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Ground heave induced by installing stone columns in clay soils

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Measurements of ground heave induced by installing stone columns in soft clay are presented in this paper. These measurements are likely to provide guidance to practitioners concerned about the possible impact of column construction on adjacent structures and services. Heaved volume, variation in heave with radius and the radial extent of heave were found to follow experience of driven piles. Simplified finite-element analyses have also been performed which indicate that the thin stiff crust typical of soft clay profiles appears to have little influence on heave, whereas soil stiffness variation with depth and column length may be influential.

1. Introduction

While ground heave caused by the undrained installation of displacement piles has received reasonable attention in the literature over the years, very little has been published on heave generated by the vibro replacement process for constructing stone columns. The potential for heave-induced damage is often a concern when stone columns are to be constructed close to movement-sensitive structures and services, such as shallow foundations and pipelines. In this paper, the authors present new data, which are likely to be of use to practitioners, on the magnitude and radial extent of heave monitored during the installation of stone columns at a soft clay site. Heave predictions from simplified finite-element simulations of the column construction process are also provided; although a perfect match with the field data cannot reasonably be expected, the exercise is helpful in identifying influences such as column length, soil stiffness variation with depth and the presence of a stiff crust (overlying soft clay).

2. Field heave measurements

Keller Foundations designed and constructed bottom-feed stone columns in 2009 to support a two-storey office block at a redeveloped industrial site at Grangemouth, Scotland. The average column length (L) was 5·5 m, and the average diameter over the column length of 550 mm was deduced from volumes of stone delivered to site, less an estimate for wastage, and with an allowance for the differences between stone densities as delivered and compacted. The reader is referred to Jarrett *et al.* (1974) for a description of the stratigraphy of the Grangemouth area, which is dominated by Carse Clay.

The 14 stone columns C1 to C14 were installed in numerical sequence in two parallel rows, C1 to C9 and C10 to C14, as shown in Figure 1. Resulting heave movement was monitored at each of 26 levelling points (again in two parallel rows, A1–A13 and B1–B13); the total accumulated heave at each point was noted after each successive installation. Subsequently, accumulated heave measurements were separated into heave values (h) specific to each individual column installation. The h values are plotted in Figure 2(a) as a function of the radial distance (r) between the measurement point and the column centreline, normalised by the average column radius $R_0 = 275$ mm. In Figure 2(b), the same data are reproduced in terms of h/R_0.

The data in Figure 2 produce a reasonably consistent set of curves. There is no evidence that the line of columns C1–C9 has had any 'blocking effect' on the heave registered due to the installation of columns C10–C14, although the movements are less than 5 mm at the closest heave measurement for these columns of $r/R_0 = 15$ approx. Given that accumulated heave data for several columns can be uncoupled to give consistent heave data for the individual columns, it should follow that heave fields caused by single installations could be superimposed to estimate the heave generated by multiple installations. Interestingly, Cole (1972) also found that the final magnitude of pile heave is the cumulative sum of all of the individual pile heaves caused by driving adjacent piles.

From Figure 2, it appears that the radial extent of heave around a single stone column is no greater than $r = 20R_0$. Few studies have considered the extent of heave around stone column groups, although Egan *et al.* (2008) showed that heave appeared to extend to about 5–5·5 m at right angles to a line of five stone columns (this equates to $r/R_0 \approx 18–20$ with $R_0 = 275$ mm). This is broadly compatible with experience for driven piles; Cooke and Price (1973) quoted heave movements to be small but measurable

