Vibro stone columns in soft clay soil: a trial to study the influence of column installation on foundation performance

Colonnes ballastées en sol argileux mou: essai pour l'étude de l'influence de l'installation de colonnes sur le comportement de fondations

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ABSTRACT: In order to assess the performance of a group of stone columns installed by the dry bottom-feed method, as support for a raft foundation, an instrumented trial has been undertaken at the Bothkennar soft clay test bed site. Close examination of the ground response to treatment has shown significant disturbance, including approximately 0.4 m heave at the surface of the treated ground, high pore water pressure response and a prolonged period of dissipation. Ground recovery was continuing one year after treatment. The implications for similar installations are discussed.

RÉSUMÉ: Afin d'évaluer la performance d'un groupe de colonnes installées selon la méthode d'approvisionnement à sec par le bas et supportant une fondation sur radier, un test instrumenté a été entrepris au site d'expérimentation sur argile molle de Bothkennar. L'observation détaillée de la réponse au traitement de sol a montré une perturbation significative du terrain, dont un gonflement de 0.4 m environ à la surface du sol traité, une forte hausse de la pression interstitielle ainsi qu'une période de dissipation prolongée de celle-ci. Le retour du sol vers son état initial se poursuivait encore un an après le traitement. Les implications pour des installations similaires sont discutées.

1 INTRODUCTION

Vibro stone columns may provide an economic alternative foundation solution to piling for low-rise development, particularly on deep soft soil deposits where end-bearing piles may be prohibitively expensive or impracticable. While design must be based on the use of suitable soil parameters, the successful application of stone column treatment will depend crucially on an awareness of how the ground may be modified during the installation process.

Historically, where treatment has been carried out in sensitive soft clay soils the wet top-feed, or vibro-replacement, system has been used. Environmental concerns, technological advances and economic considerations have resulted in the wet technique being largely superseded by the dry bottom-feed method. Watts & Serridge (2000) describe a previous instrumented trial of dry bottom-feed vibro at the EPSRC soft clay test bed site at Bothkennar, Scotland. The effects of the installation of isolated small numbers of stone columns in this moderately sensitive soil were measured and the relationship between the spacing and depth of treatment and the performance of pad foundations subsequently constructed on the treated ground were investigated. The trial showed that columns can be properly constructed but disturbance of the natural clay crust resulted in larger foundation settlements at low applied loads than at the same level in untreated ground. At higher loading the stone columns significantly reduced the rate and magnitude of settlement compared to a foundation in the untreated crust. Where foundations were built and tested below the natural crust, stone columns provided substantially better performance than measured in untreated soil.

In order to assess the performance of a much larger number of columns installed to support a raft foundation, a further instrumented trial has been undertaken. The objectives of this investigation were to examine, by visual observation and monitoring of installed instrumentation, ground response to the installation by the dry bottom-feed technique of a large concentration of partially penetrating stone columns. The design of the vibro ground treatment is briefly discussed, ground instrumentation systems are described and observations made during and subsequent to the stone column installation are presented. The observed behaviour is compared with design predictions for the treatment and the implications for future use in similar circumstances are discussed.

2 GROUND TREATMENT

2.1 *Ground conditions*

The geotechnical characteristics of the soil deposits at the Bothkennar site have been well documented by, for example, Nash et al (1992) and Hight et al (1992). In summary, a firm to stiff silty clay crust about 1.1 m thick is immediately underlain by a thin band of shells in a soft clay matrix. Below the shelly band is dark grey very clayey silt/very silty clay whose shear strength increases linearly from about 18 kPa just below the desiccated crust to 55 kPa at depth (measured using in-situ vane). Bothkennar clay has a low permeability (around 10^{-9} m/s), moderate sensitivity (typically 5), and a Plasticity Index typically about 40%. The natural crust forms part of the soil profile and has played an important role in this trial. Hand vane measurements indicate the undrained shear strength varies substantially from about 120 kPa at 0.2 m bGL (below Ground Level) to 40 kPa at the base of the crust. Ground water level is normally close to the surface and during the trial period the ground water level varied between GL and 0.3 m bGL.

