Reinforcing of soft cohesive soils with stone columns

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Summary
Stone columns, which consist of granular material compacted in long cylindrical holes, are used as a technique for improving the strength and consolidation characteristics of soft clays. This paper briefly describes the history of their use and their empirical design. A series of model experiments were run at Cambridge, using radiographic techniques to determine the actual behaviour of a single column in a uniform normally consolidated clay. Simple results of plasticity theory were used to predict the limiting load. The good agreement between experiment and theory enabled the effectiveness of stone columns, used both singly and in groups, to be deduced.

Stone columns significantly reinforce soft ground. Unlike pile foundations they make very efficient use of the soil near the surface. They are ideal for light loads; however, they are less effective at supporting heavy loads because they cannot transfer the applied stresses to the deeper layers of soil.

Introduction
All soil mechanics engineers are familiar with the problems involved in the design of foundations on soft soils and are doubtless aware that the techniques of improving the characteristics of soft soils are unfortunately very limited. They usually involve some combination of the following:

1. Piling, using friction alone in deep deposits or point bearing on a hard stratum underlying the soft clay.
2. Preloading, to reduce settlements and stiffen the soil.
4. Replacing the soft soils, either with stronger material or a buoyant foundation.
5. Stone columns.

The first four methods are well understood and have been widely used for some time. Although stone columns were well known in France in the 1830s they have only recently been rediscovered and the mechanism of their behaviour under load is not well understood.

When a rigid pile in a cohesive soil is loaded, it settles, developing end bearing pressures and cohesive resisting stresses up the side (Fig. 1). A stone column, made of compacted cohesionless granular material similarly develops end bearing pressures and the cohesive resisting stresses. However the stone column also bulges and so it must be supported by lateral stresses exerted by the soil (Fig. 2).

As far as the authors can determine, stone columns were first used in 1830 by French military engineers to support the heavy foundations of the ironworks at the artillery arsenal in Bayonne (Moreau et al 1835). The arsenal was founded on soft estuarine deposits. The columns were two metres long, 0.2 m in diameter and supported loads of 10 kN each. They were constructed by driving stakes into the ground, withdrawing them, then back-filling the holes with crushed limestone. Moreau suggested that these columns reduced the expected settlements by a factor of four.

Stone columns were forgotten until the 1930s when they were rediscovered as a by-product of the technique of vibroflotation for compacting granular soils. Compact granular columns were formed within the granular soil mass process according to Huemer (1939) could more than double the bearing capacity of a site. Even so, it was not until the early 1960s that the vibroflotation technique was used for forming stone columns in cohesive soils (Dullage, 1969). Today the columns can be made up to 15 m long, from 0.5 to 1.5 m dia and will support loads of up to 300 kN.

The modern technique of forming stone columns consists of first forming the hole with an instrument known as a vibroflot. This is essentially a long (10 m) thin (0.5 m dia) steel tube which vibrates at a high frequency of 50 or 60 Hertz. It is suspended from a crane as shown in Fig. 3. Out of the bottom of this tube flows air or water under pressure. The hole is formed in a matter of minutes by the combined action of vibration and jetting. The instrument is withdrawn either fully or partially and the hole backfilled with gravel, about 2 to 3 cm in diameter. The backfilling is done in layers and the gravel is compacted by lowering the vibroflot on to the top of the fill. Water or water and mud mixtures are used as the jetting fluid when the soil is too soft for the sides of the hole to remain unsupported. In this process a considerable portion of the soil is washed away and is not displaced radially whereas in harder soils, where air is used as the jetting fluid, almost all of the soil is displaced radially.

Stone column design has always been semi-empirical. The French found experimentally that arching transferred the vertical loads to the side of the column and then suggested that the ultimate lateral stress the soil could withstand was equal to the bearing capacity of a surface footing. Unfortunately they did not record any experiments which would confirm their suggestion. They seemed more concerned with reducing settlements under their masonry structures.

Modern practice recognises that the granular material in the circular column is confined by a radial stress just as though the column was in a triaxial ap...
paratus (fig 4). The simplifying assumption made that a row of these columns act as a long wall of sand loaded continuously along the top, in which case the maximum lateral resistance of the soil is taken to equal the passive pressure at a depth appropriate to column failure is expected to occur. It is conceivable that a strip footing with closely spaced columns could act in this way. This approximate approach does allow the maximum column load to be estimated (Greenwood, 1958). A significant difference between the old French and modern practice is that nowadays the columns are treated partly as end bearing piles and are therefore founded where possible on a firm layer. However Moreau et al observed in their experiments on sand confined in a rectangular box that, provided the depth/width ratio was more than two, none of the applied load was transferred to the bottom of the box. Therefore they did not attempt to find their columns on a firm layer.

