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An evaluation of partial depth dry bottom-feed vibro stone columns to support shallow footings in deep soft clay deposits

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Colin J. Serridge, BSc (Hons), MSc.

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Dedications

This research is dedicated to both the memory of my late father James Serridge, and grandparents – Delia and Joseph Conroy and also to my mother Josephine Serridge and wife Louise Serridge for their tireless support, patience and encouragement throughout this research experience.

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ABSTRACT

FACULTY OF SCIENCE AND TECHNOLOGY

DOCTOR OF PHILOSOPHY

AN EVALUATION OF PARTIAL DEPTH DRY BOTTOM-FEED VIBRO STONE COLUMNS TO SUPPORT SHALLOW FOOTINGS IN DEEP SOFT CLAY DEPOSITS

COLIN J. SERRIDGE

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Abstract

Ground Improvement using vibro stone columns is gaining increasing acceptance on marginal soft clay sites as a sustainable foundation solution, particularly for lightly loaded low-rise structures supported by shallow, narrow footings. Most experience in this context however has been with widespread loads and use of the wet top-feed stone column technique, which has now been largely superseded, on environmental grounds, by the dry bottom-feed technique, and for which no significant published field trial data currently exists in deep soft clay deposits in the context of shallow, narrow footings. This research is therefore principally concerned with evaluating both the ground response to installation of partial depth vibro stone columns using the dry bottom-feed method in a deep moderately sensitive soft clay soil, together with the influence of parameters such as stone column spacing and length, founding depth within a thin surface 'crust', and also foundation shape on the performance of narrow footings subsequently constructed and subjected to incremental loading, over the installed stone columns, at the Bothkennar soft clay research site in Scotland. Comparisons are made with footings constructed within the surface 'crust' at Bothkennar without stone columns.

Whilst stone columns were satisfactorily constructed with the dry bottom-feed technique at Bothkennar, it was evident that the vibroflot should not remain in the ground for longer than is necessary, in order to avoid excessive soil disturbance. For this reason construction of partial depth stone columns to a more uniform diameter, without construction of an 'end bulb', is advocated. Stress ratio was found to increase significantly with increasing length of stone column and also applied load, up to a maximum value of around 4.0. Moreover, for a trial footing founded at the base of the 'crust', stresses attracted by the columns were higher than all other columns where founding depth (level) was at shallower depth in the crust. A significant stress transfer was also measured beneath the toe of columns intentionally installed shorter than the minimum design length predicted by the Hughes and Withers (1974) approach at all applied loads, but not for columns equal to, or longer than minimum design length, confirming the predictions of this laboratory-based approach at the field scale. The stress measurements recorded by the field instrumentation demonstrate that the behaviour of the composite stone column-soil-foundation system is complex, with simultaneous and interdependent changes in pore pressures, soil stress ratios and resulting stiffness of both soil and columns.

Whilst observed settlements exceeded those predicted, with larger foundation settlements observed at low applied loads over stone columns than at the same loading level in untreated ground, principally due to soil disturbance and accelerated consolidation effects during initial loading, at higher applied loadings however the stone columns significantly reduced the rate and magnitude of settlement compared to a foundation in the untreated 'crust'. It is therefore clear that the stone columns 'reinforced' the weak soil, providing a significantly increased factor of safety against bearing failure.

Keywords: soft clay; vibro stone columns; dry bottom-feed stone columns; instrumentation; stress ratio.

Exposed stone column (1.8 m depth) constructed by the dry bottom-feed method in the soft Bothkennar Clay.

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List of symbols (notation)

A; A_o area of 'unit cell' for a stone column in an infinite grid; unit area per column

a $(A_o/\pi)^{0.5}$, equivalent radius of foundation area per compaction

A_c cross-sectional area of stone column

A_c/A area replacement ratio (ratio of area of stone column to unit cell area)

A/A_c area ratio (reciprocal of area replacement ratio)

ACV aggregate crushing value

A_f area of footing

A_r area replacement ratio; area ratio

A_s area of soil within a unit cell (sometimes denoted a_s)

B width of foundation (sometimes denoted B); slope of failure plane

C_{avg} cohesion of composite foundation;

c'_{avg} average cohesion of soil

C cohesion of soil

C′ effective cohesion of soil

C & D construction and demolition waste

C_c compression index

CE extremly high plasticity

C_h horizontal coefficient of consolidation

CPTU cone penetration test using piezocone

C_u undrained shear strength of soil

C_u/P' Skempton’s equation(s)

C_v vertical coefficient of consolidation

$C\alpha$ secondary compression

D stone column diameter (also denoted *D*); founding depth; constrained modulus

d/D depth/diameter ratio

D_c constrained modulus of stone column

D_s constrained modulus of soil

d_e unit cell

D_e equivalent diameter of unit cell

d_f equivalent thickness of stone wall (Van Impe and De Beer, 1983)

E_{50} Young’s modulus at half the maximum deviator stress.

E Young’s modulus; elastic modulus

E'	drained elastic modulus
e	void ratio
E_c	modulus of deformation of stone column; Young's modulus of stone column
E_{conc}	Young's modulus of concrete
E_c/E_s	ratio of stiffness of column to soil; modular ratio
E_o	elastic modulus of composite stone column-soil in a unit cell
e_o	initial void ratio
E_{oed}	oedometric modulus
E^{ref}	reference stiffness (after Killeen and McCabe, 2010)
E_s	Young's modulus for soil material; modulus of deformation of soil
E_s/E_c	ratio of stiffness of soil to column
E_u	undrained elastic modulus ; Young's modulus under undrained conditions
E_{ur}	Young's modulus for unload-reload
F	footprint replacement ratio
F'_c ; F'_q	cavity expansion factors
f_d	depth factor (Priebe 1995)
FOS	factor of safety
F_R	friction ratio
G; Gs	shear modulus
H	soil layer thickness
h_c	critical depth
H_p	heave ratio
$\mathrm{H_p}$	heave percentage
HS	Hardening Soil
h_w	depth of water table below ground level
I	length of granular column
I_q	influence factor L = $\pi/4$ for circular plate
I_L	Liquidity index $I_L = (w-w_p)/I_p$
I_p	Plasticity index $I_p = w_L - w_p$; Displacement influence factor
I_r	rigidity index = G/(C + p tan ø), where G is the shear modulus of surrounding soil and p is the mean isotropic effectice stress of the equivalent failure depth
K	lateral earth pressure coefficient (also denoted *K*)
k	constant relating A/A_c to unit cell geometry
K_a	earth pressure coefficient

K_c earth pressure coefficient for stone column (between at rest and active)

k_h permeability in horizontal direction

k_{ho} horizontal permeability

K_o lateral earth pressure coefficient at rest; also denoted (K_o)

K_p earth pressure coefficient

K_{pc} Rankine passive (horizontal) earth pressure coefficient of the stone column

K_{ps} Rankine passive (horizontal) earth pressure coefficient of the soil

K_s earth pressure coefficient for soil (between at rest and active)

$k_{v;}$ k_{vo} vertical permeability

L length of foundation; length of stone column

L_c critical depth to bulging (typically minimum 1.5 D)

L_{cri} critical length

L/D stone column length to footing diameter/width

L/d stone column length to diameter ratio

LL liquid limit

LI liquidity index

M constrained modulus; slope of the critical state line; ratio of column length to diameter (Lc/D) after Hughes and Withers, 1974

m power for stress level dependency (used to control relationship between soil stiffness (E) and corresponding confining stresses (*p*)); moisture content

m_v coefficient of volume compressibility

N_c bearing capacity factor

n settlement improvement factor; reduction factor; ratio

n settlement improvement factor = $s_{untreated}/s_{treated}$

n_{meas} measured settlement improvement factor

n_o Priebe's basic settlement improvement/reduction factor (also denoted n_0)

n_{pred} predicted settlement improvement factor

n_1 Priebe's n_0 amended for column compressibility

n_2 Priebe's n_0 amended for soil and stone column unit weights

N_b, bearing capacity factors (based on $ø'_{avg}$)

N_c, bearing capacity factors (based on $ø'_{avg}$)

N_d bearing capacity factors (based on $ø'_{avg}$)

n_{pred} predicted settlement improvement factor

OCR overconsolidation ratio

P	surcharge; mean isotropic effective stress at the equivalent failure depth
P	perimeter ratio
ρ_b	bulk density (partially saturated)
P_c	stress on stone column; load carried by stone column
P.I.	plasticity index
P_o	imposed load from foundation; effective overburden pressure
P_s	stress on soil between stone column
P_{ult}	ultimate design load of column
P_z	piezometer
Q_b	average plate bearing pressure
Q_c	safe capacity of stone column
Q_{ult}	ultimate bearing capacity of stone column
q	deviator stress (σ_1 - σ_3); applied foundation pressure
q_a	allowable bearing pressure
q_c	cone tip resistance in cone penetration test; Dutch cone tip resistance
R; r_o	radius of stone column (or pile).
r	distance of measuring device from column centre; radial distance from column or pile centre-line
r_u	pore pressure ratio
R_u	pore pressure
S	centre to centre spacing; stone column/pile spacing
S_{col}	settlement of the granular column
SCP	sand compaction pile
SCR	stress concentration ratio
S_h	horizontal deformation of stone wall (equal to that of soft layers)
S_r	stress ratio; stress concentration ratio; settlement ratio
S_{rv}	ratio of vertical stress in stone column divided by vertical stress in soil
S_t	undrained shear strength
$s_{treated}$	settlement of ground treated with stone columns
$s_{untreated}$	settlement of ground without stone columns
S_v	vertical settlement of stone wall (equal to that of soft layers)
S/S_s	settlement ratio where S is expected footing settlement and S_s is total settlement for an unilimited grid beneath an unlimited area.
t	time

W moisture content

W_I liquid limit

W_P plastic limit

x critical depth at which end bearing and bulging failure occur simultaneously

YSR yield stress ratio (overconsolidation ratio measured in oedometer tests)

z depth of soil

α inclination of failure surface

γ unit weight of soil

γ_b bulk unit weight of soil (partially saturated)

γ_c bulk unit weight of stone column material (partially saturated)

γ_d dry unit weight

γ_s average settlement

Δ_p increase in vertical stress

Δ_q nett loading

Δ_s average settlement under plate at Q_b (average plate bearing pressure)

Δ_u excess pore water pressure

Δ_{umax} maximum excess pore water pressure; free field vertical effective stress

ε strain

ε_v vertical strain

ζ angle to the horizontal of the shear surface of a truncated cone of the soil, after Brauns (1978)

ζ_s failure angle of composite stone column-soil, after Brauns (1978)

μ pore water pressure (also denoted μ_o) ; degree of pore pressure dissipation

σ total stress; stress applied to the unit cell; major principal stress.

σ' effective stress

σ_c stress on stone column

σ_h horizontal total stress

σ'_h horizontal effective stress

σ_{ho} in-situ horizontal stress

σ_o' major principal effective stress.

σ_o average stress over unit cell; average vertical stress in unit cell

σ'_p pre-consolidation pressure

σ_r passive resistance

σ_{ro} total in-situ lateral stress

σ_s stress in surrounding clayey soil; stress in soil

σ_v ultimate column capacity; vertical toe stress

σ'_v ultimate vertical effective stress in stone column as it bulges

σ_{vc} average vertical stress applied to the stone column (or attracted by column).

σ_{vo} in-situ vertical stress

σ'_{vo} vertical effective stress in stone column; free-field vertical effective stress; effective overburden pressure

σ_{vs} average vertical stress applied to the soil within the unit cell.

σ_{vz} depth at which vertical stress is zero (Hughes and Withers, 1974); change in stress with depth within a stone column.

σ_1 major principal total stress

σ_3 minor principal total stress

σ'_1 major principal effective stress

σ'_3 minor principal effective stress

τ shear stress

τ_f shear strength or maximum shear stress

υ Poisson's ratio

υ' drained Poisson's ratio

υ_b shape factors for treated ground

υ_c Poisson's ratio of the stone column; shape factor for treated ground

υ_d shape factor for treated ground

υ_s Poisson's ratio for soil

υ_u undrained Poisson's ratio

φ' angle of shearing resistance ; angle of internal friction of stone column aggregate

φ_c angle of internal friction of the stone column material

φ'_c effective angle of internal friction of the stone column material

φ_{comp} angle of shearing resistance of stone column-soil composite

φ'_o average effective angle of internal friction of a soil-stone column composite in a unit cell.

φ'_r residual angle of shearing resistance

φ_s angle of internal friction of the stone column material

φ'_s effective angle of internal friction of the stone column material

φ_u undrained angle of internal friction

ψ angle of dilatancy.

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Chapter 1 Vibro stone column ground improvement

1.1 Introduction

Increasing pressure to develop marginal sites with underlying deep deposits of soft clay has forced geotechnical engineers and ground engineering practitioners into looking for ways to improve soft soil properties in order to make the soils suitable for foundation construction. Historically, one of the first options considered was the use of deep foundation piles, in which all foundation loads would be carried on piles normally extending through the soft soils to a competent end bearing stratum or, in the case of very deep deposits of soft clay, using friction alone. Piles have proved to be an appropriate engineering solution because they provide the necessary factor of safety to ensure that there will be no significant problems during the life of a structure. However, particularly in deep soft soil deposits, end bearing piles may be prohibitively expensive or impractical and probably unsustainable (Serridge, 2001). In the context of lightly loaded structures, such as low-rise housing, light industrial units and storage tanks, the situation whereby the most economical (and sustainable) solution is to improve the weak in-situ soil using ground improvement is arising more frequently (Serridge, 2001). Ground improvement (ground treatment) covers any method by which the ground, whether natural or disturbed in some way by anthropogenic processes, has its performance for any specific (geotechnical) purpose enhanced (Rogers, 2012). Phear and Harris (2008) state that ground improvement comprises approximately thirty different methods of ground treatment, including modification, chemical alteration, reinforcement with steel or geosynthetics, strengthening by drainage, densification by vibration or consolidation, the use of electro-osmosis and the use of the observational technique. Typically the technical goal is reinforcing and/or stiffening in the case of soft soils. Of the various ground improvement techniques attempted to date, one of the most commonly used in the UK is the vibro-stone column technique, which is increasingly being considered for the development of marginal sites with deep soft clay deposits and where some settlement can be tolerated, to provide an economic alternative solution to the traditional approach of deep foundation piles (Serridge, 2001).

Vibro stone column treatment typically comprises the placement of lines, rows or groups of stone columns beneath load-bearing walls or column bases, or on a grid pattern arrangement beneath ground bearing floor slab areas and raft foundations and other widespread loads such as embankments (Figure 1.1). Treatment points along each length of footing are generally at about 2 m maximum centres in clay soils for narrow footings (staggered rows are adopted for wider footings, > 1.0 m), typically with a wider grid spacing under ground-bearing slabs, with actual spacings dependent upon soil properties, loading and performance requirements. Figure 1.2 shows a standard stone column layout for a simple strip foundation and suspended ground floor slab arrangement (National House Building Council (NHBC) Chapter 4.6, 2011). Figure 1.3a shows a typical stone column layout beneath part of a warehouse unit where the stone columns have been installed beneath the pad foundations and intervening strip foundations together with the ground floor slab areas. Figure 1.3b shows stone columns (exposed at formation level prior to blinding and foundation construction), which have been installed in soft clay soil beneath the alignment of a wide strip foundation.

Figures 1.4a illustrates the stresses generated in the soil surrounding a single stone column installed in a soft clay subject to load and a group of stone columns under widespread loads. Stresses associated with a friction pile under load are given in Figure 1.4b. This behaviour is common to both piles and stone columns or any similar columnar structure of stiffness which differs from that of the soil. The differing behaviour of concrete piles and stone columns arises from the hugely different ratio of their stiffnesses to that of the soil. Whilst piles are about 10,000 times stiffer than the soil, stone columns are only about 2 to 20 times stiffer (Greenwood, 1991). Unlike piles the relative stiffness between column and surrounding ground can change significantly as load is applied. The weaker stone column, with no tensile strength, compresses and bulges in response to load and the contact stresses increasingly bringing into play the (passive) resistance (σ_r) of the soil, analogous to a pressure-meter (Greenwood, 1991).

1.2 Vibro techniques

As with many Civil Engineering practices and techniques, historical evidence of ground improvement application is not uncommon. Stone columns can be regarded as one such example. One of the earliest records of granular column ground improvement systems which has been located in the literature is a reference by Moreau and Moreau (1835), describing the application of 'granular' columns to soft estuarine soils to improve bearing capacity and control settlements. One of these projects used granular (sand) columns for the first time to support the heavy foundations for the ironworks at the artillery in Bayonne, France. The military engineers had been compelled to attempt this approach because timber piles (the preferred solution), would have rotted in the estuarine deposits. The columns were 2 m long, 0.2 m in diameter and supported loads of 10 kN each. It is understood that stakes were driven into the soft soils and then withdrawn with the resultant bores immediately filled with sand. On other projects crushed limestone aggregate was also used. It has been suggested that the columns reduced expected settlements by a factor of up to four. It is salutary to realise that the French Engineers had discovered that the 'granular' columns transferred their load by arching to the side of the columns, therefore there was a maximum useful length beneath isolated foundations.

Whilst the modern origins of vibro techniques were conceived in Germany in the mid 1930’s, commencing with vibrocompaction for in-situ densification of loose sands, the development of vibro-stone column techniques for finer-grained soils did not take place until the 1950’s (Slocombe, 2001). This permitted the application of vibro techniques to a much wider range of soil types (Figure 1.5), notably fine-grained soils, including soft clays (Serridge and Slocombe, 2012). Vibro stone column techniques were introduced into Great Britain and France in the late 1950’s and have been used extensively worldwide (Serridge and Slocombe, 2012). Serridge (2006b) and Serridge and Slocombe (2012), provide a good over-view of vibro ground improvement techniques.

1.2.1 Vibroflot equipment

The principal piece of equipment used to carry out vibro (ground improvement) techniques (whether for in-situ densification or construction of stone columns) is the vibroflot (also referred to as a vibrating poker or depth vibrator), which is either suspended from a crawler crane or mounted on a leader attached to a base machine, dependent upon the specific application. The essential features of the vibroflot are presented in Figure 1.6. The vibroflot equipment comprises three main components – the vibrator head, the isolator and the extension tubes. The main component used to achieve compaction is the vibrator. Vibrations are produced close to the base or tip of the vibrator, principally in the horizontal plane. These are induced by a series of rotating internal eccentric weights mounted on a shaft driven by a hydraulically or electrically powered motor located in the upper part of the vibrator casing. Follower or extension tubes of similar or smaller diameter to the vibrator are attached to permit treatment of soils to varying depths. An elastic coupling isolates the vibration from the extension tubes. Stabilising fins maintain stability of the vibroflot in the bore, by preventing rotation influenced by torque when the vibroflot is worked hard in the ground, which would otherwise lead to twisting or snagging of any external hydraulic or electric cables. The fins also assist in the transmission of vibration to the introduced stone aggregate. Whilst the basic components of the equipment have changed very little over the years (Figure 1.7), there have been significant developments in the reliability (with extended life-spans and reduced maintenance) and power ratings of the equipment, with the objective of achieving greater efficiency in densification and stone column production (Slocombe et al., 2000).

1.2.2 Applicability

The vibroflot may be used for both in-situ compaction of cohesionless soils and for forming stone columns in fine-grained soils, as described previously (Figure 1.5). It is generally recognised that in-situ vibro compaction is appropriate for granular soils with a total fines content (particles finer than 0.06 mm) of not more than 15% of which the clay and fine silt content (particles smaller than 5 microns) should be less than 2% (Slocombe et al., 2000). The actual mechanism of improvement is a function of whether the soils are essentially granular (coarse grained) or apparently cohesive (fine-grained) and by inference, in the case of the latter – slow draining.

The principle of vibro techniques in granular soils is based upon particles of coarse grained (non-cohesive) soil being re-arranged into a denser state by means of the dominantly horizontal vibration(s) from the vibroflot (vibrating poker). The resultant reduced voids ratio and compressibility and corresponding increased angle of shearing resistance result in the acceptance of higher loadings (readings) with lower associated settlements and increased seismic resistance. Within fine-grained soils the cohesion between soil particles (more specifically pore water effects in slow draining soils), dampens the vibrations and prevents re-arrangement and compaction occuring. Improvement is achieved by 'reinforcing' the soil with 'stiffer' stone column elements.

During the stone column construction in fine-grained soils the introduced coarse granular column material is pressed radially into the soil so that it is displaced beyond the diameter of the vibrator. The column of compacted dense granular material forms, together with the surrounding soil, a composite stone column-soil mass, with enhanced shear strength and bearing capacity, together with a corresponding reduction in settlements, attributed to the 'stiffening' effect of the stone columns. Stone columns in fine-grained soils also assist in the dissipation of excess pore water pressure under applied load or surcharge, which accelerates the consolidation process, together with providing stability control (Slocombe, 2001; Serridge, 2008). The vertical free-draining granular columns with high angle of internal friction, which effectively act as 'reinforcing elements' enhance soil undrained shear strength and factor of safety against bearing capacity failure.

1.2.3 Construction methods

The terminology for vibro stone column techniques has not always been applied in a consistent manner (with the terms vibrocompaction, vibroflotation, vibroreplacement and vibrodisplacement all applied to stone columns) and some attempt at addressing this can be found in Building Research Establishment (BRE) Document BR 391 – Specifying vibro stone columns (2000). This specification employs terms that reflect the fundamental principles of top or bottom-feed to describe the method of stone aggregate supply or delivery, and wet or dry to describe the 'jetting' medium. This has given rise to the following terminology:

- *dry top-feed*
- *dry bottom-feed*
- *wet top-feed*

All of the three vibro stone column techniques use similar types of vibroflots, (normally hydraulically or electrically driven). The stone aggregate is typically handled by front end loader (excavator) shovels with a side tip facility. The construction technique adopted is a function of ground conditions, bore stability and environmental constraints.

The *dry top-feed* technique (Figure 1.8a and 1.8b) can be used in a range of soil types, but relies on the bore remaining stable during column construction, with a general absence of groundwater within the treatment depth range. The vibroflot penetrates the soil by shearing and displacing the soil around it. Air flush via jets in the nose cone of the vibrator are used to overcome suction forces. Upon reaching the required depth the vibroflot is then completely withdrawn from the bore to facilitate introduction of a charge of granular backfill (aggregate) from the surface. Aggregate is tipped into the bore and the vibroflot then re-penetrates to within a short distance of the original depth displacing the aggregate laterally and downwards, thus compacting it. The procedure is repeated in approximately 0.5 m lifts until the stone column construction is completed to the surface. Stone column diameters of about 500-600 mm are typically achieved, dependent upon soil and vibroflot equipment properties (Slocombe et al., 2000; Serridge and Slocombe, 2012). In the context of fine-grained soils the technique is typically recommended for clays with undrained shear strengths in the range 30-60 kN/m^2 (Greenwood and Kirsch, 1984).

The *dry bottom-feed* technique (Figure 1.9a and 1.9b) is used in weak typically fine-grained soil profiles with a high water table, where bore stability cannot be guaranteed during stone column construction with adoption of the conventional dry top-feed method. The dry bottom-feed technique is typically employed in soils with undrained shear strengths in the range 15-30 kN/m^2. The vibroflot penetrates the soil in the same manner as in the dry top-feed technique, the main difference being that the vibroflot has a stone feed pipe attached to it, allowing aggregate to be introduced to the tip of the vibroflot, without it having to be removed from the bore. Hence the vibroflot by remaining in the ground supports the potentially unstable bore. An air lock in the tremie pipe system allows compressed air to assist discharge of aggregate. The vibroflot unit is mounted on leaders attached to a dedicated base machine which provides pull down assistance during penetration of the vibroflot into the ground. During initial penetration the stone tube is charged with stone aggregate and when the required depth is reached the vibroflot is withdrawn a short distance from the base of the bore and a charge of stone aggregate is discharged (with air pressure assistance) at the tip of the vibroflot, which is subsequently compacted on re-penetration of the vibroflot. This cycle is repeated, in approximately 0.5 m lifts, until a compact stone column is constructed to the surface. Since the technique is employed in weaker soils than those in which the *dry top-feed* technique is used, stone aggregate consumption is higher and stone column diameters larger than that associated with the *dry top-feed* technique and typically in the range 600-800 mm based upon discussions with Specialist Ground Improvement Contractors.

Historically the *wet top-feed* technique (Figure 1.10a and 1.10b) has been employed in saturated fine-grained soils (with undrained shear strength less than 15-20 kN/m^2). To a large extent it has now been replaced by the dry bottom-feed technique on environmental grounds (Serridge, 2001), but is still being employed in saturated granular soils deemed too silty for in-situ vibro compaction and in some very soft sensitive clay soils. The vibroflot penetrates to the required depth using a combination of the vibratory action of the vibroflot and high pressure water jetting via the nose cone in the vibrator head. When the required depth is reached, the vibroflot is sometimes surged up and down to flush out the bore. The vibroflot is then held a short distance off the bottom of the bore and the water pressure reduced sufficiently to allow a nominal outflow of water (and suspended fines) at the surface, such that excess hydrostatic pressure and outward seepage forces support the uncased bore sufficiently long enough to form an annular space surrounding the vibroflot and permit construction of a stone column. A charge of aggregate is introduced, while the vibroflot is still in the bore, down the annulus against the continuing low pressure upflow of water. This is subsequently compacted by re-penetration of the vibroflot into the introduced aggregate. The vibroflot is then lifted sufficiently to allow introduction of further charges of aggregate, which are then compacted and the cycle repeated until a continuous column of compacted stone is formed to the surface. Typical gradings for the granular material are within the range 25 mm to 75 mm. Because an element of the soil is removed or flushed out with this technique, larger stone columns are typically achieved with the wet method than with the dry techniques, with column diameters typically in the range 750-1100 mm, based upon discussions with Specialist Ground Improvement Contractors. The wet top-feed technique requires consideration of provision of water supply (there is a high water demand) drainage ditches, settlement lagoons and contamination of the site surface with fines as well as final disposal of effluent in a manner acceptable to the statutory authorities.

