Settlement of floor slabs on stone columns in very soft clays

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Investigations of serviceability failures of building floor slabs supported by stone columns in very soft clays prompted a review of the methods for predicting the settlement reduction achieved by the columns. Data from both successful case histories and serviceability failures indicate that, in practice, settlement reduction for widespread loading of very soft clays may be both quite limited and overestimated by current design methods. Analysis of long-term settlement observations from two serviceability failures suggests that the reduction in secondary settlement achieved by the stone columns was also quite limited. Based on these findings it is considered that the range of loadings and thicknesses of very soft clay for which stone columns would be viable for widespread building loads is likewise limited. A review of the serviceability failures led to the conclusion that inadequacies in the procurement of the design and construction of the stone columns contributed to the failures.

Notation

A	plan area of unit cell for a stone column
A_c	cross-sectional area of stone column
A_i	area improvement = A/A_c
A_r	stone replacement = A_c/A
A_s	plan area of soil within unit cell
C_a	coefficient of secondary consolidation
C_c	compression index
C_h	horizontal coefficient of consolidation
C_u	undrained shear strength
D	stone column diameter
E_s	deformation modulus of clay soil
e_o	initial void ratio
H	thickness of clay treated with stone columns
I_f	settlement improvement factor = S_p/S_t
L	length of stone column
m_v	coefficient of one-dimensional compressibility
n	stress concentration ratio = σ_c/σ_s
p	bearing pressure
q	bearing ratio = p/C_u
R_f	settlement reduction factor = S_t/S_p
S_p	primary consolidation settlement without stone columns
S_r	settlement reduction = $1 - R_f$
S_s	secondary consolidation settlement
S_t	primary consolidation settlement with stone columns
S_T	total settlement = $S_t + S_s$
s	stone column centre-to-centre spacing
t	elapsed time
t_1	start time for secondary consolidation period

t_2	finish time for secondary consolidation period
Δ_u	excess pore pressure
σ_c	vertical stress carried by stone column
σ_s	vertical stress carried by clay soil
σ'_v	vertical effective stress
σ'_{v0}	in situ vertical effective stress
ϕ'_c	coefficient of friction of stone column

1. Introduction

In the preface to the 1975 *Géotechnique* symposium on ground and soil improvement, Burland *et al.* (1975) noted that nearly all the papers had been produced by specialist contractors and that they concentrated on successes, with any failures remaining unrecorded. In a keynote paper to the 2001 symposium, Charles (2002) commented that, despite the passage of 25 years, little had changed and ground improvement had developed in a largely empirical, contractor-led fashion and could best be described as an experience-based technology. McCabe *et al.* (2009) reviewed a large number of successful case histories of stone columns in soft soils and concluded that Priebe's method of determining settlement reduction (Priebe, 1976, 1995) provided a safe lower bound for stone columns supporting wide loaded areas. McCabe *et al.* (2009) also noted that there was no consensus on the extent to which stone columns might reduce secondary settlement.

Over the past 30 years the author has been involved in both the procurement of stone columns and the investigation of serviceability failures of buildings founded on clay soils reinforced with stone columns. Common factors in the failures were the

presence of very soft clays and widespread loading imposed by way of a ground-bearing floor slab within a building supported on piled foundations. These factors typically combined to produce large predicted settlements for the untreated ground, generally in excess of 100 mm, and serviceability limits on maximum settlements of 50 mm or less, thus requiring the stone columns to reduce settlements by at least 50%. The requirement for stone columns to achieve such large settlement reductions increases the risk of failure, which can also arise from inadequate characterisation of the ground and inappropriate design and construction procurement (Charles, 2002).

These serviceability failures raised doubts concerning the ability of stone columns to meet serviceability settlement limits for building floor slabs on very soft clays. This led to a review of the theoretical methods for determining settlement reduction and, in particular, the apportionment of applied vertical stress between stone columns and clay soil (the stress concentration ratio). Comparison of theory with field measurements of the stress concentration ratio suggests that, for widespread loading of very soft clays, the maximum settlement reduction that can be achieved in practice may be quite limited.

