

# Observed installation effects of vibro replacement stone columns in soft clay

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**ABSTRACT:** The installation of vibro replacement stone columns in any soil causes significant changes to the stress regime around each probe location. In soft clays, the principal effects are caused by a cavity expansion mechanism as the probe is introduced into the soil and the stone column constructed. This paper presents observations of changes in soil volume and pore water pressure during stone column installation.

The implications for the most suitable column construction techniques and the soil strength limiting column construction arising from the observations are discussed. The data presented are also of interest to those validating numerical models of stone column installation, and issues relevant to the practical application of numerical modeling are considered.

## INTRODUCTION

This paper considers the observed effects of both single and multiple stone column installation in soft clay and demonstrates that successful ground improvement can be achieved with this technique. A detailed explanation of the various Vibro Replacement and Compaction techniques available is provided by Sonderman & Wehr (2004) and is not repeated here. This paper concentrates on stone columns installed using the dry bottom feed method, which is the most widely used technique in the UK for routine projects on soft clay sites, often preferred nowadays to the wet method. A significant advantage of the dry bottom feed method over the wet method is that a supply of water flush is not required with the attendant requirement of handling and disposal of the wet spoil. Numerous case studies illustrating the use of the dry bottom feed method for a wide range of projects have been published, for example, Cooper & Rose (1999), Wehr (2006), McNeill (2007), Castro (2007) and Egan (2008).

Three aspects of the installation process are considered in this paper: (a) the ground response during the installation of a dry bottom feed stone column, (b) changes to the soil stress field and pore water pressure during the equalization period following stone column construction and (c) the interaction effects resulting from the construction of both small groups and large grids of columns.

## THE CONCEPTUAL MODEL FOR INSTALLATION OF A SINGLE COLUMN

An example of a modern bottom feed poker is shown in Figure 1. The main body of the poker is broadly cylindrical with a number of fins attached which counteract the tendency for the poker to rotate during column construction. The vibrating motion is generated by an electric powered rotating weight housed within the poker casing.

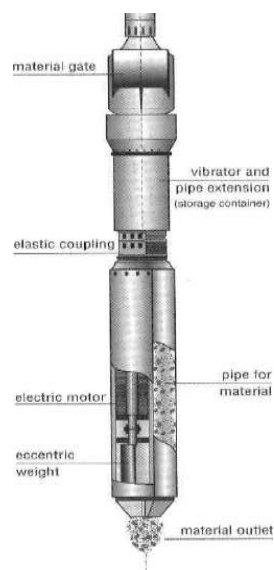


Figure 1 a typical bottom feed poker

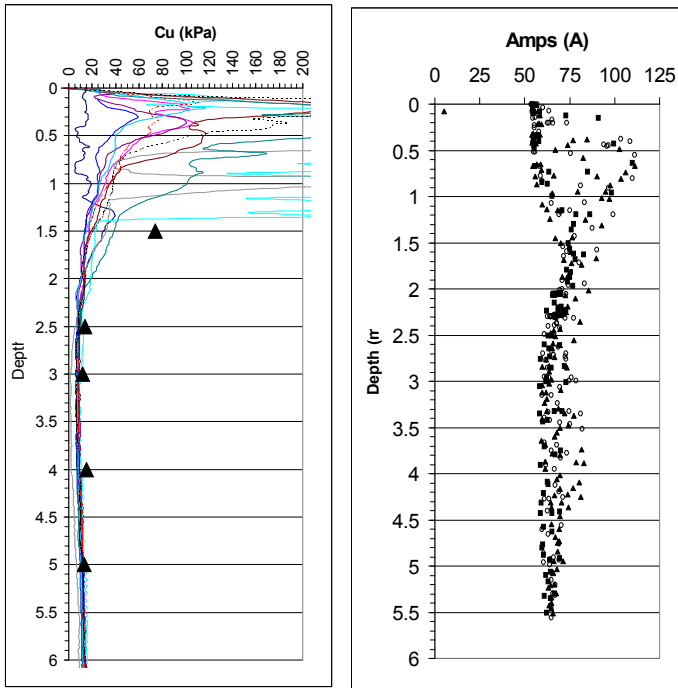


Figure 2. Poker current demand compared with undrained shear strength

The stone is fed to the base of the poker body and compressed air flush is used to assist the stone delivery. The air flush, which is channeled to the bottom end of the poker, assists in creating a small annulus between the poker and the ground, reducing frictional drag. The air flush also counteracts the suction that the poker would generate as it is withdrawn from the base of the column back up the bore, and which could lead to the surrounding soil moving back into the previously created void.