It is perhaps important to recognise that apart from the differences in diameter for the dry and wet vibro stone column techniques described above, the principal difference between the dry process and the wet process is in the way in which the bores are formed and stone columns subsequently constructed. In the dry process, since the soil is displaced sideways and not removed from the bore, local shearing inevitably takes place and hence plastic deformation. In the wet process, the removed soil space can be viewed as being occupied by the stone aggregate and the compaction during the stone column installation pushes the stone further into the periphery soil until an equilibrium is reached. Therefore, the surrounding soil undergoes less significant shear strain as a result of the attenuation of both vibratory stresses and ramming pressure (Greenwood, 1976a).

Vibro stone column techniques are increasingly being used in near-shore marine applications, working off barges or pontoons, sometimes using larger cranes reaching out from existing quays. A useful review of vibro stone column techniques in marine applications is provided by Serridge (2010).

1.3 Other granular column techniques

Other construction methods have also been used to construct granular columns. The Sand Compaction Pile (SCP) was developed in Japan in the 1950’s, independently of the vibro stone column technique in Europe and the United States and was originally designated for loose sandy soils for stability and settlement control, and to mitigate liquefaction potential during earthquake events. Murayama (1962) established the application of the SCP method in soft clays and since the 1970’s its application has become one of the mainstreams of ground improvement techniques for soft coastal marine clays for both offshore and on-shore structures (principally coastal reclamation projects) in Japan, for stability and settlement control. Whilst in excess of 500 research papers on many aspects of the SCP method have been published in international and domestic conferences and journals (Kitazume, 2005), because many research studies have been written in Japanese the accumulated research efforts, practical experience and know-how, have not been widely disseminated outside Japan. However, Aboshi et al. (1979) have provided a very good overview of the applications of the SCP method in soft clays and Kitazume (2005) provides a very comprehensive state-of-the-art review of the SCP method in Japan. The SCP method is essentially a more sophisticated version of a casing driver (Figure 1.11). The technique involves driving a steel casing (usually a 600 mm diameter thick-walled steel tube or pipe), with a plug of sand at its tip, to the base of the soil layer to be improved, using a heavy vibrating hammer. The casing is filled with sand and withdrawn in short lifts, compacting the sand as it is partially re-driven and enlarging the sand column diameter. The cycle is repeated until a compacted sand column reaches the ground surface. An automatically-controlled SCP driving system was developed in 1981 accommodating the vibration effect on soil properties. According to Kitazume (2005), the design method (approach) for SCP is similar to that for stone columns.

Different methods of granular (stone or sand) column construction have been developed in India and these include the rather crude but practical method of bored stone column and pre-assembled stone column construction techniques (Nayuk, 1982). Whilst the operation was quite slow, it has been useful in developing countries utilising indigenous equipment in contrast to the more sophisticated vibroflot technology. However, there is evidence to show that modern vibroflot technology is being increasingly used in India with associated faster and more efficient construction in a range of applications in soft ground (Raju, 1997).

Encased granular columns can be used as a ground improvement and bearing system in very soft (ultra soft) soils, for example peat or sludge with undrained shear strengths as low as 2 kN/m^2 (Kempfert, 2003), and where there would be concerns over satisfactory lateral restraint for conventional stone columns. The concept of encasing stone columns in geotextiles (geotextile wrapped or encased columns) can be traced back to work by Van Impe and De Beer, 1983; Van Impe, 1989. Currently, the most commonly used method appears to be a displacement method (Geotextile Encased Columns (GEC)), (Figure 1.12a and 1.12b) in which a steel pipe with two base flaps is vibrated into the ground displacing the soft soil. Upon reaching the required design depth a geotextile sock is introduced into the inside of the casing and subsequently filled with sand. The casing is then vibrated out of the ground compacting the sand and leaving the geotextile encased column (GEC) behind. The GEC's are arranged usually in a triangular grid pattern. The typical diameter of the columns is 800 mm and axial spacing of the columns is typically 1.7 to 2.4 m, hence the resulting replacement ratio ranges from 10 to 25% (see also Raithel et al., 2002). GEC's are typically installed to the level of an end bearing layer (Alexiew et al., 2008).

1.4 Application of vibro stone columns in soft soils

While there are no upper limits regarding the depth of treatment by vibro stone columns, usual values fall within (though not limited to) the range 3 to 15m (McKelvey and Sivakumar, 2000). Historically, it has been suggested that the dry top-feed technique should not be used in fine-grained soils with undrained shear strengths of less than 30 kN/m^2 (Greenwood and Kirsch, 1984; NHBC Chapter 4.6, 1989; NHBC Chapter 4.6, 2011 among others). For soils weaker than this, the wet top-feed technique would have been required (historically), although as intimated previously, this has been largely superseded by the dry bottom-feed technique on environmental grounds (Serridge, 2001). In terms of an acceptable lower-bound undrained shear strength, a value of 15 kN/m^2 has typically been used for vibro stone column applications (see BRE BR 391 Specifying vibro stone columns, 2000). However, developments in vibroflot technology (together with monitoring and quality control systems) have permitted much weaker soils to be treated in certain applications (Raju, 1997; Wehr, 2006), with undrained shear strengths as low as 4-5 kN/m^2, in Poland, Germany and Malaysia, although it should be recognised that this has been under earthwork embankment structures, where large settlements are typically anticipated and with the objective(s) of accelerating consolidation settlements (in a similar manner to pre-fabricated vertical band drains), and providing stability control. Wehr (2006) emphasises the importance of automated monitoring of the construction process if such low-strength soils are to be treated. Consideration also needs to be given to soil sensitivity when selecting the most appropriate technique as saturated sensitive soils can undergo significant disturbance and remoulding when exposed to vibrations. It is recommended that the vibro stone column technique should not be used in soils having sensitivity values (ratio of undrained shear strength to remoulded shear strength) greater than 5 (Baumann and Bauer, 1974; Goughnour and Bayuk, 1979a; Mc Kelvey and Sivakumar, 2000).

There are two principle applications of vibro stone columns:

- *Structural Foundations*: The stone columns permit the adoption of conventional shallow foundations for housing developments, industrial warehouses, supermarkets (including ground bearing floor slabs in the case of the latter) etc. Although the use of the stone column technique in peaty or organic soils are generally avoided (see BRE BR 391 Specifying vibro stone columns (2000); Serridge and Slocombe (2012)), sites containing thin, superficial organic or peat layers may also benefit from improvement by vibro-stone columns but only if the ratio of layer thickness to column diameter is less than two (Barksdale and Bachus, 1983, BRE BR 391 Specifying vibro stone columns, 2000). Slocombe (2001) provides some guidance on what can be achieved in terms of bearing pressure and settlement with stone columns in soft alluvial clays - bearing pressures in the range 50-100 kN/m^2 and a settlement range of 15 to 75 mm are quoted for normal foundations (see Table 1.1).

- *Embankment support and stability* : Vibro stone columns have also been used for improving soft soil beneath highway and railway embankments and in such cases the shear strength of the column is of utmost importance. In addition to strengthening the toe of the embankment, the stone columns prevent rotational and linear type stability failures. Settlements associated with stone column reinforced soft soils beneath embankments are significantly higher than under structural foundations (as previously intimated). However, the bulk of the consolidation settlement is expected to take place during, and shortly after, completion of embankment construction (typically within 3-6 months of embankment construction), due to the significantly shorter drainage paths provided by the stone columns for effective pore-water pressure dissipation during staged load application. Hence the remaining primary consolidation settlements for the finished road or rail track surface (for example) should typically fall within around 25-50 mm (Mc Kelvey and Sivakumar, 2000).

In both of the foregoing cases it is necessary to ensure that settlements fall within normal serviceability limits for the structure and consideration should be given to the implications of any secondary consolidation creep, particularly if one is dealing with very soft or organic clays. The majority of UK vibro stone column projects are either in filled, often cohesive (fine-grained) soils or in natural soft fine-grained soils, where it is often very difficult to quantify engineering design parameters. As intimated previously the vibro stone column technique is generally intended to 'reinforce' and stiffen the soil and in so doing seek out weak spots in order to enhance bearing capacity and control total and differential settlements. Appropriate soil geotechnical characterisation is therefore important.

1.5 Ground characterisation

Because of the large influence of the soil conditions on performance, ground improvement using vibro-techniques requires a more extensive site and ground investigation programme compared to more conventional deep foundation solutions (Serridge, 2008). The most common types of ground-related problems encountered relate to soil strata boundaries, i.e. geometry not as anticipated, and the geotechnical properties (characterisation) of the soil profile. Site and ground investigation requirements for vibro-techniques are covered by various documents, e.g. BRE BR 391 Specifying vibro stone columns (2000). According to Egan (2008), a carefully planned site investigation using the piezocone (CPTU) enables the ground to be quickly and cost effectively characterised in soft ground and pertinent design parameters to be obtained for the vibro design. The derivation of constrained modulus (*M)* and over-consolidation ratio (*OCR)*, may readily be undertaken on the basis of the CPTU data obtained and the following recommendations detailed below should, according to Egan (2008), be adopted when planning a CPTU investigation for the design of vibro stone columns in soft clay:

- 'The investigation should always be undertaken using a piezocone (CPTU) as the static cone does not enable the full range of parameters to be derived;

- The CPTU investigation should always be supplemented by boreholes to confirm the CPTU interpretation;

- Where experience allows the derived design parameters should be benchmarked against those obtained at similar sites.'

1.6 Specifications

The traditional use of shallow foundations for low-rise structures (particularly for housing) has helped to shape the local ground improvement (treatment) industry in the UK (BRE IP 5/89 (1989)). Much expertise is kept within Specialist Contractors and larger or specialised Consultants (BRE BR 391, 2000) and despite the publication of specifications in the 1980's and 1990's in the UK (albeit at least 20-25 years after introduction of the vibro stone column technique into the UK), it has only been in more recent specifications, e.g. BRE BR 391 (2000), where lack of guidance on vibro stone column techniques has been addressed. In terms of the chronology of UK Specifications, there have been four primary vibro stone column specifications:

- *ICE Specification for Ground treatment (1987)*. It appears that the desire or objective at the time was to have a short, straightforward specification without unnecessary clauses dealing with contract condition matters and as far as possible to keep the document technical. This gave rise to two documents: one containing the specification and one containing the notes for guidance. The specification itself covered: General Requirements; Vibrocompaction/ Vibroflotation; Vibrated stone columns; Dynamic compaction/consolidation; Deep drains and Testing ground treatment.

A 'Particular Specification' was built into each of the process (technique) and testing sections. This arrangement of the document allows a Specifier to insert, where applicable, the design loads and settlement criteria and the particular requirements for the project in question. There was also some recognition of the importance of ground treatment and the guidance notes do have a fairly significant paragraph on the topic. However, it is evident that the documents are quite contractual (conflicting to some extent with the original objective of being technical), and whilst they do deal with materials and workmanship associated with stone columns they do not attempt to specify design criteria. The Notes for Guidance describe suitable materials and gradings for the stone aggregate and state that stone columns shall be located within 150 mm of the plan positions shown on the layout drawings. The notes for guidance form a reasonable, although not detailed, discourse on ground treatment for engineers new to the processes.

- *NHBC Chapter 4.6 : Vibratory Ground Improvement Techniques.* This was first published by the National House Building Council (NHBC) in 1989. The document lays down the technical requirements and performance standards for the use of vibro techniques acceptable under the NHBC warranty scheme and was intended for supervising engineers and professionals associated with the procurement, execution and supervision of vibro stone column techniques. The latest update in 2011 involved consultation with two of the largest Specialist (Vibro) Ground Improvement Contractors in the UK and revision was deemed necessary for four reasons:

  1). The technique was being used in ground conditions not envisaged during the original drafting and this has manifested itself as an increase in costly claims.

  2). There was a need to take account of newer vibro techniques (i.e. dry bottom - feed technique) and current industry practice.

  3). Some aspects of the NHBC performance standards were not being fully adhered to, e.g. the need for appropriate validation testing and the need for a suitably experienced independent (Supervising) Engineer.

  4). The sustainability agenda started to promote the use of recycled materials.

The NHBC document concentrates on the use of stone columns and applications for low-rise housing. Two foundations are recognised: a strip foundation - suspended floor slab arrangement and a raft foundation. Full depth treatment of made ground deposits is required with the former and partial depth treatment permitted with the latter. Until relatively recently, NHBC Chapter 4.6 was the only relatively detailed document to cover a broad range of design issues associated with vibro stone columns, including site investigation; suitability of ground conditions and materials and workmanship. Consequently NHBC Chapter 4.6 has been quoted widely when specifying vibro works outside the particular application for which it was intended.

- *BRE BR 391 Specifying Vibro Stone Columns (2000)*. This document was prepared with guidance from a steering group which included senior representatives from the main UK specialist ground improvement contractors plus wider industry consultation. This specification (with incorporated notes for guidance) was intended for use by Specialist Contractors, Consulting Engineers and other building professionals concerned with the design, procurement and supervision of vibro stone column treatment. It provides a technically prescriptive specification for vibro stone columns, including design procedures (not evident in earlier specifications), which is based on accepted best practice. The specification is appropriate for most general applications. The Notes for Guidance and information provide supporting technical information for the provision of Site Investigation data, design of treatment and verification of design in order to achieve an optimum solution for effective economic ground improvement (treatment) by the installation of vibro stone columns. The Notes define recognised and agreed limits of applicability for particular variants of the process in certain ground conditions and strongly encourage the provision of adequate and appropriate site investigation information on which a safe and economic vibro design can be properly based. Guidance is given on the suitability of imported materials for stone columns, and associated works and the purpose and choice of quality control and assurance measures are explained.

It is interesting to note that BRE BR 391, like the 1987 ICE Specification, acknowledges, that whilst thicker peat deposits cannot be accommodated by vibro stone columns, thin deposits can be given consideration. Thickness, depth and lateral variations of such deposits have to be considered very carefully in relation to the size of foundation and its loading. To the author's knowledge on some sites covered by up to 2.0 m of peat, the peat has been excavated along the lines of the foundation over a 2.0 to 3.0 m width and replaced with granular fill, vibratory stabilisation then being carried out in the normal way. Alternatively narrow widths have been used, more closely aligned to footing dimensions, but with the stone filled excavations lined with a geotextile basket, (Bevan and Johnson,1989), Figure 1.13a and 1.13b.

- *BS EN 14731 Execution of Special Geotechnical Works: Ground treatment by deep compaction (2005).* This European Standard is described as applicable to the planning, execution, testing and monitoring of the following ground treatment and methods: deep vibratory compaction to densify the existing ground; vibrated stone columns to form a stiffened composite ground structure by insertion of a densified granular material with a diameter greater than 0.6 m and less than 1.2 m; methods in which depth vibrators, containing oscillating weights which cause horizontal vibrations, are inserted into the ground; methods in which compaction probes are inserted into the ground using a vibrator which remains at the ground surface and which in most cases oscillates in a vertical mode.

Whilst the Specification does not contribute anything more than BRE BR 391 (2000) and is less prescriptive, the document does make reference to the new European Standards in respect of aggregates, which BRE Report No. 391 (2000) does not and it advocates the use of recycled aggregates providing that quality control measures comparable to those applied to primary aggregates are adopted. Additionally, whilst advice on post treatment testing is similar to that given in BRE Report No. 391 (2000) the document makes the important point that the time that has elapsed between treatment and testing will have a significant impact on the test result, i.e. in the context of pore pressure dissipation in soft clay soils.

1.7 Environmental considerations

Whilst it is rare for noise issues to be a problem with vibro-stone column techniques, vibration levels need to be considered when working close to existing structures and services so that these are not adversely affected. The safe working distance would appear to depend on a number of factors including type of ground, the vibroflot power rating and the nature and state of repair of the structure, particularly if any basement structures are present, and also plant access. Discussions with geotechnical specialists suggest that as a guide, minimum stand-off distances of 1-3 metres are often appropriate. However it is evident that each situation must be assessed on its own merits and pre-condition surveys should be considered with appropriate observation and documentation prior to commencement. In appropriate cases, expert advice should be sought prior to commencement of treatment. Where required, vibration monitoring may need to be carried out during the treatment process with prior agreement of threshold (vibration) levels.

Penetrative ground improvement techniques such as vibro-stone columns can generate potential pathways for contaminant migration on brown-field sites. This raises environmental concerns, particularly if sensitive ground waters or underlying aquifers are present due to potential pollutant linkages, i.e. source-pathway-receptor linkages. An innovative approach to address this issue has been the development of 'vibro-concrete plug' technology, incorporating the introduction of lean mix concrete into the basal section (toe) of the stone column, thereby isolating any pathways for downward migration of contamination via the stone columns. Further guidance on pollution prevention in this context can be found in Environment Agency Report NC/99/73 (2001) and Serridge (2006a).

1.8 Monitoring, testing and quality control

BS EN 14731 (2005) states that a plan for the supervision of vibro stone column works should be available at the ground treatment (improvement) site and as a minimum, the following shall be noted (recorded): written procedures, which include a list of critical control parameters; site and ground conditions, and significant departures from the design basis; any obstructions in the ground which hinder or prevent penetration of the ground by the vibrating tool. Evaluation of the effect on the design of any changes to specified treatment procedures where unforeseen conditions are encountered or new information about soil conditions becomes available, together with agreement of actions prior to the change being made is also highlighted in BS EN 14731 (2005). Other parameters which it is considered advisable to monitor and record, and which it is also intended to address in the research objectives, include:

- depth of penetration at each location;
- time required to reach maximum depth and details of times and depths during withdrawal;
- vibrator power consumption during penetration and compaction of granular material or soil for depth vibrators;
- any unforeseen conditions and obstructions encountered;
- presence of heave or settlement of ground surface;
- the quantity of stone aggregate used in the construction of each column;

The majority of vibro-stone column projects in the UK involve the support of structures. Therefore some form of post-installation validation testing is typically required. Typical tests include plate load tests; zone load tests and embankment loading (surcharge load) tests. Moseley and Priebe (1993) provide a tabulated suitability rating for testing in the context of vibro-techniques (Table 1.2).

Short duration plate load tests are carried out as a fairly routine procedure using a circular steel plate with diameter comparable to the design stone column diameter, normally standardised at around 600 mm, and typically completed within two hours. Load is applied to the test column to between 1.5 times and 3.0 times the design pressure over the plate area, and is then followed by unloading, to determine load-deformation behaviour. The load is often applied via a hydraulic jack system using a crane or vibro installation rig as reaction. The tests are principally regarded as a quality control test to assess level of workmanship with regard to stone-column installation (see Figure 1.14). They can also be used for determination of stone column deformation modulus, but cannot be used for design or to predict long-term movements of structures which stress a large number of columns and the intervening ground.

To predict movements of structures built on the treated ground it is necessary to load a representative area that includes a number of columns and the intervening or surrounding ground in the same way that the structure will apply the load to the treated (stone column reinforced) ground. It is also necessary to maintain the load for a reasonable period to obtain an indication of the rate of settlement (creep) in the long-term, subsequent to the immediate response of the application of the load. Such tests are called *zone load tests or area tests* (Figure 1.15). A concrete slab is typically cast over a number of columns and loaded to 1.5 times (or more) working load. The tests are more meaningful than the plate load tests since they apply load to the composite structure of both stone columns and soil, but are far more expensive because of the cost of importing kentledge to site.

Skip tests (Figure 1.16) *or dummy foundation test* provide an intermediate test between zone load tests and plate load tests and may be more practical and economic on smaller projects, such as low-rise housing projects. A model footing can be cast over two stone columns (and intervening soil) and loaded with kentledge. However, this approach has been largely superseded by a dummy foundation test employing a portable (stiffened steel plate) footing, with the vibro stone column installation rig used as reaction load. In the skip test a small area is loaded by a waste skip filled with sand. Larger stresses can be applied by placing a second skip on top of the first, but are limited typically to loads from walls of two-storey housing. For practical and Health and Safety reasons the dummy footing test is more commonly used than the skip test.

Embankment loading tests (surcharge load tests) by virtue of a wider and greater stress depth influence can impose loads over a larger and deeper area than plate load tests, dummy foundation tests and zone load tests for a longer period of time. These are particularly useful for road embankments on soft ground where time-dependent performance may be important, to ensure post road construction settlements fall within acceptable limits. As with untreated soft soils, the rate of loading should be carefully controlled and monitored via appropriate field instrumentation. This size of test facilitates better assessment of the performance of widespread loads.

Until relatively recently (mid-late 1990's), there have been no test methods available to adequately measure improvement in soil stiffness resulting from stone column installation in soft clay soils. The Continuous Surface Wave (CSW) technique, Matthews et al. (1996), Menzies and Matthews (1996) have a number of potential advantages over other testing methods. The equipment is portable and data acquisition rapid and is non-intrusive. Furthermore, it is argued that the non-invasive nature of surface wave testing for the measurement of stiffness means that the area under test is not disturbed and that the stiffened formation created by the ground improvement is not destructured by the measuring technique. Destructuring of the ground under test is thought to be a contributing factor where traditional measurement techniques (pressuremeter or static cone penetration testing using the piezocone), do not indicate post treatment increase in stiffness. An example of the effectiveness of the CSW technique is demonstrated in Figure 1.17 for a site in Kilwinning, Scotland (UK) where vibro stone columns were installed in historic soft silty clay fill (made ground). Immediate stiffness increase in the cohesive soils after stone column installation can be observed with the CSW technique, undertaken by the researcher (author), which contradicts popular thinking on the issue. Moxhay et al. (2001) also provide a few useful examples of application of the CSW technique in the context of vibro stone column projects.

1.9 Design aspects (terminology)

Settlement analysis approaches consider that a typical stone column in an infinitely large group acts in concert with its tributary area of soil and the columns and soil together are described as the *unit cell*. The true hexagonal tributary area is approximated as an equivalent circle (Figure 1.18a). Thus the ratio of the stone column area to the unit cell area, i.e. the proportion of native soil area replaced by stone column, and which is referred to as the area replacement ratio (A_r) Figure 1.18b is given by:

$A_r = A_c / A_o$ -- 1.1

where: A_c = section area of stone column; A_o (sometimes defined as A) represents frequency of stone columns per unit area of foundation. A_s in Figure 1.18a is the remaining area of soil within the unit cell. The reciprocal ratio A_o/A_c is described as the area ratio,

The area ratio is close to the square of the column diameter D over an equivalent diameter of the unit cell D_e. For an equilateral triangular layout of stone columns D_e is equal to 1.05S and for a square pattern D_e is equal to 1.13S, where S is the centre to centre spacing between stone columns. Balaam (1978) examined these approximations and concluded that they were sufficiently accurate to be used with confidence. In the context of settlement analysis the basic assumptions of the unit cell idealisation are as follows :

- Vertical surcharge stresses remain constant over an infinitely wide loaded area ensuring theoretical validation of the concept of a unit cell.
- Shear stresses at the boundaries of the cell are insignificant so that boundaries can be approximated to be smooth (frictionless).
- Settlements for both the column and the clay are equal in the unit cell.
- The principal stresses in the unit cell are vertical, radial and tangential.
- Rigid boundaries persist.

The model of a unit cell loaded by a rigid plate is thus analogous to a one-dimensional consolidation test, in that the unit cell is confined by a rigid frictionless wall and the vertical strains at any horizontal level are uniform.

The working unit cell area (area of influence of one stone column) is defined as:

$$A_o = A_c + A_s \quad \text{--} \quad 1.2$$

where,

A_o - unit cell area,usually idealised as circular (Figure 1.18a and b)
A_c - area of stone column
A_s - area of soil within a unit cell

For the composite unit cell the vertical forces are in equilibrium so that,

$$\sigma_o \, . \, A_o = \sigma_{vc}.A_c + \sigma_{vs}.A_s \quad \text{--} \quad 1.3$$

where,

σ_o - average vertical stress on unit cell
σ_{vc} - average vertical stress applied to the stone column

σ_{vs} - average vertical stress applied to the soil within the unit cell

The distribution of the stress in the column (σ_{vc}), soft clay soil (σ_{vs}) and average stress (σ_o) is illustrated in Figure 1.18c. Studies have shown that when soft ground reinforced with stone columns is loaded, stress re-distribution occurs. This can be explained by the fact that when loaded the vertical settlement of the stone column and surrounding soil is approximately the same, leading to stress concentration in the stone column, which is stiffer than the surrounding cohesive soil. A stress concentration ratio (SCR or S_r) (or stress concentration factor -SCF) is used to express the distribution of the vertical stress within the unit cell and is defined as:

$$SCR = \sigma_{vc}/\sigma_{vs} \quad \text{------------} \quad 1.4$$

The magnitude of the stress concentration is influenced by the relative stiffness of the stone column and the surrounding soil. The average stress (σ_o) over the unit cell is given by:

$$\sigma_o = \sigma_s a_s + \sigma_s(1\text{-}a_s) \quad \text{------------} \quad 1.5$$

$$\text{i.e. } \sigma_o = \sigma_s \quad \text{------------} \quad 1.6$$

For a given stress concentration ratio, the stress on the stone column (σ_c) and the stress in the surrounding clayey ground (σ_s) are given as follows:

$$\sigma_c = SCR\sigma_o / 1+ (SCR\text{-}1)\ a_s \quad \text{------------} \quad 1.7$$

$$\sigma_s = \sigma_o / (1+ (SCR\text{-}1)\ a_s \quad \text{------------} \quad 1.8$$

where:

σ_o = stress applied to the unit cell due to applied load
a_s = area replacement ratio

Additional useful diagrams relating to stone column layouts and unit cell idealisations are provided in Figures 1.19 and 1.20.

