CHALLENGES FACED IN 3-D FINITE ELEMENT MODELLING OF STONE COLUMN CONSTRUCTION

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Abstract
In all areas of geotechnical engineering involving an 'installation' and corresponding ground displacement, the changes to the stress regime taking place tend to be poorly understood. The installation of Vibro Replacement stone columns in soft clays and silts is no different, and there is a shortage of high quality experimental data and proven mathematical approaches to inform the stone column designer of the implications of the installation process. In this paper, some conclusions are drawn regarding current analytical and numerical capability to model stone column installation.

Keywords: analytical modelling, cavity expansion, installation, numerical modelling, Vibro Replacement stone columns

1. Introduction

Vibro Replacement is a technique used to improve the bearing capacity and settlement performance of a wide variety of soil types. In recent years, its application to marginal ground conditions in Ireland, such as soft clays and silts and loose fills, has grown. McCabe *et al.* (2007) and McNeill (2007) provide an explanation of the Vibro Replacement technique, illustrate suitable soil types and design concepts, as well as providing some Irish case histories.

The stress changes that construction of groups of columns impose on the surrounding ground cannot be readily quantified, and there is a dearth of high quality field measurements that might help to validate any mathematical prediction approach used. The importance of construction effects to ground improvement design has been acknowledged with the advent of a new Marie Curie Research Training Network called GEO-INSTALL from April 2009, following on from the current AMGISS (Advanced Modelling of Ground Improvement in Soft Soils) network. The AMGISS project has concentrated primarily on developing constitutive models to reflect common soft soil characteristics such as anisotropy, destructuration and bonding, whereas the new GEO-INSTALL network will focus on the numerical issues associated with large strain modelling of displacement piles and stone column installation.

In this paper, some lessons learned from undrained cavity expansion theory are presented. The ability of 3-D finite element analysis to model the column construction process is assessed based upon a comparison with cavity expansion theory. Possible methods of building in an installation effect before the loading phases of numerical analyses are suggested.

2. Installation Process

Well-constructed stone columns may be formed in soft silts and clays by inserting a bottom feed depth vibrator poker into the ground (typical range of diameters 430-800mm). The poker penetrates under the action of its own vibrations, assisted with the pull-down facility of the rig if necessary. The vibrations are predominantly horizontal, and tend to densify materials having in excess of 90% granular (sand and/or gravel) content. However clays, silts and mixed ground conditions are not immediately densified by the vibrations; the improvement comes from filling the hole formed by the poker with inert crushed stone or gravel and compacting in stages from the base of the hole upwards. The performance of the stone column depends largely on the lateral resistance of the soil around it. The poker acts as an investigating tool in this respect and will match the quantity of stone supplied to the consistency of the soil layer encountered. When using a standard 430mm diameter poker, the final stone column diameter is typically 500-600mm.

There are two different approaches that may be adopted to construct stone columns. The stone may be tipped in controlled amounts from the ground surface and compacted in layers through repeated penetration and withdrawal of the poker; this system is referred to as 'Top Feed'. Alternatively, the stone may be fed from a hopper through a delivery tube along the side of the poker. This 'Bottom Feed' method is preferred when hole collapse is likely, such as when there is a high water table in granular soils and in weak clay soils. A schematic of the Bottom Feed process is shown in Figure 1. More detailed information is given by Sondermann and Wehr (2004) and others.