2.2 *Treatment design*

The vibro stone column treatment was designed to support a raft foundation with the simple geometry of an 8.1 m x 8.1 m square lightly reinforced slab with a thickened edge-beam around its perimeter; the geometry was chosen to allow comparison with a similar raft on untreated ground (Chown & Crilly, 2000). The vibro design anticipated a maximum perimeter loading of about 53 kN/m run, assumed to be carried entirely by the stone columns. Column design was based on the method suggested by Hughes and Withers (1974), taking the internal shearing resistance of the compacted column material, $\varphi' = 42.5°$, undrained shear strength for the surrounding soil, $c_u = 20$ kPa and an initial column diameter of 0.65 m, giving an ultimate column load capacity of 175 kN. Column spacing is commonly

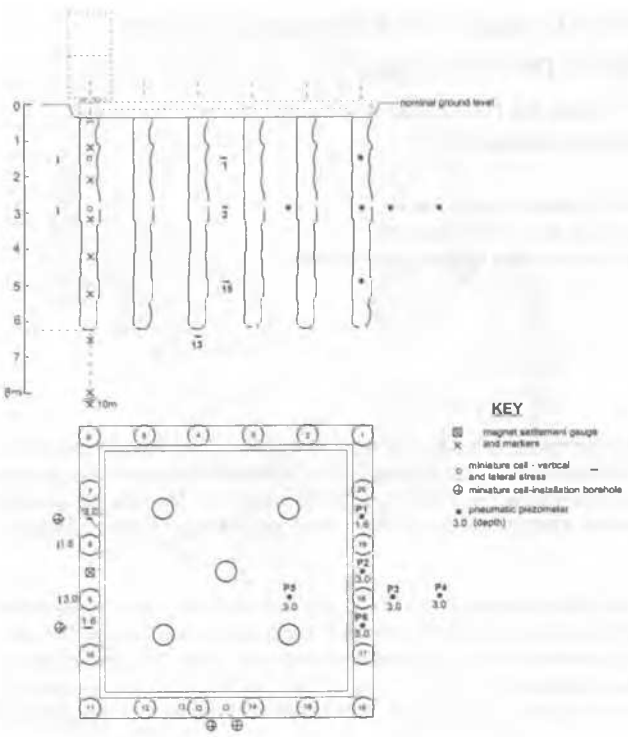

Figure 1. Layout of stone columns and positions of instruments.

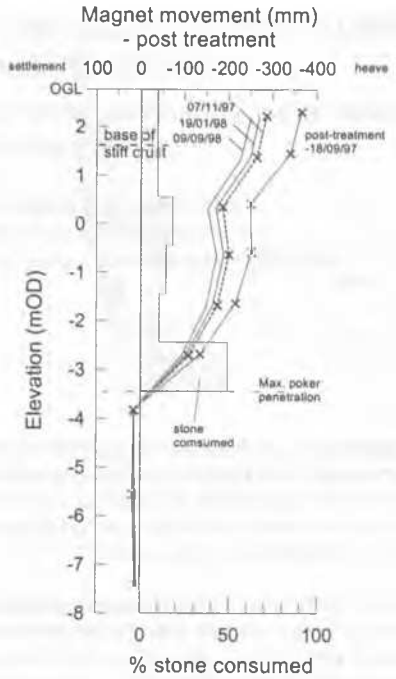

Figure 2. Stone consumption and vertical ground movements during and after column installation.

calculated based on a factor of safety of at least 2.0 against column over-stressing. Foundation geometry dictated columns at raft corners, and intermediate spacing under the edge beam of 1.5 m gave a factor of safety of 2.2 based on the proposed maximum line load. Estimated settlements were based on foundation performance without treatment and the use of appropriate settlement reduction factors according to Priebe (1995). The method suggested by Hughes and Withers (1974) for calculating the minimum column depth required to prevent end bearing failure at the toe of the column occurring before bulging failure at a critical depth near the top of the column, gave a minimum depth of 5.5 m. While a foundation of this size would be expected to increase stresses in the ground to a significantly greater depth than this, practical and economic reasons often dictate a relatively shallow treatment to provide a stiffened upper layer on which low-rise structures are founded. A depth of treatment to 5.7 m below foundation level was adopted as standard for this and the earlier trial. Figure 1 shows the layout of treatment in relation to the proposed foundation and the intended positions of the kentledge loading blocks.

2.3 *Instrumentation*

Instruments were installed prior to ground treatment to assess the effect of column construction. Eight BRE miniature cells (Watts & Charles, 1998) were installed to measure horizontal earth pressures close to the proposed top of selected stone columns and vertical earth pressure between and beneath the toe of some columns. Six pneumatic piezometers were installed close to columns and immediately outside the treated area. A magnet extensometer was installed between column positions. Figure 1 shows the position of all the instruments in relation to the stone columns.