Stone columns are used for three distinct loading situations. Firstly they are used as single columns to support point loads such as small pad footings, in which case the column has equal lateral restraint all round the side; secondly to support strip footings, the column being restrained more in the direction parallel to the strip, and thirdly to support widespread loads. In this case the column has equal lateral restraint all round the side, but as the soil settles under the large load the lateral resistance of the ground increases.

Stone columns also have secondary roles such as:

1. Acting as sand drains and thus speeding up the process of consolidation.
2. Replacing the soft soil by a stronger material such that the toes of embankments can be strengthened.
3. Initial compaction of the soil during the process of installation. This certainly is effective in stiff non-saturated materials, for instance fill material. However it is doubtful whether it is of benefit in soft soils in which the holes are jetted with water.

Clearly, to explain the different roles of stone columns in all the various loading situations is a complex task. However the approach taken in this paper is to simplify the problem by treating the complete system as the sum of two parts; firstly the behaviour of the soil alone, ignoring any influence of the columns, and secondly the behaviour of the columns, ignoring any changes in the properties of the surrounding clay as a result of consolidation. While this method makes no allowance for the stiffening of the clay as it consolidates vertically, it does provide a simple rational solution to the problem. This may be conservative but when tempered with practical experience forms a more satisfactory basis for design than the almost complete empiricism based solely on field experience.

Because of the difficulties in fully instrumenting a field column, without altering its characteristics, and with the uncertainties of obtaining the relevant parameters of the soil, model experiments were run using materials whose properties were well established. From the results of these tests a design procedure has been established.

Experiments

This section is an account of a series of model experiments in the laboratory, the purpose of which was to examine the behaviour of a single stone column made from Leighton Buzzard sand. The clay (kaolin) was first one-dimensionally consolidated, then kept under a constant stress. Loads were applied to the top of the column only (fig 5). Displacements in the clay and sand were measured by taking radiographs of lead shot markers placed inside the column and the clay. Of course for a model test to be fully representative all the gravity stresses in the soil should be reproduced. This is only possible in a centrifuge; however the authors assumed that tests using clay in a uniform anisotropic stress field would yield sufficient information to determine the mechanism of behaviour.

The model columns were 150 mm long and ranged in diameter from 12.5 mm to 38 mm. They were constructed in the clay after it had been consolidated to the desired pressure. For comparison a 38 mm dia footing test was done on the surface of the clay without the stiffening effect of a stone column. The behaviour of this loading test corresponds to a deep footing loading in the field. The apparatus used for these experiments is shown in fig 6. The tests were stress controlled and sufficient time was allowed between successive increments of load for full dissipation of the pore pressures.

Fig 7 shows the variation of the vertical applied stress on the 38 mm column and footing, divided by the undrained shear strength of the clay, with the settlement of the footing expressed as a percentage of the diameter. It is clear that the bearing capacity of the footing on the column is much greater than that of the footing on the clay. Only when the vertical displacement of the top of the column was about 25 mm (58 per cent of the column diameter) was the ultimate applied stress of 22 times the undrained cohesion, c, reached.

Figs 8 and 9 show the pattern of vertical and radial deformation within the column for three different vertical displacements of the top of the column (9.8 per cent, 21 per cent and 58 per cent of the column diameter). In fig 9 the radial displacements are converted into nominal strains by dividing by the initial column radius. It can be seen that the considerable vertical and lateral distortion which occurs at the top of the column rapidly diminishes with depth. At failure about four diameters length of the columns were being significantly strained.

This is also apparent in fig 10 which is a tracing of a superposition of radiographs taken before the 38 mm column was loaded and at a vertical displacement of 53 per cent of the column diameter. The original and final position of a selection of lead shot are connected by arrows. The outer line, which marks the limit of 1 per cent radial strain in the clay, indicates that only the clay within a cylinder with a diameter of about two and a half times that of the column is significantly affected by the loading. This suggests that the columns could act independently if placed more than two and a half diameters apart.

Two other observations reinforce this
idea. Firstly, at the end of the test on the 38 mm column the moisture contents were found for the six positions A to F in fig 10. The results are:

A 51 per cent  D 57 per cent
B 55 per cent  E 57 per cent
C 59 per cent  F 58 per cent

These show that considerable consolidation has occurred near the column but at a distance 1.5D from the centre the effect is not noticeable. Secondly, the readings of the load cell in the wall adjacent to the bulge show that over the entire test the lateral stress has increased by 4 kN/m², i.e. an increase of 7 per cent.

The model columns both increased the rate and reduced the size of settlements.