The electrical current drawn by the poker during initial penetration of the ground and subsequent compaction of the stone provides an indication of the strength and stiffness of the ground. Figure 2 compares the measured draw of electrical current plotted against depth during the construction of a 5.5m long column in soft alluvial clay. Also shown is the profile of undrained shear strength from the site (estimated from cone penetration test data). The increase in current draw with increase of the undrained shear strength of the host soil is clear. As there are a number of other factors which can affect the current draw of the poker it is not possible to develop a simple and universal relationship with soil strength. However the ability to measure and record the current consumption does offer a qualitative indication of the quality of column construction in soft soils where overworking of the ground (a sign of poor workmanship) should be avoided. This is one of the reasons why modern dry bottom feed rigs, fitted with advanced instrumentation, can be used where hitherto the wet system would have had to be adopted.

As the poker is lowered into the ground the soil is mainly displaced laterally as a cavity is opened up. Some vertical smearing of the soil adjacent to the poker may occur, although there is little evidence that this is significant, and some vertical heave of the ground surface can result. The magnitude of heave is highly dependent on the arrangement of columns being constructed as discussed below.

## HEAVE

Most experienced practitioners are aware that the installation of stone columns into soft clay can generate a degree of surface heave, particularly if construction control is inadequate and overworking of the ground takes place. Some degree of surface heave during the installation of stone columns using the dry bottom feed method is inevitable and demonstrates that the stress regime around the column is being altered and improvement of the ground is being achieved and is not detrimental to the final foundation performance. For this reason heave is rarely measured (indeed it is not necessary to do so). Nonetheless the few case studies where heave was recorded are of interest as they show that the amount of heave is a function of the column size, spacing, extent and construction method.

Castro (2007) reported a conical zone of surface heave with a maximum value of 290mm following the installation of a test group of seven columns.

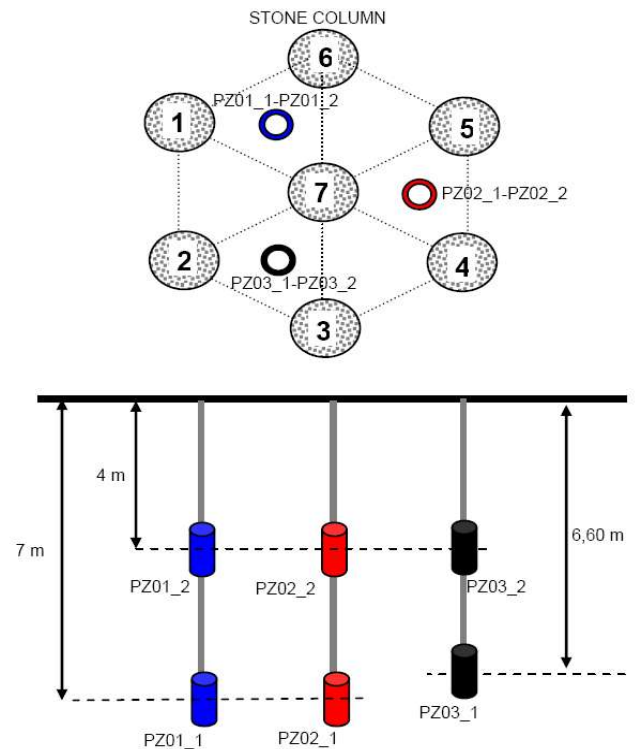


Figure 3. Stone column and instrumentation layout reported by Castro (2007)

The columns were 0.8m in diameter, 9.0m long installed at 2.8m centres through soft to firm alluvial clay by Keller Terra in Spain. Figure 3 shows the column layout and the locations of piezometers installed prior to the stone column construction. It is unclear if the heave reported was a local maximum and the overall volume of heave was not reported.