In this paper, long-term settlement records are presented for two serviceability failures involving the use of stone columns to reduce settlements of building floor slabs on very soft clays. Analysis of these case histories indicates that, while the stone columns may have effected reductions in both their rate and magnitude, large secondary settlements of the treated ground still occurred. This suggests that the ability of stone columns to reduce secondary settlement may also be quite limited and, as a consequence, their efficacy for floor slabs on very soft clays.

A review of the procurement of the design and construction of the stone columns for the serviceability failures suggests that shortcomings in procurement contributed to an inadequate assessment of the viability of stone columns to achieve the required serviceability limits and, as a result, to the failures.

2. Design of stone columns for settlement reduction

2.1 Equilibrium equation and stress concentration ratio

For design purposes, the unit cell (the area of soil surrounding a single stone column) is assumed to lie within an infinitely large group of stone columns that fully penetrate the soil and bear on an incompressible stratum. The load apportionment between the columns and the surrounding soil is represented by the equilibrium equation

$$1. \quad pA = \sigma_c A_c + \sigma_s A_s$$

in which A is the plan area of the unit cell, p is the bearing pressure, A_c is the cross-sectional area of the column, σ_c is the vertical stress carried by the column, A_s is the plan area of soil and σ_s is the vertical stress carried by the soil.

The reduction in vertical stress for the soil is governed by the proportion of the plan area of the cell within which soil is replaced by stone, represented by the stone replacement A_r

$$2. \quad A_r = \frac{A_c}{A}$$

and the stress concentration ratio n , given by

$$3. \quad n = \frac{\sigma_c}{\sigma_s}$$

Combining Equations 1, 2 and 3 yields

$$4. \quad \sigma_s = \frac{p}{[1 + (n - 1)A_r]}$$

Using conventional one-dimensional consolidation theory, the settlement of very soft normally consolidated clays treated with stone columns is given by

$$5. \quad S_t = \frac{C_c}{(1 + e_o)} \log_{10}[(\sigma'_{vo} + \sigma_s)/\sigma'_{vo}]H$$

where S_t is the primary consolidation settlement of the treated clay, H is the thickness of treated clay, σ'_{vo} is the in situ vertical effective stress, C_c is the compression index and e_o is the initial void ratio.

From Equations 4 and 5, it follows that the settlement reduction, as represented by the settlement reduction factor R_f

$$6. \quad R_f = \frac{S_t}{S_p}$$

in which S_p is the primary consolidation settlement of the untreated clay for very soft normally consolidated clays, is given by

$$7. \quad R_f = \frac{\log_{10}\{(\sigma'_{vo} + p/[1 + (n - 1)A_r])/\sigma'_{vo}\}}{\log_{10}[(\sigma'_{vo} + p)/\sigma'_{vo}]}$$

From Equation 7 it can be seen that the settlement reduction factor for very soft normally consolidated clays is dependent not only upon the stress concentration ratio (n) and the stone

replacement (A_r) but also upon the initial effective stress in the clay (σ'_{v0}) and the magnitude of the bearing pressure (p), with greater settlement reduction indicated for larger initial effective stresses and smaller bearing pressures. With increasing initial effective stress and decreasing bearing pressure, the settlement reduction factor rapidly approaches

$$8. \quad R_f = \frac{S_i}{S_p} = \frac{1}{[1 + (n-1)A_r]} = \frac{\sigma_s}{p}$$

Equation 8, which applies to overconsolidated clays (primary consolidation settlement directly proportional to applied bearing pressure), gives a slightly unconservative estimate of settlement reduction for very soft normally consolidated clays. It is, however, well suited for comparative studies of the stress concentration ratios predicted by theory with those achieved in practice.

Figure 1 shows the settlement reduction S_r ($S_r = 1 - R_f$) plotted against stone replacement (A_r) predicted using Equation 8 for a range of stress concentration ratios (n) from 2 to 25. Also shown in Figure 1 are alternative representations for S_r (i.e. the improvement factor $I_f = 1/R_f$) and A_r (i.e. area improvement $A_i = 1/A_r$) used in the stone column literature.