1.10 Foundation design over stone columns

Table 1.1 (Slocombe, 2001) provides a guide as to what can be achieved in terms of load bearing capacity and settlement performance for a range of soil types and applications with adoption of stone columns. It is important to reiterate that each site has to be assessed on its own merits, including consideration of the most appropriate foundation, whether rigid or flexible, based upon prevailing ground (soil) conditions and performance requirements.

Because the stone columns are formed by working against the overburden pressure the stone aggregate at the top of the constructed columns is not as compact as at depth. For this reason the main load bearing foundations are typically placed at a minimum depth of 600 mm below the level from which the ground improvement was carried out in order to fully realise the stone column performance. In cohesive soils in particular it is recommended that there should be appropriate allowance for minimum top and bottom mesh reinforcement. The implication is that a strip footing (foundation) should be designed as a continuous beam on unyielding rigid supports, indeed this is the implication in NHBC Chapter 4.6 (2011). However, stone columns yield and the load is shared in some proportion between columns and the intervening soil, and this recommendation may be inappropriate. Prior to the publication of the NHBC Chapter 4.6 guidelines it was not uncommon for strip foundations to residential blocks to be constructed without reinforcement or with nominal bottom reinforcement only. Wood et al. (1996); Watts et al. (2000) investigated the performance of strip footings on heterogeneous fill materials. It was concluded that the stone columns improved the performance of the strip foundation, although measured stress ratios were lower than computed values. Bending moments, deduced from the measured strains, suggested that hogging moments were unlikely to occur for a strip foundation on stone columns with a thickness of at least 400 mm. Such research has not currently been extended into soft clay soils and it appears to be common practice to allow for at least nominal mesh reinforcement where dealing with such soils, to address any potential hogging moments which may occur around stone column positions during intial foundation loading. NHBC Chapter 4.6 (2011) requires both top and bottom mesh reinforcement, irrespective of soil type, i.e. granular or cohesive (fine-grained). Levelling off and proof rolling of the sub-grade following completion of vibro ground improvement is normally acceptable prior to construction of ground bearing floor slabs.

1.11 Research aims and objectives

The research incorporates field trials of partially penetrating dry bottom-feed vibro-stone columns supporting shallow narrow footings typical of low-rise lightweight structures, at the Bothkennar soft clay research site in Scotland, to advance the state of understanding of vibro stone column applications in this context. Confirmation of the general behaviour of stone columns historically has been obtained from field data. Historically, early specially commissioned field trials were difficult to implement and were costly and so other investigations have exploited contract works (live projects) opportunistically. However, these were often undertaken with limited preparation time and with site investigation data obtained for the contract purposes rather than specifically for the field trials. Relevant soil parameters were not always available in adequate detail for subsequent analysis. Moreover, none of these field trials had addressed either dry bottom-feed vibro stone column installation effects or the performance of partially penetrating dry bottom-feed vibro stone columns in a deep soft sensitive clay profile. Selection of the Bothkennar soft clay research site for this PhD research thesis provided the opportunity for such a trial.

The specific objectives of the research reported herein have been to:

- Compare and contrast published empirical approach(es) to vibro-stone column design - principally bearing capacity and settlement reduction aspects and undertake a forensic review of reported successful and unsuccessful case histories, including what can be learnt from these. Actual stone column field trial performance at Bothkennar is compared with pre-trial predictions of post treatment settlements, based upon one of the most commonly used settlement prediction methods adopted in European Ground Improvement practice - the Priebe (1995) approach. Whilst the method contains many simplifying assumptions and some empiricism in its formulation (Bouassida et al., 2008), it is nevertheless a widely-used approach. Furthermore, the field results are compared with predictions made (verified) by numerical Finite Element Analysis using the Plaxis 3D geotechnical software package and any limitations or requirements for further development of the software package for such analysis are explored.

- Investigate the largely unknown (and poorly understood) behaviour and settlement performance beneath shallow narrow footings, of partially-penetrating vibro-stone columns installed by the dry bottom-feed method in a deep (sensitive) soft clay deposit. Specific considerations include ground response to stone column installation and the influence of stone column length and spacing, founding depth within a thin surface crust and also foundation shape on foundation performance under incremental loading (with applied loadings to replicate those generated by low-rise lightweight structures).

1.12 Structure of thesis

Chapter 1 provides an introduction to vibro stone column techniques, applications and specifications, and highlights the importance of ground characterisation, monitoring, testing and quality control. The more commonly used design aspects and terminology are introduced and an outline is then provided of the aim and structure of the research.

Chapter 2 contains a review of the literature relating to vibro-stone column techniques in soft clay soils and looks at the mechanics of vibro-stone column behaviour, with emphasis on bearing capacity and settlement. Current design methods for vibro-stone columns are examined and compared. Links are drawn between the complex and sometimes contradictory behavioural observations and the relative unreliability of some of the current design methods.

Chapter 3 provides an overview of case histories of vibro-stone column applications in soft clay soils. There is limited case history information on the successful application of the dry bottom-feed technique in soft clay soils. The emphasis is therefore on these cases, particularly the more novel applications. A forensic review of some cases of unsatisfactory performance is also provided as it is considered much can be learned from situations where problems have arisen.

Chapter 4 covers the research programme undertaken, including the reasons for the research project and selection of the Bothkennar site. Justification and design of the field trials to assess the performance of different arrangements of partial depth vibro stone columns is provided as well as the reasons for: selection and implementation of the field instrumentation used and the approach to the numerical analysis.

Chapter 5 covers the equipment, field instrumentation, significant soils and materials properties, together with installation and test procedures relevant to the field trials.

Chapter 6 focuses on the response of the soft, sensitive Bothkennar Clay to stone column installation. Ground displacement (together with ground heave) and associated stress and pore pressure changes arising from the formation of cavities during vibroflot penetration and subsequent construction of compacted stone columns are investigated. Direct and indirect methods of assessing stone aggregate consumption are also compared.

Chapter 7 is concerned with the effect of various values of significant parameters, e.g. replacement ratio, treatment depth and strength of formation layer, on performance of partial depth stone columns in a deep soft sensitive clay deposit. Field trial data concerning the behaviour of narrow (isolated) strip footings subject to incremental loading at Bothkennar is analysed in terms of impact of stone column length, column spacing, foundation shape and founding depth in a thin surface crust on foundation performance. The deformed shape of selected columns has been investigated to understand the failure mechanism under vertical loading. The results are compared with the load-settlement performance of footings constructed in the surface 'crust' at Bothkennar, where no-vibro stone column installation had been carried out.

Chapter 8 contains output and review of Finite Element Analysis (FEM) undertaken using a 3D geotechnical software package to model the Bothkennar field trial data. The restrictions and limitations of the current software are explored (in the context of vibro stone columns) and future research needs are proposed.

Chapter 9 summarises the key conclusions arising from the research work and contains suggestions and recommendations for future research.

Soil type	Bearing pressure (kN/m²)	Settlement range after treatment (mm)
Made Ground: (cohesive and mixed granular and cohesive)	100–165	5–25
Made Ground: (granular)	100–215	5–20
Natural Sands or Sands and Gravels	165–500	5–25
Soft Alluvial Clays	50–100	15–75

Table 1.1: Typical improvements achievable in terms of load bearing capacity and settlement after vibratory stabilisation (after Slocombe, 2001).

Test method	Granular	Cohesive	Comments
McIntosh Probe	*	*	Pre-/post-treatment essential. Can locate obstructions prior to treatment.
Dynamic cone	**	*	Too insensitive to reveal clay fraction. Can locate dense layers and buried features.
Mechanical cone	***	*	Rarely used in UK.
Electric cone (CPT)	****	**	Particle size important. Can be affected by lateral earth pressures generated by treatment. Most appropriate test for seismic liquefaction evaluation.
Boreholes and SPT	***	**	Efficiency and repeatability of test important. Recovers samples (e.g. split spoon sampler).
Dilatometer	***	*	Rarely used.
Pressuremeter	***	*	Rarely used.
Small plate test	*	*	Does not adequately confine stone column. Limited stress depth. Affected by pore water pressures.
Large plate test	**	**	Better confining action.
Skip test	**	**	Can maintain test for extended period.
Zone loading test	****	****	Best test for realistic comparison with foundations.
Full-scale (e.g. surcharge load test)	*****	*****	Not commonly used. Tends to be confined to highway embankment projects.

* Least suitable ***** Most suitable

Table 1.2: Suitability rating for test methods applied to vibro stone column techniques (from Moseley and Priebe, 1993).

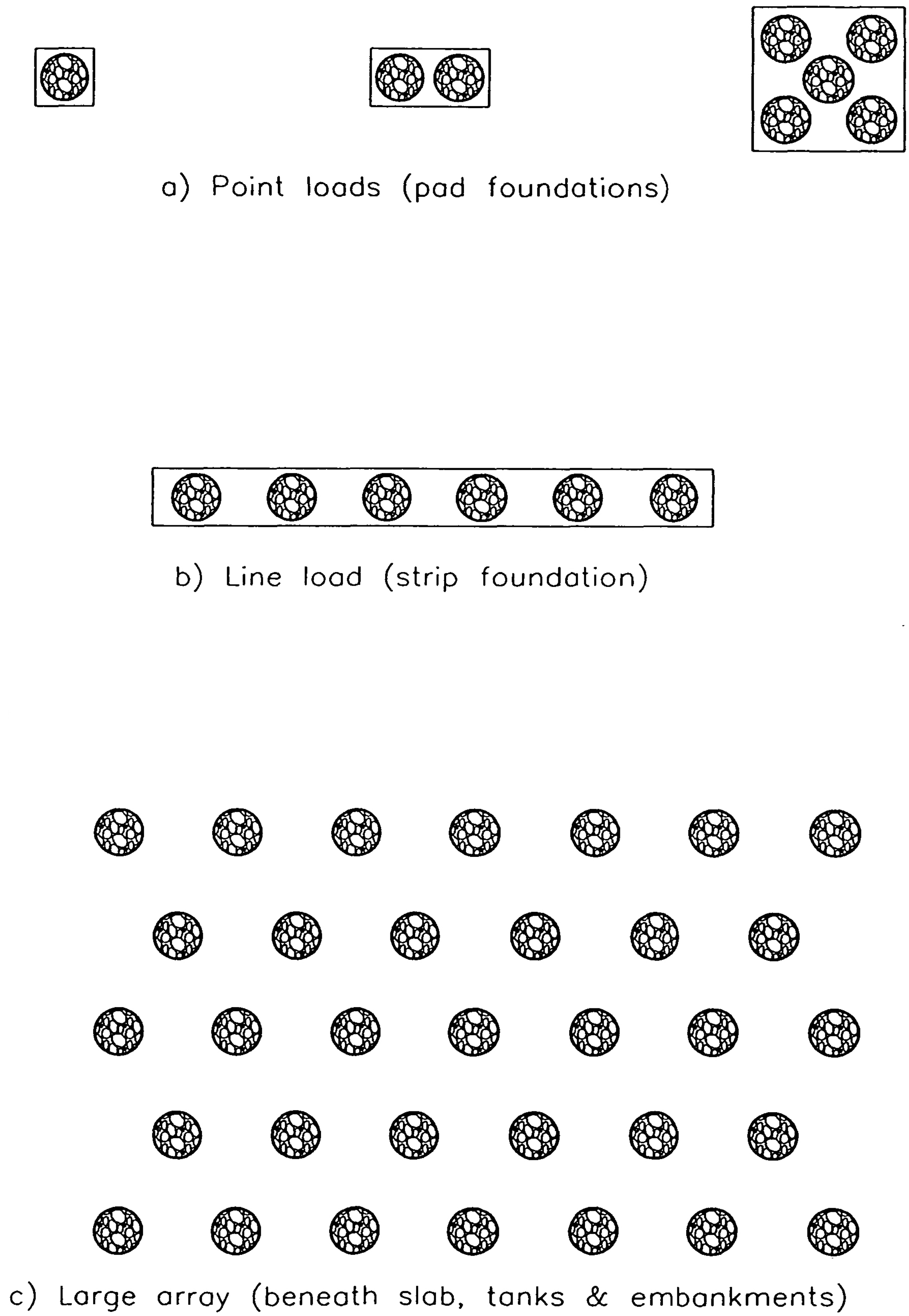


Figure 1.1: Stone column layouts (a) point loads; (b) line loads; (c) widespread load, (after Saadi, 1995).

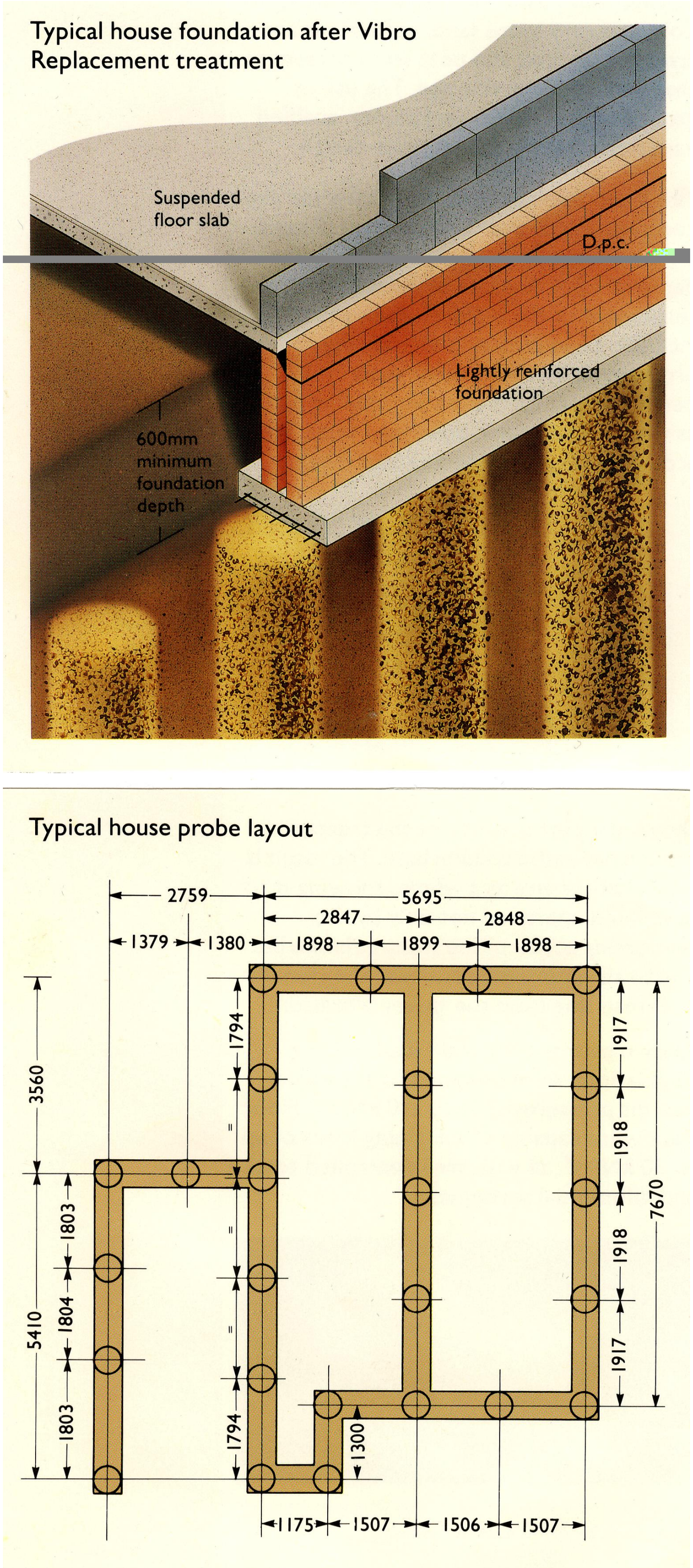
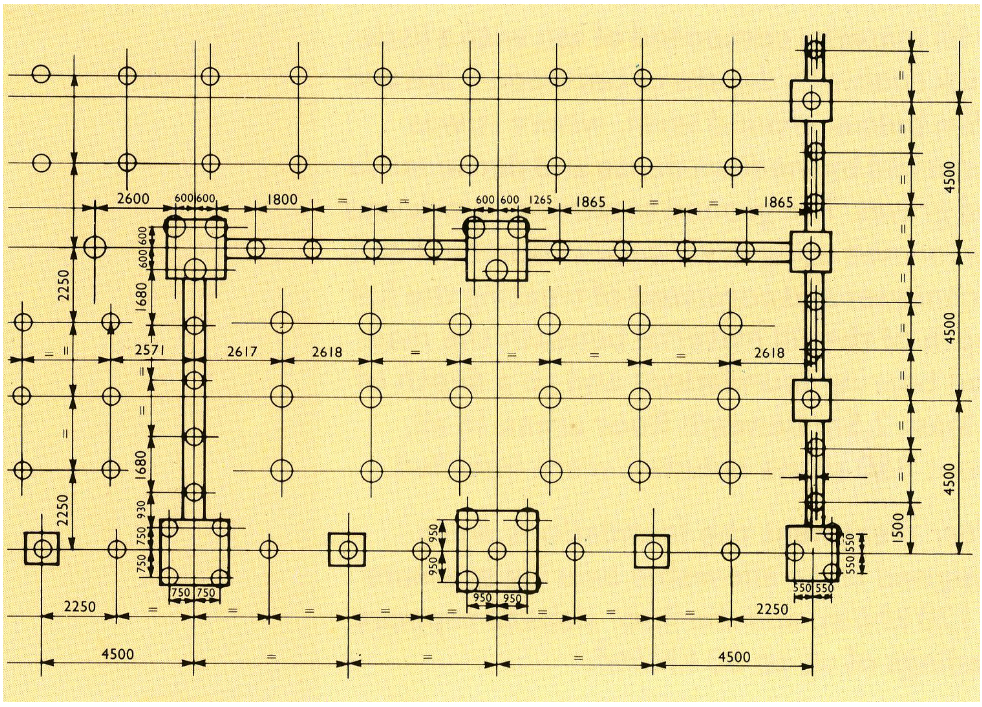


Figure 1.2: Stone column layout beneath strip footings for two-storey house (dimensions in mm), from Keller Brochure (2008).

a)



b)

Figure 1.3: (a) Stone column layout beneath foundations and slab of warehouse unit. (b) Stone columns exposed at subgrade level prior to (wide) strip foundation construction (Bauer Internal document – Serridge, 2000).

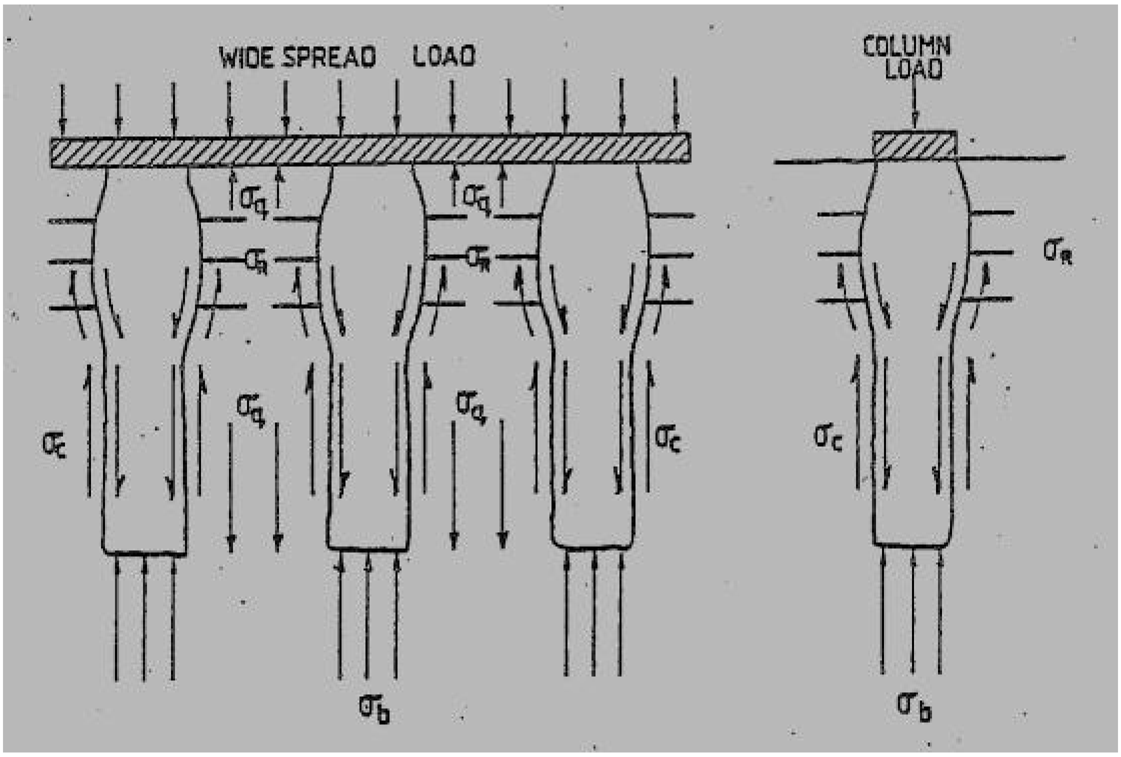


Figure 1.4: (a) Stresses associated with a stone column under an isolated footing and stone columns under a widespread load (after Greenwood, 1991). ($\boldsymbol{\sigma}_R$ = radial soil stress).

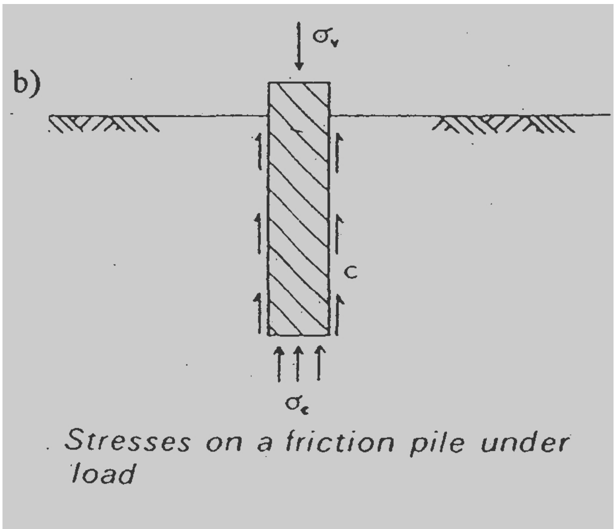


Figure 1.4: (b) Stresses associated with concrete piles, (after Greenwood, 1991).

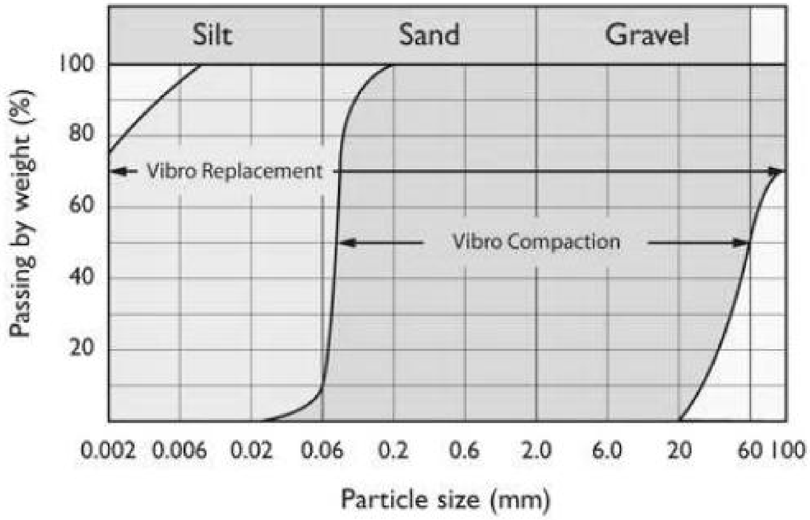


Figure 1.5: Grading envelope for vibro (replacement) stone columns (after Serridge and Slocombe, 2012).

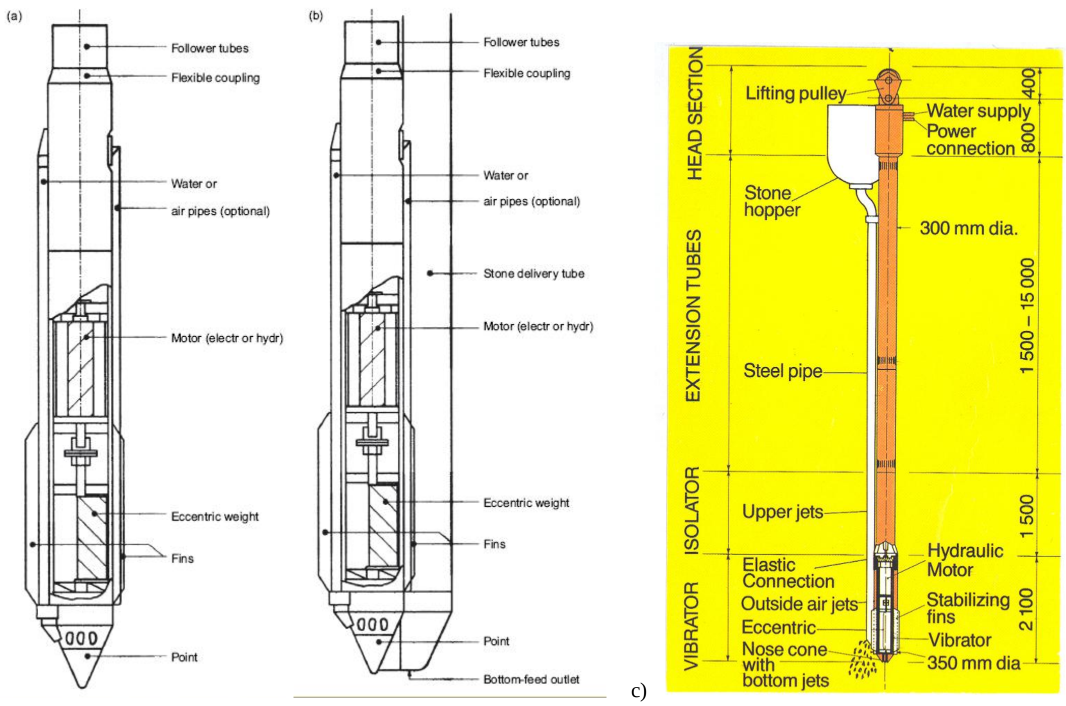


Figure 1.6: (a) Top and (b) bottom-feed vibroflots (vibrating pokers), after Slocombe et al., 2000. (c) bottom-feed vibroflot showing isolator detail (from Bauer technical brochure, 2001).