Watts et al (2000) reported different amounts of heave for different column densities in the soft Carse clay at the Bothkennar test site in Scotland. (Details of this site are fully discussed in Geotechnique, Volume 42, 1992). The columns were installed to a length of 6m using a Bauer HBM4 bottom feed rig. The installation procedure followed the now dated practice of ramming a large volume of stone into the base of the columns creating an enlarged toe of up to 1.25m diameter. Extensometers revealed that most heave was generated by the construction of the bulb, emphasising that this practice is to be avoided if heave is to be controlled. No heave was recorded for two columns located at 1.5m centers (although 150mm of lateral displacement was measured by inclinometer 0.5m from the columns). Three columns at 2.0m centers produced an average local increase in ground surface of 107mm; four columns at 1.5m centers produced an average of 143mm of heave. A maximum of 425mm heave was recorded due to installation of 25 columns 6m long to support an 8.1m square concrete test raft. The total volume of heave recorded for these 25 columns equated to 27% of the stone introduced into the ground.

Measurements of heave have also been made during a few contracts performed by Keller Ground Engineering in the UK. In Contract A in Lincolnshire, an average of around 150mm of surface heave was measured during the installation of

a large grid (80m x 70m) of 5m long 450mm diameter columns in soft alluvial clay ( $20\text{kPa} < C_u < 50\text{kPa}$ ,  $30\% < w < 70\%$ ). The columns were installed on a 2m by 2m square grid and the volume of heave equated to 75% of the volume of stone in the columns. In another contract (Contract B), three separate rows each comprising five columns (550mm diameter and 5.5m long) were installed in Carse clay. Contours of the average magnitude of heave, estimated from a grid of levels taken around each column group, are shown in Figure 4. The average maximum heave was around 70mm along the centreline, diminishing with distance from the columns. In this case, the volume of heaved ground was estimated to be approximately 35% of the volume of stone installed in the columns. While the phenomenon of heave is occasionally referred to in the literature, very little quantitative guidance is available to estimate how much heave can be expected for any given column arrangement.

Defining the Perimeter Ratio  $P$  and Heave Ratio  $H_p$  according to equations [1] and [2] below appears to allow a relationship to be developed, and values of  $P$  and  $H_p$  are shown in Table 1 for the case histories discussed above.

$$P = \frac{\sum \text{Column perimeters in a group}}{\text{External perimeter of the group}} \quad (1)$$

$$H_p = \frac{\text{Volume of heaved ground}}{\sum \text{Volume of stone columns}} \quad (2)$$

Table 1. Data on observations of heave during column installation

| Reference          | Number of columns in group | Column length<br>m | Average column diameter<br>mm | Arrangement                                  | Perimeter ratio, $P$ | Heave percentage, $H_p$<br>% |
|--------------------|----------------------------|--------------------|-------------------------------|--|----------------------|------------------------------|
| Castro (2007)      | 7                          | 9                  | 800                           | 2.8m spacing                                 | 1.05                 | 12                           |
| Watts et al (2000) |                            |                    |                               |  |                      |                              |
| 8.1m raft          | 25                         | 6                  | 940                           | 1.5m centers around edge of raft 5 in centre | 2.28                 | 27                           |
| 2 Columns          | 2                          | 6                  | 940                           | -  | 0.99                 | 0                            |
| 3 Columns          | 3                          | 6                  | 940                           | -  | 0.99                 | 3                            |
| 4 Columns          | 4                          | 6                  | 940                           | -  | 1.21                 | 5                            |
| Keller Contract B  | 5                          | 5.5                | 550                           | Single line 1.5m centers                     | 0.63                 | 35                           |
| Keller Contract A  | 'Infinite'                 | 5                  | 450                           | Infinite 2m square grid                      | 6.6                  | 75                           |