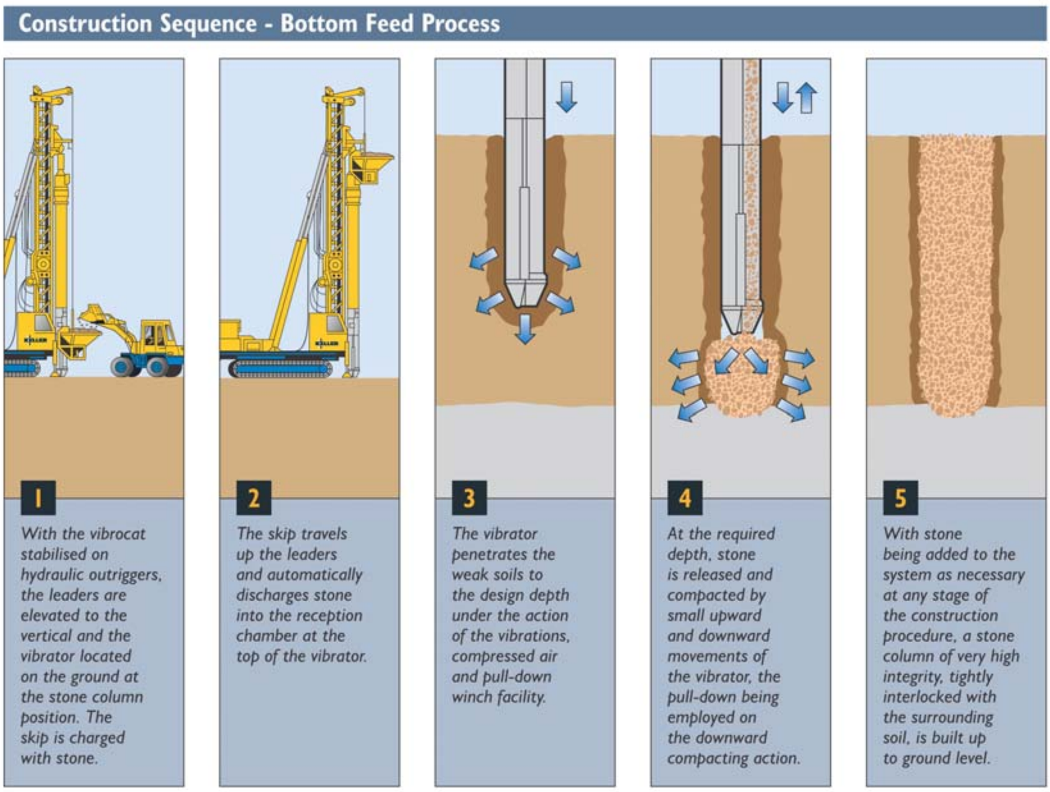

Figure 1 – Bottom Feed Vibro Replacement (courtesy of Keller Ground Engineering)

3. Lessons from Cavity Expansion Theory

Yu (2000) illustrates that, with the exception of a zone at the top of a pile affected by the ground surface, the soil displacement due to a single pile installation may be modelled by analytical cavity expansion theories. Much of the soil is displaced predominantly in the radial direction, so cylindrical cavity expansion (CCE) theory is most appropriate. Measurements of the radial movement of the soil near a pile mid-depth taken from the field tests of Cooke and Price (1978) and the model tests of Randolph *et al.* (1979) show good agreement with theoretical solutions. Spherical cavity expansion applies locally where ground displacement is affected by the pile tip.

The effect on the ground of constructing a stone column in fine soil may be estimated from the undrained CCE formulation of Gibson and Anderson (1961). Since the poker expands a hole from an initial diameter of zero to the poker diameter (430mm) and subsequently to the diameter of the compacted column, the expansion is effectively infinite. On this basis, Egan *et al.* (2008) suggest that modelling stone columns using CCE requires the additional assumption that the equivalent cavity expansion pressure reaches its limit value (p_{lim}) during installation, quantified in Gibson and Anderson (1961) by:

$$p_{\lim} = p_0 + c_u\left(1 + \log_e\left[\frac{E}{2c_u(1+\upsilon)}\right]\right) \qquad (1)$$

where p_0 is the initial lateral total stress, c_u is the undrained shear strength, E is Young's modulus and ν is Poisson's ratio.

From a finite element (FE) modelling perspective, it is of interest to examine the development of cavity pressure as a function of lateral expansion. The soil parameters used for the prediction are shown in Table 1 and pertain to the Carse clay at Bothkennar, Scotland (Nash *et al.*, 1992). CCE predictions are plotted in Figure 2; the cavity pressure p is normalised by p_{lim} and the current cavity radius a is normalised by the starting radius a_0. The curve shown in Figure 2 corresponds to mid-depth along a 5m long column; the curve in normalized form is relatively insensitive to the depth (small changes to p_0/p_{lim}, i.e. at $a=a_0$) and to the initial value of a_0. It can be seen that considerable expansion is required to approach the limit pressure; ≈$0.15a_o$ expansion is required to reach $0.8p_{lim}$ and ≈$0.4a_o$ expansion is required to reach $0.9p_{lim}$. On this basis, it would appear that FE models would need to incorporate considerable lateral expansion to capture undrained installation adequately.