The stress concentration ratio is dependent upon the radial restraint provided to the column by the surrounding soil and is thus an interdependent function of the strengths and deformation moduli of the soil and the column. In practice, the radial restraint also depends on the spacing of the columns and the soil yielding and consequential excess pore pressures generated by their installation (Castro, 2007; Castro and Sagaseta, 2012; McCabe *et al.*, 2009). Hughes and Withers (1974) considered that stone columns would act as a group

with spacing (s) to diameter (D) ratios less than 2.5 while Bergado and Lam (1987) cite a limiting s/D of 3.

Figure 1 also shows a line representing equality of stone replacement and settlement reduction, corresponding to $n = 2-3$, being a suggested practical lower bound for settlement reduction. Experience-based practical upper bounds suggested by Barksdale and Bachus (1983), St. John *et al.* (1989) and Charles (1993) broadly correspond to $n = 5$, indicating a practical maximum settlement reduction of about 60% for 35% stone replacement.

2.2 Theoretical methods for settlement reduction

Theoretical design methods solve the equilibrium equation by using various constitutive models and assumptions for shear strength and deformation parameters to determine the stress concentration ratio. Baumann and Bauer (1974) produced design curves for a variety of soil types, with that for soft silty clay shown in Figure 2. This method implies stress concentration ratios between 13 and 33. Priebe (1976, 1995) produced design curves for stone column friction angles (ϕ'_c) varying from 35° to 45° . The curves for $\phi'_c = 35^\circ$ and 40° are shown in Figure 2, implying respective n values of 5–6.5 and 6.5–8.5, indicating a theoretical reduction in stress concentration ratio with reducing stone column friction angle. The methods of Baumann and Bauer (1974) and Priebe (1976, 1995) predict settlement reductions equal to the ratio of vertical stress on the soil to the applied bearing pressure (Equation 8) and have both been used, apparently successfully, by stone column contractors over the past 40 years.

Barksdale and Bachus (1983) developed design charts for predicting the settlement of soft compressible clays treated with stone columns using a finite-element unit cell model.

Figure 1. Settlement reduction as a function of stone replacement and stress concentration ratio

Figure 2. Comparison of methods for determining settlement reduction

The analyses indicated stress concentration ratios increase with increasing soil compressibility and column length (L) to diameter (D) ratios, with predicted n values in the range 2–10. Charles (1993) (see also Charles and Watts (2001)) presented an elastic analysis based on columns with $\phi'_c = 37^\circ$. The resulting design curve implied n values between 4 and 5.5 (Figure 2).

McCabe *et al.*'s (2009) view that Priebe's (1995) method with $\phi'_c = 40^\circ$ provides a safe lower bound for settlement reduction in practice suggests that stress concentration ratios between 6.5 and 8.5 also represent a range of safe lower bounds for stone columns in soft soils.

3. Stress concentration ratios achieved by stone columns in practice

3.1 Successful case histories

Case histories with field measurements of the stress concentration ratio are relatively rare. Table 1 presents data from six such case histories. In five cases, the stress concentration ratio was determined from direct measurement of vertical stress on the columns and the clay and in a single case from measurement of settlement with and without stone columns. The case histories are all examples of stone columns in very soft clays and include three test embankments, one full-scale embankment, one test loading and a storage tank.

Figure 3 presents the field measurements of stress concentration ratio (n) plotted against the undrained shear strength (C_u) of the clay soils. While the measured stress concentration ratios are similar to those predicted by Barksdale and Bachus (1983), they all lie within the lower half of the finite-element predictions. The range of measured n values is similar to that measured by Charles and Watts (1983) in large-scale oedometer tests using direct measurement of both stresses in the stone columns and the soft clay soils and settlement reduction.