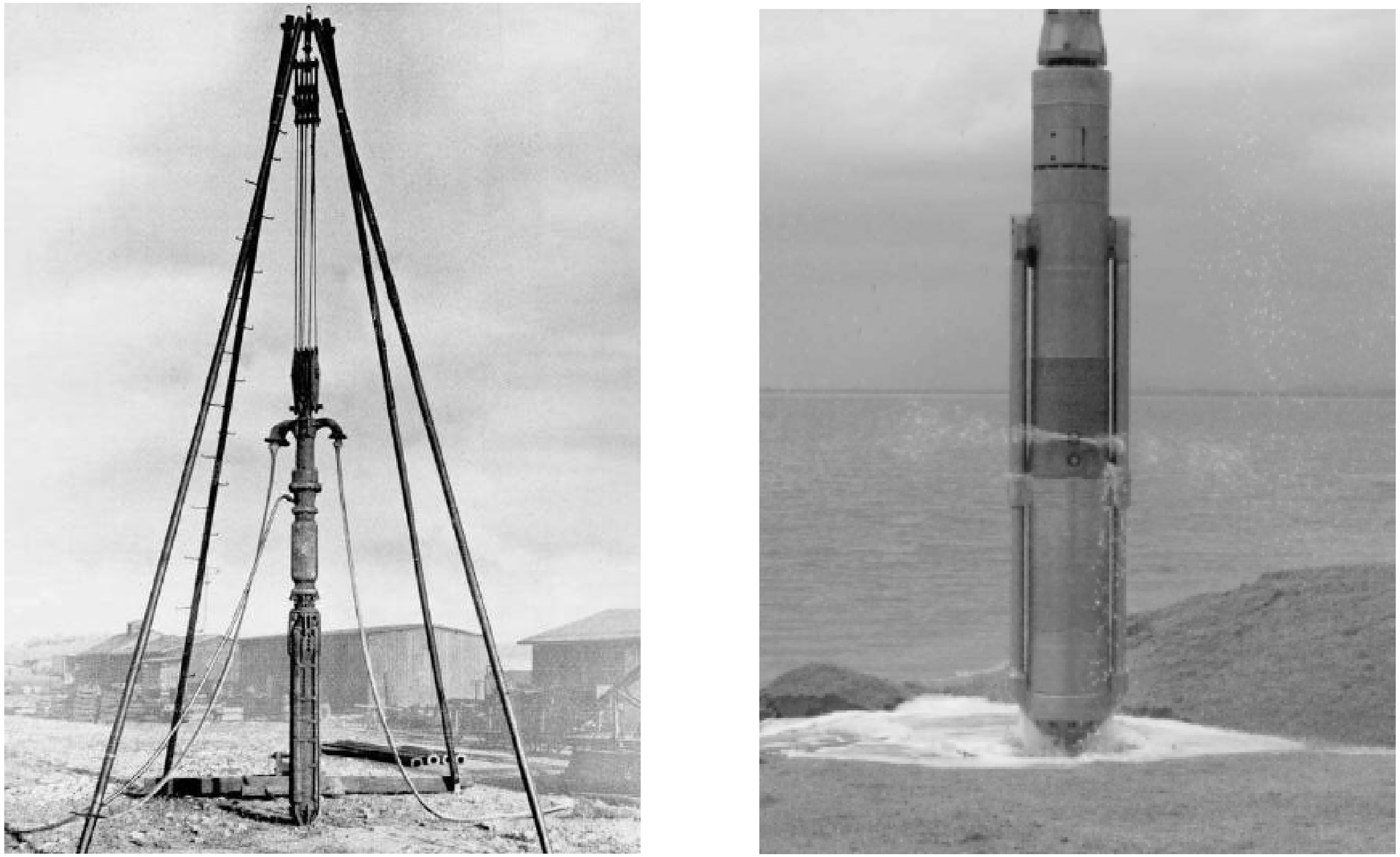
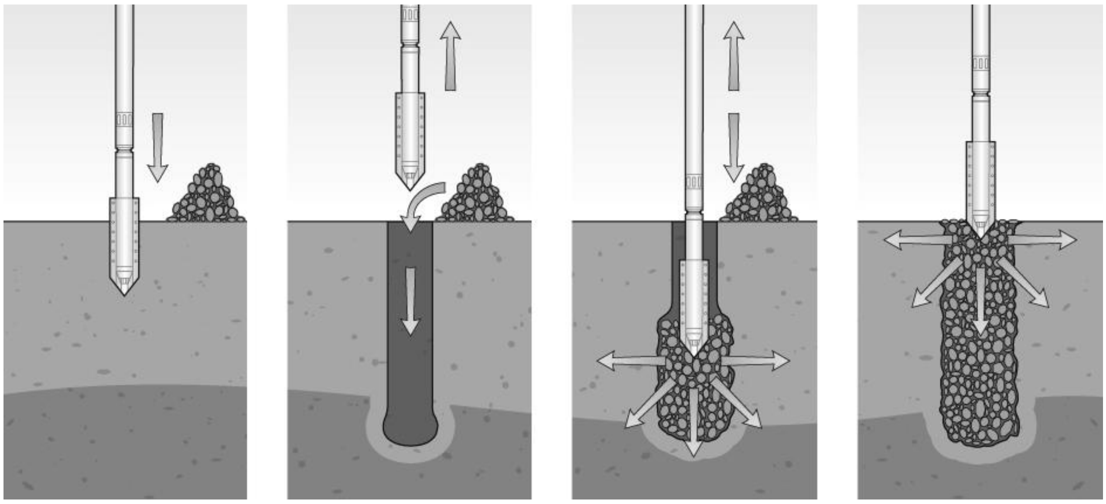
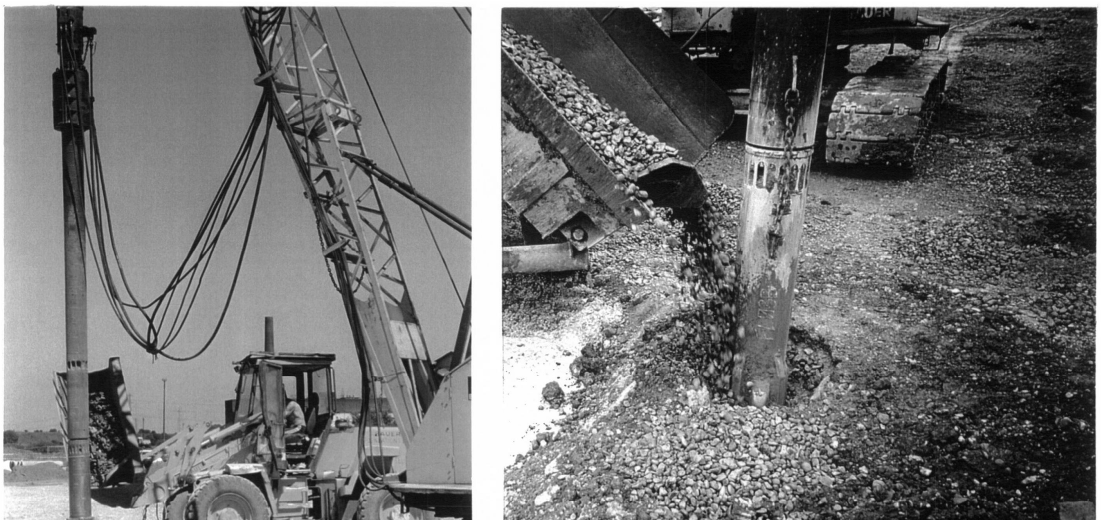


Figure 1.7: Original 1940’s vibroflot unit; (left) and modern vibroflot equipment (right), after Wehr (2007) .

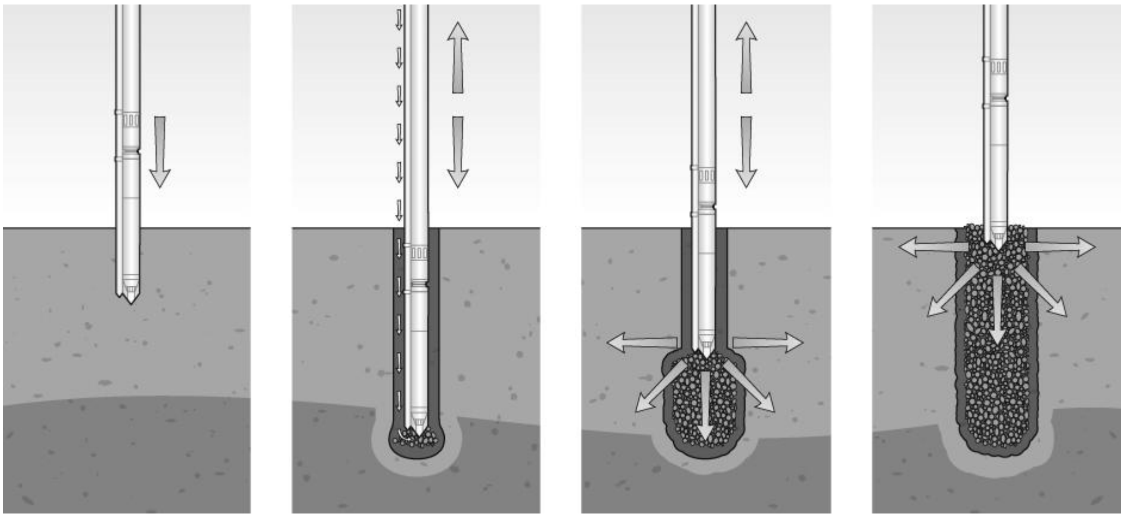


a)



b)

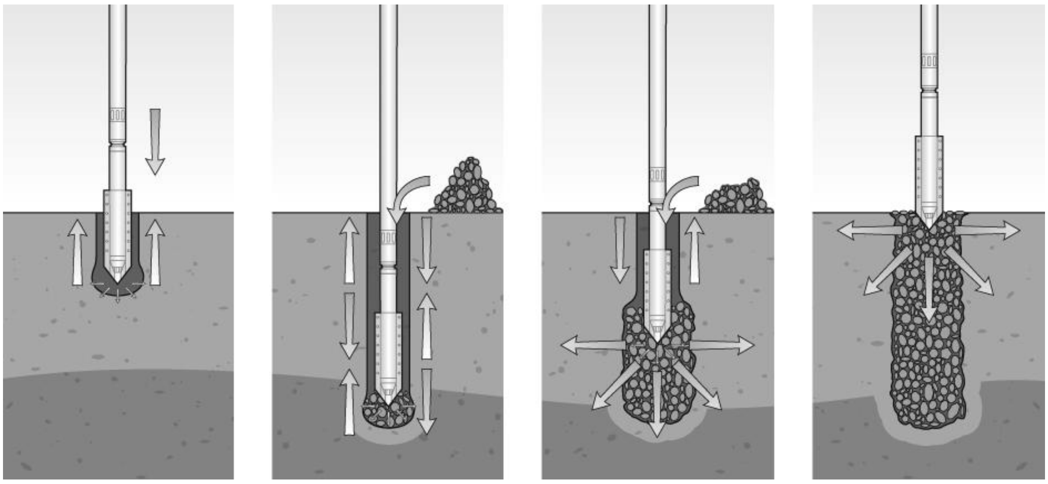
Figure 1.8: (a) Dry top-feed stone column installation sequence, (after Serridge and Slocombe, 2012). (b) Dry top-feed stone column installation in the field.



a)

b)

Figure 1.9: (a) Dry bottom-feed stone column installation sequence, (after Serridge and Slocombe, 2012). (b) Dry bottom-feed stone column installation in the field.



a)

b)

Figure 1.10: (a) Wet top-feed vibro stone column installation sequence, (b) Wet top-feed stone column installation in the field, (after Serridge and Slocombe, 2012).

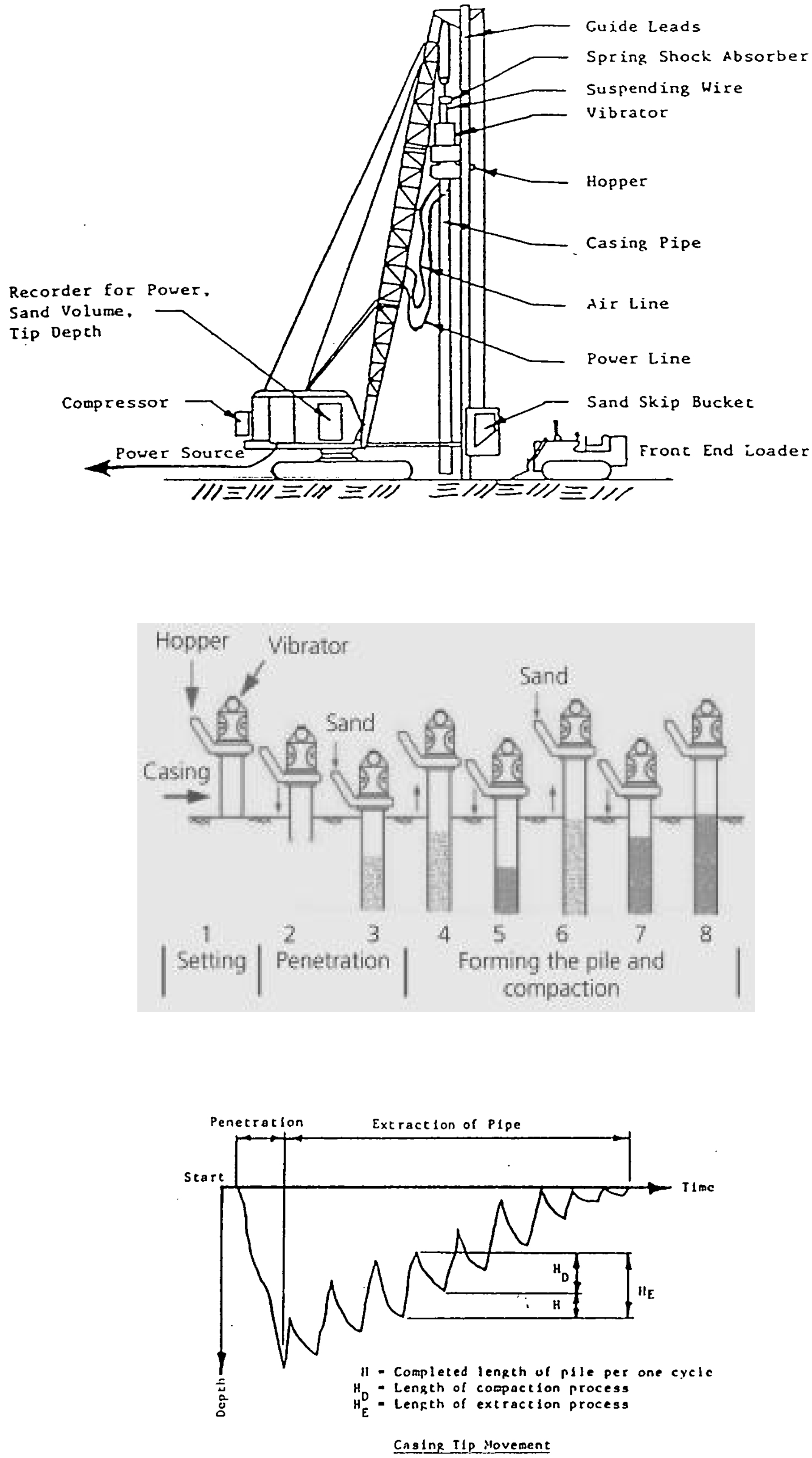
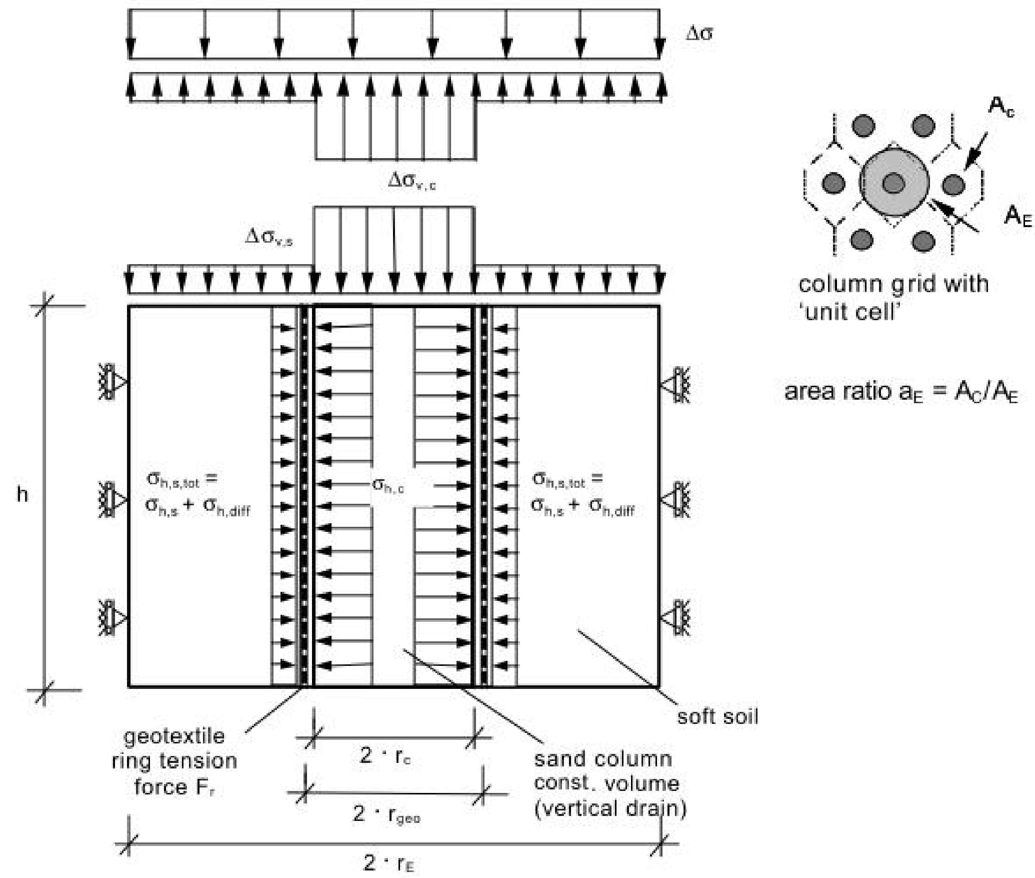


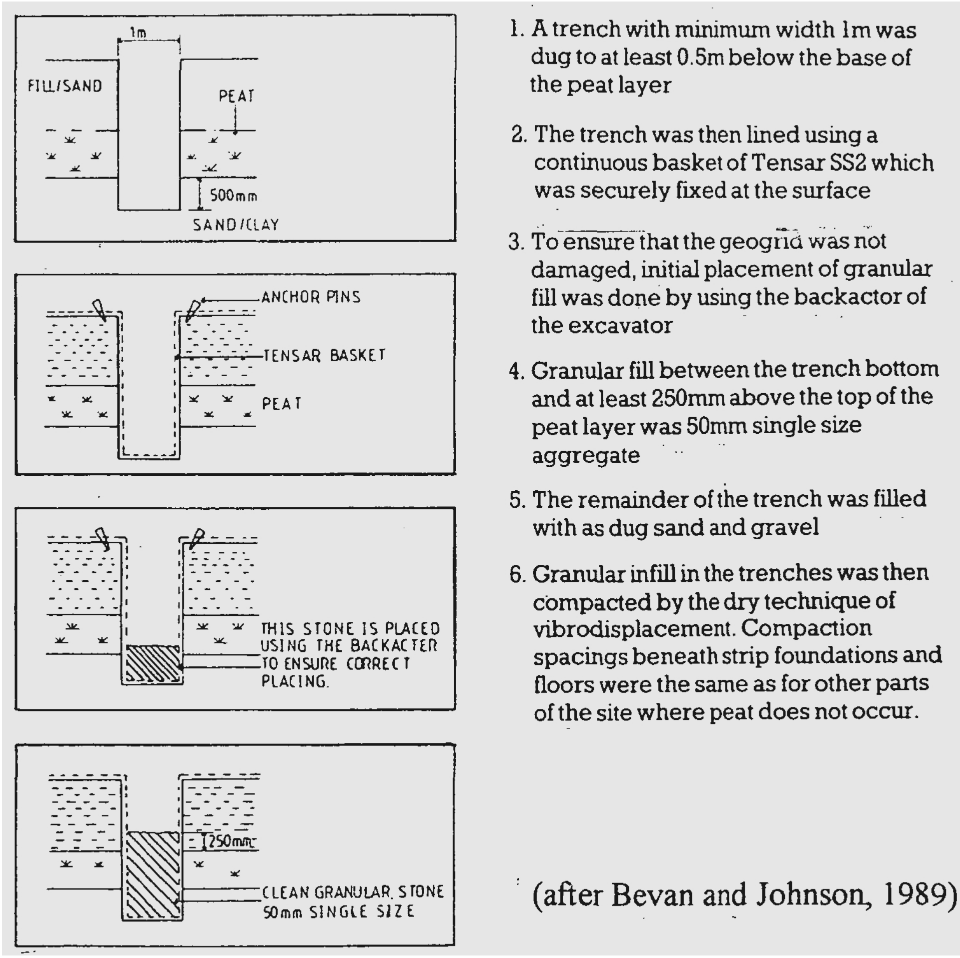
Figure 1.11: The Compozer equipment and construction sequence for sand compaction piles (SCP) (after Aboshi et al., 1979).



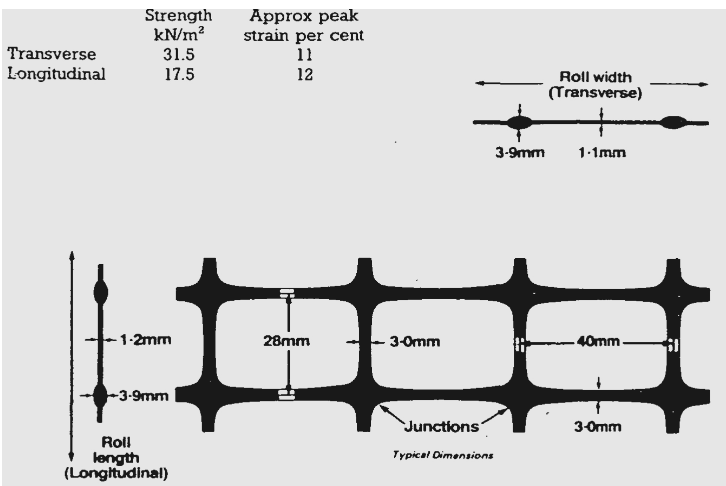
a)

b)

Figure 1.12: (a) Geotextile encased column (GEC) design philosophy and (b) Installation equipment (after Kempfert , 2003).



a)



b)

Figure 1.13: (a) Procedure for installing geogrid lined granular baskets through peat deposits prior to vibro and (b) Geogrid (basket) properties (after Bevan and Johnson , 1989).

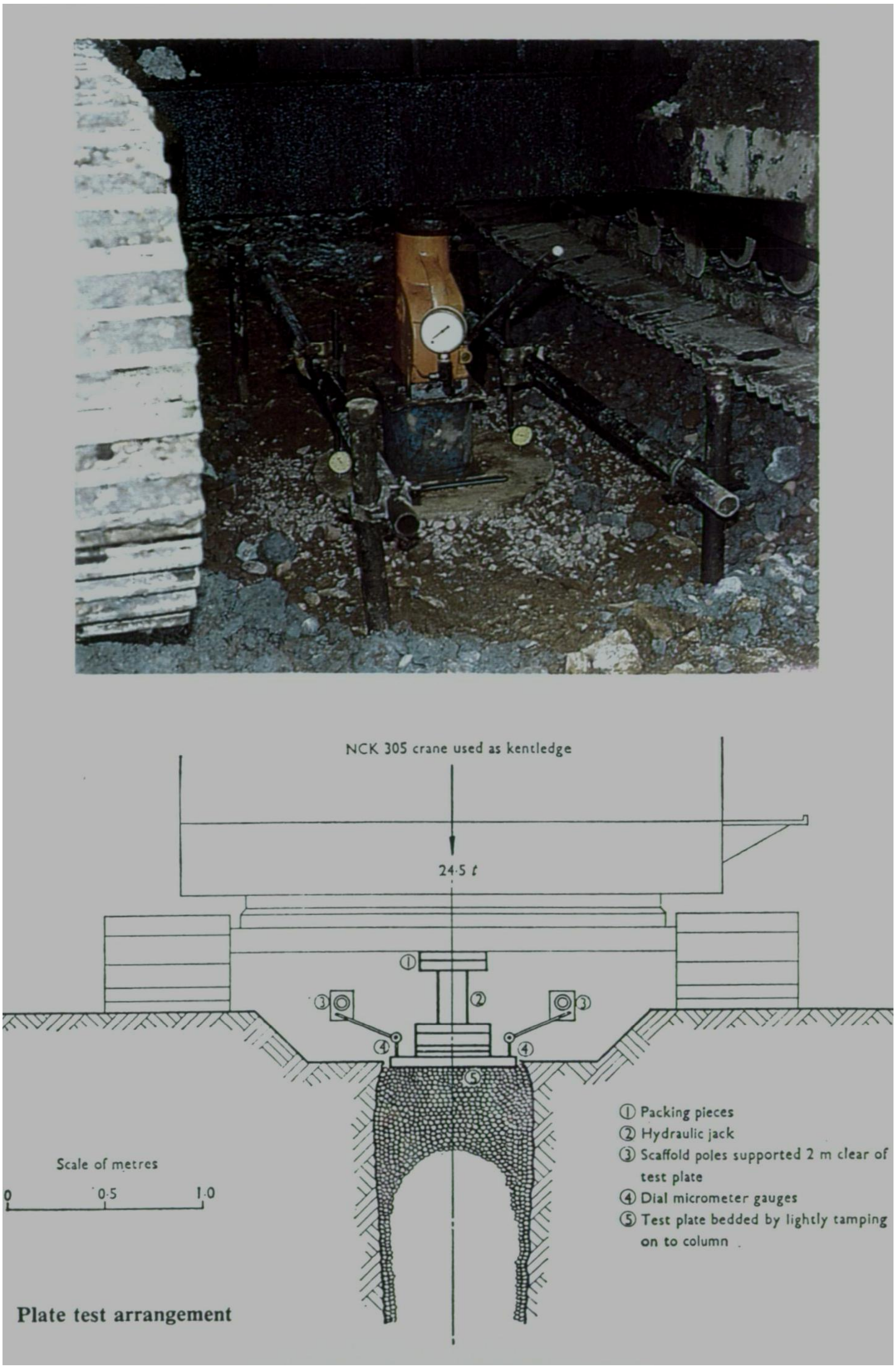


Figure 1.14: Short duration plate load tests, (after Greenwood, 1976a).