Table 1 – Bothkennar parameters used in CCE model

Undrained shear strength, c_u	14 + 2.3z (z=depth in m)
Young's modulus, E	3050 + 1447z (z=depth in m)
Earth pressure coefficient, K_o	0.5

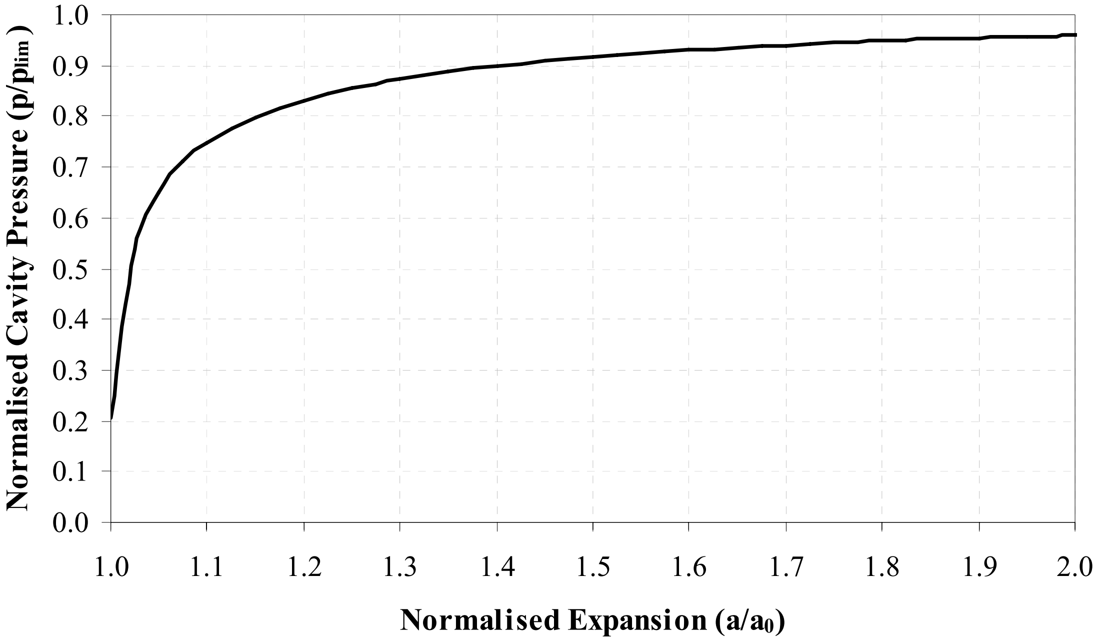

Figure 2 – Variation of cavity pressure with radius (undrained CCE)

The lateral variations of radial horizontal stress (based on Gibson and Anderson, 1961) and pore water pressure (Randolph *et al.*, 1979) predicted by CCE when $p=p_{lim}$ are plotted in Figure 3 and pertain to column mid-depth. The Randolph *et al.* (1979) expression for excess pore water pressure in the plastic zone is given by:

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

where R is the radial extent of the plastic zone. In Figure 3, r is the radial distance from the centre of the column. There is some field evidence that maximum excess pore pressures measured in the ground around a stone column during installation conform to eqn. (2) (i.e. Castro 2007, Egan *et al.* 2008). This, in addition to similar evidence for driven piles, gives some credence to the use of CCE predictions to appraise the performance of FE analysis.

4. Finite Element Analysis

Few researchers have attempted to capture the effects of column construction in numerical modelling by a CCE type approach. Guetif *et al* (2007) model installation in PLAXIS 2-D by defining a cylinder of 'dummy material' about the axis of symmetry with purely elastic properties and a nominal Young's Modulus (E=20kPa). 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 to those of stone.

The use of PLAXIS 3-D Foundation is preferable when modelling pad and strip foundations, where the 3-D nature of a problem needs to be captured. However, it is known that PLAXIS 3-D has greater limitations than PLAXIS 2-D in terms of modelling larger strains. With this in mind, a simple PLAXIS 3-D model of a 5m long 600mm diameter stone column was developed, supporting a 1m × 1m square footing (modelled as a floor in PLAXIS). Various degrees of lateral expansion were applied to the column (a/a_0 = 1.03, 1.06, 1.1, 1.33, 1.67) and the radial total stress and pore pressure variation with radius was noted. The stone was modelled as a Mohr Coulomb (MC) material while the more advanced Hardening Soil (HS) model was used to represent the Bothkennar soft clay. In the latter case, the parameters used are as consistent as possible with those deployed in the CCE model. All parameters are listed in Table 2 and some of the Bothkennar parameters are taken from Kamrat-Pietraszewska *et al* (2008). Undrained installation (without consolidation) was considered for the purposes of this paper.