There are no indications that, in practice, n increases significantly with increasing L/D , increasing compressibility, decreasing C_u or increasing stone replacement. Greenwood (1991) reported measured stress concentration ratios both increasing and decreasing with increasing bearing ratio ($q = p/C_u$). The bearing ratios for the case histories varied between 4 and 8.7 but with no indication of any significant influence on the overall average value of the stress concentration ratio.

One case history (Munfakh *et al.*, 1984) records stress concentration ratios increasing from 2.8 to 4.5 at, respectively, the end of construction and completion of primary consolidation, giving an overall mean of 3.6, which corresponds to a settlement reduction of 40%. The same settlement reduction was obtained from measured settlement of the treated and untreated ground, supporting the view that the use of Equation 8 does not introduce significant errors into the calculated settlement reduction for very soft normally consolidated

clays. The rapid primary consolidation in this case (3 months) was typical of the case histories, suggesting a relatively small contribution from secondary consolidation settlement to the measured primary settlements of the treated ground.

The case history data, as presented in Figure 4, indicate that stress concentration ratios in practice correspond to the practical upper bound settlement reduction at low values of stone replacement, but reduce with increasing stone replacement approaching the practical lower bound settlement reduction for stone replacements in excess of 25%.

McCabe *et al.* (2009) reviewed the results of a large number of successful short-term case histories for widespread loading of soft soils from predominantly embankments and storage tanks. Only two of the case histories (Greenwood, 1970; Munfakh *et al.*, 1984) were based on reliable settlement measurements for the untreated ground, the remainder being predictions based on calculation. McCabe *et al.* (2009) considered the calculated settlement reductions to generally accord with Priebe (1995) with $\phi'_c = 40^\circ$, while noting that – for the two case histories with settlement reduction based on measured settlement of untreated ground – the settlement reduction was significantly less than that predicted by Priebe (1995). From an examination of all the data, McCabe *et al.* (2009) concluded that improved settlement reduction was achieved using dry bottom-feed installation (newer case histories) than using wet top-feed installation (older case histories). As such, it was concluded that Priebe (1995) with $\phi'_c = 40^\circ$ provided a safe lower bound estimate of settlement reduction for soft soils using the dry bottom-feed method.

Debats *et al.* (2013) provide a detailed case history concerning the construction of storage tanks on ground ranging from loose silty sands to loose/soft sands/clays and very soft clayey silts. Ground improvement was carried out using a combination of wick drains, stone columns and pre-loading. Settlement reductions achieved by the stone columns were as shown in Table 2, the data from which are shown in Figure 4, indicating that, for the loose silty sand, settlement reduction approximated to Priebe's (1995) method with $\phi'_c = 40^\circ$; for the loose/soft sands/clays to just below the suggested practical upper bound and for the clayey silt to the suggested practical lower bound. Dry bottom-feed was used for the stone columns in the first stage of the project but for the second stage, reported above, wet top-feed was considered preferable.

3.2 Serviceability failures

Data from two serviceability failures are presented in Table 3 with corresponding long-term settlement observations presented in Figure 5 and Figure 6. Ground conditions for both cases were similar, comprising very soft to soft clay, with an upper firm crust overlain by historic mixed granular/cohesive fill, underlain by relatively incompressible stiff/dense soils. Both buildings consisted of a piled superstructure and a ground-bearing floor