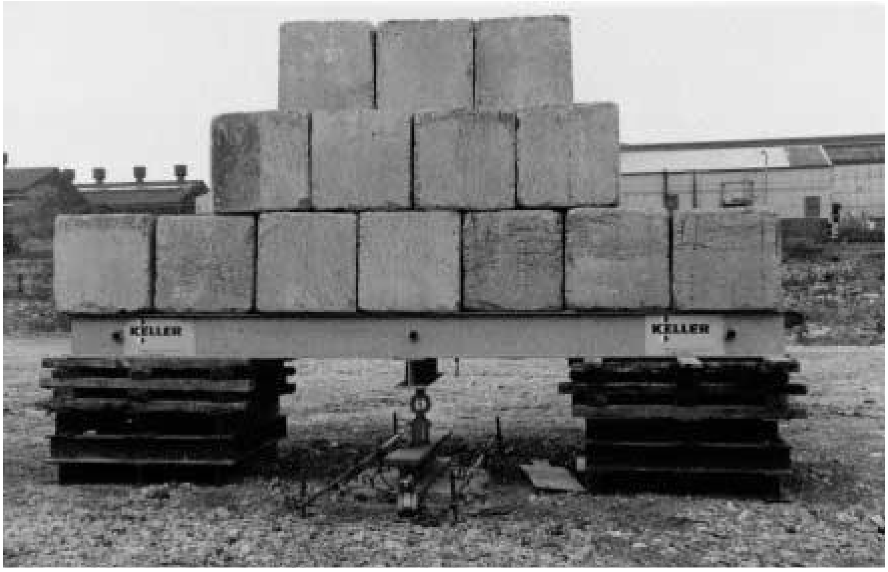


Figure 1.15: Zone load test (using concrete kentledge blocks as reaction load), after BRE 391 Specifying vibro stone columns (2000).

Figure 1.16: Skip tests (using sand filled skips), after BRE 391 Specifying vibro stone columns (2000).

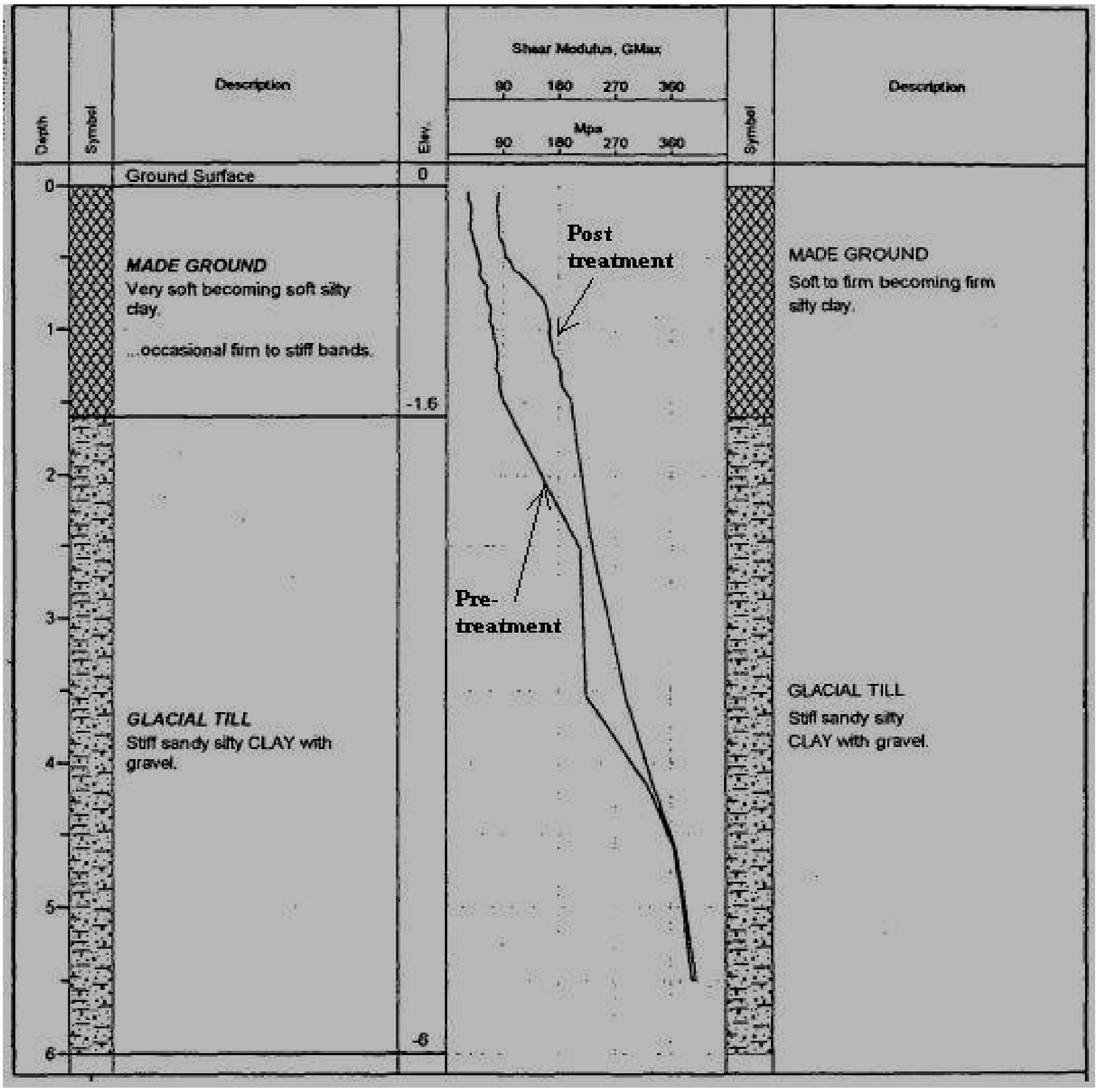
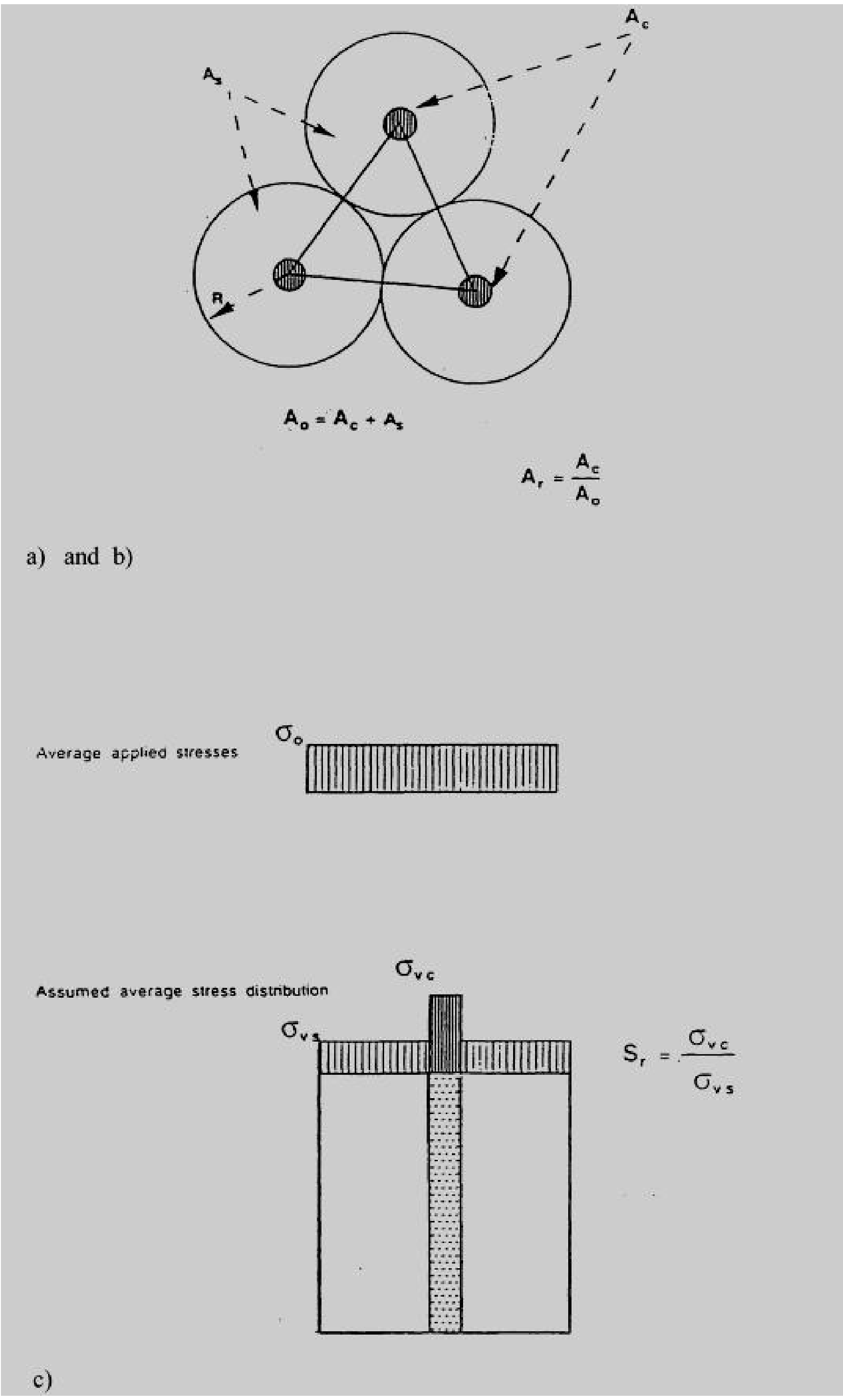


Figure 1.17: Continuous surface wave (CSW) testing (shear modulus versus depth) demonstrating enhancement in shear modulus after stone column installation at Kilwinning , Scotland (after Serridge, 2001).



A_o = Area per stone column.
A_c = Area of stone column.
A_s = Area of soil.
A_r = Area replacement ratio.
S_r = Stress ratio.
σ_o = Average applied stress.
σ_{vs} = Average vertical stress attracted by soil.
σ_{vc} = Average vertical stress attracted by stone column.

Figure 1.18: (a) Unit cell concept; (b) Area definition; (c) Vertical stress equilibrium (after Saadi, 1995).

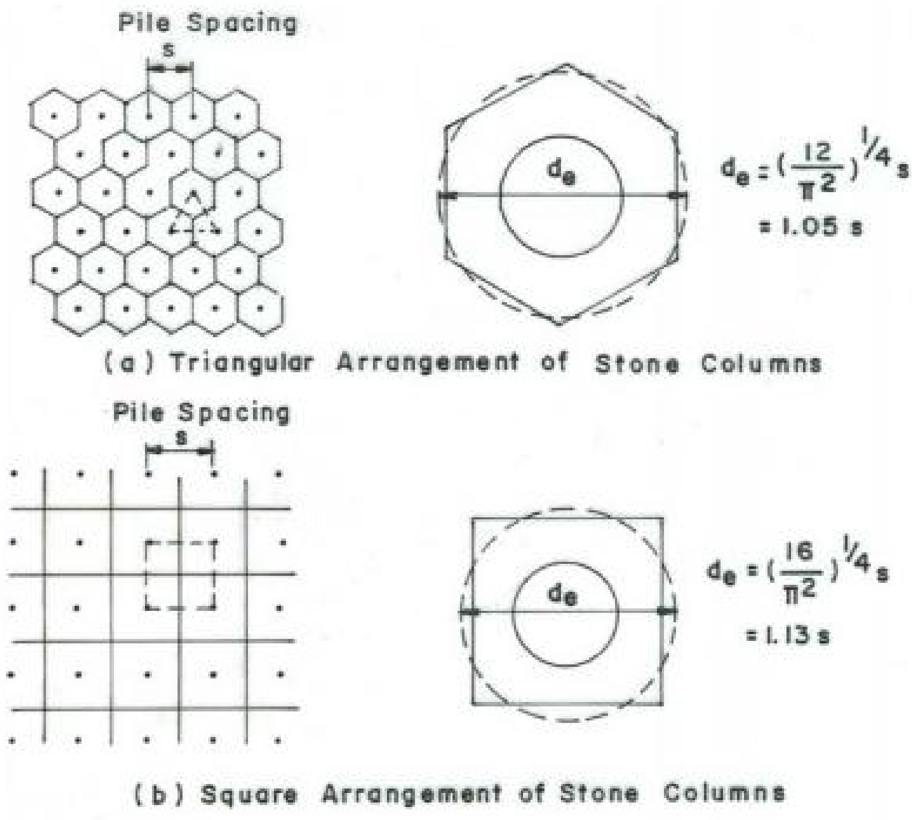


Figure 1.19: Typical layout of stone columns (a) Triangular grid arrangement (b) Square grid arrangement (after Balaam and Booker, 1981).

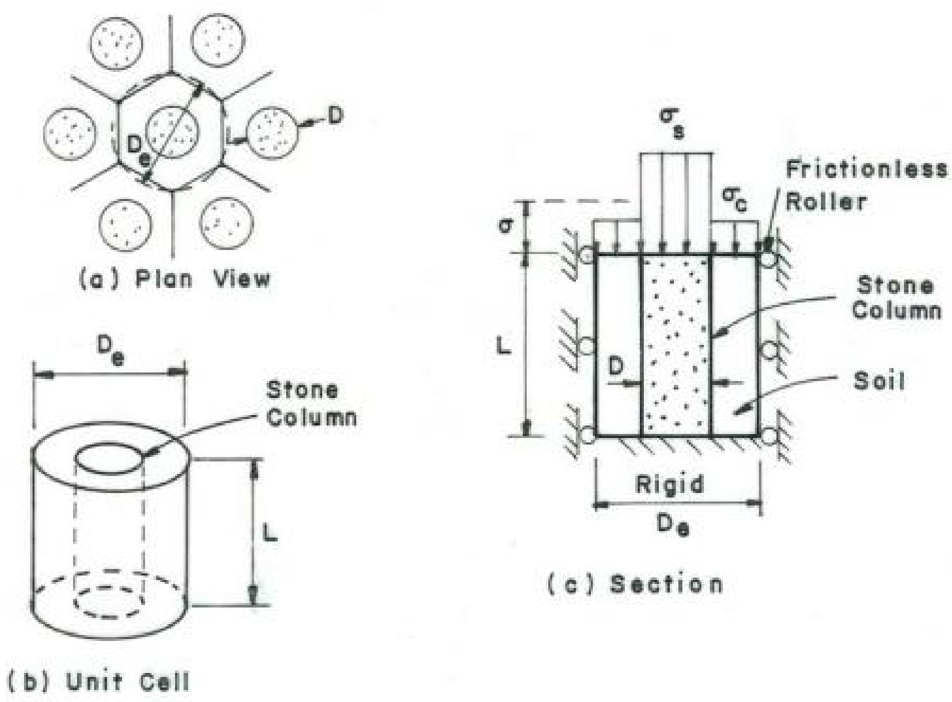


Figure 1.20: Unit cell idealisations (after Bachus and Barksdale, 1983).

Chapter 2 Previous investigations of stone column behaviour

2.1 Introduction

During the last four to five decades, the complexities of (vibro) stone column-soil interaction have been approached and investigated by a number of researchers in various ways, in an attempt to improve understanding and facilitate more economic design. The analytical techniques and investigations used in these research approaches have included (physical) laboratory modelling and full scale field trials, together with numerical and theoretical (analytical) methods. The evaluations of the bearing capacity (including ultimate load capacity of stone columns) and the reduction of settlement due to the presence of stone columns have been proposed by several authors. The aim of this chapter is to undertake a review of the more significant work undertaken in these areas and discuss their general development.

2.2 Laboratory modelling

It is evident that one of the main objectives of laboratory modelling is to validate theoretical models and attempt to simulate the field situation. Cohesive (fine-grained) soils used in the modelling are typically either remoulded or consolidated from a slurry. The latter necessitates use of specialised apparatus and equipment (Burland, 1969; Hughes and Withers, 1974), since removing consolidation loads causes swelling, which in turn creates problems because the soil becomes over-consolidated and the soil history is unknown. Hu (1995) notes that to determine load-settlement characteristics of the stone column-soil composite, rigorously speaking, a model has to be designed in such a way that all geometrical dimensions and material properties should be reduced by appropriate scale factors and identical to those found in the (field) prototype. It is important to recognise however, that in no physical model is it possible to maintain complete similarity of all parameters that govern the prototype response. Modelling with a geotechnical centrifuge (Schofield, 1980), provides the possibility of maintaining prototype stress levels. Such equipment is expensive and not always available in research establishments however, but has yielded some very useful research data where it has been utilised. The laboratory-based analyses may be sub-divided into those which have focussed on isolated stone columns and those which have investigated the performance of groups of columns. Whilst it has been argued that the latter is most relevant to the behaviour of stone column foundations in the field, the single column studies have dominated, but nevertheless appear to have provided a satisfactory basis for understanding the composite stone column-soil behaviour. Moreover, a number of design procedures currently used by ground engineering practitioners appear to have been derived from single column analyses.

The laboratory model tests of Hughes and Withers (1974) on columns of Leighton Buzzard sand in kaolin clay probably constitute the greatest advance in knowledge about the behaviour of stone columns in soft clay soils and arguably represents one of the most influential laboratory model studies in the understanding of the load carrying capacity of individual stone columns, in addition to providing a catalyst for many subsequent research projects. The laboratory testing comprised the construction of a series of isolated (model) sand columns with a length of 150 mm and diameters within the range 12.5 to 38 mm, in one dimensionally consolidated kaolin clay beds. Stress controlled load was subsequently applied to the column surface area only in each test. For comparison purposes, a 38 mm diameter model footing test was carried out on a plain (unreinforced) clay sample. Radiographic (X-ray) techniques were used to determine the displacement of pre-placed lead shots within the clay and granular column(s) at various levels (Figure 2.1). Results of a suite of experiments showed that the presence of the granular column reduced the magnitude of expected settlements. In addition, the bearing capacity was significantly greater that that of a similar unreinforced (untreated) foundation. From their radiographs of displacements of the lead shot markers (Figure 2.1), Hughes and Withers postulated that the strength properties of the material surrounding the column were improved within a distance of 2.5 times the column diameter. Hughes and Withers also demonstrated that under vertical load application, a single isolated column bulges near the top to facilitate generation of extra lateral confining stress (Figure 2.1) and stated that the columns' ultimate strength is governed primarily by the maximum lateral reaction (passive resistance) of the soil within the zone of column bulging and that the extent of any vertical movement is limited. Essentially the stone column in the ground behaves similar to a column in a triaxial cell, confined by a radial stress. Such (bulging) behaviour was also idealised by the authors as analogous to the expansion of a pressuremeter, in which a cylinder is expanded against the walls of a borehole. At the point where horizontal resistance of the soil reaches its limiting value, indefinite expansion of the column occurs and it fails. By adopting Gibson and Anderson's (1961) elasto-plastic theory for expansion of a cylindrical cavity, the authors established a relatively simple method to facilitate estimation of the load carrying capacity for a single (isolated) stone column, as a function of undrained shear strength of the clay and the internal friction angle of the column material. A simple plasticity method was also developed to determine the vertical stress distribution by assuming that the limiting value of shear stress along the side of a column is equal to the initial undrained shear strength of the clay, and that it is constant over the column length. Estimation of the ultimate vertical effective stress, σ'_v, carried by a stone column as it bulges was given by:

$$\sigma'_v = (1+ \sin \varphi'_s/1- \sin \varphi'_s)(\sigma_{ro} + 4C_u - \mu) \quad \text{--} \quad 2.1$$

where:

φ'_s is the friction angle of the stone column material, σ_{ro} is the total *in situ* lateral stress, C_u is the undrained shear strength and μ is the pore water pressure.

On the basis of this analysis, the authors defined a critical length at which end bearing and bulging failure will occur simultaneously in a single column, corresponding to about four times the column diameter.

For prediction of minimum stone column length, i.e. the column length required to prevent end bearing failure at the toe occurring before bulging failure near the top of the column, assuming C_u is constant over the length of the column, the following expression was used to calculate the depth at which vertical stress (σ_{vz}) in the column will be zero:

$$\sigma_{vz} = \sigma_v + M (\gamma_c.D-4C_u) \quad \text{---} \quad 2.2$$

where:

M = the ratio of column length to diameter (L_c/D)

σ_v = the ultimate column capacity

γ_c = unit weight of stone column material

D = diameter of stone column

C_u = cohesion of soil

Furthermore, the authors stated that the additional length of a column beyond this critical depth (length) will not enhance the load bearing capacity, but may have some use in reduction of settlement. The plasticity approach itself proposed by the authors is relatively simple and straight forward and still in common use today.The representativeness of a model column, with a diameter of only around 38 mm, in the Hughes and Withers (1974) analysis, compared to approximate full field-scale diameters, typically of the order of 600-800 mm in soft clay, is questionable however. In the case of the Hughes and Withers (1974) model, vertical deformation at failure was around 58% of the column diameter compared to values of 10-15% obtained from field trials (Thorburn, 1975). However, the basic expression (2.1) is independent of any scale effects and in this regard it is only the failure load predictions based on the laboratory model tests which perhaps need to be treated with some caution. It is important to note that although many assumptions were made by Hughes and Withers (1974) in order to simplify the analysis, their method of predicting the ultimate load carrying capacity of a single column is still widely used because of its simplicity.

Charles and Watts (1983) conducted large scale laboratory tests in a floating rig oedometer (1 m in diameter) to assess the effectiveness of an isolated granular column, (Figure 2.2a) including investigation of different column diameters, in reducing vertical compression of soft clay (Figure 2.2b). As would be expected, the vertical displacement of the clay under effectively a rigid foundation load was significantly less when reinforced with stone column material, compared to unreinforced clay. This was attributed to the fact that the column was significantly stiffer and carried a much greater proportion of the applied load, thus reducing the vertical stress in the surrounding clay. As the column diameter increased, the load carried by the column also increased and consequently the compressibility of the clay layer decreased (Figure 2.2b). The authors found that the principal stress ratio can reach a 'peak' value with a small diameter column (low area ratio A/A_c), but that the ratio was well below the 'peak' value for a larger diameter column (high area ratio A/A_c). In order to achieve a significant reduction in compressibility the authors recommended a higher area ratio is required - greater than 30%. The author (researcher) is of the opinion that such high ratios may not be practical at the field scale, particularly where the dry bottom-feed technique (a displacement process) is used in soft clay, due to potential for excessive soil disturbance and heave during column installation.

Bachus and Barksdale (1984) used the unit cell concept in their laboratory - based stone column research when undertaking a vertical load test on an end bearing stone column in a physical unit cell chamber (108 mm in diameter and 305 mm in height). The purpose of the unit cell was to model uniform loading over an infinite array of stone columns. Load was applied in increments to the column and its tributary clay, i.e. the unit cell, and settlements were recorded at various loading levels. Bachus and Barksdale recommended an area replacement ratio of 40% to achieve significant settlement reduction. Furthermore they found that the stress concentration ratio (SCR or S_r) fell within the range 2.8 to 4.2 and remarked that it remained reasonably constant with time and load level. Again it is important to highlight that replacement ratios as high as 40% may not be practical in the field situation in soft clay with vibroflot equipment, for reasons previously described.

Hope (1988) investigated the behaviour of granular (sand) columns in a remoulded clay (glyben). Single columns within the fine-grained (cohesive) soil were subjected to fairly rapid loading and therefore essentially undrained conditions. In contrast to the modelling by Hughes and Withers (1974), which was based primarily on drained conditions and noting that the stress history of the fine-grained (host) soils for the columns were different in both the Hughes and Withers (1974) and Hope (1988) research programmes, it was found that the bearing capacity of a single column was governed by the undrained shear strength of the host soil and the load-settlement plots were almost identical in both cases .