This can be seen in fig 11 which shows the variation of the applied stress and settlements with time of the footing and the column.

For example, a bearing stress of 160 kN/m² (the ultimate stress on the sand column divided by a factor of safety of three) intersects the curves shown in fig 11 at G and H. It can be seen that the settlement of the column is virtually complete after 380 minutes whereas the footing takes 1480 to reach equilibrium under this stress. Therefore the column increases the rate of settlement by a factor of four. If points H and G are projected down to the lower part of fig 11, it can be seen that settlements are reduced by about a factor of six.

The results clearly show that the ultimate strength of an isolated column loaded by its top only is governed primarily by the maximum lateral reaction of the soil round the bulging zone and that the extent of vertical movement within the column is limited. For the particular clay and sand used in the experiment the vertical movement does not go below four column diameters. It would appear for this particular case that if the length diameter ratio is less than four then the columns would fail in end bearing before bulging.

If the bulge is idealised as a cylindrical expansion into the clay then it resembles a pressuremeter test in which a cylinder is expanded against the side of a borehole. The results of many pressuremeter tests indicate that as the column expands the radial resistance of the soil reaches a limiting value at which indefinite expansion occurs. If the soil is treated as an
elasto plastic material then the limiting stress is given by Gibson and Anderson (1961) as
\[
\sigma_{rL} = \sigma_{r0} + c[1 + \log_6 \frac{E}{2c (1 + \mu)}]
\] (1)
where \( \sigma_{r0} \), \( E \), \( \mu \) and \( c \) are, respectively, the total in situ lateral stress, the elastic modulus, Poisson’s ratio and the undrained cohesion. In other words the stone column can be thought of as being confined in a triaxial stress system where the cell pressure is limited. Therefore there is an ultimate load the column can carry.

From a detailed examination of many field records of quick expansion pressure-meter tests it appears that equation (1) can be approximated by
\[
\sigma_{rL} = \sigma_{r0} + 4c + u
\] (2)
where \( u \) is the pore pressure.

The published data of drained tests in normally consolidated clays is very limited. However the records of a few full tests by Wroth and Hughes (1973) suggest that equation (2) predicts the limiting pressure reasonably accurately.

If the sand in the bulb region near the top of the column is at a critical state of stress then:
\[
\sigma'_{r} = \frac{(1 + \sin \phi')}{(1 - \sin \phi')} \sigma_v
\] (3)
where \( \phi' \) is the angle of internal friction of the column material.
\( \sigma_v \) the vertical effective stress
\( \sigma_{r} \) the lateral effective stress.

Therefore the ultimate vertical stress which the column can carry as it bulges laterally is given by
\[
\sigma'_{r} = \frac{(1 + \sin \phi')}{(1 - \sin \phi')} (\sigma_{r0} + 4c + u) \] (4)

Substituting measured values of cohesion, in situ lateral stress, the pore pressure and the angle of internal friction of the sand into this equation gives the ultimate stress the model column could support.

i.e. for \( c = 19.1 \) kN/m\(^2\); \( \sigma_{r0} = 54 \) kN/m\(^2\);
\( u = 0; \ \phi' = 35^\circ \)
\( \sigma'_{r} = 482 \) kN/m\(^2\) and \( \gamma = 25.2 \)
\( c \)
This compares very favourably with the observed value of \( \sigma'_{r} \) of 22.

If a load factor of 3 is assumed and \( \sigma_{r0} = 3 \) the stresses predicted by equation (4) can be compared with the field observations of Thorburn and MacVicar 1968 as expressed in their fig. 12.

Their results have been expressed in terms of allowable load on a single stone column is constant in all soils then the stress on these columns can be compared directly with equation (4) as shown in fig 12. Clearly there is reasonable agreement with soft soils but in stiffer soils the results are more conservative. However Thorburn 1974 has observed from measurements of excavated stone columns that in the stiffer soils smaller columns are formed and therefore the allowable stress is considerably higher than would be anticipated from a constant column size. Based on these measurements his allowable stress is remarkably close to the value predicted by equation (4). The other line on fig 12 shows for comparison the allowable strength of a surface footing.

The end bearing mode of failure concerns only the equilibrium of the vertical forces on the column. Clearly, if the vertical load exceeds the shear resisting forces along the side of the column and the ultimate bearing pressure at the base, the column will push through the soil. For simplicity the limiting value of the shear stresses up the side of the column are taken to be equal to the undrained cohesion of the soil.