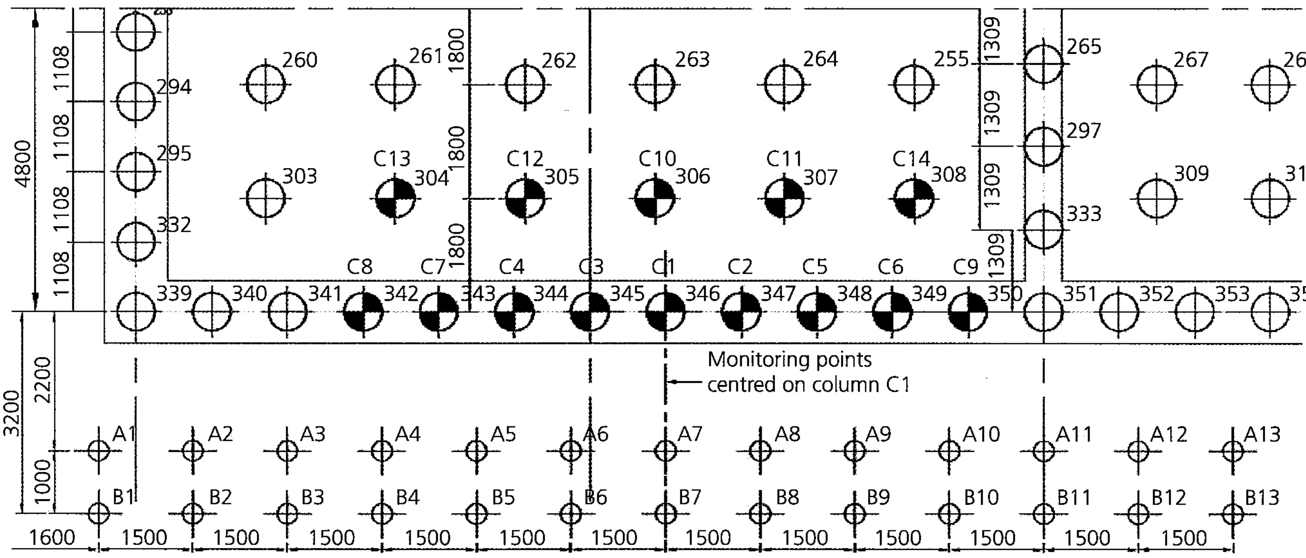

Figure 1. Settlement monitoring at the Grangemouth site

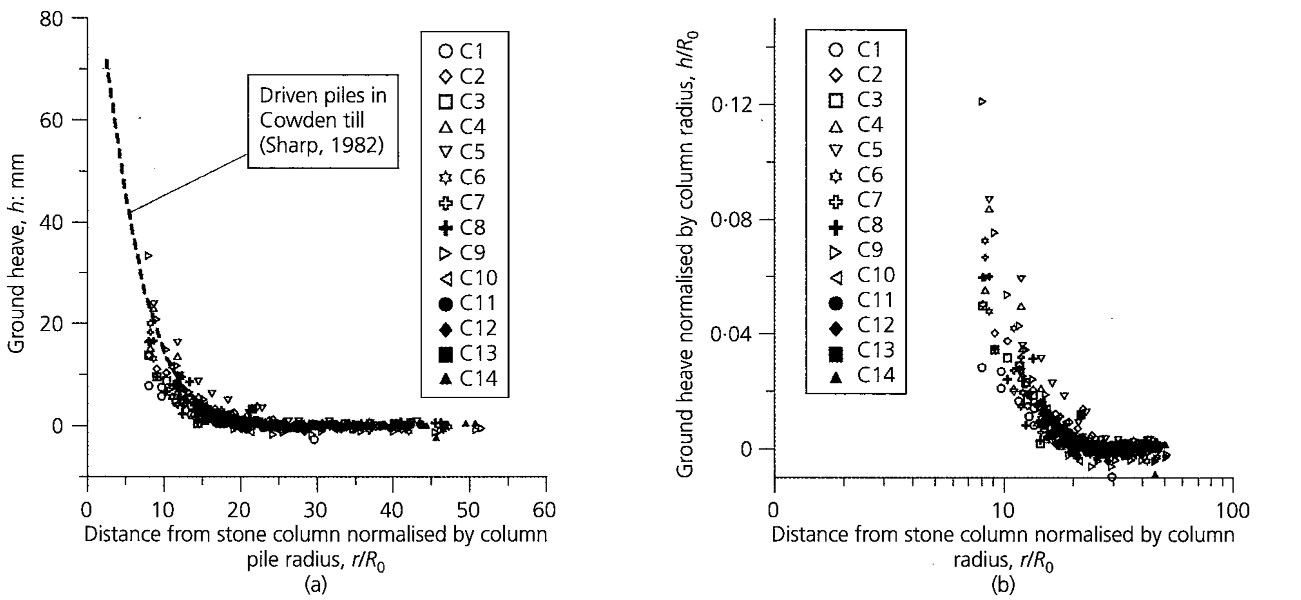

Figure 2. Measured data for Grangemouth site: (a) heave; (b) normalised heave

beyond $r/R_0 = 20$. Excess pore pressures measured in the vicinity of single jacked piles in clay (McCabe *et al.*, 2008) show $r/R_0 \approx 20$ as a limit of radial influence.

Also superimposed in Figure 2(a) are field data by Sharp (personal communication, 1982, referenced by Gue, 1984), who measured vertical heave around 203 mm diameter closed-ended piles driven in stiff Cowden till. The data are derived from a normalised plot of h/R_0 against r/R_0, adopting the curve most representative of typical stone column aspect ratios. The magnitudes of heave around the piles, although marginally higher, have similar radial extent.

A fourth-order polynomial (Equation 1, with $a_4 = 0{\cdot}0007$, $a_3 = -0{\cdot}0587$, $a_2 = 1{\cdot}8$, $a_1 = -24{\cdot}129$, $a_0 = 122{\cdot}41$) was used to approximate the 'average' measured h–r/R_0 trend measured at the Grangemouth site, with an assumed variation for $r/R_0 < 7$ where no measured data were available, using Sharp's data as a rough guide to the shape.

$$h = a_4\left(\frac{r}{R_0}\right)^4 + a_3\left(\frac{r}{R_0}\right)^3 + a_2\left(\frac{r}{R_0}\right)^2 + a_1\left(\frac{r}{R_0}\right) + a_0 \quad (1)$$

When a volume integration was performed (between $1 < r/R_0 < 20$) on Equation 1, the integrated heaved volume was found (within the accuracy of the assumptions made) to be $100 \pm 10\%$ of stone column volume, as might be expected for an undrained installation, and this is in keeping with the 90–100% cited by Gruber (1994) and quoted by Kirsch (2008) for stone columns. Driven piling experience is similar; Adams and Hanna (1971) and Massarsch (1976) both showed that all the embedded