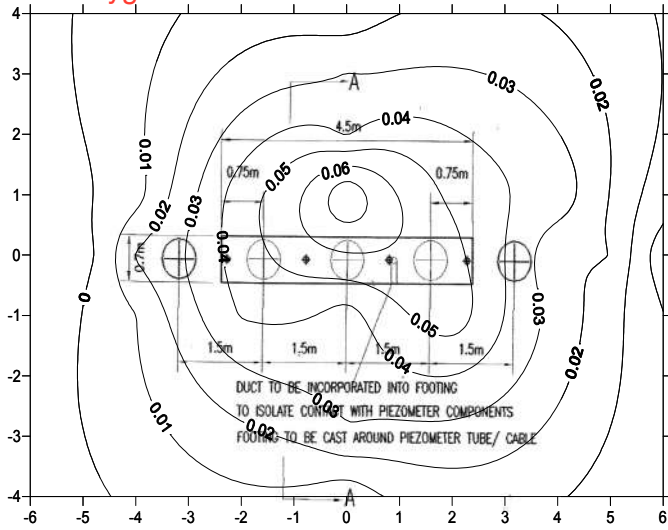


Figure 4. Contours of heave around a line of stone

The data from Table 1 are plotted in Figure 5 and while it is recognised that the database is limited the data shown an interesting trend suggesting some sort of relationship between column layout and amount of heave exists. Further collection of field data is recommended so that this relationship can be substantiated.

#### Heave Summary

It is clear from a number of observations that the process of constructing stone columns causes some lateral and vertical soil displacement, and that the proportion of lateral to vertical strain is a function of the lateral confinement of the host ground and adjacent stone columns. Single columns, rows and small groups of columns exhibit less vertical heave than large grids of columns. It also appears that that the closer the column spacing for any given column arrangement the greater the magnitude of heave. In considering these data, variations in ground conditions and installation practice between different sites is not considered and at this stage and the trends shown in Figure 5 should be considered indicative rather than definitive.

#### TOTAL STRESS AND CAVITY EXPANSION

The penetration of the vibro poker into the ground and subsequent charging and compaction of the stone aggregate may be modelled mathematically by the theory of cylindrical cavity expansion, such as that developed by Gibson and Anderson (1961). As the process of stone column installation is relatively quick in saturated soft clays, undrained cavity expansion is most applicable. Cavity expansion theory comes in various forms, taking account of (i) the required rigour in modelling soil behaviour (elastic, elastic-plastic or critical state solutions),

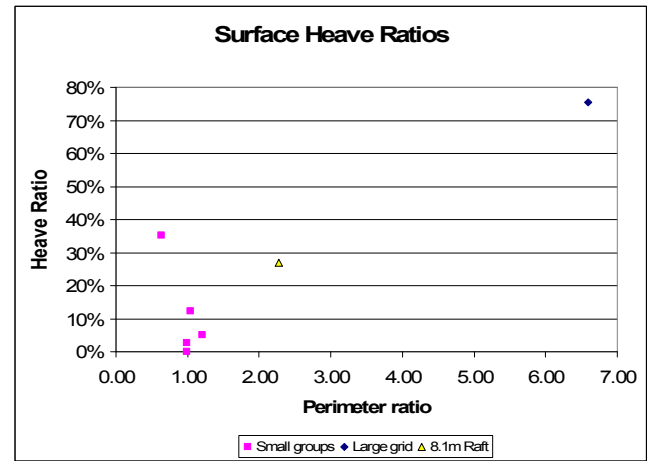


Figure 5 Ground surface heave ratios for stone column installation.

- (ii) whether the medium is finite or infinite, and
- (iii) the effect of various initial radii on the cavity stresses.

Using Gibson and Anderson's (1961) elastic-plastic theory, the zone of soil closest to the cavity is in a plastic state and the soil beyond remains in an elastic state. The radial extent of the plastic zone,  $R$ , may be calculated as:

$$R = ae^{\frac{p - \sigma_r}{2c_u}} \quad (3)$$

where  $p$  is the cavity pressure,  $\sigma_r$  is the radial total stress at radius  $r$ ,  $c_u$  is the undrained shear strength and  $a$  is the radius of poker. Cavity expansion is effectively infinite in the case of a penetrating poker (starting from zero initial radius and expanding to radius  $a$ ), in which case it is reasonable to assume that the pressure in the cavity will have reached the limit pressure ( $p_{lim}$ ), calculated according to:

$$p_{lim} = \sigma_{h0} + c_u \left( \frac{E}{2c_u(1 + \nu)} \right) \quad (4)$$

where  $\sigma_{h0}$ ,  $E$  and  $\nu$  are the free field horizontal total stress, Young's modulus and Poisson's ratio respectively. In this instance, the radial stresses in the plastic ( $r < R$ ) and elastic ( $r > R$ ) zones will be given by equations [5] and [6] respectively:

$$\sigma_r = \sigma_{h0} + c_u \left( \frac{R}{r} \right)^2 \quad (5)$$

$$\sigma_r = \sigma_{h0} + c_u - 2c_u \ln \left( \frac{r}{R} \right) \quad (6)$$



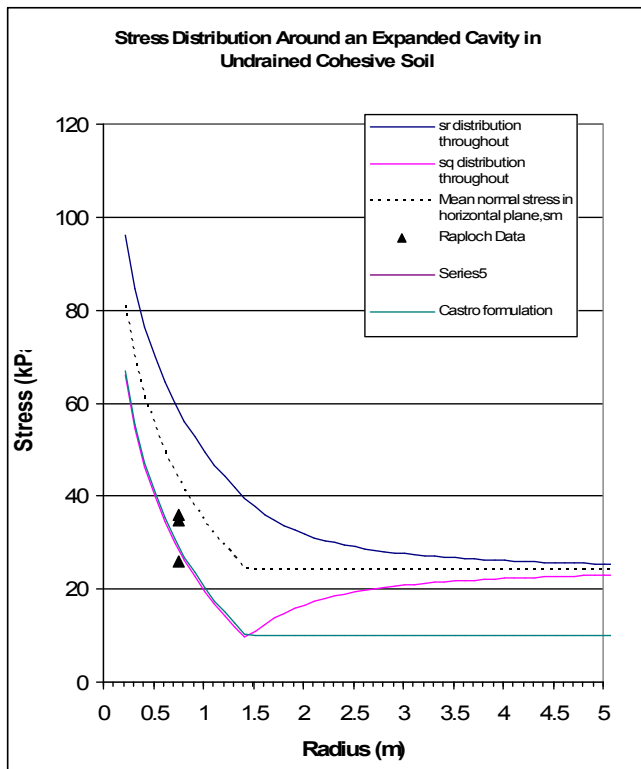


Figure 6. Stress field around a stone in soft Carse clay

The above equations were used to calculate the stress field at 2m depth for Contract B. At this site the clay was very soft with an undrained shear strength,  $c_u$ , of around 10kPa at this depth. The stress field is shown in Figure 6, from which it can be observed that the radius of the plastic zone is 1.4m, beyond which the mean horizontal stress is unchanged from the initial (pre-column installation) values.

## PORE PRESSURES AND CAVITY EXPANSION

### Observations During Column Construction

Kirsch (2003), Gab *et al* (2007) and Castro (2007) measured the pore pressure response to the installation process in three separate studies. Figure 7 reproduces the data shown by Castro (2007) as an example of the general trends. In all three cases the pore pressure rose extremely rapidly as the poker first penetrated the ground generally reaching a peak when the probe achieved the same depths as the piezometer. This is in keeping with general trends observed for displacement piles (i.e. McCabe *et al* 2008). Sharp and significant fluctuations in readings characterise the curves during column construction and are attributed to a range of causes including contact pressure fluctuations as the poker is raised and lowered into the ground (low fre-

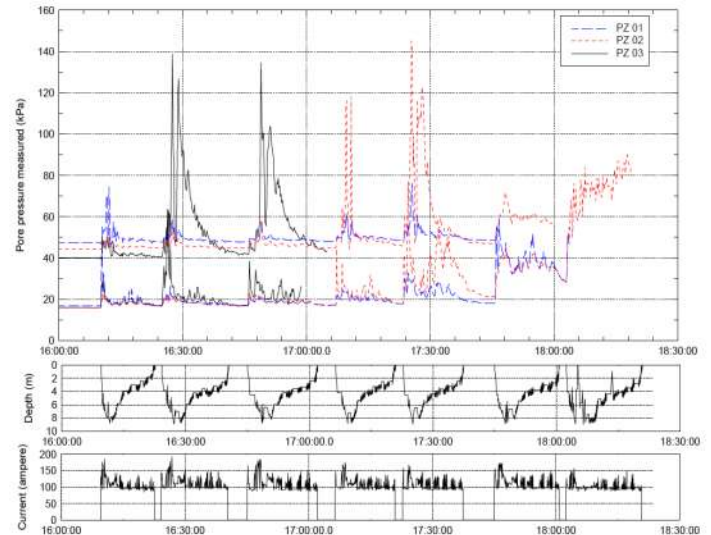


Figure 7. Pore water pressures measured during stone column construction (Castro (2007))

quency), cyclic vibrations from the poker (high frequency  $\sim 50$ Hz), possibly effects from escape of air flush into the ground and general construction vibration.