Hu (1995) carried out a comprehensive investigation of a soft clay reinforced by a large group of granular columns, incorporating laboratory-based displacement controlled vertical loading tests, with the objective of assessing the actual failure mechanisms of the column-clay composite under predominantly a rigid circular footing. The influence of significant parameters including soil (clay) undrained shear strength, area replacement ratio (A_r), column length, method of installation and relative flexibility of the footing on the performance of 'stone' column reinforced clay soil, was investigated through a total of 25 model tests. Vertical loads were applied to footings constructed on one-dimensionally (1-D) consolidated kaolin samples, 300 mm in diameter and reinforced with an array of fine sand columns (with diameters ranging from 11-17.5 mm). The author observed that the initial strength of the clay had a significant influence on the capacity of the foundation constructed over the stone column reinforced soil. Furthermore the area replacement ratio (A_r) was also found to be a significant parameter influencing overall performance of the stone column reinforced soil: its value reported as significantly affecting the extent of column interaction, degree of consolidation in the soft clay and the proportion of applied stress carried by the stone column and intervening soil (stress concentration ratio - S_r). Hu (1995) recommended than an area replacement ratio (A_r) of greater than 24% was required to achieve a significant increase in load capacity. Contact stresses were noted to be higher in the columns than the intervening soil. Average values for the S_r fell within the range 1 to 5 with evidence of the value increasing as the applied load increased. It is evident that no definitive pattern emerges with regard to the change in S_r with increasing pressure. In some cases S_r decreased, notably at the early stages of loading, whilst in other cases it remained reasonably constant or increased as the applied stresses (foundation pressures) increased. Whilst Hu's (1995) analysis assumed that the tests were being carried out under fully drained conditions, he postulated that the composite (stone column-soil) sample was sheared at a relatively fast rate so that excess pore pressures probably had insufficient time to dissipate. Furthermore the samples were not confined in the vertical direction since there was no surcharge on the surface.

It was observed that increasing the length of the column also enhanced the stiffness of the reinforced ground: a short column (Figure 2.3) was noted to punch into the clay below the column toe in addition to developing localised bulging. The punching behaviour was found to be eliminated if the length of the column was increased (Figure 2.4 and 2.5). This was in line with observations reported by Hughes and Withers (1974). An increase in load capacity of around 15% was observed when the L/d (column length to column diameter) ratio was increased from 5.7 to 9.1. No further improvement or benefit was observed when this ratio was increased to 14.5. Despite these observations and comments, Hu (1995) suggested that the critical length (defined as the depth at which end bearing failure and bulging failure occur simultaneously) was controlled by the ratio of column length to footing diameter (L/D). Penetration of the column toe was only observed to occur when the L/D ratio was less than 1. Upon completion of the laboratory load testing regime, Hu (1995) meticulously exhumed the sand from the samples using a vacuum arrangement and subsequently took plaster casts of the resultant bores with the objective of investigating more closely the deformed shape of the columns. This was the first time that such an approach had been attempted. A typical example of these plaster casts is shown in Figure 2.5, which demonstrates buckling and bulging deformation, particularly in longer columns. In general agreement with the measured stresses, the columns adjacent to the centre-most columns exhibited the most prominent deformed shape (Figure 2.5). Differing load bearing behaviour of a group of columns and a single column was demonstrated**.** Most of the bulging, shearing and lateral deflection was observed to have occurred within a 'conical-shaped' region directly beneath the model footing. The stresses and deformations in the stone column reinforced foundation were reported by the author as being complex, but suggested that the 'reinforced' soil initially undergoes elastic deformation, then as loading progresses the deformation becomes elasto-plastic and the overall stiffness reduces. The depth of this failure wedge increased as the area ratio increased. Hu (1995) proposed a four stage failure mechanism, shown schematically in Figure 2.6a and 2.6b. An elastic conical zone, defined as Zone 1 (I), exists immediately beneath the footing. Within this region bulging was not considered to occur since the surrounding clay provides adequate confinement to the columns. Within Zone 2 (II), immediately below Zone 1 (I), plastic deformation takes place, where bulging, shearing, bending and buckling of columns are observed. Hu (1995) refers to Zone 3 (III) as a 'retaining unit', providing lateral support to the wedge underneath the footing, and Zone 4 (IV) as an 'extension' zone. The work seems to have correlated well with previous stone column research and emphasises the importance of parametric variation on the performance of the foundation.

Over the last 10 years a significant amount of laboratory-based research has been carried out at Queens University, Belfast (UK), principally developing the work of Hu (1995), utilising new materials and novel techniques to facilitate further insight into the performance and patterns of failure of small groups of stone columns in soft clay. The laboratory modelling research reported by Mc Kelvey (2002) utilised a relatively new artificial transparent clay-like material and which revealed probably for the first time, the process of deformation that takes place in a stone column reinforced clay bed in real time, during actual loading (Figure 2.7). Three sand columns, 25 mm in diameter, were installed in a triangular arrangement beneath the circular footing (100 mm in diameter) and in a row beneath a strip footing to depths of 150 mm and 250 mm. The results of the tests showed that columns can fail in three different ways: bulging, punching and bending, with punching more prevalent in short columns, whilst bending was predominant in 'perimeter' columns located beyond the centre of the footing. Bulging was generally more common in long columns, as shown in Figure 2.7. Beneath the rigid footing, the central column in the stone column group deformed or bulged relatively uniformly, whilst the edge columns bulged away from the neighbouring columns, also shown in Figure 2.7. A similar pattern of behaviour was observed by Hu (1995). The presence of the granular columns was shown to improve the stiffness and therefore load-carrying capacity of the soft clay layer, in line with earlier research (Hughes and Withers, 1974; Hu, 1995). Mc Kelvey et al. (2004) observed that in the case of 'short' columns ($L/d < 6$), bulging took place over the entire length of the columns and they punched into the clay beneath their bases. In the case of the long column ($L/d > 10$) the columns deformed significantly in the upper region whereas the bottom portion remained undeformed. This suggests that there was little or no load transfer to the base in longer columns, with failure arising from bulging or shear. For columns longer than about six times their diameter ($L/d = 6$) no further increase in capacity for small column group configurations was demonstrated. This was similar to observations made by Hu (1995) for large group configurations, but greater than the $L/d = 4$ suggested by Hughes and Withers (1974). This again suggests that the length of the columns beyond the optimum value may be more significant in terms of settlement design criteria than load carrying capacity. Beneath a rigid footing supported on long columns, the columns were observed to accept a higher proportion of the applied load than the intervening clay, whereas for the footing supported on short columns, the stress concentration ratio was reported as significantly smaller.

Black et al. (2006) and Black et al. (2007a) ; Black et al. (2007b) developed the work of Mc Kelvey et al. (2004) further by investigating settlement performance of stone column reinforced soft clay, and an L/d ratio of 8 was cited for serviceability criteria. This may be associated with the fact that a novel protocol was adopted in an attempt to mitigate problems associated with a lack of control of pore water pressure under foundation loading and frictional resistance, which had led to non-uniform soil stiffness (strength) properties in previous research, where soil beds were prepared and restrained in one-dimensional consolidation chambers during foundation loading. Samples were therefore initially prepared by one-dimensional consolidation and subsequently transferred to a more spohisticated large triaxial cell for re-consolidation under isotropic stress conditions. This system allowed for the confining and pore water pressures, and offered the additional benefit of a non-rigid 'free' lateral boundary, i.e. boundary conditions could be regulated. This work also identified a 'block failure mechanism' was prevalent in small groups whereby the columns and confined clay region acted as a single entity. The research by Black et al. (2007a) and Black et al. (2007b) also demonstrated that vibro stone column design in clay soils can be flexible: settlement can equally be controlled using short columns at relatively high area replacement ratios, or longer columns at smaller area replacement ratios. An optimum area replacement ratio (A_r) of 30-40% was suggested to exist for control of settlement. The settlement performance of a small group of columns was considered to be highly influenced by inter-column and footing interaction effects.

2.3 Full – scale field trials

The importance of full-scale field testing cannot be underestimated. Unfortunately, most of the field trial data that has been published on vibro stone columns in soft clay, with a few exceptions, e.g. Serridge (2001) who investigated widespread load beneath a raft foundation at Bothkennar, were carried out through contract works in which not only was preparation time limited, but also site investigation data together with soil laboratory data were obtained for contract purposes rather than for research purposes, so often not sufficiently robust in respect of permitting detailed analysis and interpretation.

One of the earliest full scale field trials reported in the UK literature is provided by Greenwood (1970), who described the results of trials at Bramerhaven (Germany) for motorway slip road embankment(s) application. Both vibro stone columns and sand columns were constructed for comparison purposes. The stone and sand columns were installed to an average depth of 6.0 m by the wet top-feed technique through a layer of soft clay and peat into a fine uniform sand layer (Figure 2.8a and 2.8b). The stone columns were constructed using gravel of size 30-70 mm with the sand backfill having a grain size of 0-3 mm. The combined total thickness of the clay and peat layers was about 3.0 m, with geotechnical properties summarised in Figure 2.8b. Unfortunately, there appears to be limited information on the actual geometry of the peat deposit. Reported average stone column diameters were 1.2 m, with columns installed on a 2.3 m triangular grid spacing (A_r = 25%). For the stone column, sand column and untreated areas, average settlements (Figure 2.8c) and range of settlements after 15 months following embankment construction were respectively 468 +/_ 31 mm, 645 +/_75 mm, and 765 +/_ 13 mm. Reduction of settlements was around 15% where sand columns were installed and 40% where stone columns where installed, representing settlement improvement ratios of 1.18 and 1.6 respectively. It is evident that a closer column spacing would also have resulted in less settlement. It was considered that local yielding of the soil, probably within peat inclusions, had caused less settlement reduction than would be predicted by elastic theory. The field trials demonstrate the importance of using a coarse aggregate backfill rather than sand in the column construction to lend better rigidity (stiffness) to the column. It is important to highlight that the settlement against time records showed only marginal improvement in the rate of drainage due to the presence of the granular columns, but this was attributed by the author to the nature of the peat (and its inherent low coefficient of compressibility and the proximity of the treated area to the untreated area).

Hughes et al. (1976) undertook field scale trials to verify the theory proposed by Hughes and Withers (1974) in their laboratory modelling. The parameters required were: the undrained shear strength of the soil, the in situ lateral stresses in the soil, the radial pressure-deformation characteristics of the soil and the angle of internal friction of the stone column. Cambridge pressuremeter and menard pressuremeter tests were conducted to determine the in-situ lateral stress and the radial deformation properties of the soft clay soil. Field vane tests, Dutch cone and undrained triaxial tests were used to obtain shear strength parameters. A single 730 mm diameter column was installed by the wet top-feed technique through soft clay strata to the level of a firm stratum at a depth of 10 m. A rigid 660 mm diameter circular steel plate was used to apply vertical load to the column. The test was considered to be undrained since it only took around 30 minutes to complete. Good agreement was obtained between predicted and measured load-settlement curves and demonstrated the occurrence of shear transfer between the column and surrounding clay. Considering the column as a 'pile' Hughes et al. (1976) defined a critical length for an isolated column, at which end bearing and friction are equated. Beyond this length the column was considered not to contribute extra benefit in terms of enhanced ultimate load, but contributed to reducing settlements by penetrating to a firm stratum. Based upon the site specific soil and column parameters the critical depth (zone of anticipated bulging) translated to about four column diameters, similar to observations by Hughes and Withers (1974). Following completion of the test the column was excavated so that its deformed shape could be examined. It was observed that the deformed shape (Figure 2.9) was similar to that described by Hughes and Withers (1974) (see Figure 2.1). Hughes et al. (1976) stressed the importance of column diameter when estimating the load capacity of a stone column. It is perhaps important to recognise that whilst common in France, it is currently rare to see pressuremeter testing employed in soft clay soils in the UK.

Goughnour and Bayuk (1979b) reported the results of a field trial on a group of columns installed using the wet top-feed technique, through very soft sensitive silts and clays in Hampton, Virginia (U.S.) (Figure 2.10). The columns were installed to an average depth of 6.4 m and on an approximate 1.8 m grid pattern with a recorded average diameter of 1.1 m (representing an A_r of 33%). A vertical load test was undertaken to simulate embankment loading conditions. In-situ shear vane testing showed that the average undrained shear strength at a location within the stone column area lay approximately midway between lowest and median values recorded in ground immediately outside the trial area prior to stone column installation. Load cells placed on top of stone columns and intervening clay soil prior to application of load recorded stress concentration ratios of between 2.6 and 3.0. Pore pressure measurements indicated that a large stress increase at the completion of load application occurred at a depth equal to half the width of the loaded area. The load test was terminated when total recorded settlement had reached 300 mm in the centre of the test area, with no total failure of the ground reported (Figure 2.11). Field measurements were subsequently compared with predictions using an elasto-plastic theory (Goughnour and Bayuk, 1979a).

Bergado and Lam (1987) reported the results of field trials to investigate the behaviour of granular 'piles' (columns) with different densities and containing different proportions of sand and gravel, installed in soft Bangkok clay by the compozer method (see Section 1.3). Table 2.1 shows that for the same granular (stone column) materials the ultimate bearing capacity increases with number of blows per layer during installation attributed to an increase in density and angle of internal friction. The resulting load-settlement curves for the different proportions of gravel and sand are compared (Figure 2.12) and indicate a higher ultimate capacity for pure gravel and which equates to the higher reported friction angles in the literature for compacted gravels compared to those for compacted sands. The average deformed shape of the granular columns was described as typically bulging type and it was observed that the maximum bulge occurred near the top of the column. The authors indicate that with an initial diameter of 300 mm, the measurements of bulging recorded were in close agreement with the field observations of Hughes et al. (1976).

Greenwood (1991) reported the results of field trials to assess the suitability of stone columns for limiting settlements for a southern approach embankment to the Humber Bridge (UK), and which included some interesting and very significant observations. Prior to construction of an 8.0 m high trial embankment comprising rolled chalk fill, stone columns were installed by the wet top-feed technique on a 2.25 m triangular grid pattern and extending through a stiff surface crust and underlying soft alluvial soils to the level of competent glacial till (Figure 2.13a), resulting in stone column lengths of around 9.0 m. Following stone column installation, approximately 1.0 m of overburden was removed to facilitate both direct determination of stone column diameter and installation of pressure cells on and between stone columns (bedded on around 150 mm of sand), to facilitate measurement of stress ratio during and subsequent to embankment construction. An average stone column diameter of 775 mm was determined and which for the spacing defined above equated to an area replacement ratio (A_r) of around 10%. A further sand layer was placed and levels reinstated to original ground level using crushed chalk fill. Staged embankment construction was achieved using compacted chalk fill. Post compaction density determinations yielded an average value of 2.08 tonnes/m^3 and with this uncharacteristically high value interpreted as indicative of collapse of the chalk structure having taken place under the applied rolling (compaction) stresses during placement. Additional instrumentation installed included piezometers and inclinometers, together with induction settlement gauges at three levels (with rod settlement gauges encased in independent tubes extending through embankment fill layers (Figure 2.13a)). Recorded settlements under the centre of the embankment are shown in Figure 2.13b. A settlement reduction factor of 1.3 was reported. The clear step in settlement in the weakest alluvial stratum was observed when the maximum embankment height had been achieved, interpreted by the author as representative of plastic bulging. Measured stresses and stress ratios are shown in Figure 2.13c and Figure 2.13d respectively. During the trial, stresses recorded on the soil between columns was initially high at around 80 kN/m^2 and remained fairly constant during embankment construction, only showing a tendency to rise in the final stages of loading when overburden weight exceeded the pre-stress imposed by the compaction rollers so that only the columns would reflect the increase in weight (stress) beforehand and with recorded settlement approaching around 1.0 m. The author considered that heavy compaction of the chalk fill created intense local direct stress, partially maintained by capillary suction, giving rise to a residual stress comparable to that in an over-consolidated crust, estimated to be around twice the overburden pressure above the (pressure) cells, with the compacted fill effectively behaving as a raft with sufficient rigidity to span the columns and supported by the fall in column stress as pore pressures dissipated after the initial increment of loading. Uniform yielding of both column and soil to applied load and associated pore pressure dissipation should have increased the resistance to bulging, with corresponding settlements increasing rather than levelling off. If the described rafting mechanism was taking place the author states that pore pressure dissipation would have permitted radial consolidation as bulging occurred with consequent reduction of stress on the column as observed and recorded. With further increase of load to the second plateau bulging would be exacerbated and the rafting mechanism partially over-come, shedding more load onto the intervening soil. The conclusion drawn from this detailed case study was that the nature of the loading conditions has a significant effect on the performance of stone columns. Figure 2.13c illustrates that due to pre-stressing the soil barely changed with applied load. Extra load went into the columns, and stress ratios increased, thus reversing the expected behaviour.

Greenwood (1991) describes a field trial at a site in Uskmouth, Scotland (UK) incorporating the installation of an isolated stone column through a soft normally consolidated clay profile, to the level of a firm stratum at 11 m depth (Figure 2.14a). During column installation by the wet top-feed technique, the installation was interrupted to allow 600 mm diameter stress gauges to be incorporated within the column length at depths of 1.83 m and 3.66 mm (Figure 2.14a) and also at the surface to allow stresses to be determined at three levels in the stone column during loading. Load was applied to the stone column via a 660 mm diameter steel plate by a 30 tonne hydraulic jack using an appropriate reaction load to facilitate application of a maximum applied stress of 630 kN/m^2. Settlements were measured until values of 25 mm were recorded, occurring at a surface bearing stress of 330 kN/m^2, and it was evident that both settlement and stress readings were allowed to become sensibly constant at each increment. A Chin plot of the recorded settlement by the author suggested an ultimate strength of 704 kN/m^2. Following completion of the test the upper 4 metres of column was carefully excavated and its diameter measured at between 810 and 890 mm with a mean of 850 mm and the maximum diameter recorded at 2m depth below ground level, i.e. below the soil crust. At the same time hand shear vane tests were undertaken in the clay surrounding the column, which in the upper zone at least, showed enhanced values of cohesion by a factor of approximately 1.5. It is important to note that whilst bulging resistance was calculated at 630 kN/m^2 on the basis of initial soil strength: this would arguably have been enhanced if stone column installation resulted in a real soil strength gain as implied by the post installation hand vane tests. From Figure 2.14b it is evident that at higher loadings the upper cell registered higher stress levels than those applied at the surface which on face value would seem impossible. A possible explanation proposed by the author was that stress re-distribution due to deformation below the crust caused the crust to transfer its weight to the column by skin friction. In the early stages of loading little stress was transferred deep into the column because the skin friction against the strong soil crust sustained the stresses distributed from the steel test plate. With an average crust cohesion of 45 kN/m^2 over a two metre depth the author indicated that the plate stress would have had to reach about 420 kN/m^2 (Figure 2.14b) before this was fully mobilised as reflected in the changing gradients of the settlement plots. The test clearly confirmed the Hughes and Withers (1974) hypothesis and as indicated by the author also clearly demonstrated the practical influence of a stiff crust over soft material.

2.4 Numerical modelling

Many of the historic 1970's finite element modelling approaches developed have been based on unit cell idealisation where the variable boundary conditions of different unit cells are neglected, in order to facilitate understanding of the settlement behaviour of the foundation and the interaction between columns and surrounding soil. Whilst the unit cell approach is relatively simple, it is important to recognise that it relies heavily on important assumptions and which may not be applicable to every situation, e.g. under flexible loading conditions, and which eventually led to the development of homogenisation finite element approaches in the mid 1980's. These are based on the assumption that the stone column material is uniformly distributed throughout the full column reinforced zone, i.e. a homogeneous composite material prevails. This effectively means that there will be a volume replacement ratio as opposed to an area replacement ratio. The primary advantage of this technique over the unit cell method would appear to be non-restriction of boundary conditions. It can also model the yielding in both column and soil materials (Schweiger and Pande, 1986). Some examples of both the unit cell and homogenisation approach are discussed below:

2.4.1 Unit cell methods

Balaam et al. (1977) undertook finite element analysis of large groups of stone columns employing the unit cell concept, assuming load was applied to both column and the surrounding soil. Undrained settlements were found to be small and were therefore neglected. The ratio of modulus of the stone column to that of the clay (modular ratio) was assumed to vary between 10 and 40, and the Poisson's ratio of each material was assumed to be 0.3. A coefficient of at rest earth pressure $K_o = 1$ was used. Only about 6% difference in settlement was found between elastic and elastic-plastic response. The amount of stone column penetration into the soft clay layer and the diameter of the column were found to have a significant effect on settlement (Figure 2.15) the modular ratio of stone column to soil was considered of less importance.

Barksdale and Bachus (1983) presented a series of design curves (for predicting primary consolidation settlement), obtained from finite element analysis, (Figure 2.16) claiming that the finite element program could solve small or large displacement, axisymmetric or plane strain problems. For a non-linear analysis, load was applied in small increments and computation of incremental total stresses were undertaken by solving a system of linear, incremental equilibrium equations for the system. By assuming the uniform stress condition in the stone and clay, only one vertical column of elements was used to model the stone and one to model the soil. Field observations have shown that under surface loading, both column and clay deform horizontally (Munfakh, 1984), the implication being that the rigid boundary condition assumption adopted in the unit cell analysis is not correct. To overcome this, an attempt was made by the authors in the unit cell model to place a soft compressible boundary in lieu of the original rigid incompressible boundary to the unit cell (Figure 2.17a and b).

2.4.2 Homogenisation techniques

Schweiger and Pande (1986) applied homogenisation analysis to the investigation of the performance of flexible and rigid rafts supported by stone columns. Analysis was undertaken under drained conditions. In the finite element mesh, elements directly beneath the 15 m diameter footing analysed were assigned properties corresponding to stone column reinforced clay. Load-settlement curves, taking into account both dilatant and non-dilatant behaviour were generated for the flexible and rigid rafts respectively. For the latter, this represented settlement reductions of 25% and 50% for replacement ratios of 10% and 30% respectively. The analysis also showed the development of plastic zones under the flexible raft. As the applied stress increases, this plastic region extends deeper into the foundation. As might be expected, the plastic zone underneath a rigid footing was shown to be relatively small.

Pande (1994) and Lee and Pande (1994) developed the homogenisation approach further by assuming that both stone column and clay behave elasto-plastically. They indicated that the stone column obeys the Mohr-Coulomb criterion with a non-associated flow rule whilst the clay was represented by the modified Cam-clay model. Results from the numerical analysis were compared to the experimental data from model tests by Hu, 1995. The foundation geometry and material properties in both laboratory and numerical studies were compatible. Although the numerical analysis appears to over-predict the initial stiffness of the composite ground there was reasonably good agreement between the observed ultimate loads. The load settlement curve from the finite element analysis showed the reinforced ground behaving elastically until it reaches a well-defined peak; this is followed by plastic yielding. Lee and Pande (1994) attributed this softening behaviour to the generation of tensile stresses beneath the periphery of the stiff footing, taking the state of the soil to the dry side of the critical state model. This effect, together with the non associated flow rule of Mohr-Coulomb criterion, influenced the overall behaviour of the foundation.

Pande (1994) suggested that an area replacement ratio (A_r) of 24% was the upper limit for stone column foundations. Beyond this, he postulated that an increase in the area replacement ratio would not lead to further increase in bearing capacity. This somewhat contradicts Hu's (1995) findings, since the experimental results shows that an area replacement ratio greater than 25% was required for significant improvements in bearing capacity. This may be due, in part, to the fact that for low area replacement ratios the clay in the model tests might not have been fully drained, whereas the homogenisation analysis of Pande (1994) assumed a fully drained condition.

Lee and Pande (1994) found that vertical stresses beneath the footing were higher towards the edges. In order to provide cost savings the authors suggested varying the length of the stone columns, proposing short columns under the centre of the footing and longer columns towards the periphery, where stresses were higher. It appears that reducing the length of the central column had a positive effect on the load carrying capacity of the foundation. Furthermore the softening behaviour observed in the constant length tests was significantly reduced. It is important to recognise that laboratory experimental studies are required to validate such hypotheses, and whilst Hu (1995) does address this to some degree, the importance of field trials for such validation should not be under-estimated.

2.4.3 Other finite element studies

Killeen and Mc Cabe (2010) presented the results of a parametric study using the PLAXIS 3-D Foundation (Version 2.2) software to model the behaviour of rigid square pad footings supported by stone columns in order to investigate key factors relevant to the design of small groups of stone columns. The Bothkennar soil profile was adopted for the modelling. Details of the parametric study are given in Table 2.2. For a drained analysis for the ground profile modelled: settlement performance was noted to continue to improve beyond L/d = 10 and this improvement was more pronounced for groups of columns with a low area replacement ratio (A_r). Columns closer to the footing edge were found to perform better for short columns (L/d <10) than for columns closer to the centre, but the 'n' value (settlement reduction factor) converges with depth and long stone columns are relatively insensitive to column spacing. The stiffness of the stone backfill was noted to have a significant influence on the settlement performance of a footing supported by a large number of stone columns. However, as the number of supporting columns were reduced, so did the influence of the column stiffness; for a given area replacement ratio (A_r), and increased number of columns supporting a footing led to an increase in the proportion of group columns that have full confinement, i.e. behave like a unit cell, resulting in enhanced settlement performance of the footing.

2.5 Bearing capacity and settlement analysis of stone column reinforced ground

2.5.1 Introduction

The composite nature of stone column reinforced ground compels a working together of stone column and soil, resulting in a load sharing mechanism between soil and column, and by inference the load bearing behaviour of the composite ground is therefore influenced by the behaviour of both stone column material and soil, as is the settlement behaviour. Reviewing available literature, case histories and discussions with Specialist Contractors and Practitioners, the author has been able to briefly summarise the chronological development and current state-of-the-art in respect of the design of stone column reinforced foundations – relating specifically to bearing capacity and settlement.

2.5.2 Load carrying capacity

Thorburn and MacVicar (1968) proposed an empirical approach based upon the relationship between the allowable working load on a single column and the undrained shear strength of the soil (Figure 2.18). The chart appears to be based mainly on the lead author’s experience, as a Consultant, of stone columns applied to strip footings for the foundations of low-rise buildings in Glasgow during the early 1960’s. The authors recommended the use of their design curves assuming that all building loads are carried by the stone columns. Since the intervening soil will invariably carry some of the load it is claimed that there was a built-in factor of safety. Quantification of this factor of safety is difficult since it will depend upon the type of foundation and soil-structure interaction. However, this empirical design procedure is likely in certain cases to lead to highly over-conservative design.