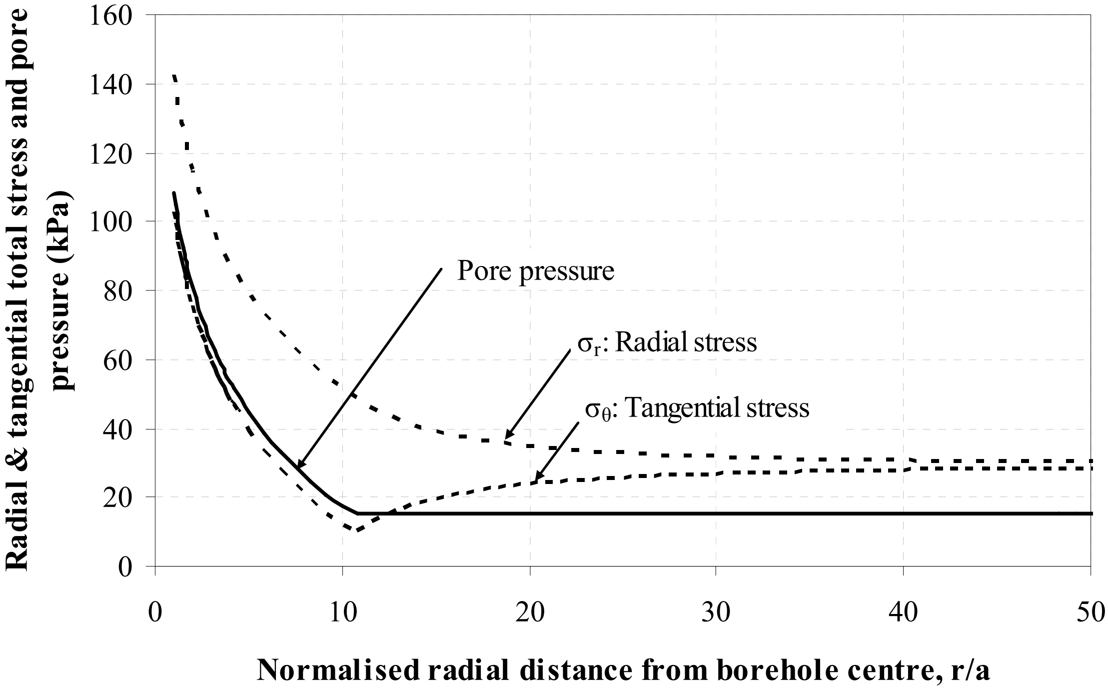

Figure 3 – Undrained CCE predictions for radial total stresses and pore pressures

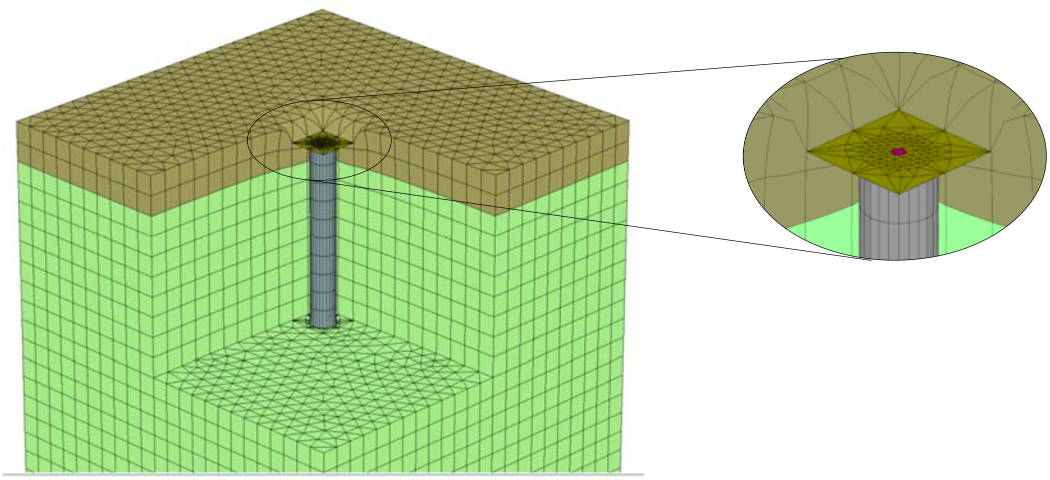

Figure 4 – Mesh used for PLAXIS 3-D finite element analysis

Table 2 – Parameters used to model Bothkennar soil profile in PLAXIS 3-D

HS Parameters	Crust	Carse Clay
Depth (m)	0-1	1-20
γ (kN/m^3)	19.0	16.5
E	1.37	2.00
POP (kPa)	30	-
OCR	-	1.5
K_0	0.7	0.5
K	0.02	0.02
ν'	0.2	0.2
Λ	0.3	0.3
M	1.51	1.51