Case history	Reference	Loading	C_u : kPa (range/median)	q (min/ median C_u)	L : m	D : m	L/D	s : m	s/D	Grid ^a	A_r : % ^b	S_r : % ^c	n ^d
Bremerhaven, Germany	Greenwood (1970)	Test embankment 45 kPa	7–15/11	6.4/4.1	6	1.2	5	2.3	1.9	TR	24.7	39 (R)	3.6 (D)
Rouen, France	Vautrain (1977)	Embankment 150 kPa	15–35/25	10/6	11	1.0	11	1.7	1.7	SQ	27.2	33 (D)	2–3, average 2.8 (M)
Hampton, USA	Goughnour and Bayuk (1979)	Test loading Concrete slabs 115 kPa	7–25/16	16.4/7.2	6.4	1.1	5.8	1.8	1.6	TR	33.9	38 (D)	2.6–3.0, average 2.8 (M)
New Orleans, USA	Munfakh <i>et al.</i> (1984)	Test embankment 60 kPa	5–25/15	12/4	19.8	1.11	17.8	2.1	1.9	TR	25.3	56 (C) 40 (D)	$n = 4.75(D)$ 2.6–3.0 initial, 4.0–5.0 final, average 3.6 (M)
Canvey Island, UK	Greenwood (1991)	Storage tank 130 kPa	~15	8.7	10	0.75	13.3	1.5	2.0	TR	22	40 (R) 52 (D) 54 (C)	$n = 3.6 (D)$ 5.6 (M) $n = 6.3 (D)$
Humber Bridge, UK	Greenwood (1991)	Test embankment 145 kPa	11–25/18	13.2/8.1	9	0.775	11.6	2.25	2.9	TR	11	31 (D) 23 (C)	5 (M) $n = 3.1 (D)$

^aTR, triangular grid; SQ, square grid

^bAll stone columns installed using wet top-feed process

^c(R) = recorded: S_r based on recorded settlement of treated and untreated areas (Bremerhaven) or on recorded settlements of treated area and extrapolated recorded settlements of untreated areas (New Orleans). (C) = calculated: S_r based on recorded settlement of treated areas and predicted settlement of untreated area. (D) = derived: S_r based on measured n using Equation 8

^d(D) = derived: n based on recorded/calculated S_r using Equation 8. (M) = measured: n values based on recorded vertical stresses in stone columns and clay soils

Table 1. Field measurements of the stress concentration ratio in
very soft clays

Figure 3. Field measurements of stress concentration ratios for very soft clays

Figure 4. Performance of stone columns in practice

slab (oversailing the pile caps) supported on stone columns bearing on the underlying stiff/dense soils. Stone column design conformed to Priebe's (1995) method with $\phi'_c = 40^\circ$, giving predicted settlements of 25 mm and 30 mm for the treated ground under the design loadings. Long-term settlement observations suggested maximum settlements, under significantly lower working loads, of 143 mm and 132 mm respectively after 25 years.

3.2.1 Serviceability failure 1

Design of the stone columns was carried out by the stone column contractor using assumed values of soil compressibility (E_s). Floor loading comprised high-bay racking with settlement-related problems occurring within months of first loading, with both tilting and distortion of the racks in the vicinity of the external walls. Settlement observations

	S_r : %	n
Loose silty sand	55	8
Loose/soft sands/clays	37.5	4.5
Very soft clayey silt	17	2.2

$A_r = 17.5\%$, $L/D = 17$, $s/D = 2.3$, n from Equation 8

Table 2. Variation in settlement reduction with fines content (after Debats *et al.* (2013))

indicated that localised serviceability failure occurred when maximum settlements exceeded about 40 mm.

The primary consolidation curve in Figure 5 was constructed to fit the long-term settlement observations and corresponded to a horizontal coefficient of consolidation (C_h) of about $1 \text{ m}^2/\text{year}$. The analysis suggested primary consolidation to be complete within about 1.5 years, with primary settlement (S_p) of 90 mm and 53 mm of long-term secondary settlement (S_s). The latter equates to a coefficient of secondary consolidation (C_α) of 0.015, being in good agreement with published field and laboratory values for very soft clays (Simons, 1974; Simons and Menzies, 2001).

Calculated primary settlement of the untreated ground under working load, using coefficient of compressibility (m_v) values based on the limited site investigation data and experience, suggested a settlement reduction of 31% to have been achieved by the stone columns, this being similar to that predicted by Priebe (1995) with $\phi'_c = 40^\circ$ and equivalent to a stress concentration ratio of 6.

Failure was the result of both a significant design underestimate of S_t and failing to consider secondary consolidation.