Vesic (1972) proposed a theoretical approach to bearing capacity based upon cylindrical cavity expansion theory and includes both cohesive and cohesionless soils. The behaviour of the material was assumed to be elastic initially and then plastic once the strength is reached. The expression for the ultimate resistance (cavity pressure) is thus:

$$\sigma_{rl} = c'F'_C + p\,F'_q \quad \text{--} \quad 2.3$$

where: F'_C and $F'q$ are cavity expansion factors, which are functions of the internal friction angle of surrounding soil and the Rigidity Index $I_r = G/(c + p \tan Ø)$, where G is the shear modulus of surrounding soil and p is the mean isotropic effective stress at the equivalent failure depth.

The Hughes and Withers (1974) approach to bearing capacity has been discussed previously in Section 2.2.

Thorburn (1975) undertook some further development of the Thorburn and MacVicar, (1968) method and presented an empirical approach for the prediction of the allowable stress on a single stone column by relating it to the undrained shear strength of the soil as shown in Figure 2.19. The approach seems to correlate well with the Hughes and Withers (1974) approach, as described in Section 2.2. When calculating the allowable stress Thorburn (1975) assumed that the effective diameter of the column would decrease as the strength of the soil increased (Figure 2.19). As Figure 2.19 shows, the approaches seem to correlate well for soft soils; however in stiffer soils the high stresses predicted by Thorburn are probably as a result of the reduced cross-sectional area of the column, which would be expected in stiffer soil.

Greenwood (1975) considered that the maximum column bearing capacity is achieved when the ratio of applied stress on the column to passive restraint at the critical depth is a maximum i.e. the peak stress is first achieved at critical depth. On this basis, he provided a solution for estimating the bearing capacity of a single column in a group using passive earth pressure coefficient as:

$$q_{ult} = K_{ps}(\gamma z K_{pc} + 2c\sqrt{K_{pc}} + xqK_{pc}) \quad \text{---} \quad 2.4$$

where: γ is the total unit weight of soil; K_{ps} and K_{pc} are the ratio of horizontal passive stress to vertical stress in the stone column and soil respectively; c is the undrained cohesion of the soil and z is the depth of soil. x is the critical depth where bulging and end bearing failure occur simultaneously. Greenwood (1975) considered a hexagon-shaped unit cell for individual columns in a group and modified equation (2.4) by using an area replacement ratio parameter so that:

$$p_s = qA - K_{pc}aK_{ps}[\sigma_{vs} + x_q] / A\text{-}a \quad \text{--} \quad 2.5$$

where: A = the total area of the unit cell

a = the total area of soil in the cell. Other parameters refer to function (2.4)

For a structure with low settlement tolerance, Greenwood suggested that all load should be considered to be carried by the columns only in order to be on the safe side. Although equation (2.5) seems to consider the column and surrounding soil since the area ratio is introduced, the critical depth is still determined on the basis of the single column behaviour so that virtually no group effect was considered at all.

Brauns (1978) proposed an approach based on Vesic's (1972) theory. He assumed no side friction existing between column and clay and no volume changes for a single stone column in cohesive ground, thus the ultimate lateral stress that can be mobilised in a stone column by surrounding cohesive soil is given by:

$$\sigma_{rl} = c\,(1 + \log I_r) + \sigma_v \quad \text{---} \quad 2.6$$

where:

$I_r = G/C_u$. A comprehensive comparison of these approaches based on cavity expansion theory was also made by Brauns (1978) (Figure 2.20). Other analyses and approaches by Brauns (1978) are also summarised in Figure 2.20.

Barksdale and Bachus (1983) recommended the use of 'past experience and good engineering judgement' in parallel with the theoretical approach, i.e. on the basis of the Vesic's (1972) cavity expansion theory, when determining the design working load of a stone column. Their approach for estimating the bearing capacity of a single stone column was presented in the following form:

$$q_{ult} = c_u N_c \quad \text{---} \quad 2.7$$

where: q_{ult} is the ultimate bearing capacity of the stone column, c_u is the undrained shear strength of the *in situ* material and N_c is a bearing capacity factor for the stone column.

The choice of N_c is semi-empirical and depends on the compressibility of the surrounding soil. N_c is stated as usually ranging between 18 and 22 depending on the compressibility of the soil surrounding the column, with a higher value of N_c reflecting a higher soil stiffness. The authors recommend that the stress concentration should be taken into account, when estimating N_c from field tests. They also suggest that the ultimate bearing capacity of the surrounding soil may be taken as $5c_u$ with an upper limit of $\alpha_c\sigma$, where α_c is the ratio of stress in the clay, σ_c, to the average stress over the tributary area, σ. Despite acknowledging its conservatism the authors believed it sufficient to mutiply N_c by the number of columns in the group to determine the capacity of the foundation. In terms of the ultimate bearing capacity of a group of stone columns underneath a rigid pad or strip footing, this may be considered as dependent upon the lateral resistance, σ_3, of the block of soil beneath the footing and the composite (soil-stone column) shear resistance along the inclined shear surface. This theoretical approach assumes that the foundation fails on a straight failure surface and that the strength is fully mobilised in both the column and the soil. The cohesion of the composite foundation, c_{avg}, the slope of the failure plane, β, and the lateral resistance of the soil block, σ_3, can be determined using the following equations.

$c_{avg} = (1\text{-}A_s)c_u$ -- 2.8

$\beta = 45^\circ + \varphi'_{avg}/2$ -- 2.9

$\sigma_3 = \frac{\gamma_c B \tan\beta}{2} + 2c_u$ -- 2.10

where: c_u is the undrained shear strength of the *in situ* soil, A_s is the area replacement ratio, φ'_{avg} is the composite angle of internal friction, γ_c is the unit weight of the soil and B is the width of the footing. If σ_1 (or q_{ult}) and σ_3 are the principal stresses then for achievement of equilibrium of the soil block, the following relationship is valid:

$q_{ult} = \sigma_1 = \sigma_3 \tan^2\beta + 2c_{avg} \tan\beta$ -- 2.11

This analysis considered foundation geometry and the geotechnical properties of the column and soil materials. Barksdale and Bachus (1983) recognised that column bulging was not taken into account, and on this basis recommended that this approach (method) should only be applied to soils which have undrained shear strengths greater than 30 kN/m^2. For groups of stone columns in softer soils the ultimate bearing capacity should be predicted by multiplying q_{ult} from Equation 2.7 by the number of columns in the group.

Priebe (1991) also presented two methods for determination of the bearing capacity for stone column reinforced soil based upon German DIN Standards, for predicting the bearing capacity of a footing supported by a limited number of stone columns. The first method (Method 1) requires the average angle of internal friction, φ'_{avg}, which may be determined using design charts in the most recent DIN Standard 4017 (2006). An average cohesion, c'_{avg} is then also calculated along the assumed failure line of the foundation. The following equation can then be used to estimate the design load, $P_{ult.}$

$P_{ult} = A_f = (c'_{avg} N_c \upsilon_c + \gamma_d N_d \upsilon_d + \gamma B N_b \upsilon_b)$ -- 2.12

where: A_f is the area of the footing, c'_{avg} is the cohesion of the soil, γ is the unit weight of the soil, d is the footing depth and B is the footing width. N_c, N_d and N_b are bearing capacity factors (based on φ'_{avg}) and υ_c υ_d υ_b are shape factors for the treated ground.

Method 2 used other design charts where the failure line of the untreated ground is extended below the footing, to a depth equivalent to an assumed footing width, B. (Priebe used the German DIN Standards 4017 to determine the approximate failure line). The design load is calculated using the following relationship assuming bearing capacity factors, shape factors and the angle of internal friction of the untreated ground:

$$P_{ult} = A_f\,(c' N_c \upsilon_c + \gamma_d N_d \upsilon_d) \text{ ---------- } 2.13$$

In this context, A_f is the area of the footing using the assumed width, B.

Priebe (1991) also compared these two approaches to that of Barksdale and Bachus (1983), using a worked example. The three methods yielded similar results. Priebe (1993) subsequently developed a design chart, to determine the proportion of load carried by a stone column in an infinite grid, m, as a function of area ratio and friction angle of the column material.

2.5.3 Settlement reduction

For vibro stone column design in soft soil, settlement criteria are normally the governing factor, generally ahead of bearing capacity. Calculation of post-treatment settlement is important since vibro stone columns are principally applied in soft clay soils and work with the soils rather than by-passing them (as is the case with piled foundations), so assessment of what can be realistically achieved is important. Different techniques have been developed to estimate settlements ranging from settlement of single columns loaded at their top only to settlements of footings on large groups of columns subject to uniform load using the unit cell concept. It is also important to recognise that settlement prediction can only be as accurate as the site investigation information upon which it is based (Serridge, 2008). One of the earliest approaches to assessing settlement of stone column reinforced soil was by Mattes and Poulos (1969), who presented an analytical solution for preliminary settlement prediction of a single compressible column in a semi-infinite mass. Using this approach, which was based on linear elastic theory, the settlement was calculated as follows:

$S_{col} = [P/(E_s.I)].\ I_p$ -- 2.14

where,

S_{col} = settlement of the granular column

P = total applied vertical load

E_s = Youngs modulus of the clay soil

I = Length of granular column

I_p = Displacement influence factor

I_p depends on the column stiffness factor whose value is equal to the ratio between the Youngs's modulus of the column material to that of the surrounding soil. The drained and undrained values of Young's modulus were measured using soil tests and implemented according to the loading condition. The authors highlighted that the major part of the total final settlement occurs as immediate settlement. Using the Mattes and Poulos (1969) approach, it was demonstrated that the difference between drained and undrained settlement does not exceed 10% such that the influence of void ratio is insignificant. Hughes and Withers (1974) provided a relatively simple elastic method to calculate the linear distribution of vertical stress by simply taking the limiting stresses up the side of a single column to be the undrained cohesion of the soil so that a depth z

$\sigma_{vz} = \sigma_v + (\rho d - 4C_u)$ -- 2.15

where d is the diameter of the column. From this the critical depth at which end bearing and bulging failure occur simultaneously was determined to be about 4.1d. Such a result is supported by Mattes and Poulos (1969) in their linear elastic analysis for a single floating compressible 'pile' where the 'pile stiffness factor' for a stone column is taken to be in the range 30 to 50.

Since 1970, however, several empirical approaches to the analysis of stone column reinforced clay soils have been proposed. The most common, recognised by Aboshi and Suematsu (1985) include: Greenwood method; Priebe Method; Equilibrium Method; Incremental Method; Granular Wall method and Finite Element Method. These approaches, all of which are based upon unit cell idealisation, with the exception of the finite element method, which has been discussed previously in Section 2.4, are discussed below. Brief mention is also given to the Baumann and Bauer (1974) approach, which although more recognised and exploited for determining the proportion of applied foundation stresses carried by the stone columns and intervening soil, also had a settlement aspect attached to it.

Greenwood Method (1970)

The earliest experience based attempt to predict the settlement of ground reinforced with stone columns was proposed by Greenwood (1970). Empirical curves, which are based on field experience, for estimating consolidation settlements under widespread loads for uniform soft clay layers reinforced with (granular) stone columns, as a function of undrained shear strength and stone column spacing, were presented by Greenwood (1970) as shown in Figure 2.21. The design curves are based on the assumption that the columns are founded on a firm stratum. Immediate settlement and shear displacements were neglected. As can be seen from Figure 2.21 the magnitude of the composite ground settlement is not only related to the column spacing and the clay soil (undrained) shear strength but also the method of stone column installation. According to Figure 2.21 stone columns installed using the dry top-feed process must be installed closer together to achieve the same settlement reduction which would be attained using the wet top-feed process. This is attributed to the fact that smaller diameter stone columns are formed by the dry process, because of the differing methods of installation for vibroreplacement (wet) and vibrodisplacement (dry) techniques (see Chapter 1). Although immediate settlements and shear displacements are neglected, this approach compares well with many of the more recent numerical and theoretical methods of settlement prediction. It is a useful chart for preliminary assessment, because of its relative simplicity. Greenwood and Kirsch (1984) noted that care must be exercised, however, when contemplating designs outside the range of data from which the curves have been developed.

Baumann and Bauer (1974) considered the total settlement divided into two parts: firstly immediate settlement of the stone column, S_1, for which no volume change of the soil is assumed and secondly, consolidation settlement, S_2, where Terzaghi's classical one-dimensional consolidation theory is adopted. The approach is similar to Priebe's (1976) method but uses a weighted modulus as a modification. This approach, which considered the load only applied to the column area, was adopted to determine the vertical stress distribution under the centre line of a circular footing. This is quite an unrealistic representation of the field situation and it was recognised by the authors that the approach was only an 'analytical model'.

Priebe Method (1976)

Priebe (1976) proposed an analytical approach for estimation of settlement of stone column reinforced soil, essentially based upon elasticity and Rankine earth pressure theory, with the assumption of an infinite grid of stone columns beneath a rigid raft by considering a stone column within a cylindrical elastic half space with no change in lateral stress with depth. The unit cell approach was adopted and the analysis involved the summation of settlements of discrete slices of the unit cell to predict the overall settlement performance of foundations on an infinite grid of stone columns. The following idealized assumptions were made: a) the stone stone column material was incompressible (within elastic half space) with the columns allowed to deform in shear failure only, and with the surrounding soil still behaving in a quasi-elastic stress-strain manner or pattern; b) equal vertical settlement was experienced by stone column and soil; c) stresses were uniform in column and soil; d) the column was end-bearing on a rigid stratum. The bulk density of the column and soil is essentially neglected, hence they will not experience failure in end bearing with any settlement of the loaded area resulting in bulging of the column, which remains constant over its entire length.

The improvement achieved with the 'reinforcing' action of the stone columns is evaluated on the assumption that the column material effectively shears from commencement of loading whilst the surrounding soil reacts elastically. Furthermore, the soil is assumed to have been displaced to such an extent during the installation process that its initial resistance corresponds to the liquid state, i.e. the coefficient of earth pressure corresponds to K = 1. The results of the evaluation, taking Poisson’s ratio, μ =1/3, which was considered adequate for the state of final settlement in most cases, is expressed as the basic improvement factor n_0 as a function of area ratio and angle of internal friction of the column material (φ') (Figure 2.22).

Priebe defined an improvement factor as the ratio of settlement of untreated to treated ground S/S_t, which is expressed as:

$$\frac{S}{S_t} = 1 + A_s \,.\left[\frac{\frac{1}{2} + 2\,v\,.\frac{1 - A_s}{v + A_s}}{K_{ac}\,.2v\,.\;\frac{1 - A_s}{v + A_s}}\right] \qquad \text{-- 2.16}$$

where A_s is the area replacement ratio and K_{ac} is the active earth pressure coefficient of the column material. The Poisson's ratio, υ is taken to be 1/3. This solution is normally presented in the format of a design chart (Figure 2.22). In the above calculation, the effective stress resulting from the soil overburden is neglected. Priebe (1976) claimed that this is on the safe side. Further improvements were made to the design charts to take account of the compressibility of the column material and the effect of overburden (Priebe, 1993;1995), Figure 2.23a-c. Additional design charts were developed to predict the settlements of isolated pad and strip footings supported by a finite number of stone columns, based on the performance of a large grid of columns (Figures 2.24a and b). The curves are used to determine settlement ratio, S/S_s where S is the expected footing settlement and S_s is the total settlement of an unlimited column grid beneath an unlimited area.

Incremental method

The incremental method, developed in a theoretical study by Goughnour and Bayuk (1979a), also adopts the unit cell concept and provides an extension of earlier approaches by Priebe (1976), for example. The theory idealises the stone column as behaving elastically until yielding and then undergoes plastic deformation (Figure 2.25a). The effective stress path of the clay is assumed to commence at the K_o line and to be bi-linear as stresses increase and consolidation progresses. The ratio of horizontal to vertical stress, K, varies between K_o and $1/K_o$ (K_o represents the lateral resistance provided by the surrounding clay), Figure 2.25b. The analysis is undertaken for successive disc-shaped increments of the unit cell model that make allowance for the changes in confining pressure with depth. The vertical strain is calculated for each element under two separate conditions, with the first assuming that the column is rigid-plastic and incompressible, while the second considers the stone columns as a perfect elastic material and with the vertical strain adopted taken as the larger of the two computed values. Radial consolidation of the clay is considered using a modification of the Terzaghi one-dimensional consolidation theory. A more complete and detailed description of the theorectical approach is described by Goughnour (1983). It is important to recognise that the approach requires detailed iterative calculations which makes the analysis very complex. Goughnour (1983) did extend the research which culminated in the production of a series of design charts for ease of hand calculation Figure 2.26. The incremental method was employed to predict settlements associated with the field trials at Hampton, Virginia (US) (Goughnour and Bayuk, 1979b, see chapter 2, section 2.3). Good agreement of settlement prediction and field observation was realised in the central columns, although predictions at the corner seemed somewhat conservative and appeared to over-estimate the settlement.

Equilibrium Method

The equilibrium method has been used in Japan for prediction of settlement of sand compaction pile (SCP) reinforced clay under a flexible raft. The theory was initially put forward by Aboshi et al. (1979). Barksdale and Bachus (1983) and Barksdale and Goughnour (1984) have also provided input. Barksdale and Bachus (1983) recommended that the approach offered a simple yet realistic engineering approach for estimating settlement associated with the introduction of vibro stone columns into soft clay soils. The following assumptions were identified as necessary in developing the equilibrium method: (1) the extended unit cell idealization is valid; (2) the total vertical load applied to the unit cell equals the sum of the force(s) carried by the stone and the intervening soil, which basically implies that an equilibrium condition is maintained in the column-clay interface; (3) the vertical displacement of stone column and soil is equal, and (4) a uniform vertical stress due to external loading exists throughout the length of stone column, or else the compressible layer is divided into increments and the settlement of each is calculated using the average stress increase in the increment. Following this approach, as well as other methods, settlement occurring below the stone column reinforced ground must be considered separately; usually these settlements are small and can often be neglected (Barksdale and Bachus, 1983). Based upon 1-D consolidation theory, the settlement reduction ratio, the ratio of settlement of the stone column treated ground, S_t, to that of an untreated ground, S, in a given vertical increment is expressed as:

$$\frac{S_t}{S} = \frac{\log_{10}\left(\frac{\sigma_o + \mu_c\,\sigma}{\bar{\sigma}_o}\right)}{\log_{10}\left(\frac{\bar{\sigma}_o + \sigma}{\bar{\sigma}_o}\right)} \quad \text{-- 2.17}$$

where, the σ_o σ, and μ_c represent the average initial effective stress in the clay layer, the effective stress due to load and ratio of stress in the clay and the column respectively. In summary, this simple approach makes the level of settlement reduction of stone column reinforced soil a function of stress ratio, the initial effective stress and the magnitude of applied stress, with a recommendation by Barksdale and Bachus (1983) that the stress ratio should be estimated from field measurements or past experience, since no realistic analytical solution was available at the time. As a consequence, the approach is viewed as semi-empirical.

Granular wall method

Van Impe and De Beer (1983) presented a relatively simple method for estimating the improvement in the settlement behaviour of soft clay soils due to the presence of stone columns, defined as the granular wall method. The method considers two cases: (1) that the stone columns deform, at their limit of equilibrium, at constant volume; (2) that the stone columns are deforming elastically under the applied foundation load. These mechanisms are summarised in Figures 2.27a-c where D = diameter of the stone column a = shortest centre to centre spacing. d_f represents the equivalent thickness of the stone wall and in Figure 2.27b is given by:

$d_f = \pi.D^2/(4ab)$ -- 2.18

In Figure 2.27b the shear stresses between column and surrounding soil are neglected and the resistant layer beneath the soft layer is considered to be incompressible.

For case 1 above, in the context of Figure 2.27b, this means:

$d_f \, . \, H = (d_f + 2S_h).(H\text{-}S_v)$ --- 2.19

where:

S_v = the vertical settlement of the stone wall (equal to that of the soft layer)
S_h = the horizontal deformation of the stone wall (equal to that of the soft layer)
H = the initial height of the stone walls.

The interaction between the in-situ soil and the stone column is demonstrated by a rheologic model, Figure 2.27a. In this model the load is transferred onto the column and soil. The mid-section of the model represents the interaction between stone column and soil. The bulging of the column creates a reaction force in the soil which laterally supports the column and increases the proportion of load carried by the soil. The authors indicate that the only parameters required are the spacing of the columns and their diameter, the angle of shearing resistance φ' of the stone column material, the oedometer modulus of the soft soil and its Poisson's ratio. The improvement parameters are deduced from diagrams similar to those in Figure 2.28. It is mentioned that the computation method has been applied to large storage tanks on soft soil improved with stone columns and that measurements of the settlements proved the indicated computation method to be very reliable although no supporting data is provided. The approach proposed by Van Impe and De Beer (1983) seems to mostly have been applied in Belgium but does not appear to be in common use outside this country.

2.5.4 Comparison of settlement approaches

Attempts have been made by several researchers to provide and plot comparisons of the various settlement approaches (including numerical approaches) for vibro stone columns, e.g. Balaam and Poulos (1983); Mitchell and Huber (1985); Greenwood and Kirsch (1984); Slocombe (2001), Charles and Watts (2002), Ambily and Ghandi (2007) and Mc Cabe et al. (2009). Some of the more significant settlement approaches for stone column design have been discussed in Section 2.5.3. The well recognised settlement approach comparison chart by Greenwood and Kirsch (1984) is given in Figure 2.29.

The settlement approaches compared in Figure 2.29 were principally elastic methods or based on variants of elastic theory and useful discourse on the data and their interpretation is given by Greenwood and Kirsch (1984). In Figure 2.30, Charles and Watts (2002) compare the stiffening effects of stone columns. Some of the more recent significant work is discussed below:

Slocombe (2001) provided a comparison of the Baumann and Bauer (1974) and Priebe (1976, 1995) approach. Key parameters for the Baumann and Bauer (1974) approach were identified as:

K_s = assumed to lie between K_o and K_p
K_c = assumed to lie between K_a and K_o but usually taken as K_o
E_s/E_c = ratio of stiffness of soil to column

with key parameters for Priebe (1976;1995) identified as:

K_{ac} = active earth pressure coefficient for column
K_o = assumed to be 1 for all soils
φ' = friction angle of stone column
E_c/E_s = ratio of stiffness of column to soil
υ_s = Poisson's ratio for the soil

It was suggested by Slocombe (2001) that one of the issues with the Baumann and Bauer (1974) approach is that the usually quoted values of K_s do not appear to follow the normally accepted trend of lower values with finer-grained soils. This results in the Baumann and Bauer (1974) method predicting a settlement improvement ratio of about 40% higher in clay soils than the Priebe (1995) approach (Figure 2.31). From the authors (researchers) own experience and also discussions with ground improvement Specialists and Practitioners, the Priebe (1995) approach is considered more realistic in clayey (cohesive fine-grained) soils and the Baumann and Bauer (1974) approach more appropriate for essentially granular soils.

Mc Cabe et al. (2009) has compared settlement data from a number of sites, with settlement performance captured in the form of a settlement improvement factor *n*, defined as :

$n = S_{untreated}/S_{treated}$ -- 2.20

where: $S_{untreated}$ represents settlement (of the loaded zone) without stone column treatment and $S_{treated}$ is the corresponding settlement with stone column treatment. The data captured comprised two components identified as follows – (1) projects where $S_{treated}$ and (a reference value of) $S_{untreated}$ were both measured, so the value of *n* was completely measurement based (Table 2.3); (2) projects where $S_{treated}$ values were measured but $S_{untreated}$ values were not, but instead either predicted analytically or from experience of observations and measurements in similar ground conditions (Table 2.4)
.

The area ratio, A/A_c (A_r) values, based upon Priebe (1995), 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, were used to capture the concentration of the column array in an infinite grid (as discussed in Chapter 1). The A/A_c is deduced from the column diameter D and spacings according to:

$A/A_c = k\,(s/D)^2$ -- 2.21

where: k is $4/\pi$ and $2(\sqrt{3})/\pi$ for square and triangular grids respectively.

A/A_c data are also given in Tables 2.3 and 2.4 for widespread loading cases, with A_c either measured directly or indirectly from stone consumption records. Values of 'n' from Tables 2.3 and 2.4 were plotted by Mc Cabe et al. (2009) against A/A_c in Figure 2.32 for the widespread loading cases (together with three localised loading cases, two of which were for square pad footings and one of which was for a rectangular strip footing). In order to provide a point of reference for the data Priebe's basic n_o curve for a friction angle $\varphi' = 40^\circ$ for the stone column was added (with Poisson's ratio of the soil $\upsilon_s = 0.33$ assumed, as is normal practice). This assumption appears to have been made due to case or site-specific values of φ' not generally being presented in the literature.

The value was also regarded as fairly typical for design in soft clays. Moreover, additional parameters needed for predicting Priebe's (1995) n_1 and n_2 factors to account for column compressibility and soil and column unit weights repectively were not readily available. Attention therefore focussed on n_o values as previously intimated.

Reference to Figure 2.33 shows that whilst there is a spread of data around the Priebe (1995) n_o curve the match can be described as reasonably good, particularly given that there was inconsistency of conditions across all data sets (test sites) and given the inevitable uncertainties associated with the assumptions that had to be made when analysing the published data. It is nevertheless clear that Equation 2.22 predicts the shape (trend) of the measured n-A/A_c variation reasonably well, despite there being insufficient resolution from the published data to take account of all the factors that would have influenced the degree of settlement control.

$$n_0 = 1 + \frac{A_c}{A}\left[\frac{5 - \frac{A_c}{A}}{4(1 - \frac{A_c}{A})\tan^2(45 - \frac{\phi'}{2})} - 1\right] \qquad \text{--- } 2.22$$

Mc Cabe et al. (2009), considered these factors to include:

(a) Variations in prediction methods used to determine *n* values in Table 2.4.
(b) Uncertainty as to 'as-constructed' column diameter and spacing.
(c) The stage of loading or time period after loading at which *n* is measured is not consistent throughout all the case studies.
(d) Settlement recorded at the ground surface will reflect the settlement over the treated depth range plus any additional settlement generated below the columns and which may be more relevant to partial depth treatment. It is not always possible to separate out these contributors to overall settlement. In some instances settlements may therefore have been under-estimated.

