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A review of field performance of stone columns in soft soils

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The behaviour of stone columns has yet to be captured fully by analytical and numerical techniques, and predicting column behaviour in soft cohesive soils brings specific challenges. This paper provides a comprehensive review and assessment of some aspects of field performance of stone columns in soft clays and silts, from both published and unpublished data and for loading over both wide areas (i.e. embankments) and small areas (i.e. footings). In particular, a new database of settlement improvement factors is developed, which gives an indication of the reliability of a traditional analytical approach, as well as distinguishing clearly between the performance of the preferred dry bottom feed system and other column construction systems. The paper gives evidence highlighting the key construction issues central to the successful performance of treated ground. Some data are also presented on pore pressure and total stress changes as a result of column installation and loading, and interpretation of these data benefits from a direct comparison with driven piles.

NOTATION

A	area of 'unit cell' for a stone column in an infinite grid
A_c	average cross-sectional area of stone column
A/A_c	area replacement ratio
B	pile width
c_h	horizontal coefficient of consolidation
c_u	undrained shear strength
c_v	vertical coefficient of consolidation
D	stone column diameter
D_p	poker diameter
E_c	Young's modulus for column material
E_s	Young's modulus for soil material
E_c/E_s	modular ratio
h	distance above pile base
K	lateral earth pressure coefficient
K_0	lateral earth pressure coefficient at rest
k	constant relating A/A_c to unit cell geometry
n	settlement improvement factor = $s_{untreated}/s_{treated}$
n_{meas}	measured settlement improvement factor
n_{pred}	predicted settlement improvement factor
n_0	Priebe's basic settlement improvement factor
n_1	Priebe's n_0 amended for column compressibility
n_2	Priebe's n_1 amended for soil and stone unit weights
R	radius of stone column or pile
R_{eq}	equivalent radius of a square pile $= B/\sqrt{\pi}$
r	radial distance from column or pile centreline
s	stone column (centre-to-centre) spacing
$s_{treated}$	settlement of ground treated with stone columns
$s_{untreated}$	settlement of ground without stone columns
T_h	time factor for horizontal/radial consolidation
t	time
U	degree of pore pressure dissipation
Δu	excess pore water pressure
Δu_{max}	maximum excess pore water pressure
YSR	yield stress ratio (overconsolidation ratio measured in oedometer tests)
ν_s	Poisson's ratio for soil
ϕ'	angle of friction of stone
σ'_{v0}	free-field vertical effective stress

1. INTRODUCTION

An ability to create solutions tailored to meet specific settlement and/or bearing capacity requirements has led to the emergence of vibro flotation as one of the world's most widely used forms of ground improvement. Its versatility is reflected in the literature by the variety of ground conditions that have been treated successfully, including granular deposits,[1,2] weak natural cohesive deposits,[3–5] hydraulically placed dredged fills,[6] mixed made ground[7,8] and ground prone to liquefaction during seismic events.[9] Vibro flotation is an 'umbrella' term for two distinct processes.

(*a*) Vibro compaction is used in ground with fines ($<$ 0·06 mm) content less than 15%;[1] the poker's (predominantly) horizontal vibrations densify the coarse ($>$0·06 mm) material.
(*b*) Vibro replacement is used in fine soils in which the density is not enhanced by the vibrations themselves. Well-compacted columns, typically of crushed stone or gravel, are constructed in the voids formed in the ground by the poker, improving bearing capacity and drainage while reducing absolute and differential settlements.

Slocombe *et al.*[1], BRE[10] and Sondermann and Wehr[11] elaborate on the vibro flotation equipment and processes.

Mounting pressure to develop soft and very soft clay and silt sites has focused attention on the potential of vibro replacement stone columns to offer both technically and economically viable solutions in these deposits. The use of stone columns in these ground conditions poses very specific challenges. Ground displacement and associated stress changes arising from (a) the formation of an array of cavities generated by the penetration of the poker and (b) the subsequent construction of compacted columns are critical factors that are only beginning to be understood.[5] The effects of plasticity index and sensitivity on stone column performance have not been widely investigated, and to date there is no consensus on the extent to which stone columns arrest secondary settlement. In general, settlement is a more stringent design criterion than bearing capacity in these soils, and the most popular settlement prediction method in European ground improvement practice[12] contains many simplifying assumptions and some empiricism in its formulation.[13]

Considerable advances have been made in developing constitutive models for soft natural soils for use in conjunction with finite-element software to incorporate characteristics such as creep, anisotropy, destructuration and bonding.[14–16] However, such models are not routinely used by industry practitioners in ground improvement design, and, in general, finite-element approaches are limited significantly by their inability to capture installation effects satisfactorily, especially the three-dimensional versions.

Under these circumstances, it was worthwhile to carry out a detailed review of published and some unpublished records of stone column field performance. In this paper, information is collated from around 30 case studies in which stone columns were used to improve soft cohesive soils with undrained shear strengths c_u below 30 kPa in most cases and below 15 kPa in some cases. There is a dearth of published data on certain aspects of stone column performance, so the paper concentrates on those aspects for which a reasonable body of data is available. In particular, a new database of settlement improvement factors (i.e. the settlement of untreated ground divided by the settlement of treated ground) for soft cohesive deposits is presented and interpreted. Findings relating to pore pressures and total stresses are also presented. Overall, the review gives confidence that the vibro replacement method can be successfully applied to many soft cohesive sites, provided adequate care is paid to the construction technique and to workmanship. The quantitative evidence presented in the paper provides a useful reference for practitioners when designing future stone column ground improvement schemes in soft cohesive deposits.