MC Parameters	Stone
Depth (m)	0-5
γ (kN/m^3)	19.0
E	72000
ν	0.2
c' (kPa)	0.1
φ (°)	45
ψ (°)	15

Only the data for a/a_0 = 1.03, 1.10 and 1.33 lateral expansions are presented in Figure 5 for brevity, where they are compared with cavity expansion values. The theoretical curve for Figure 5(a) is based upon Randolph *et al* (1979). The variation of the radial stress with radial distance from the centre of the stone column is adapted from Gibson & Anderson (1961) in Figure 5(b). There are two theoretical curves shown in Figure 5(b), one representing the radial stress assuming $p=p_{lim}$ (i.e. what field conditions would suggest occurs) and the other representing the radial stress due to the specified lateral expansion. It is observed that as the lateral expansion increases, the PLAXIS data tends towards the p_{lim} curve as expected. The general trend of how the radial total stress increase varies with radius r is captured quite well.

However, the PLAXIS 3-D radial total stress and excess pore pressure predictions are irregular and do not vary smoothly, especially for $a<r<2a$. In fact at certain r/a values, PLAXIS 3-D returns several different values of pore pressure and radial stress, indicating major problems in handling the lateral expansion. It was found that these anomalies still arose regardless of the mesh density chosen; further work is needed to decipher whether these problems are due to a violation of small strain theory, the magnitude of the strains imposed in relation to the size of the elements or other effects. It is of concern that anomalies arise even for the smallest expansion (a/a_0 = 1.03) and worsen as the extent of lateral expansion increases. Clearly the scatter at the level of expansion needed to reach p_{lim} would be unacceptable. In addition, the radial extent of the anomalies is such that there is no prospect of modelling installation effects in groups of columns that are at typical field spacings (i.e. 1.2-2.3m). This work demonstrates that even with a relatively sophisticated soil model, PLAXIS 3-D is currently incapable of capturing the installation effects properly. Although measurements were not in purely fine materials, Elshazly *et al.* (2007) and Kirsch (2008) show that successful modelling of load tests on column-reinforced ground requires the inclusion of a post-installation improvement to the lateral earth pressure coefficient, K.

This paper has concentrated solely on undrained expansion; as the post-consolidation stresses will not be correctly modelled if PLAXIS 3-D returns poor undrained cavity expansion predictions. Further work could be focussed on PLAXIS 2-D with large undrained expansions followed by a consolidation phase to estimate the permanent changes to K values in the ground. Until a more suitable approach is developed, an approximate approach would be to initiate the model with an increased K (higher than K_0 up to a maximum value of perhaps K_p =1 in the undrained case) in the vicinity of the columns.

