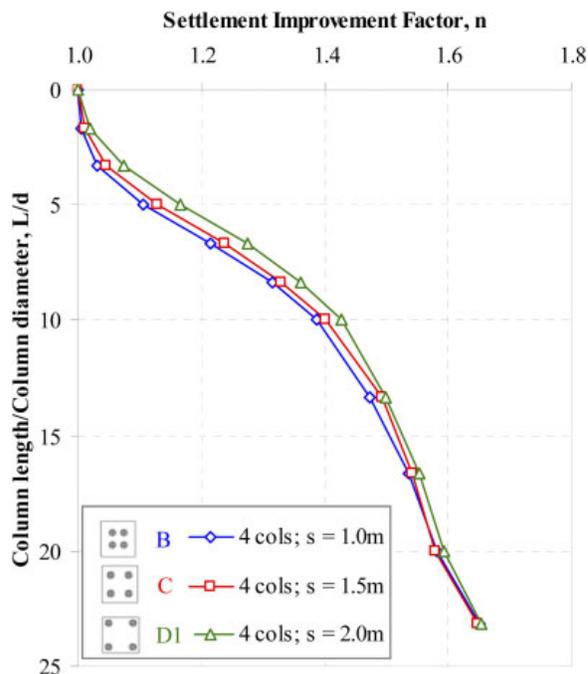

Figure 3. Effect of column spacing (or position in relation to the edge of the footing) on settlement performance.

stress distribution beneath rigid footings in clay is such that higher stress concentrations develop towards the edges, so columns placed here have the potential to absorb more load and develop improved n values.

Also, the stress concentrations that develop decay rapidly with depth and this may explain why the influence of column spacing is restricted to short columns. A finite element study by Wehr (2006) to examine the group behaviour of stone columns demonstrates that shear zone develop at the edges of a pad footing and extend to a depth beneath the centre of the footing. Positioning the columns closer to the edge of the footing, and thus closer to these shear zones, may also explain the enhanced settlement performance.

4.3 Influence of column deformability

Figure 4 demonstrates that the stiffness (E_{50}) of the stone backfill has an influence on the settlement improvement behaviour of stone column reinforced foundations. A reduction in the stiffness of the column leads to a reduction in settlement improvement, which is similar to findings from finite element modelling on an embankment (Domingues et al., 2007) with columns having $A/A_c \approx 5$.

Interestingly, from Figure 4, the effect of column deformability on settlement appears to be much more pronounced for the 9 column group (F = 3.5) than for the two groups with higher F values.

4.4 Column confinement

Figure 5 compares three footings sizes, with the number of columns chosen to maintain equal footprint
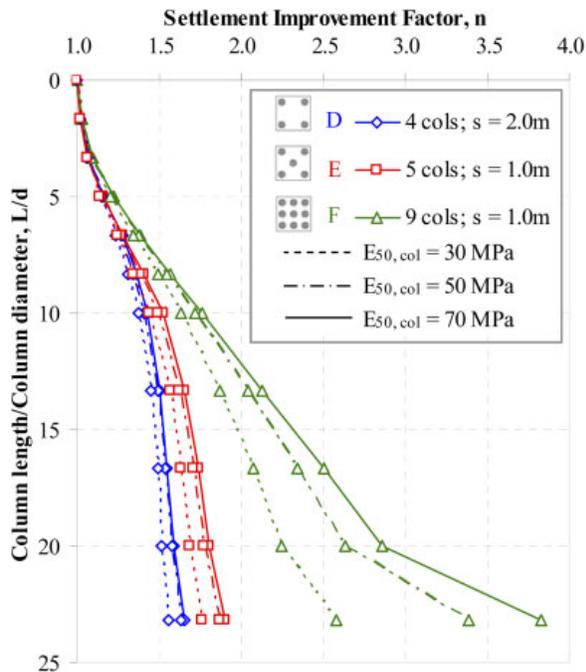
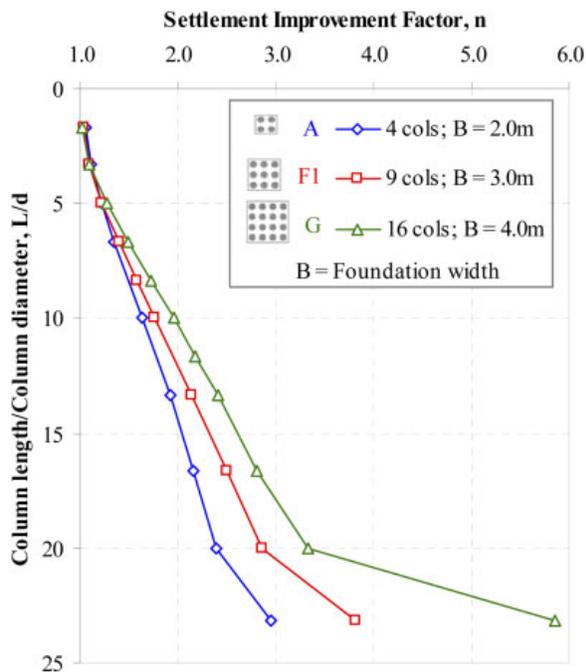Figure 4. Effect of column stiffness upon settlement performance.