2. STONE COLUMN CONSTRUCTION METHODS

The correct choice of stone column construction method and proper on-site implementation are the keys to successful improvement of soft and very soft soils. The main methods available for vibro replacement are discussed below, where the terms 'top feed' and 'bottom feed' describe the method of stone supply, and 'wet' or 'dry' relates to the jetting medium.

2.1. Construction methods

2.1.1. Dry top feed. Dry top feed is predominantly used for shallow to medium treatment depths of coarse and more competent fine deposits for vibro compaction or vibro replacement stone columns, where the hole formed by the first penetration of the poker remains stable as the column is constructed. This method often uses controlled air flush to aid construction, and is commonly used for lightly loaded to heavily loaded developments, but is rarely suitable for use in soft cohesive soils.

2.1.2. Wet top feed. Wet top feed is used for medium to deep treatment below the water table, and for the treatment of softer cohesive deposits for vibro compaction or vibro replacement stone columns. In cohesive soils, water flush helps remove arisings from the void formed by the vibrating poker and maintain its stability. Wet top feed is used less frequently nowadays, as disposal of the flush arisings is often troublesome, so it is often reserved for larger-scale moderately loaded to highly loaded developments.

The wet top feed system has been used with success in soft cohesive soils: for example, Mitchell and Huber[17] and Munfakh *et al.*[18] describe successful treatment for a wastewater facility and a trial embankment respectively, in which only limited intrusion of fines from the treated ground was noted within the columns, mainly around the periphery. However, McKenna *et al.*[19] suggest that the softening of the void lining by water flush, combined with the use of a coarse backfill stone (38 mm single-size crushed limestone), may have been responsible for the failure of a central portion of a 7·9 m high test embankment on soft clay treated with stone columns. Water flush may have resulted in a soft clay slurry encroaching on the stone columns and reducing the effectiveness of the drainage properties. It should be noted that Greenwood[20] puts forward alternative explanations for the collapse: that is, that the columns performed as friction piles with little or no bulging, but were of inadequate depth.

2.1.3. Dry bottom feed. Dry bottom feed[21] has now largely replaced the wet top feed method since its development in the late 1970s, and is predominantly used for treatment of water-bearing and soft cohesive deposits (usually for the vibro replacement process). Controlled air flush is used to aid construction and maintain stability of the void formed by the vibrating poker.

There is considerable evidence that the dry bottom feed method, now the preferred construction technique in soft soils, can successfully treat ground with c_u well below 15–20 kPa frequently quoted as the lower practical limit of the system's applicability.[10,22] Wehr[23] gives several examples (in Poland, Germany and Malaysia) where stone columns were installed in soils with c_u as low as 4–5 kPa, but emphasises the importance of automated monitoring of the construction process if such low-strength soils are to be treated. Egan *et al.*[5] discuss the behaviour of trial strip foundations relevant to a two- and three-storey housing development in Scotland, where dry bottom feed columns were used (subsequently referred to as Keller Foundations Contract B, to be consistent with Egan *et al.*[5]). Ground conditions at the site comprised a 1·5 m thick clay crust (30 kPa $< c_u <$ 100 kPa), underlain by 12 m of soft Carse clay (average $c_u = 10$ kPa). Load tests were carried out on a number of strip footings (see Figure 1a) to demonstrate the feasibility of stone columns in these soils. A typical settlement–time graph is shown in Figure 1b (with the first 24 h of immediate elastic settlement removed, since this would occur during construction), from which it can be seen that the majority of primary consolidation settlement was complete within 8 weeks.

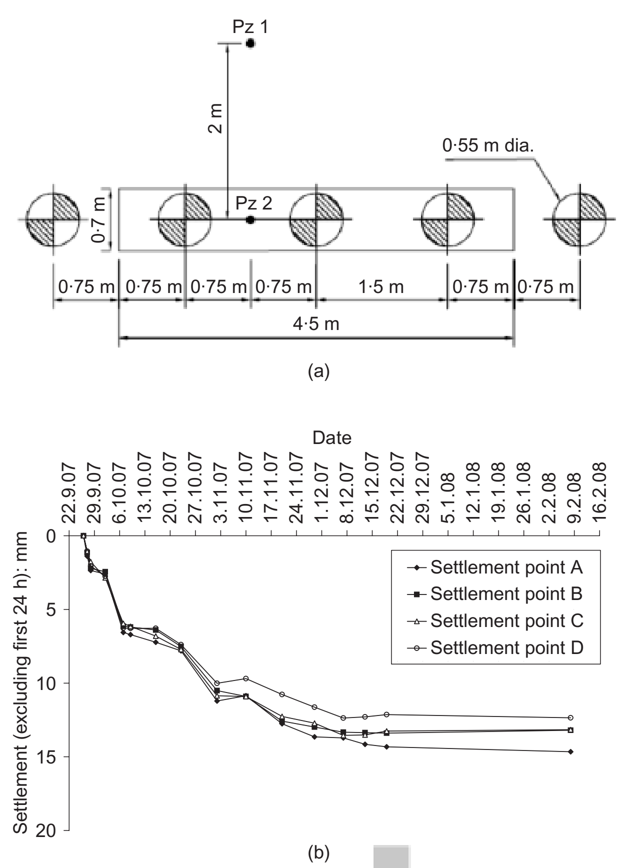

Figure 1. Keller Foundations Contract B: (a) trial strip footing (Pz = piezometer); (b) settlement–time behaviour of trial strip footing

Venmans[3] also reports successful dry bottom feed treatment for embankment support in clays with $c_u = 15$–20 kPa. An interesting comparative trial was carried out with a rarely used wet bottom feed method that actually liquefied the clay, preventing construction of a well-compacted and permeable stone column. The behaviour of the wet bottom feed trial section was actually worse than that of a reference section with prefabricated vertical drains.