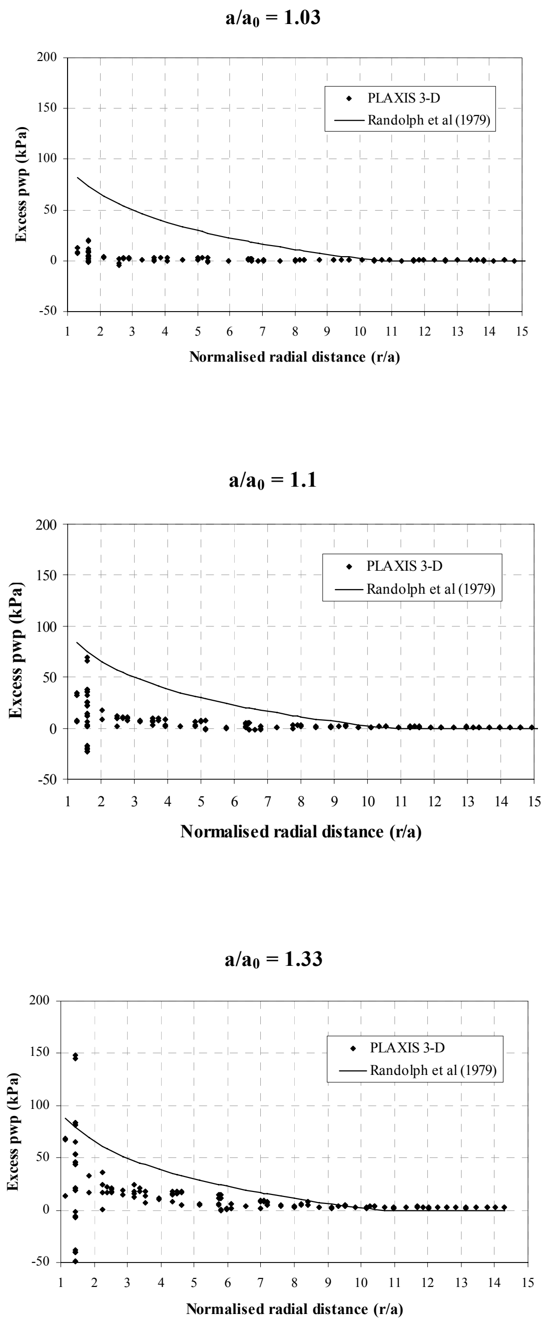

Figure 5(a) - Excess pore pressure (pwp) versus normalised radial distance for various lateral expansions

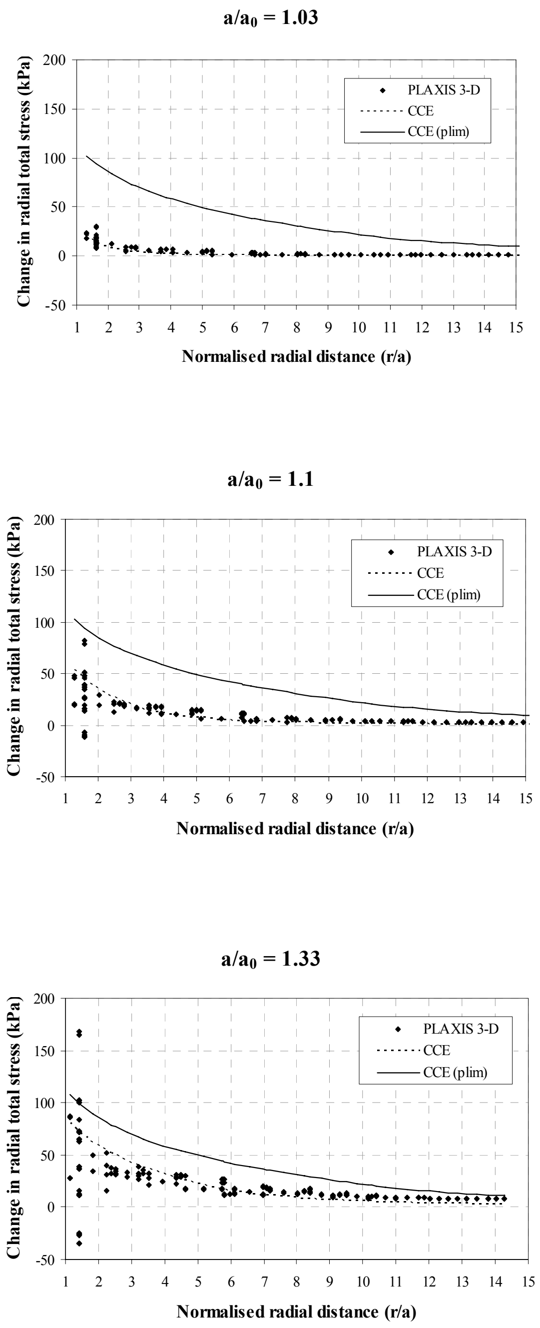

Figure 5(b) - Total radial stress versus normalised distance for various lateral expansions

5. Conclusion

This paper has illustrated that large cavity expansions need to be included in a numerical analysis if the process of stone column installation is to be captured at a fundamental level. It appears that the small strain or other limitations inherent in PLAXIS 3-D FE formulation restricts its use for modelling the installation process. Further development work needs to be carried out to provide PLAXIS 3-D with the capability of modelling larger strains. It should also be borne in mind that PLAXIS 3-D is a quasi 3-D package in that meshing in the vertical direction is basic as shown in Figure 4. In the interim, 'work-arounds' are possible that will give more realistic predictions of column behaviour, one of which is to increase the post-installation K above K_0 to a maximum of K_p=1.

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