3.2.2 Serviceability failure 2

Design of the stone columns was carried out by the stone column contractor using m_v values from oedometer consolidation tests. Floor loading was low-rise shelving with settlement-related problems again occurring within months of first loading, in terms of both excessive slopes and changes in slope of the floor slab as a result of the restraining influence of the pile caps. Settlement observations indicated that localised serviceability failure again occurred when maximum settlements exceeded about 40 mm.

The primary consolidation curve in Figure 6 was derived similarly to that of Figure 5. In this case, primary consolidation was indicated to be complete after about 2 years with $S_t = 60 \text{ mm}$ and $S_s = 72 \text{ mm}$, again equivalent to $C_\alpha = 0.015$ and in good agreement with published data. Calculated primary settlement of the untreated ground under working load suggested a

	Serviceability failure 1	Serviceability failure 2
Soil properties		
Description	Very soft to soft silty clay, locally peaty/organic	Very soft silty clay, locally peaty/organic
C_u : kPa (range)	11–24	8–16
C_u : kPa (average)	17	12
m_v : m^2/MN	0.75–1.5 (estimated)	0.85–1.9
Equivalent E_s : kPa ^a	3200–1600	2800–1250
Loading		
Dead (net): kPa	14	6
Live (design): kPa	39	15
Total (design): kPa	53	21
Live (working): kPa	29	4
Total (working): kPa	43	10
q (working) (min/average C_u)	3.9/2.5	1.3/0.8
Stone columns (dry bottom-feed method)		
L : m	6	7.25
D : m	0.64 (design)	0.50 (installed)
L/D	9.4	14.5
s : m	1.9, square grid	2.0, square grid
s/D	3	4
A_r : %	9	5
Settlement reduction		
S_r : %	31	22
n	6	6.6
Secondary consolidation		
C_α	0.015	0.015
$\log t_2/t_1$ ($t_2 = 25$ years)	1.2	1.1
S_s/S_t	0.59	1.25

^aBased on $E_s = 2.4/m_v$

Table 3. Stone column serviceability failures

Figure 5. Long-term settlement observations – serviceability failure 1

Figure 6. Long-term settlement observations – serviceability failure 2

settlement reduction of 22% to have been achieved by the stone columns, as predicted by Priebe (1995) with $\phi'_c = 40^\circ$ and equivalent to a stress concentration ratio of 6.6.

Failure was again the result of both a significant design underestimate of S_t and a failure to consider secondary consolidation.

4. Discussion

4.1 Stress concentration ratios and settlement reductions in practice

Values of the stress concentration ratio and settlement reduction based on measured vertical stresses on stone columns and the surrounding very soft clay are towards the lower end of the range predicted by finite-element analyses. In practice, the stress concentration ratio has also been found to increase from a minimum upon first loading to a maximum at the end of primary consolidation, but with the overall average decreasing with increasing stone replacement.

McCabe *et al.* (2009) presented installation pore pressure data for stone column groups in soft clays showing that, at a distance of five diameters from the centre of a column, the excess pore pressure (Δu) generated by the column installation exceeded the original in situ effective stress (σ'_{vo}), approaching twice σ'_{vo} at the column perimeter. The data also suggested that excess pore pressures were cumulative as each successive column was installed.

McCabe *et al.* (2009) also included installation pore pressure data reported by Castro (2007), which, although following similar trends with respect to radial distance and cumulative effects, were of lesser magnitude and dissipated rapidly post-column installation. Castro and Sagaseta (2012) presented a more detailed analysis of Castro's (2007) data, showing that the soft to firm silty clay contained thin layers and lenses of sand with very high field values of the horizontal coefficient of consolidation, to which they attributed the relatively lower, rapidly dissipating pore pressures.

Based on the pore pressure data of McCabe *et al.* (2009) and Castro and Sagaseta (2012), the installation of stone column groups in very soft clays could be expected to result in initial excess pore pressures greater than σ'_{vo} throughout the treated clay. As a result, it could also be expected that $\Delta u/\sigma'_{vo}$ would increase with decreasing column spacing (s/D) and that the stress concentration ratio on first loading would correspondingly decrease with decreasing s/D .