Stone columns and surcharging in combination is also an option in soils where c_u values are considered marginal. Raju *et al.*[24] show average increases in c_u from 8 kPa to 32 kPa when this combined approach was used to improve clayey silt mining slimes supporting an embankment height of up to 15 m in Malaysia. The average treatment depth was 13·5 m, column diameter was 1·05 m, and the spacing of columns on a square grid was 1·9 m. Bhushan *et al.*[25] use a similar combined approach for two storage tanks on soft cohesive ground. These columns were 12·2 m in length and spaced on a 2·4 m triangular grid.

2.1.4. Other methods. Other construction methods have also been used: for example, some case histories exist describing bottom rammed[26] columns, which have evolved from the classic Franki pile. A tube is bottom-driven to the required penetration depth. As it is withdrawn, successive charges of stone are introduced and progressively forced into the surrounding soil by intensive ramming. This method would be neither practical nor economical for soft soils.

2.2. Construction defects and overworking

Development of automated rig instrumentation (Figure 2) has been very beneficial in facilitating a high level of construction control for columns formed in soft cohesive deposits. Bell[27] highlights the dangers of poor construction technique for a routine project in the UK in mixed ground conditions incorporating clay. Excavation revealed that some columns were very poorly constructed, with many having diameters much smaller than the poker diameter. This would have been realised at the time of construction if stone consumption had been monitored and compared with expected quantities, or if poker current demand had been used to check the compaction and verify that the column had been constructed in discrete lifts. The relationship between the poker current demand and c_u for the installation of dry bottom feed columns at a soft clay site[5] is shown in Figure 3 (and pertains to initial penetration).

A skilled practitioner will recognise that the benefit of a tightly compacted stone column may be offset by the extent of disturbance caused by imparting excessive energy to the ground. For this reason, stone columns in soft cohesive deposits should ideally be approximately uniform in diameter over their length. This may be facilitated by constructing to a stone consumption rather than a rig compaction energy criterion. A previously adopted approach of ramming stone at the base of a column to form an enlarged base[28] has become

Figure 2. In-rig instrumentation for construction quality control

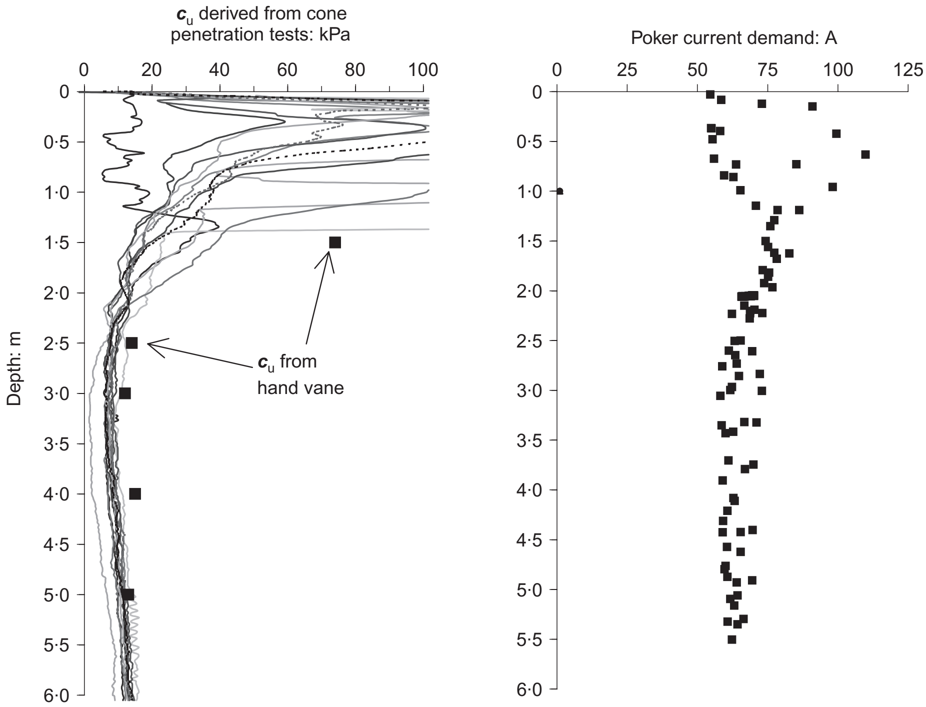

Figure 3. Relationship between undrained shear strength and current demand of poker[5]

discredited for this reason. In this trial,[28] at Bothkennar, Scotland, ~360 mm of heave is reported at the surface of treated ground, which occurred with the development of high pore water pressures and long dissipation periods for a group of 25 bottom feed stone columns supporting an 8·1 m square raft foundation with a thickened perimeter edge beam. The disturbance is attributed to the excessive (pressure and duration) use of air flush during the non-standard construction of expanded bases for the partial-depth stone columns; 60% of the total stone volume was consumed over the bottom third of the column length. In a separate study at the same site, settlement performance of footings was found to deteriorate when columns were used to support them, due to overworking of the firm clay curst with the poker.[29] Practitioners need to be aware of the very significant benefit to performance offered by even a relatively thin firm or stiff crust overlying soft soils. It should be emphasised that modern practice of carefully controlling column construction has contributed significantly to the execution of many successful projects at soft clay sites over the last 10 to 15 years.

3. DEVELOPMENT OF A NEW SETTLEMENT DATABASE

The ability to meet absolute (and probably more importantly differential) settlement criteria is normally the governing factor for vibro replacement design in soft soil, and therefore most published case histories concentrate on settlement performance ahead of bearing capacity. In any case, a measurement of bearing capacity requires a failure to occur. The authors have developed a new database with settlement information from a total of over 20 case histories in cohesive or predominantly cohesive soils. While most of these cases relate to loading of areas that are wide in relation to the column length (subsequently referred to as 'widespread loading'), such as embankments and storage tanks, three of the cases relate to footings supported by small column groups.