2.4 *Construction*

Stone columns were installed using a Bauer HBM4 bottom-feed rig that incorporates a leader-mounted vibrating poker with a 200 mm diameter pipe to deliver stone directly to the tip of the poker. The poker penetrated the ground by the combined effort of the vibratory action of the poker and pull-down applied from the leader. Air jetting at the tip of the poker was used to reduce friction and collapsing of the bore due to suction as the poker was withdrawn. After penetrating to design depth, stone was fed via a travelling hopper system to the poker tip and compacted in short sections back to ground surface. In addition to the twenty-five columns installed to support the raft foundation, a further two columns were installed using the same method at two locations a short distance from the main treatment. The purpose of these columns was to carry out post-installation investigation by excavation.

3 OBSERVATIONS DURING AND AFTER TREATMENT

3.1 *Column construction*

The construction sequence of each column was carefully monitored and included stone consumption with depth. After penetrating to design depth, a procedure was adopted to construct a stone 'bulb' to form a firm base from which the rest of the column could be constructed to the ground surface. This procedure consumed a high volume of stone in comparison with the rest of the column. Figure 2 shows the average percentage of total stone consumed with depth, illustrating the quantity of stone consumed by the base 'bulb'. Assuming the diameter of the base 'bulb' is not greater than the distance between column centres and that it does not extend higher than the test column excavations, an average base diameter of 1.25 m can be calculated (Watts & Serridge, 2000).

3.2 *Ground movements*

During and subsequent to the ground treatment measurements of vertical ground movement were made using the magnet extensometer. The gauge was monitored from the ground surface using a normal graduated tape probe mounted on a BRE portable ground frame and the position of each magnet marker was related to a stable deep site datum by precise levelling. Several sets of measurements were made to establish the gauge was stable before treatment began and further measurements were made immediately after treatment and periodically over a one-year period prior to foundation construction. Figure 2 shows that large upward movements (heave) occurred in the soil between

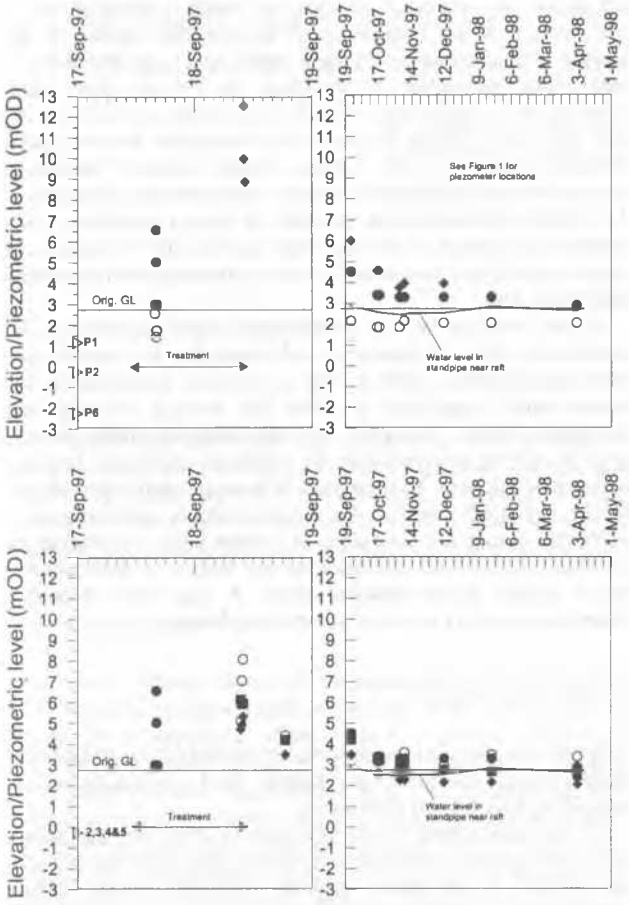

Figure 3a & b. Changes in pore water pressure during and after column installation.

columns as a result of the ground treatment while only a relatively small amount of settlement took place below the maximum treatment depth. The largest upward displacements occurred in the lowest part of the column where a substantial proportion of the stone was placed and also immediately below the crust, with only a small increase in crust thickness indicated. Figure 2 also shows that over 100 mm 'recovery' of the heave occurred over the subsequent one-year period before foundation construction and loading.