volume of piles appears as heave. In studies where measured heave volume is reported to be lower than the embedded volume, this is generally because heave is measured only within a pile group perimeter (for instance), and/or the full extent of heave volume has not been captured.

3. Numerical modelling

3.1 Model details

Axisymmetric (2D) finite-element (FE) simulations were carried out using Plaxis v9.01 (Brinkgreve *et al.*, 2008) to assess whether some factors influencing heave could be captured using routine analyses within the capability of practitioners who have some FE modelling experience.

The vibro replacement installation process involves two distinct parts: (a) penetration of a vibrating poker to displace soil and form a cavity; and (b) compaction of stone within the cavity by successive withdrawals and reinsertions of the poker. In the FE analyses, this complex process was modelled by simply expanding a cylindrical cavity uniformly along the cavity wall from a nominal small radius of 0·02 m to a final radius $R_0 = 0{\cdot}275$ m in undrained mode (similar to the approach used by Guetif *et al.*, 2007), for example. Three column (or 'cylinder') lengths (L) were used: 2·75 m, 5·5 m and 8·25 m corresponding to L/R_0 ratios of 10, 20 and 30, respectively. For stability reasons, the tip of the column was modelled as a wedge element with an angle of 45°, which avoids any numerical problems in this area without compromising the FE output. The 2D mesh, shown in Figure 3, consists of more than 2100 15-noded elements, with greater refinement in close proximity to the column cavity. Since large strains are generated during cavity expansion, updated mesh calculations were necessary. FE software updates not only the mesh nodes' coordinates (and the position of the stress points) but also the stiffness matrix, and applies a correction procedure