This is confirmed by the case history data (Figure 7(a)), which show that the stress concentration ratio decreases significantly with decreasing s/D . This is in marked contrast to stone column theory, which predicts near-constant or rising values of the stress concentration ratio (Figure 2). Priebe's (1995)

Figure 7. Effect of column spacing on stress concentration ratio and settlement reduction: (a) stress concentration ratio n plotted against inverse column spacing ratio D/s ; (b) settlement reduction S_r plotted against stone replacement A_r

design curves show that, in theory, reducing ϕ'_c from 45° to 35° reduces the stress concentration ratio by about 35%. This suggests that the cause of the observed decrease in the average stress concentration ratio with decreasing s/D is a decrease in stress concentration ratio upon first loading as a result of increasing installation pore pressures and decreasing in situ effective stress.

Figure 7(b) shows that the effect of the average stress concentration ratio decreasing with s/D is to limit the amount of settlement reduction that can be achieved for very soft clays in practice to about 40% at 20% stone replacement. A suggested design curve for widespread loading on very soft clays based on the case histories is presented in Figure 7(b). The design curve equates to that of Priebe (1995) with $\phi'_c = 40^\circ$ at 5% stone replacement, then mirrors the suggested practical upper bound to about 17.5% stone replacement, beyond which, at 20% stone replacement, it peaks at a settlement reduction of

about 40%, moving ever closer to the suggested practical lower bound as the stone replacement approaches 35%.

Priebe's (1995) method with $\phi'_c = 40^\circ$ would thus appear to be valid for very soft clays with relatively widely spaced stone column groups ($s/D \geq 3$), but to become increasingly unconservative with decreasing s/D and very unconservative for stone replacements greater than about 15% ($s/D \leq 2.5$). The data of Debats *et al.* (2013) also show the Priebe (1995) method with $\phi'_c = 40^\circ$ to be suitable for relatively free-draining sands/clays with low and/or rapidly dissipating installation pore pressures as reported by Castro and Sagaseta (2012). Debats *et al.* (2013) reported settlement reduction for very soft clayey silt to be close to the suggested practical lower bound.

4.2 Influence of stone columns on secondary consolidation

When very soft clays, which typically exhibit some degree of ageing (Bjerrum, 1967), are subject to loading, primary consolidation is accompanied by an increase in the rate of secondary consolidation. It thus follows that primary settlement of very soft clays will include secondary settlement, the magnitude of which will increase with increasing primary consolidation time.

Stone columns significantly reduce primary consolidation times and should therefore reduce the amount of secondary consolidation that occurs during the primary consolidation period. In theory, the reduced vertical stress in the soil should also reduce the initial rate of secondary consolidation, albeit dependent on the magnitude of the applied vertical stress relative to the degree of ageing (apparent pre-consolidation pressure).

The case histories indicate that primary settlement reduction can be predicted using the proposed design line of Figure 7(b) despite the fact that the settlements must, in practice, include an element of secondary consolidation. The apparent lack of significant secondary settlement accompanying primary consolidation seems likely to reflect the benefit of the stone columns in both effecting rapid primary consolidation and reducing the vertical stress in the clay.

Despite the serviceability failures involving different very soft clays, the long-term settlement records are very similar and in both cases a reasonable predictive assumption would have been the commencement of secondary consolidation at completion of primary consolidation. While it is possible that the stone columns reduced the long-term secondary settlements by effectively delaying their onset, they were still of considerable magnitude.

4.3 Efficacy of stone columns for building floor slabs on very soft clays

The results of the case histories suggest that stone columns on very soft clays are most efficient for low stone replacements

with a maximum potential settlement reduction of about 40% achieved at 20% stone replacement. For a serviceability settlement limit of no more than 50 mm, stone columns are therefore unlikely to be viable for building floor slabs with respect to untreated primary consolidation settlements much greater than 85 mm.