Settlement performance is captured in the form of a settlement improvement factor n, defined as

$$n = \frac{s_{\text{untreated}}}{s_{\text{treated}}} \tag{1}$$

where $s_{\text{untreated}}$ is the settlement (of the loaded zone) in the absence of stone column treatment, and s_{treated} is the corresponding settlement with stone column treatment. The database necessarily comprises two parts, distinguished as follows.

(*a*) Table 1 lists the projects where s_{treated} and (a reference value of) $s_{\text{untreated}}$ have both been measured, so the value of n is entirely measurement based.

(*b*) In Table 2, s_{treated} values have been measured; however, $s_{\text{untreated}}$ values have not been measured but instead have been predicted either analytically or from experience of measurements in similar or local ground conditions.

The quantity used by Priebe[12] to capture the concentration of the column array in an infinite grid is referred to as the area replacement ratio, A/A_c where A is the plan area of the 'unit cell' attributed to a single column, and A_c is the cross-sectional area of one column. The value of A/A_c may be deduced from the column diameter D and spacing s according to

$$\frac{A}{A_c} = k\left(\frac{s}{D}\right)^2 \tag{2}$$

where k is $4/\pi$ and $(2\sqrt{3})/\pi$ for square and triangular column grids respectively. A/A_c values are also given in Tables 1 and 2 for the widespread loading cases, with A_c either measured directly or derived from stone consumption records.

Reference	Site location	Material treated	Average treatment depth: m	Stone column spacing: m	Treatment configuration	Average diameter: m	Treatment method	Loading type	Area replacement ratio, A/A_c	Settlement imp. factor, n
Cooper and Rose[4]	Bristol, UK	Clay	4·35	2·10	TR	0·602	BF	Embankment	13·40	1·85
			4·35	1·50	TR	0·602	BF	Embankment	6·84	2·55
Watts *et al.*[7]	Bacup, Lancashire, UK	Clay, ash, made ground	3·50	1·80	L	0·6	DTF	Test strip	4·78	1·47
Munfakh *et al.*[18]	New Orleans, USA.	Clay	~20	2·10	TR	1·11	WTF	Embankment	3·95	1·70
Greenwood[30]	Bremerhaven, Germany	Clay, peat	6·0	2·30	TR	1·2	WTF	Embankment	3·85	1·63

WTF, wet top feed; DTF, dry top feed; BF, bottom feed; TR, triangular.

Table 1. Case histories in which both s_{treated} and $s_{\text{untreated}}$ values have been measured

3.1. Widespread loading

Values of n (from Tables 1 and 2) are plotted against A/A_c in Figure 4 for the widespread loading cases. In order to provide a frame of reference for the data, Figure 4 also includes Priebe's[12] basic improvement factor n_0 prediction (Equation 3; Poisson's ratio of the soil $\nu_s = 0{\cdot}33$ assumed, as is customary) adopting an operational friction angle $\phi' = 40°$ for the stone.

$$n_0 = 1 + \frac{A_c}{A}\left[\frac{5 - \dfrac{A_c}{A}}{4\left(1 - \dfrac{A_c}{A}\right)\tan^2\left(45 - \dfrac{\phi'}{2}\right)} - 1\right] \qquad (3)$$

This assumption is made since case-specific values of ϕ' are generally not presented in the literature, and $\phi' \approx 40°$ is typical of the range of values used in design[38] for weak deposits. Moreover, additional parameters needed for predicting Priebe's[12] n_1 and n_2 factors (amendments to n_0 and n_1 to account for column compressibility and soil and column unit weights respectively) are not available.

Although there is a spread of data around the Priebe[12] n_0 curve, the match is nevertheless considered to be good, given that the conditions are not consistent across all tests, and given the uncertainty associated with the assumptions that have had to be made in analysing the published data. It is clear that Equation 3 predicts the shape of the measured n–A/A_c variation reasonably well, even though there is insufficient resolution from the published data to take account of all of the factors that would have influenced the degree of settlement control. These factors include the following.

(*a*) There are variations in the prediction methods used to determine the n values in Table 2.
(*b*) There is uncertainty in the 'as-constructed' column diameter and spacing.
(*c*) The stage of loading/time period after loading at which n is measured is not consistent throughout all of the case studies. Because of the drainage effect of stone columns, primary settlement will occur at a faster rate and will be complete sooner for treated areas than for similarly loaded untreated areas. Therefore if, for example, the settlement of the untreated area is measured before primary settlement is complete, unfavourably low n values will be obtained. In the longer term the actual n values will improve, since the untreated settlement ($s_{\text{untreated}}$) will continue while treated settlement (s_{treated}) will be well advanced or even complete.
(*d*) Settlement at the ground surface will reflect settlement over the treated depth plus any additional settlement generated in the ground below the columns (the latter is relevant for partial depth treatment). A majority of the cases in Tables 1 and 2 have had full depth treatment but some have had partial depth treatment. It is not always possible to separate out the settlement arising from the treated zone in studies carried out in construction projects: hence in such situtations the degree of improvment attributed to the installation of the stone columns would probably be underestimated.

All of these different aspects are difficult to quantify, but an experienced column designer would need to take them into account.