3.3 *Pore water pressures*

During the construction of the stone columns high pore pressures were generated within the silty clay deposits. In Figure 3a pressures measured by piezometers at the same depth (3 m) within and outside the treatment area are plotted and Figure 3b shows similar measurements for piezometers between columns at different depths. Piezometer P2 is common to both plots.

4 SOIL RESPONSE TO COLUMN CONSTRUCTION

Soil conditions at Bothkennar might be considered marginal for dry bottom feed vibro-displacement treatment. The sensitivity of the soft clay varies with depth but is reported in the range 5 -10 for field vane measurements. This implies the possibility of significant reductions in soil strength due to disturbance and shearing during column construction. Historically, the wet vibro-replacement method has been used to overcome these difficulties, however the dry top feed process offers considerable practical and economic benefits.

4.1 *Soil shear strength*

Average peak field vane strengths at Bothkennar vary from about 20 kPa at 2m bGL, rising steadily to 60 kPa at 20 m bGL. During the excavation of the test columns five days after their construction measurements of undrained shear strength were made by hand vane within about 0.1 m of the edge of the columns. These suggested little or no loss of strength compared to previously measured peak values at the critical level, that is, the depth at which bulging failure of the stone column occurs. This is normally taken to be about 2 x column diameters, in this case 1.3 m below the proposed raft edge beam or 1.8 m below treatment level. The columns were found to be well formed and were not contaminated with soft clay The average column diameter calculated from stone consumption and measured in-situ is about 0.75 m, significantly larger than the 0.65 m assumed in the initial design. Possible consequences of excessive stone consumption are discussed further below. A narrow annular zone of discoloured clay surrounding the columns was probably material from a darker soil band near the column base. Some dark clay was ejected from the bore during column construction, probably as a result of excessive air jetting pressure. This observation is important and is discussed further in relation to other findings.

4.2 *Soil shear strength*

Watts & Serridge (2000) showed that there was substantial lateral soil displacement due to stone column installation at Bothkennar by the top feed vibro-displacement method. A specially designed electrolevel inclinometer installed 0.5 m from a column installation point was displaced uniformly by 150 mm over a depth of 2.5 m from the ground surface. At this location only two columns were installed in close proximity (1.5 m centres) for an isolated pad test and no significant ground surface movements were observed. Where the concentration of column installation was increased to three columns at 2.0 m centres ground surface heave was more apparent, with an average measured local increase in ground surface level of 107 mm. A greater concentration of four columns at 1.5 m centres resulted in an average rise of 143 mm. Installation of columns for the raft foundation caused considerable disturbance of the ground surface combined with general stone spillage. This area was carefully re-levelled by a mechanical digger and surface surveying showed an average rise in level over the treatment area of 426 mm. These observations suggested that the ground heave developed as the treatment progressed and the number/concentration of columns increased.

Figure 2 shows the top magnet marker, about 0.5 m below ground level between columns 8 and 9, rose by 365 mm. The gauge also shows that more than 50 % of the heave occurred within the bottom 2 m of treatment depth. Watts & Serridge (2000) also showed that about 60 % of the total stone volume was consumed at this level to build column base 'bulbs'. The average percentage stone consumed over the treatment depth is also shown in Figure 2. About 105 m^3 of stone was used in the construction of the raft columns but the average overall surface heave was only equivalent to a volume of about 35 m^3. Only very small compression of the soil below the treatment depth took place and it is evident therefore that lateral ground displacement of about 70 m^3 took place. Assuming this occurred outside the treatment area only, an overall lateral ground displacement of about 330 mm would have taken place around the outside of the treatment area. While there were no actual measurements made of lateral ground movements around the raft treatment area, the earlier measurement of 150 mm close to a single column at this site suggest an overall displacement of 330 mm is not unreasonable. However, measurements of earth pressures and pore water pressures during and subsequent to treatment may also be important factors in the overall effect observed.

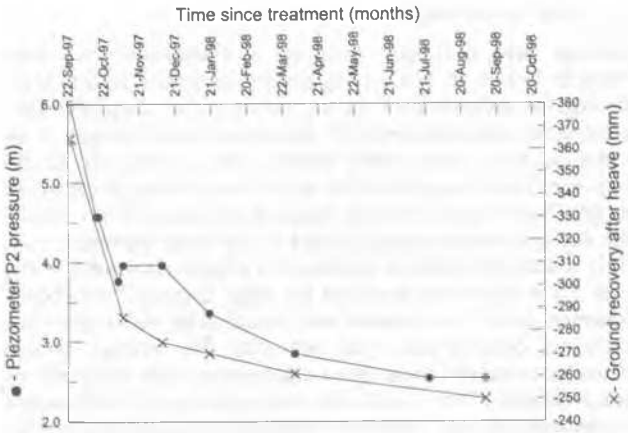

Figure 4. Pore pressure dissipation and recovery of ground heave following ground treatment.