With respect to secondary consolidation, it appears that the presence of stone columns may limit both the initial rate and overall magnitude. However, the serviceability failures suggest that, following completion of primary consolidation, a minimum allowance should be made for one log cycle of secondary consolidation for very soft clays treated with stone columns. This would equate to about 15 mm of secondary consolidation per metre thickness of very soft clay and stone columns would therefore appear unlikely to have been viable with respect to secondary settlements for very soft clay thicknesses exceeding about 3.5 m.

The serviceability failures thus suggest that, for the particular case of building floor slabs on very soft clays, the efficacy of stone columns is likely to be quite limited.

4.4 Procurement of stone columns for building floor slabs

The published successful stone column case histories predominantly relate to civil engineering earthworks, often involving surcharging and/or pre-loading. The major component of loading for earthworks is dead load and design is typically based on primary consolidation plus a degree of secondary consolidation being largely complete before any surface finishes are constructed. The benefit of any settlement reduction achieved by the stone columns is thus largely subservient to the benefits of rapid drainage and consolidation.

In contrast, the major component of loading for floor slabs of large buildings is live load. Construction times are relatively short and the finished building will generally undergo close to full primary and secondary consolidation settlements. Low serviceability settlement limits mean that risks are greater and realistic estimates of primary and secondary settlements and settlement reduction are a prerequisite for assessing the efficacy of stone columns.

In both cases of serviceability failure, a site investigation with interpretative report was procured to enable foundation design. This led to tenders being sought for the design and construction of stone columns to support the floor slabs. The only geotechnical engineering inputs were provided by the specialist site investigation contractor and the specialist stone column contractor. Only one report contained oedometer consolidation test data and neither contained any reference to secondary consolidation. In neither case were project-specific settlement limits defined and, ultimately, the stone column contractor's contracted settlement predictions were adopted. The stone

column design was provided by the contractors as part of their stone column tenders and neither considered secondary consolidation.

The failure in the serviceability case histories to provide adequate geotechnical engineering input and the stone column contractors with adequate information, either for the assessment of viability or for the design of the stone columns, is considered to reflect inadequacies in the method of procurement and to have contributed to the failures. The method of procurement is considered to have been inappropriate when considering the level of geotechnical engineering required in the design of stone columns for very soft clays and the risks associated with a failure to meet serviceability settlement limits.

5. Conclusions

From a review and comparison of the theoretical and practical performance of stone columns to support widespread loads on very soft clays it appears that the stress concentration ratio in practice decreases significantly with decreasing column spacing. This results in the settlement reduction that can be achieved being limited to about 40% at 20% stone replacement. The indications are that, while in reasonable agreement with the method of Priebe (1995) for $\phi'_c = 40^\circ$ at low values of stone replacement, the practical performance of stone columns falls increasingly short of Priebe's method as stone replacement increases.

Back-analysis of long-term settlements from two case histories of building serviceability failures suggests that, by facilitating rapid primary consolidation, stone columns limit secondary consolidation settlements during primary consolidation and, by reducing the vertical stress on the clay, may also limit the initial rate and overall magnitude of secondary consolidation. Nevertheless, large secondary settlements of the treated ground occurred in both cases.

For the serviceability failures, the low stone replacements meant that reliable estimates of treated primary settlement could have been provided by conventional calculations of untreated primary settlement combined with the method proposed by Priebe (1995). Reliable estimates of treated secondary consolidation settlements in these particular cases would have been provided by the assumption of the commencement of secondary consolidation coincident with the completion of primary settlement.

The large primary and secondary settlements associated with very soft clays and the limited settlement reductions indicated to be afforded by stone columns suggest that the range of loadings and clay thicknesses for which stone columns would be viable for building floor slabs is quite limited.

The method of procurement of the design and construction of the stone columns in the serviceability failures is considered

to have led to inadequate geotechnical engineering input to the design and to a failure to provide the stone column contractors with sufficient information for the design. These procurement deficiencies were a significant contributory factor in the failure to assess adequately the viability of the stone columns and to the subsequent failures. As such, the method of procurement is considered to have been inappropriate in relation to both the geotechnical complexity of the works and the associated risks.

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