Reference	Site location	Material treated	Average treatment depth: m	Stone column spacing: m	Treatment configuration	Average diameter: m	Treatment method	Loading type	Area replacement ratio, A/A_c	Settlement imp. factor, n
Venmans[3]	Holendrecht-Abcoude, Netherlands	Clay	5·2	1·5	TR	0·65	BF	Embankment	5·88	1·54
Greenwood[20]	Canvey Island, UK	Clay/silt	10·0	1·52	TR	0·75	WTF	Storage tank	4·53	2·38
Raju *et al.*[24]	Kajang, Malaysia	Silt, made ground	13·5	1·90	SQ	1·05	WTF + BF	Embankment	4·17	2·60
De Cock and D'Hoore[26]	Antwerp, Belgium	Peaty clay	8·5	1·60	TR	0·9	BR	Storage tank	3·48	3·00
	Oreye, Belgium	Silt	11·0	1·60	TR	0·8	BR	Storage tank	4·41	1·83
Baumann and Bauer[31]	Konstanz, Germany	Silt	5·5	1·40	TR	1·0	WTF	Raft	2·12	4·03
Watt *et al.*[32]	Teesport 104, UK	Silt	6·1	1·91	TR	1·09	WTF	Storage tank	3·38	2·80
	Teesport 165, UK	Silt	6·1	1·91	TR	1·09	WTF	Storage tank	3·38	3·43
	Hedon, UK	Clay	6·7	2·13	TR	1·06	WTF	Storage tank	3·72	2·77
Greenwood[33]	Stanlow, UK						WTF	Storage tank	3·24	5·47
Goughnour and Bayuk[34]	Hampton, USA.	Silt, clay	6·4	1·8	TR	1·1	WTF	Embankment	2·95	2·40
Raju[35]	Kinrara, Malaysia	Silt, made ground	17·0	1·80	SQ	1·2	BF	Embankment	2·86	4·00
	Kebun, Malaysia	Clay	15·0	2·20	SQ	1·1	BF	Embankment	5·09	2·50
Bell[36]	Stockton, UK	Clay	4·4	2	TR	0·615	WTF	Embankment	11·70	1·38
				2·5		0·615			18·20	1·24
				2·5		0·615			26·20	1·15
Kirsch[37]	Essen, Germany	Silt	5·0	1·70	STSQ	1·12	WTF	Storage tank	2·90	2·35
Kirsch (unpublished)	Berlin-Brandenburg Test B, Germany	Clay/silt	7·1	1·45	SQ	0·6	BF	Footing	14·30	1·94
	Berlin-Brandenburg Test C, Germany	Clay/silt	6·8	2·0	SQ	0·6	BF	Footing	7·70	2·10
Keller Foundations Contract B	Scotland	Clay	5·5	1·5	L	0·55	BF	Test strip	4·42	1·20

WTF, wet top feed; BF, bottom feed; BR, bottom rammed; TR, triangular; SQ, square; STSQ, staggered square; L, linear.

Table 2. Case histories in which $s_{treated}$ values have been measured but $s_{untreated}$ values have been predicted

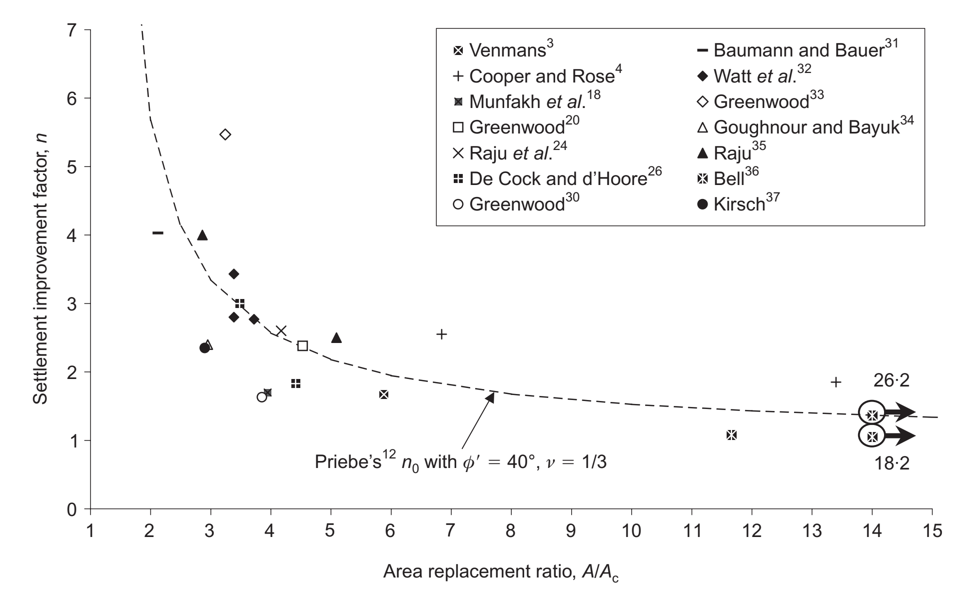

Figure 4. Settlement improvement factor against area replacement ratio for sites with widespread loading

'unit cell' conditions were assumed for the loaded columns. In Figure 5 a distinction is made between Table 1 data (true n measurements, filled symbols) and Table 2 data (open symbols); also, the different systems used to construct the columns are highlighted.

The higher n_{meas}/n_{pred} ratios in Figure 5 (i.e. denoting measurements that are better than predictions) tend to relate to bottom feed projects (rather than the top feed and bottom rammed projects), which endorses the use of the bottom feed system for treating these soft ground conditions. Figure 5 implies that a design friction angle of $\phi' = 40°$ may not always give safe settlement predictions for the top feed system, but as already stated, there may be other possible explanations besides the friction angle. For example, the deposits within the treatment zone at the Bremerhaven site[30] incorporated some peaty material.

In general, the data indicate that a design friction angle of $\phi' = 40°$ is a conservative assumption in the case of the bottom feed system. Interestingly, Herle *et al.*[38] advocate the use of higher ϕ' values (i.e. in excess of 50°) than are commonly adopted in UK and Irish design (based on shear box tests carried out on stone at high relative density levels). However, caution is advised if adopting ϕ' values from direct shear tests; good design practice should consider the degree of stone compaction and the confining strength of the soil, which again highlights the importance of good construction control and the benefits of instrumentation.