4.3 *Earth pressures and pore water pressure*

Cells installed to measure lateral total earth pressure alongside columns during installation indicated that the pressure increases were greater between columns than outside. This might be anticipated in view of the confining effect between closely spaced columns. Up to about 8 m head of water, or 80 kPa, excess pressure was measured. It is perhaps more surprising that up to 5 m head of water, or 50 kPa changes in vertical pressure were also measured during similar column installations in the same treatment area. Although the major proportion of these increases was not sustained after treatment, some residual pressures remained for several months.

Excess pore water pressures were also generated during the ground treatment. A maximum 10 m head of water, or 100 kPa of excess pressure was measured by piezometer P6 between columns at 5 m bGL, where the base 'bulbs' were formed. In contrast, the piezometer at 1.6 m depth, close to the more permeable shelly layer, indicated that little sustained excess pressure was generated by column construction at this level. Piezometer P2 at 3 m depth between columns indicated lower excess pressure than P3 located at the same depth 0.75 m outside of the treatment area. This may be due to the increased drainage available between columns. P4, 2 m outside the treatment area also registered several metres excess head of pressure and P5 indicated the lowest rise overall. The response of the piezometers and the magnitude of increases in pressure suggest that most of the apparent increases in total stresses were, in fact, changes in pore pressure. The piezometers continued to measure pressure in excess of pre-treatment values for at least two months after treatment, with pressure at 5 m depth between columns remaining elevated for up to four months. After that time small variations can be attributed to seasonal fluctuations in natural ground water levels, indicated by a standpipe piezometer installed about 5 m outside the treated area. In Figure 4 the partial 'recovery' of the ground heave induced by treatment is plotted for a period of one year. This shows the recovery to be in proportion to the original displacement.

During installation it was noted that a small amount of liquefied clay was occasionally ejected from the bore past the poker. There were also instances of air escaping from adjacent stone columns during treatment. This may have been transmitted laterally between columns along the relatively permeable shelly band below the stiff crust. Where a several columns remained flooded during periods of high ground water level, gas bubbling was noted for many months after treatment. While the dry bottom feed process does not require any additional means of maintaining bore stability if the poker remains in the soft soil, jetted air is used to reduce friction between the poker and penetrated soil. It is also essential to eliminate the risk of collapsing the bore through suction as the poker is partially withdrawn to compact charges of stone. However, it is speculated that the magnitude of the pressure applied in the specific circumstances of this trial and its duration of application, particularly in building the column base bulbs, contributed to unnecessary raising of pore water pressures in the soft silty clay. Figure 4 shows the correlation between post-treatment recovery of ground heave before foundation construction and dissipation of pore water pressure measured at 3.0 m depth between stone columns. A large proportion of this settlement occurred in the first four months after treatment but further significant movements were continuing twelve months after treatment.

It has been shown that partial-depth stone columns can be installed in this soft sensitive soil using the dry bottom feed vibro-displacement method, and a previous investigation has demonstrated significant benefits for bearing capacity and settlement control. However, it is clear that air jetting pressure may be critical and should be the minimum necessary to avoid collapse of the bore. The duration of pressure application should be the minimum required for proper column construction, in particular during the formation of a base bulb in partial-depth treatment. In addition, the need for this feature is uncertain and would require further detailed study. A high level of quality control is necessary to avoid potential problems.

5 CONCLUSIONS

This trial has provided the opportunity to study the installation of stone columns using the dry bottom feed vibro-displacement method in difficult soil conditions.

Close examination of the response to treatment has shown significant ground disturbance, including about 400 mm heave at the surface of the treated ground, high pore water pressure response and a prolonged period of dissipation. Ground recovery was continuing one year after treatment.

Constructed columns were larger in diameter than was assumed in the design.

Implications for construction of vibro stone columns in similar circumstances include:

- air jetting pressure should be the minimum necessary to avoid collapse of the bore
- the duration of pressure application may be critical and should be the minimum required for proper column construction.
- a high level of quality control is necessary to avoid potential problems.

The need for the large base bulb at the base of a partial-depth column is uncertain and requires further detailed study.

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