In all cases the pore pressures reached a quasi static level, albeit at an elevated level, immediately after completion of each stone column. Subsequent installation of adjacent columns leads to a further increase in the quasi static levels.

Dissipation of the excess pore pressure is observed, occurring at different rates, governed by the permeability characteristics of the host soil. Kirsch (2003) and Castro (2007) do not provide dissipation data beyond the end of construction of the column groups and it should be noted that the columns appear to have been constructed in a high silt content soil so may not be directly comparable to stone columns constructed in soft clay. Castro (2007) presents a finite difference analysis to estimate the rate of drainage, however the measured

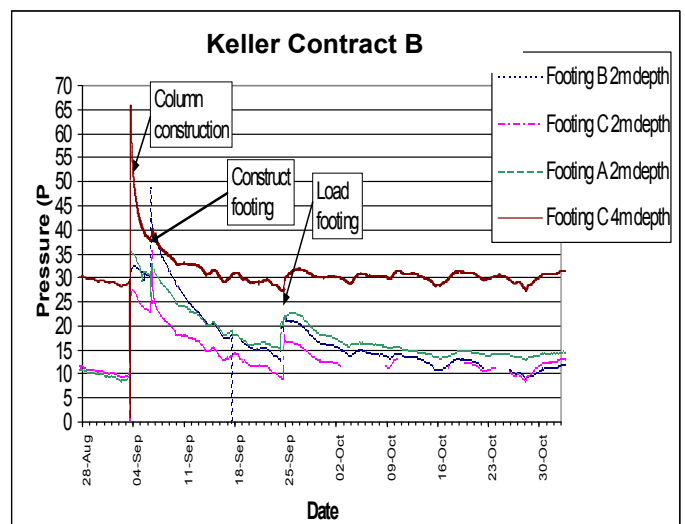


Figure 8 Piezometer response to column loading Keller Carse clay site

dissipation was approximately 10 times faster than predicted by the theory. This was attributed to the presence of ground fractures caused by the penetration of the air flush during column installation creating preferential drainage paths, but the actual reasons were not conclusively proved.

### *Following Column Construction*

Figure 8 shows the dissipation of measured pore pressures from Keller Ground Engineering Contract B. The piezometers were BAT-System instruments (supplied by BAT Geosystems AB) and comprise a porous filter tip which is inserted into the ground independently of the measuring sensor which can be inserted and removed from the filter tip many times and without causing any ground disturbance. To prevent possible damage to the measuring sensors, they were removed from the filter tips during stone column construction, being replaced immediately construction was completed. The pore pressures measured therefore are those prevailing just after completion of the stone columns. Figure 8 also shows the response of the pore water pressures over the following two months. It should be noted that a concrete footing was constructed and then loaded as shown in Figure 8, before full dissipation of the installation excess pore water pressure had occurred. Oedometer tests carried out on samples of the Carse clay indicated that  $0.6 < C_v < 1.7 \text{ m}^2/\text{yr}$  and from insitu measurements  $1.3 < C_h < 7.0 \text{ m}^2/\text{yr}$ . Watts et al (2001) report for the installation of 25 columns for a 8.1m square raft at the Bothkennar test site, a maximum 100kPa rise in pore water pressure during column construction, from a piezometer 5.0m deep installed midway between 2 columns 1.5m apart. The elevated pressure appears to have dropped back to around 30kPa within 24 hours and had returned to pre-treatment levels after about 2 months. However some elevated pressure apparently remained between two columns at a depth of 5m after four months, although it is not clear why this was the case.