Additional wet top feed data by Raman[39] not included in Table 1 or Figure 4 (owing to the large amount of data and considerable scatter, reflecting, at least in part, local variations over the several sections of expressway totalling nearly 4 km), would plot significantly above the $\phi' = 40°$ reference curve.

3.2. Widespread loading and small foundation loading combined

Predicted values of n (n_{pred}) are plotted against measured values (n_{meas}) in Figure 5. Tests that plot below the $n_{pred} = n_{meas}$ line denote measurements that exceed predictions. Much of the data in Figure 5 is taken from Figure 4 (i.e. n_{pred} is n_0 with $\phi' = 40°$). However, Figure 5 also includes data from two test strips (Watts *et al.*[7] and Keller Foundations Contract B) and two pad footings (Kirsch, unpublished). In the case of the strips, the Priebe[12] correction was applied to n_{pred} to allow for the reduced efficiency of small groups. No such correction was applied for the pads, as the loaded columns underneath the footings were surrounded by 'buttressing' columns, so

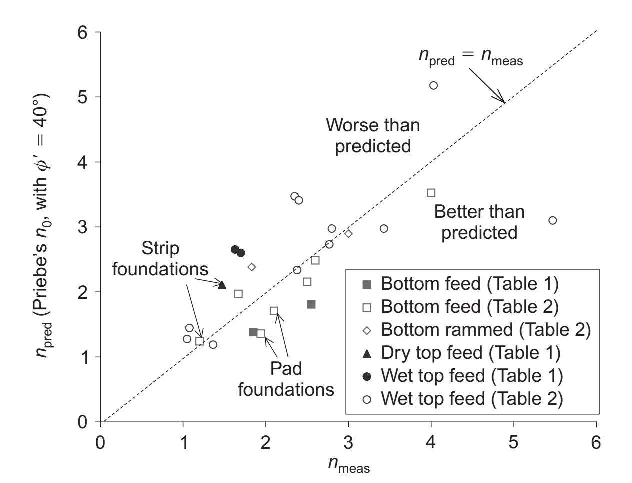

Figure 5. Predicted against measured settlement improvement factor for all widespread loading and footings

4. STRESS CHANGES

4.1. Lateral total stress measurements

There are some published studies with measured lateral/radial stress changes immediately after the installation of stone columns. However, there are few data to clarify what has happened to the total lateral stresses (and hence effective lateral stresses) by the time excess pore pressures have dissipated, which is unfortunate, as these stresses represent the initial conditions for loading. In general, permanent lateral total stress increases are known to arise after driving piles in soft cohesive soils,[40] and it seems reasonable to assume a similar situation arising during stone column construction.

The most comprehensive data on the installation of stone columns are reported by Kirsch.[41] The measurements of lateral total stress were taken during column installation within a 5 × 5 column group in silty clay (described as weak to stiff and of medium plasticity). The maximum lateral total stresses reached (soon after installation) are presented in the form of a lateral earth pressure coefficient $\boldsymbol{K}$ normalised by the at-rest value $\boldsymbol{K}_0$. Values of $\boldsymbol{K}/\boldsymbol{K}_0$ exceeded 1 to a distance of $9D_p$, reaching a maximum of $\sim$1·6 at $4D_p$–$5D_p$ (where D_p is the poker diameter). Kirsch[41] attributes the lower values closer to the column to remoulding by the poker and dynamic excitation caused by its vibrations. Watts *et al.*[28] and Serridge and Sarsby[29] both (coincidentally) report maximum horizontal total stresses during installation of $\sim$80 kPa for a raft and for small column groups respectively, falling to lower values soon after installation; however, it is difficult to interpret these data, given the aforementioned problems with the construction processes in each instance.

There is some uncertainty whether the deepest lateral total stress measurements presented by Watts *et al.*[7] relate to an ashy or cohesive fill, although the latter is suspected. No increase was registered until the poker reached the cell level, but up to 60 kPa was registered with further penetration to the design depth by a cell 900 mm from the centreline of a 600 mm diameter column. Lower increases were observed 1·5 m from the column centreline. Withdrawing the poker to compact the stone caused an immediate return to pre-penetration pressure levels, and similar elevated levels were reached once stone compaction took place. The increases disappeared again once the column was completed. With the exception of this study, none has reported equalised lateral total stress measurements, to the authors' knowledge. Such information is key to understanding how the process of column construction modifies the ground in advance of loading.

4.2. Pore pressure measurements

4.2.1. Maximum installation pore pressures. Case histories presenting measurements of excess pore water pressure generated by stone column installation along with relevant consolidation parameters are also limited. Castro[42] showed that installation pore pressures (recorded by piezometers) at any level are at their maximum approximately when the poker tip passes that level. This is broadly in keeping with equivalent data for displacement piles.[43,44] Unlike a driven pile, a poker will pass any given horizon more than once as the stone is compacted in lifts, and Castro's[42] data show a second pore pressure peak, which is lower in magnitude than the original, when the poker is again passing the level of the sensor.