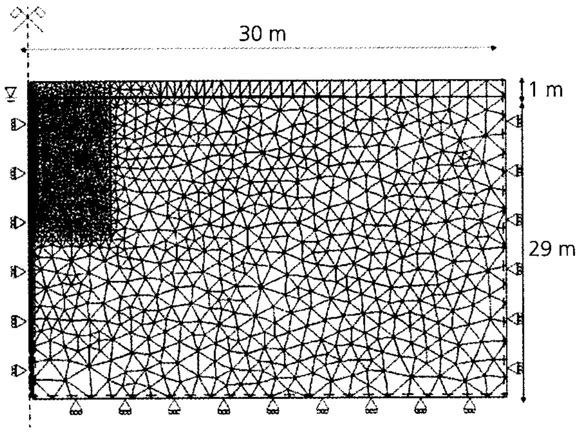

Figure 3. Typical mesh and boundary conditions used for FE analysis

to the calculations to account for large strains during simulations. The boundaries of the model were assumed to be free in both directions, and the usual checks on mesh sensitivity and extent of boundaries were applied.

As is often the case with routine projects, the ground investigation information provided for this project was not adequate to enable derivation of the full suite of parameters for FE modelling. Instead, Mohr–Coulomb (MC) soil models were developed for the Carse Clay at the Bothkennar test site, which has been the subject of many comprehensive studies (ICE, 1992) and FE analyses (Krenn and Karstunen, 2008; Kamrat-Pietraszewska *et al.*, 2008; Kamrat-Pietraszewska and Karstunen, 2009). It is acknowledged that the use of a more sophisticated model would be preferable for the clay; however, MC was adopted for this rudimentary parametric study to identify factors that might warrant further, more detailed consideration with higher-order models. Two stiffness variations with depth were considered; one having constant stiffness with depth (Clay A) and the other having stiffness increasing linearly with depth, i.e. a 'Gibson' material (Clay B). Analyses were performed to assess the influence of the stiff overconsolidated crust up to 1 m thick that typically overlies soft clays such as the Carse Clay (Clay A + crust, Clay B + crust). Relevant parameters are shown in Table 1. The groundwater table is assumed to be 1 m below the ground surface.

Depth, z: m	γ: kN/m^3	ν	K_0	ϕ': degrees	E_{ref}: kPa
0–1	19·0	0·2	0·7	37·5	4300 (if no crust) 25 000 (if crust)
1–20 (Carse Clay)	16·5	0·2	0·5	37·5	4300 (Clay A) 2840 + 1460z (Clay B)

Table 1. Geotechnical parameters used to simulate dry crust/Carse Clay profile

3.2 Modelling results

The radial variations of heave are shown (as a normalised plot of h/R_0 against r/R_0) for Clay A in Figure 4 and for Clay B in Figure 5. The general trends in Figures 2(b) and 4 are similar, although it should be noted that a perfect match could not be expected, given the simplified nature of the modelling, and the substitution of Bothkennar soil parameters for those in existence at the Grangemouth test site. The magnitude and radial extent of heave appear virtually unaffected by the presence of the 1·0 m stiff crust, so results for simulations considering the dry crust layer in the soil profile are omitted from Figures 4 and 5 for clarity. Rapid decrease of the heave magnitude with distance from the column has been captured well, although the predicted heave for the 5·5 m long column has a larger radial extent than the field measurements. Some problems in reproducing the surface heave at the lower r/R_0 range may be explained by the assumptions inherent in the cavity expansion used.