By equating the forces on a horizontal element of the column (fig 13) and assuming that \( c \) is constant over the depth of the column we obtain
\[
\sigma_{rL} = \sigma_{r} + M (2c - \mu c)
\] (5)

**Fig. 10. Tracing of the superposition of the radiographs taken before and at a vertical displacement of 25 mm. A selection of the lead shot markers are joined by arrows**

**Fig. 11. The upper figure shows the applied stress on the column and the footing against time. The lower figure shows the vertical settlement as a percentage of the footing diameter against time**

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end bearing and bulging failure occur simultaneously as 4.1D. This agrees well with the observed value of about 4D and so there appears to be little justification for refining the assumptions.

Generally, though, the cohesion of the soil is not uniform and so care must be taken in the choice of suitable values of c. For instance bulging failure depends on the cohesion of the soil near the top of the column whereas the end bearing failure depends on the cohesion at and below the base of the column.

In practical foundations the load is applied to the surrounding soil as well as the top of the column. The load on the soil will cause the clay to consolidate and increase the radial stiffness of the soil. However this is unlikely to have a marked increase on the strength of the column. Therefore the limiting load for a column can be calculated using equation (4) and the minimum length from equation (5).

Large groups of stone columns
In this section some important conclusions about the effectiveness of groups of stone columns under widespread loads are drawn. These are based on the observations made of the behaviour of single columns.

The usual problem with widespread foundations is one of reducing settlements to an acceptable level. The bearing capacity becomes a problem only at the edge of the loaded area.

The settlements are produced by several interacting factors which makes an analytical prediction of their magnitude difficult. In the case of a foundation on a thin layer of clay typically less than 6m thick, overlying hard clay or rock, an estimate of the effectiveness of stone columns can be made using the following assumptions. Firstly that the stiffening effects of the bulging column on the clay and of the consolidation of the clay on the column can be ignored. Secondly that the behaviour of a typical column within the group is the same as that of an isolated column. This approach produces an upper bound to an estimate of the settlements.

As an example, consider a large rigid raft on soft soil 4 m deep reinforced with 1 m dia columns, 4 m long, on a 2.5 m square grid (fig 15). The average properties of the clay and the behaviour of each column are taken to be similar to those used in the model experiments. Fig 16 shows the stress settlement characteristics of the columns and of the clay. The settlement of the raft can be thought of as the sum of two components. Firstly that produced by the stress on the raft supported by the columns alone. Secondly that produced by the stress on the raft supported by the clay between the columns alone. For example if the raft stress is 50 kN/m² then the settlement of the reinforced ground D, given by point A in fig 17 as 0.18 m. The stress on the columns a, corresponding to this settlement is 330 kN/m² (point B in fig 16) and that on the clay a, is 10 kN/m² (point C in fig 16). If the soil was untreated then the settlement of the raft D, for a stress of 50 kN/m² would be 0.47 m (point D in fig 16).

The effectiveness of stone columns at reducing settlements is a function of the stress on the raft. This is shown in fig 18 which is a graph of the ratio of the settlements with and without the reinforcing effect of stone columns against the applied stress. Clearly if the stress on the raft is 25 kN/m² then the columns reduce

Fig. 12. Allowable vertical stress against undrained cohesion on stone column calculated by equation (4) compared with the design curve of Thorburn and MacVicar (1968) and some recent data from Thorburn (1974)

Fig. 13. Vertical stresses on a typical element in the stone column

where M is the ratio of depth to column diameter, p, the effective density of the column material, \( \alpha_{vz} \), the vertical stress at a depth given by M.

Equation (5) produces a distribution of vertical stress with depth such that the stress decreases from a maximum of the ultimate value in bulging failure at the top to zero at some depth (fig 14). Clearly any increase in column length beyond this value will not increase the strength of the column. As the column length is reduced some of the applied vertical stress will be taken by the soil at the base and the columns will act partly as end bearing piles. If the columns are made so short that the stress at the base exceeds the bearing capacity of the soil (about 9c) then the end bearing mode of failure will occur before the bulging.

The linear distribution of stress of equation (5) gives the critical length at which
settlements by about a factor of 10. At higher stresses, for example 75 kN/m², the columns are less effective, reducing settlements by a factor of only 1.6.

In the case of widespread foundations on deep layers of clay an allowance must be made for the settlement of the soil beneath the base of the columns. It would seem reasonable to assume that this clay is one dimensionally consolidated by the average raft stress. This consolidation could dominate the total settlement but in any case stone columns are less effective in this situation.

Conclusion
This paper has examined the behaviour of stone columns and shown that it can be satisfactorily explained using simple results of plasticity theory.

Stone columns should preferably be used where a modest increase of bearing capacity is required because they use the strength of the soil very efficiently. However they are not so suitable for heavy loads because they cannot significantly stress the deeper layers.

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References