It is of interest to compare maximum pore pressures generated by stone column installation with those generated by displacement piles. Data for single piles and five-pile groups in Belfast clay[44] are reproduced in Figure 6a; other published single-pile data[45,46] are also included. Maximum excess pore pressure ratios $\Delta \boldsymbol{u}_{max}/\boldsymbol{\sigma}'_{v0}$ (where Δu_{max} is the maximum excess pore pressure reached, and $\boldsymbol{\sigma}'_{v0}$ is the free-field vertical effective stress) are plotted as a function of distance from the pile centre normalised by the pile radius, r/R (or equivalent pile radius r/R_{eq}; $R_{eq} = B/\sqrt{\pi}$, B = pile width). Acknowledging the relationship between excess pore pressure and yield stress ratio YSR ($\Delta \boldsymbol{u} \propto \text{YSR}^{0.36}$),[47] it can be seen that the pore pressure field for pile groups is of greater magnitude and radial extent than that for single piles. The McCabe *et al.*[44] data at $r/R = 1$ shows the well-established h/D effect (h = distance above pile tip, $D = 2R$) for driven piles in clay. Equivalent $\Delta \boldsymbol{u}_{max}/\boldsymbol{\sigma}'_{v0}$ data (as interpreted by the authors) for stone column case histories are

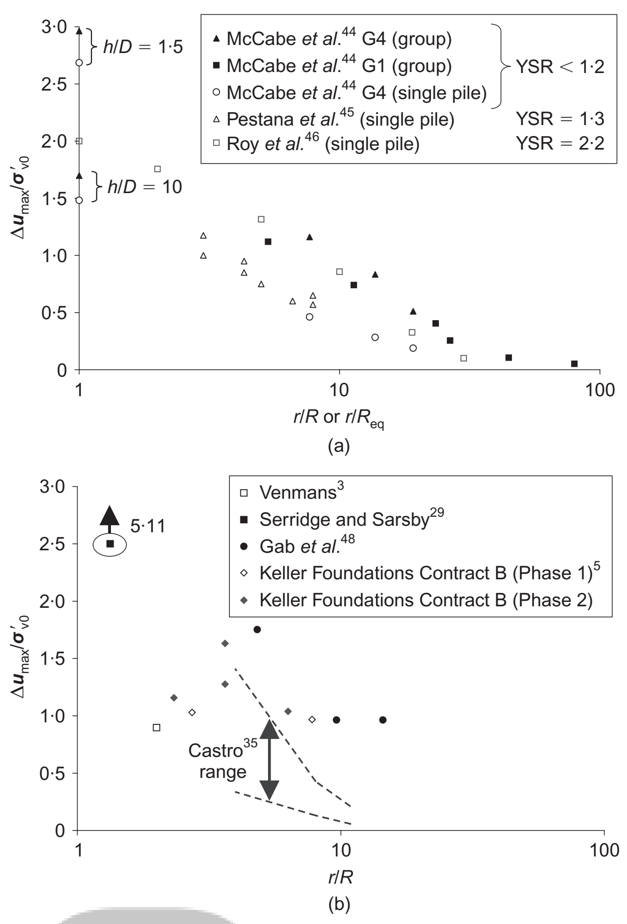

Figure 6. Maximum excess pore pressures around: (a) driven piles in clay;[33] (b) stone columns

presented in Figure 6b, with supporting information, including approximate YSR values, in Table 3. Once again r is the distance (or average distance) of the measuring device from the column centre, and R is the column radius.

Despite some scatter, it is apparent that the pore pressure fields around stone columns and driven piles are quite similar (with the exception of the data from Castro,[42] where the significant sand content of the material is likely to be responsible for the lower $\Delta \boldsymbol{u}_{max}/\boldsymbol{\sigma}'_{v0}$ values). Moreover, the higher $\Delta \boldsymbol{u}_{max}/\boldsymbol{\sigma}'_{v0}$ values from Gab *et al.*[48] reflect the greater size of this stone column group. In general, the data suggest that the pore pressures are cumulative as each successive column is installed, but the cumulative values will of course be a function of, among other things, the average duration between installations. It is interesting to note that the $\Delta \boldsymbol{u}_{max}/\boldsymbol{\sigma}'_{v0}$ value from Serridge and Sarsby[29] plots quite high compared with the rest of the data, possibly indicative of the aforementioned overworking during the formation of the columns.

4.2.2. Dissipation of installation pore pressures. Field observations suggest that stone columns accelerate the rate of consolidation of soft clays. Han and Ye[49] recorded the rates of settlement of two similar building foundations on the same site, one with stone column support and one on unreinforced clay. Over a 480-day period the degrees of consolidation for the unreinforced and reinforced cases were 66% and 95% respectively. Munfakh *et al.*[18] reported that by the time 100%

Reference	Site location	Approx. YSR	Column length: m	Average column diameter: m	No. of columns	Piezometer depth: m
Venmans[3]	Holendrecht-Abcoude, Netherlands	~1	5·5	0·65	Large grid	
Egan *et al.*[5]	Scotland, UK	1·2	5·5	0·55	5	2, 4
Serridge and Sarsby[29]	Bothkennar, UK	1·5	5·7	~0·75	4	1·6
Castro[42]	Valencia, Spain	~1	9·0	0·80	7	4, 7
Gab *et al.*[48]	Klagenfurt, Austria	~1	14·5	0·70	36	12
Keller Foundations Contract B (unpublished)	Scotland, UK	1·2	5·5	0·55	>10	3

Table 3. Case histories with installation pore pressure measurements

primary consolidation had taken place beneath a stone column supported trial embankment, only 25% primary consolidation had taken place in a corresponding untreated area. The aforementioned Raju *et al.*[24] report a sixfold increase in consolidation rate compared with unimproved ground, and accelerated consolidation rates were also noted by Bhushan *et al.*[25].

The simplified analytical approach of Han and Ye[50] may be used to quantify the extent to which stone columns accelerate drainage, and the method suggests that the drainage rate depends on the modular ratio E_c/E_s, where E_c and E_s are the Young's moduli for columns and soil respectively. If a 5 m thick double-draining layer of soft clay is treated full depth with a square grid of columns having $A/A_c = 4$ (shown by Figure 4 to be a reasonable average treatment density) and 2 m spacing, $E_c/E_s = 10$, the method predicts that the columns will increase both c_h and c_v by a factor of ~2·4. Adopting $c_h/c_v = 2$, the predictions suggest that only ~10% dissipation will have occurred in untreated ground in the time required for in excess of 90% dissipation in treated ground. This enhancement to drainage is in keeping with the case histories mentioned.