Figure 5. Effect of column confinement upon settlement performance.

replacement ratios (F = 3.5). The settlement performance of the footings improves as the footing size and the number of supporting stone columns increases.

It is well documented that an isolated column will tend to bulge when loaded. This bulging tends to occur near the ground surface, where the overburden stresses are at their lowest. Figure 6 (which is a diagonal cross section through the 4 m × 4 m group G) highlights that the outer columns beneath the pad footings tend to
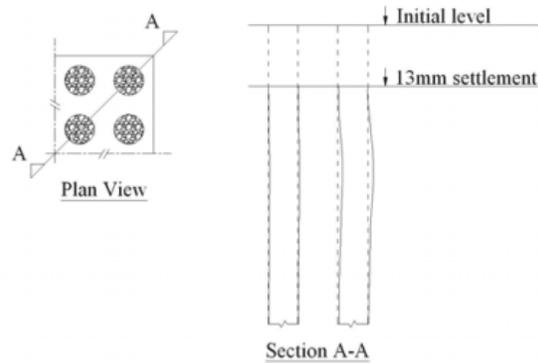

Figure 6. Mode of deformation for 4 m × 4 m group G.

bulge outwards from the footing centre and towards the unconfined side; the inner columns appear to bulge less and more uniformly. This behaviour was also observed by McKelvey *et al.* (2004), who examined the interaction between stone columns beneath strip and pad footings. As the number of columns beneath a footing increases, so does the number of columns with full confinement on all sides and therefore the average settlement performance of a group column will improve.

It should be noted that an increased footing size will stress the soil to a greater depth, which should induce more settlement. However, Figure 5 indicates that this effect is more than offset by the positive effects of column confinement.

5 CONCLUSIONS

A parametric finite element study with an advanced soil model was carried out to assess the effect of a number of key design variables on the settlement performance of rigid pad footings supported by stone columns. The following conclusions may be drawn, which are specific to a type (i) drained analysis for the ground profile modelled:

The PLAXIS 3-D output suggests that settlement performance continues to improve beyond $L/d = 10$, and this improvement is more pronounced for groups with a low footprint replacement ratio. End bearing is also significant for the F = 3.5 case, but this may be related in part to the assumption of a rigid layer.

Columns closer to the footing edge perform better for short column lengths ($L/d < 10$) than for columns closer to the centre, but the 'n' values converge with depth and long stone columns are relatively insensitive to column spacing.

The stiffness of the stone backfill has a significant influence on the settlement performance of a footing supported by a large number of stone columns. However, as the number of supporting columns reduces, so does the influence of column stiffness.

For a given footprint replacement ratio, an increased number of columns supporting a footing leads to an increase in the proportion of group columns that have

full confinement (i.e. behave like a unit cell) resulting in enhanced settlement performance of the footing.

It is acknowledged that more definite conclusions from the finite element work are pending upon the outcome of the type (ii) analyses (undrained loading followed by consolidation), and the output from the modelling in general can only be satisfactory validated by full scale field testing. The long term settlement behaviour of footings on soft soils must also consider creep.

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