### *Relevance of Observations to Cavity Expansion Theory*

Castro (2007) applies the Randolph et al (1979) formulation of cavity expansion theory applied to the installation of stone columns, suggesting that the increase in pore pressure at a radius  $r$ , is related to the undrained shear strength  $c_u$ , and radius of the plastic zone ( $R$ ) by:

$$\Delta u = 2c_u \ln\left(\frac{R}{r}\right)$$

(7)

A very good correlation with measured peak increases in pore water pressure at varying distance during the installation of a single stone column is shown. Installation of subsequent columns did not follow the theory at all well. It is probable that the changes in the soil stress field and pore water pressure distribution generated by installation of the first column violate the assumptions governing the cavity expansion theory formulation applied for installation of subsequent columns.

The curve from equation 7 is plotted in Figure 7 arising from the expansion of a 550mm diameter cavity 5.5m long plotted with measured pore water pressures for Keller Ground Engineering Contract B. A reasonable correlation is observed at a depth of 2.0m midway between two columns.

### *Implications of Pore Pressure Changes*

The increase and then decrease of pore pressures arising from column installation indicate that the ground is squeezed laterally as the columns are installed, causing an immediate undrained increase in pore water pressure which then dissipates as the clay consolidates and gains strength. From the published evidence the consolidation process appears to take within a period of a few weeks following column installation, during which time the ground between the columns increases in strength. This process will usually be complete before construction of the supported structure starts, or at least have concluded during the early stages of construction – the evidence being successful completion over many years of a large number of vibro replacement projects in soft clays.

## IMPLICATIONS FROM OBSERVED BEHAVIOUR FOR NUMERICAL MODELLING

While the installation of bored piles can be simulated in Finite Element packages such as PLAXIS, there is currently no satisfactory procedure available to model the large lateral displacements associated with pile driving or stone column installation.

Some simple attempts to capture expected changes in ground properties have been proposed, although there is great uncertainty regarding how the ground responds as high quality field data that is sufficiently extensive is limited. Some approaches that have been taken include:

- (i) increasing  $K_0$  in the vicinity of the columns to reflect the permanent increase in total stress

- (ii) Guetif *et al* (2007) model installation in PLAXIS 2-D by defining a cylinder of 'dummy material' with purely elastic properties and a nominal Young's Modulus ( $E=20\text{kPa}$ ). The dummy material is then expanded from an initial diameter of 0.5m to a final diameter of 1.1m, before the properties of the 'dummy material' are converted into those of the column material. However, this approach appears to be unsuitable for PLAXIS 3-D as its basis is in small strain theory and numerical instability/inaccuracy may arise from large element distortions.
- (iii) Prompted by experimental results, Kirsch (2003) starts with a cylindrical block having the properties of stone, before applying a radial expansion. He also suggests that the stress increase may be modelled by applying a temperature gradient to the stone column having a coefficient of thermal expansion, or the application of a strain or stress field to the finite element mesh. However the cavity expansion approach was deemed to be superior from a numerical stability point of view.

The key issue in modelling cavity expansion in a FE framework is the amount of radial strain that must be applied in the model to replicate field conditions, and importantly, whether this amount of expansion violates the assumptions in the FE theory used. There is considerable amount of field work and model development that needs to be carried out in tandem before installation effects can be included in routine design.

## CONCLUSIONS

The observed effects of stone column installation indicate that they improve the strength and stiffness of the host ground by a combination of introducing granular elements of greater strength and stiffness than the host ground. Additional benefit is derived as a function of increasing the horizontal stresses and subsequent consolidation and increase in the strength of the host ground. Simple closed form equations based on cavity expansion are useful for modeling the increases in horizontal stress and pore water pressure rise due to the installation of a single column. Where groups of columns are installed interaction of the stress fields from each column occurs in a way that can not, at present, be described by simple closed form equations. While there has been a fairly widespread use of numerical modelling for both research and project purposes, few studies have addressed the installation effects. Within the few numerical studies that have addressed installation effects different modeling ap-

proaches have been adopted and at the present time there does not appear to be a universally accepted, rigorous, means of accounting for the installation effects. What is clear is that installation effects can influence the performance of vibro replacement stone column projects, and as is the case with most geotechnical processes, construction control by experienced practitioners remains the key to successful implementation.

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