The same conclusion may be arrived at by comparing the pore pressure dissipation around stone columns with that around displacement piles. In Figure 7 the degree of dissipation U is plotted as a function of a time factor ($T_h = c_h t/4R^2$, where c_h is the horizontal coefficient of consolidation and R is the column radius, or R_{eq} for the square piles). The curves in Figure 7 are derived from measurements

(*a*) at $r/R \approx 5$ for the stone columns[5] in Figure 1
(*b*) at $r/R = 1$ (i.e. at the pile shaft) on a single jacked pile in Bothkennar clay[51]
(*c*) at $r/R = 1$ on a single jacked pile in Belfast clay[52]
(*d*) at $r/R > 5$ for a driven five-pile group in Belfast clay.[40]

The value of c_h assumed in Figure 7 is 30 m^2/year. Nash *et al.*[53] show that field c_h values can considerably exceed laboratory values in a soft cohesive soil. Lehane[51] back-figured this value of c_h from instrumented displacement pile tests in Carse and other clays, while McCabe[54] back-figured similar values from piezometer data around driven piles in Belfast clay.

It is clear from Figure 7 that post-installation consolidation is complete much faster for the stone columns than for the piles, thereby providing a further indication of the drainage benefit that stone columns offer.

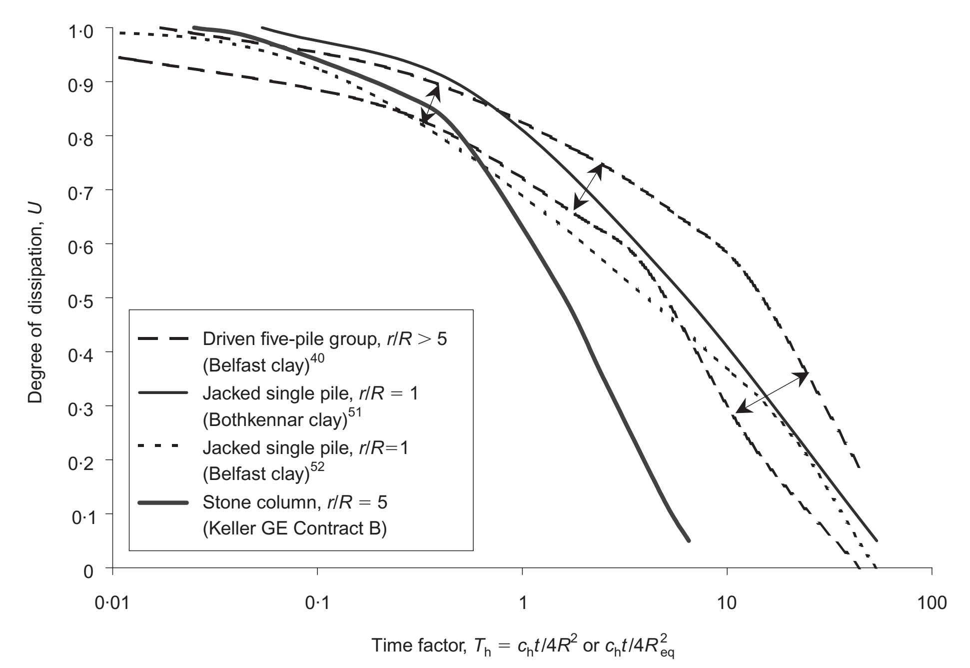

Figure 7. Comparative excess pore pressure dissipation rates for displacement piles and stone columns

4.2.3. Pore pressures during loading. Cooper and Rose[4] show that the pore pressures measured during embankment construction are very significantly below the weight of fill placed – evidence of the enhanced drainage facilitated by the stone columns. Pore pressures measured between a group of three columns at Konstanz student residences[31] increased to a maximum of 40–50% of the applied footing load, and again the columns were the key to rapid pore pressure dissipation.

5. CONCLUSION

A comprehensive literature search has distilled some very useful information and guidance for the ground improvement designer, which may be summarised as follows.

(*a*) The dry bottom feed method is the preferred approach for constructing stone columns in soft soils. Soils with c_u as low as 5 kPa have been treated successfully, and the settlement improvement database developed here suggests that this method performs better than other methods such as wet top feed.

(*b*) The Priebe[12] design method with conventionally adopted friction angles of ~40° provides a safe lower-bound estimate of actual bottom feed performance in practice. This method, which continues to be the favoured design approach of leading stone column designers, is proved to be reliable, even though it does not capture all of the fundamental soil and stress changes that take place during column construction and subsequent loading. More sophisticated approaches, such as finite-element (FE) analysis, provide another useful tool to geotechnical engineers, although FE analysis has not evolved to a stage of readiness for routine design.

(*c*) Maximum excess pore pressure fields around a penetrating poker are quite similar to those round driven piles, and the effect of group size is reflected in a similar way. However, these excess pore pressures dissipate faster with the stone columns, because they act as vertical drains.

(*d*) The evidence suggests that undrained shear strengths can be expected to increase permanently with stone column construction, provided construction standards are adequate. Lateral effective stresses seem to increase around the column because of the installation process, although higher-quality measurements are needed for greater confidence.

The literature search has identified a lack of high-quality data on aspects of stone column behaviour in soft cohesive soils, such as the stress split between soil and stone, long-term changes to the lateral effective stress regime imposed by column construction, and the effect on long-term creep settlements. The vast majority of the data pertain to large loaded areas, and not to strip or pad footings. There remains a need for high-quality instrumented field tests to improve our understanding of the factors controlling stone column behaviour.

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