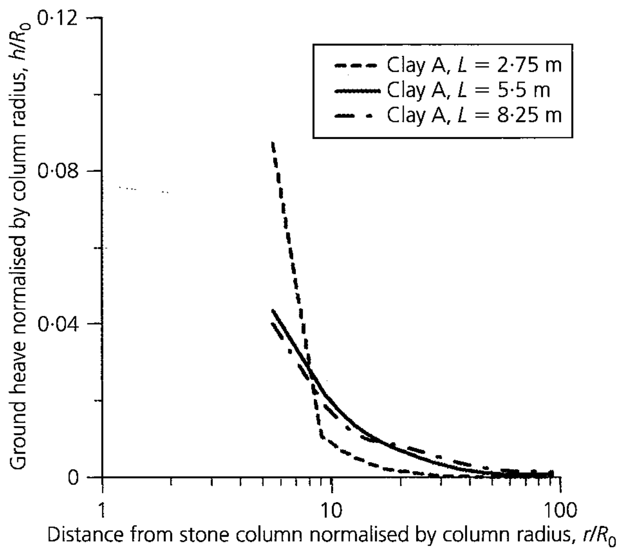

Figure 4. Plaxis heave predictions for Clay A

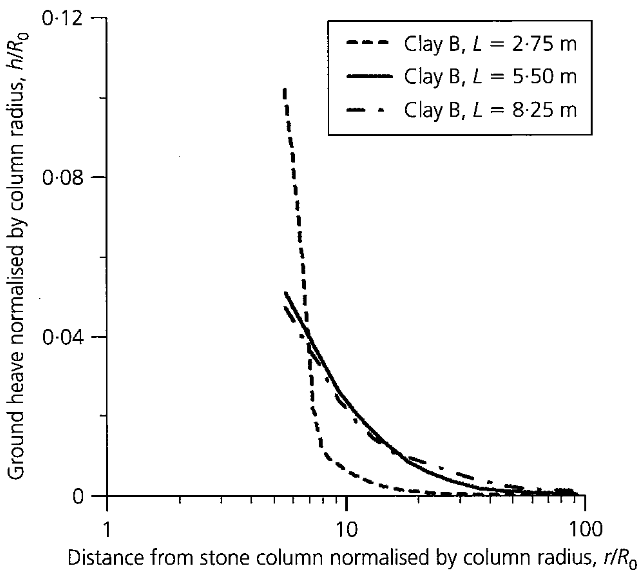

Figure 5. Plaxis heave predictions for Clay B

Figures 4 and 5 also show the effect of the modelled cavity length on the value of h/R_0. The value of h/R_0 for the shortest cavity (2·75 m long) is higher than for the 5·5 m cavity at low r/R_0 values, but decays more rapidly with radius. The trends for the 5·5 m and 8·25 m columns are quite similar, so the FE analysis appears to predict a limiting cavity length beyond which the h/R_0–r/R_0 curve is unaffected.

The effect of stiffness variation with depth within the Carse Clay can be seen by comparing Clay A in Figure 4 with Clay B in Figure 5. The differences are generally quite small, but are more pronounced for the longer cavities (5·5 m and 8·25 m) than for the short cavity (2·75 m).

4. Conclusion

In this paper, some new field data have been presented that will be of interest to practitioners concerned about damage to existing structures and services due to nearby stone column construction in soft clay deposits. Some specific conclusions include the following.

(*a*) The Grangemouth data provide evidence that superimposition may be used to estimate the cumulative heave generated by multiple stone column installations.
(*b*) The heave field generated by installing a stone column in undrained mode is very similar to that generated by driving a pile.

In addition, a simplified FE parametric study has been presented in which cavity expansion is used to represent the column installation process, and a simple elastic perfectly plastic model is used to represent the soft clay. A direct comparison between the field data and the FE output is not appropriate; however, the similar patterns obtained give confidence that the FE has not excluded any of the most important aspects of behaviour. The study has indicated that the presence of a thin crust may have little effect on the magnitude and extent of heave, and that stiffness variation with depth may be influential; however, further investigation is warranted using a model that is more appropriate for estimating heave in soft clay, such as Cam Clay.

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