Greenwood and Kirsch (1984) have also previously identified some of these contributory factors during their investigation of the comparison of settlement approaches to vibro stone column design.

2.6 Recycled aggregates in vibro stone columns

It is perhaps important to recognise that even in the mid 1990's, up to two-thirds of construction wastes were recycled for low-grade use close to site and for landfill engineering, but with only 4% used to produce recycled aggregate (RA) however (Sherwood, 1995), with limited if any application in vibro stone column techniques, and this despite the fact that recycled aggregates are referred to in the Institution of Civil Engineers (ICE) Specification for ground treatment (1987). Such low-grade recycling did little to reduce our dependency on high-quality primary aggregate (PA), so it could be argued that the more significant issue was to seek higher value application for recycled aggregates, such as in vibro stone columns (VSC's).

Pressure to demonstrate environmental sustainability within the ground improvement sector is resulting in an increase in the use of recycled and secondary aggregates in VSC's These aspects are covered in more detail by Slocombe (2003), Serridge (2005), Serridge and Sarsby (2010) and Jeffersen et al. (2010) among others, with these authors providing some examples of successful application (see Appendix 2.2 and 2.3 and 2.4 with regard to Serridge (2005); Serridge and Sarsby (2010) and Jeffersen et al. (2010) respectively). Current utilisation of recycled (and secondary) aggregates in the UK vibro ground improvement sector is around 25-30% (Serridge, 2005; Serridge and Sarsby, 2010). Ways in which this can be enhanced include changing perceptions of recycled aggregates and increasing awareness of potential sources and their sustainability, building up case histories and ensuring similar levels of quality control are implemented as those applied to primary aggregates. Early contractor involvement is also encouraged to maximise opportunities where sites are being re-developed (with concrete structures and floor slabs being broken up), with opportunities for crushing and screening and use as recycled aggregate in any vibro stone column technique being subsequently employed on site (Serridge, 2005; Serridge and Sarsby, 2010).

2.7 Concluding remarks

It is evident that a significant amount of research has been carried out in the last four to five decades and this has given rise to an improved understanding of the behaviour and capacity of stone column reinforced clay soils. This chapter has provided a review of some of the more significant research that has been carried out. It is evident that significant research on vibro stone columns in soft clay, covering both laboratory and field-based research, together with numerical modelling has been carried out. It should be recognised, however, that whilst there have been some useful developments in laboratory-based analysis of stone columns in soft clay soils (Hu, 1995; Mc Kelvey, 2002, Black et al. (2006) ; Black et al. (2007a) and Black et al. (2007b)), their exact behaviour is not fully understood beneath shallow, narrow footings with regard to both ground response to stone column installation using the dry bottom-feed technique and subsequent performance of the stone column reinforced clay soil under applied load in the field situation.

In the context of laboratory-based studies, the majority of research appears to have been carried out under unconfined loading conditions, i.e. ignoring the effect of surcharge from the overlying material, although latterly Mc Kelvey (2002); Mc Kelvey et al. (2004); Black et al. (2007a); Black et al. (2007b) and Black et al. (2010) have addressed this to some degree. In most cases, foundations for low-rise structures are supported on strip or pad footings or in some cases raft foundations. Most of the laboratory based research has considered circular footings, whereas most of the field-based research has focussed on widespread loads beneath rafts and embankments, with no significant investigation of narrow, shallow footings. It is important to recognise that laboratory-based investigations do not replicate the true field conditions in terms of calibration, scale effects and installation effects associated with a vibroflot (vibrating poker). It is therefore important to recognise that an essential component of any research and in particular any future research, is the undertaking of field trials, if valid conclusions applicable at the field scale are to be made.

Analysis and design of stone columns is based principally on simple analytical models using elasticity and plasticity theories (Greenwood,1970; Hughes at al., 1976; Balaam and Booker, 1981; 1985 ; Barksdale and Bachus, 1983; Priebe, 1976, 1991, 1995). It is evident that the relative simplicity of the Priebe (1995) method in applying a settlement improvement factor to conventional consolidation calculations makes the approach attractive to ground engineering practitioners, which accounts for its wide use and application within the vibro ground improvement sector. Numerical methods, particularly the finite-element method, have also been used for the analysis of the stone column-soil systems (Balaam et al., 1977; Balaam and Booker, 1981; Balaam and Poulos, 1983; Lee and Pande, 1994; Mitchell and Huber, 1985; Schweiger and Pande, 1986). With the advent of 2-D and 3-D numerical geotechnical software packages, it is evident that there has been limited investigation of the field behaviour of stone columns using this approach. A number of full-scale field trials have also been performed on stone columns installed in soft soils (Greenwood, 1970; 1991; Hughes et al. (1976); Munfakh et al. (1984); Mitchell and Huber (1985), Bergado and Lam (1987) amongst others. Laboratory-based analysis, as intimated above, has also been undertaken to investigate the behaviour of the composite stone column-soil system (Hughes and Withers, 1974; Charles and Watts, 1983; Hope, 1988; Hu, 1995; Mc Kelvey, 2002; Mc Kelvey et al., 2004; Black et al., 2006; Black et al., 2007a; Black et al., 2007b and Black et al., 2010). Whilst several case histories have been reported in the literature relating to vibro stone columns in soft clay it is proposed to review this separately in conjunction with a review of unsuccessful case histories, which although not widely publicised, nevertheless provide important learning tools.

The current research has been undertaken with the objective of trying to address some of the issues raised in the preceeding paragraphs relating to lack of research data.

Group	1			2			3			4		5	
No. of pile	G1	G2	G3	G4	G5	G6	G7	G8	G9	G10	G11	G12	G13
Proportion of sand in volume	1.0			1.0			1.0			0.3		0.0	
Proportion of gravel in volume	0.0			0.0			0.0			1.0		1.0	
Blows per compacted layer	20			15			10			15		15	
In-situ average density (t/m)	1.73	1.71	1.66	1.64	1.51	1.67	1.47	1.53	1.50	1.91	1.96	1.76	1.79
Average	1.70 t/m^3			1.61 t/m^3			1.50 t/m^3			1.94 t/m^3		1.74 t/m^3	
Friction angle (degree)	39.1	38.4	37.2	37.0	36.0	37.6	35.1	36.2	35.6	37.4	37.9	42.5	44.7
Average	38.2^o			36.9^o			35.6^o			37.7^o		43.3^o	
Ultimate Load (tons)	3.50	3.25	3.25	3.25	3.00	3.00	2.25	2.25	2.00	3.25	3.00	3.50	3.75
Average	3.33 tons			3.08 tons			2.17 tons			3.13 tons		3.63 tons	

Table 2.1 Properties of granular columns (after Bergado and Lam, 1987)

Test name	Ftg size (m)	k (−)	s (m)	F (−)	$E_{50,col}$ (MPa)
A	2 × 2	4	1.0	3.5	70
B	3 × 3	4	1.0	8.0	70
C	3 × 3	4	1.5	8.0	70
D1	3 × 3	4	2.0	8.0	70
D2	3 × 3	4	2.0	8.0	50
D3	3 × 3	4	2.0	8.0	30
E1	3 × 3	5	1.0	6.4	70
E2	3 × 3	5	1.0	6.4	50
E3	3 × 3	5	1.0	6.4	30
F1	3 × 3	9	1.0	3.5	70
F2	3 × 3	9	1.0	3.5	50
F3	3 × 3	9	1.0	3.5	30
G	4 × 4	16	1.0	3.5	70

Note: k = number of columns; s = column spacing; E_{50} = stiffness of column material; F = footprint replacement ratio ($F=A_f\ kA_c$ where A_f = footing area, k = number of supporting columns and A_c is the cross-sectional area of each column).

Table 2.2: Details of parametric study (after Killeen and Mc Cabe, 2010).

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

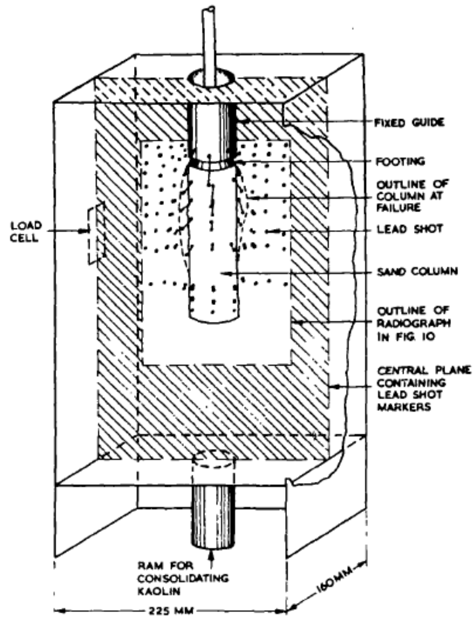
WTF, wet top feed; DTF, dry top feed; BF, bottom feed; TR, triangular.

Table 2.3 : Case histories with $S_{untreated}$ and $S_{treated}$ measured, after Mc Cabe et al. (2009).

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 2.4 : Case histories in which $S_{treated}$ values have been measured but $S_{untreated}$ values have been predicted, after Mc Cabe et al. (2009).



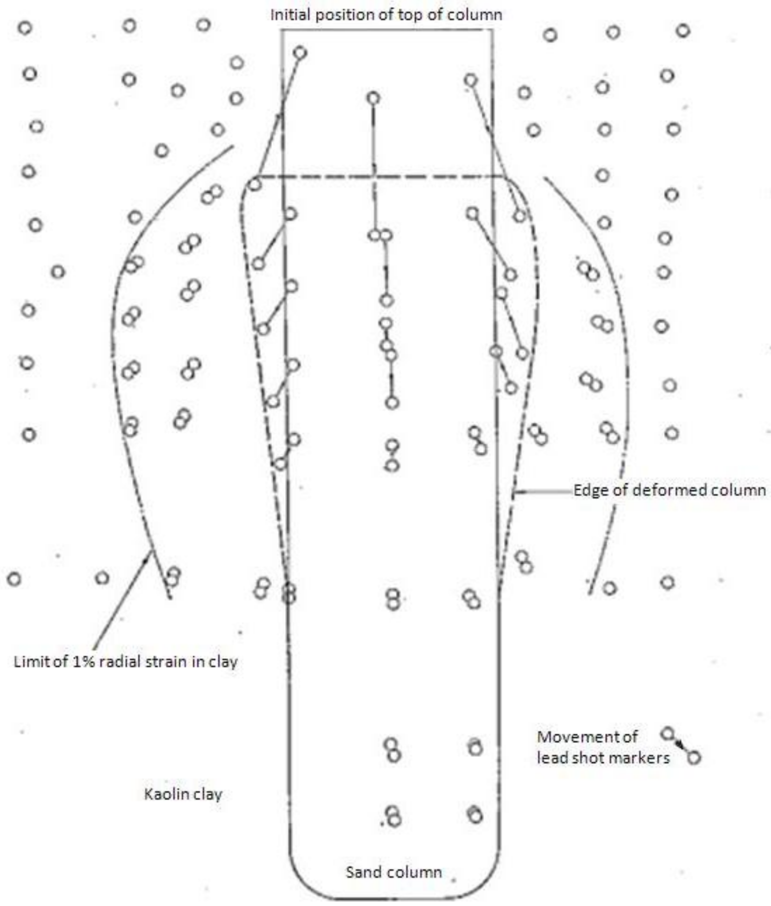
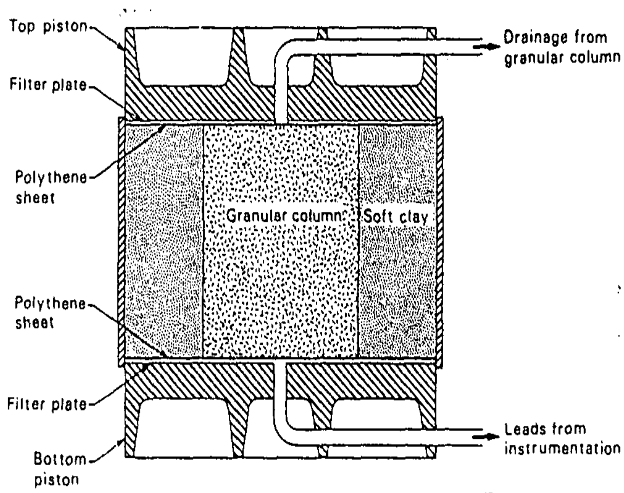
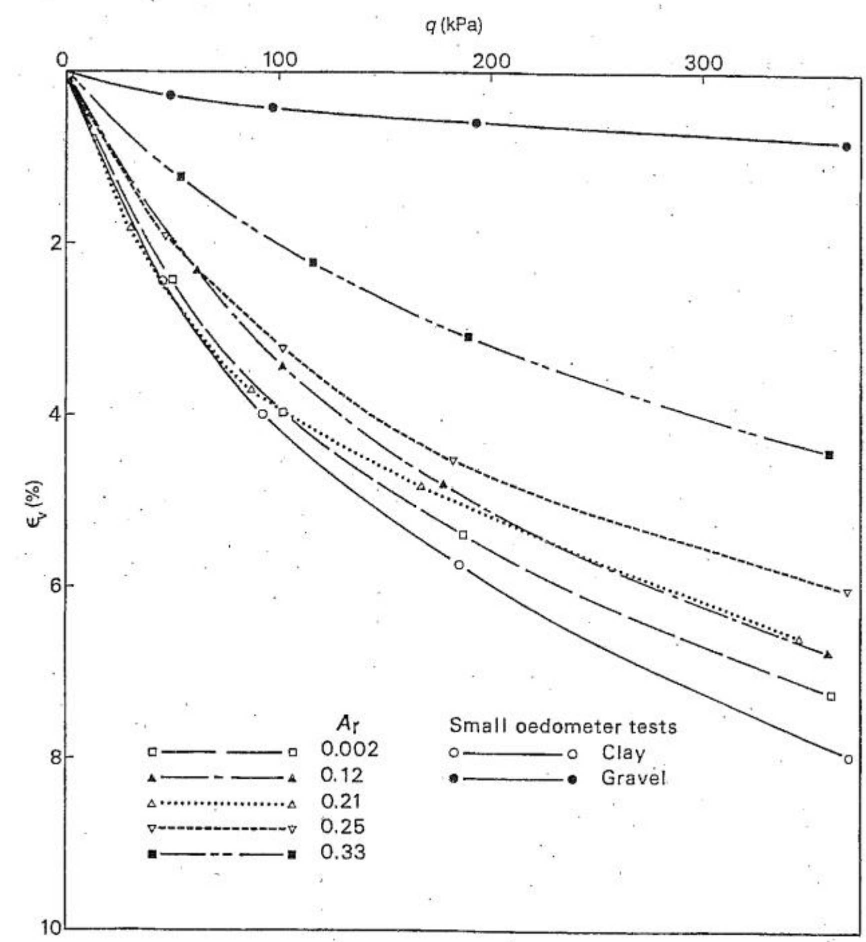


Figure 2.1: Deformation of 38mm diameter laboratory model column for 25mm displacement with limit of 1% radial strain in clay shown (after Hughes and Withers, 1974)

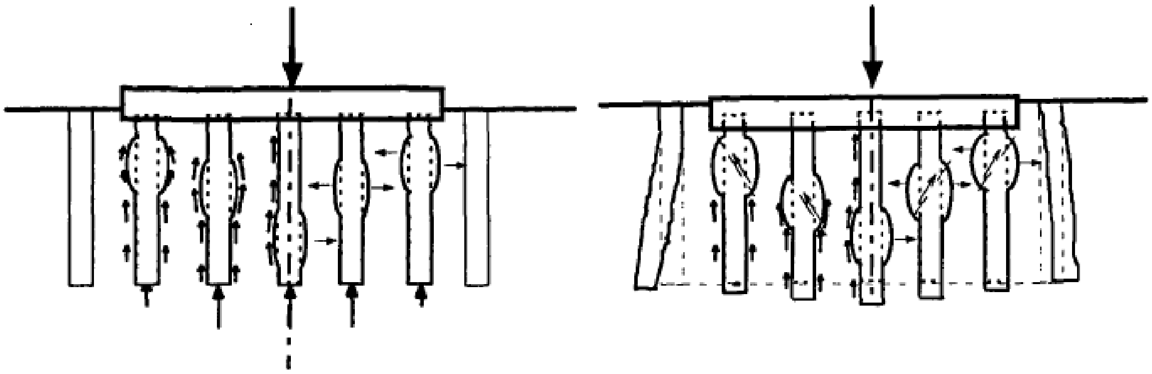
a)



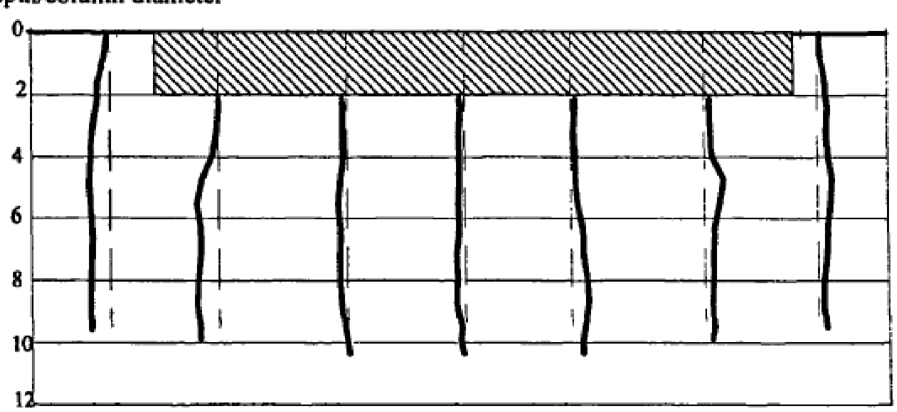


(b)

Figure 2.2: (a) 1.0 m diameter floating rig oedometer for investigating reinforcing effect of different stone column diameters in soft clay. (b) Plot of results for different Area ratio (A_r) values (after Charles and Watts, 1983).

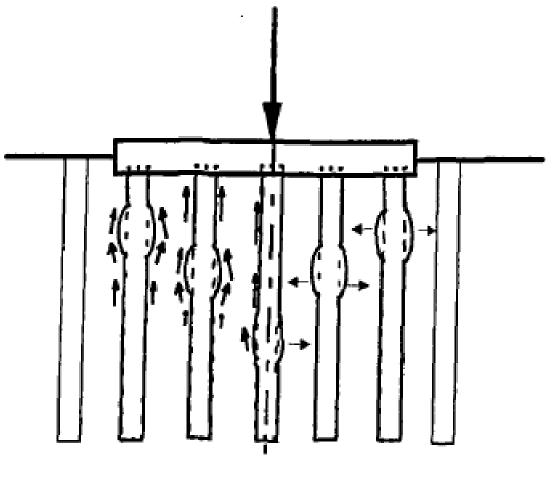


(a) before end bearing failure (b) after end bearing failure

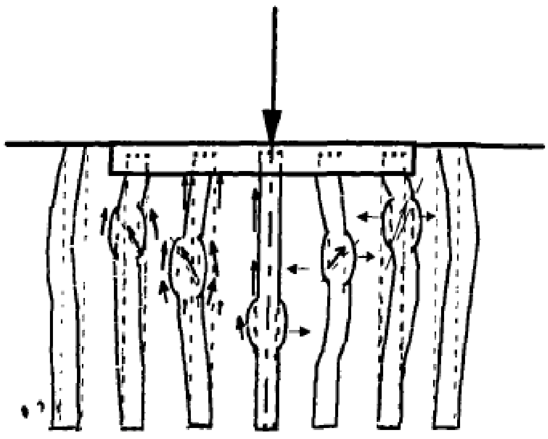


(c) horizontal movements deduced from centrelines of columns (TS-08)

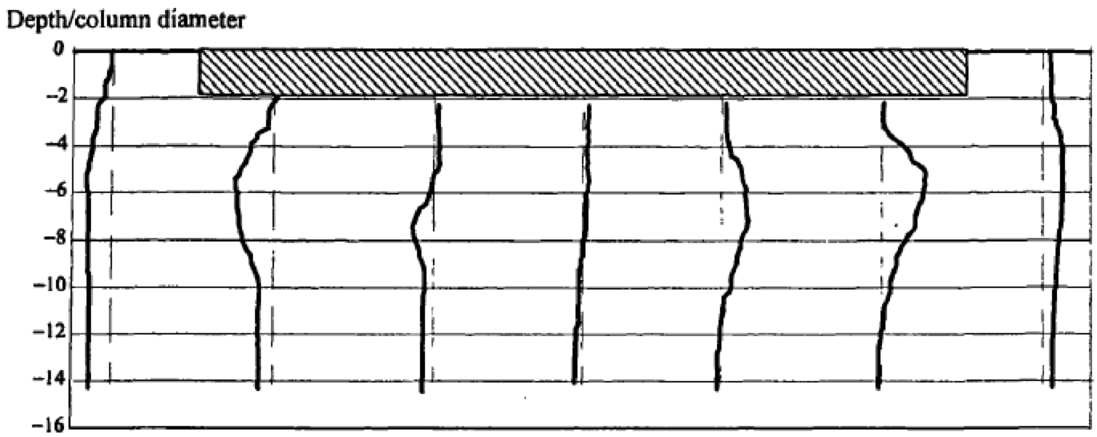
Figure 2.3: Suggested mode of deformation for short columns (a) before end-bearing failure; (b) after end bearing failure; (c) horizontal movements deduced from centre-lines of columns (TS08), after Hu (1995).



(a) before overall failure;

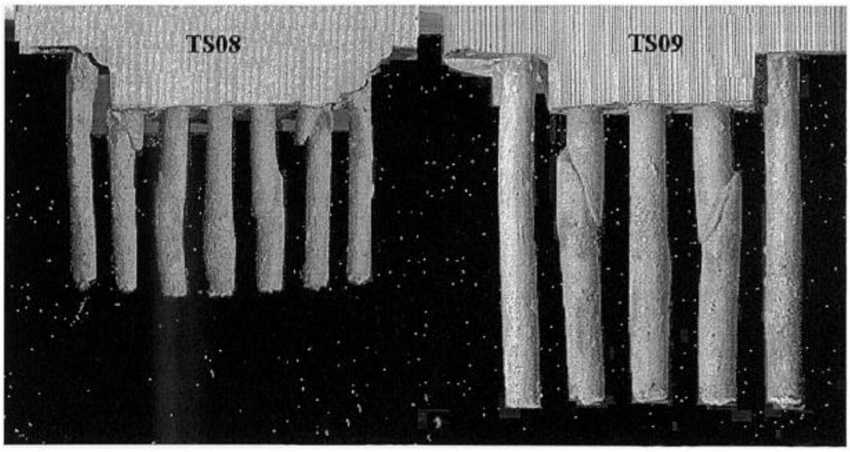


(b) after overall failure;

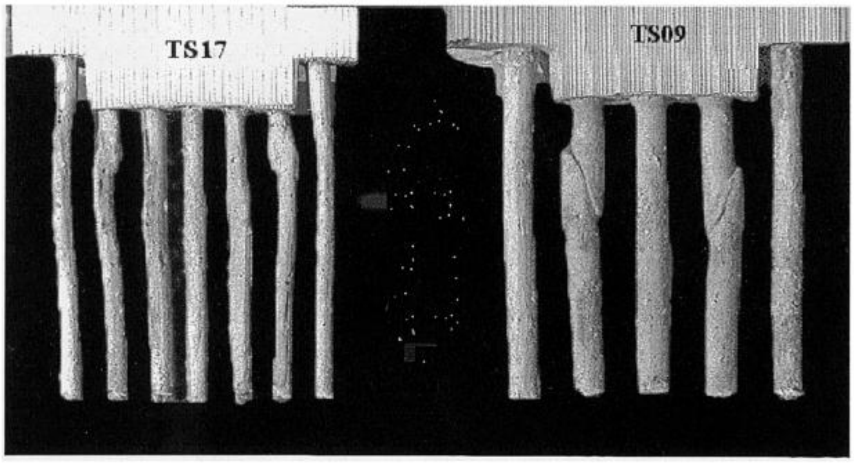


(c) horizontal movements deduced from centrelines of columns (TS-17)

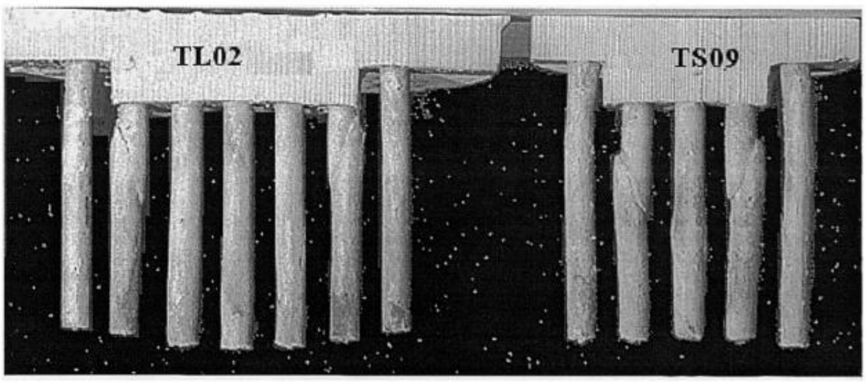
Figure 2.4: Suggested mode of deformation for long columns (a) before overall failure; (b) after overall failure; (c) horizontal movements deduced from centre-lines of columns (TS17), after Hu (1995).



(a)

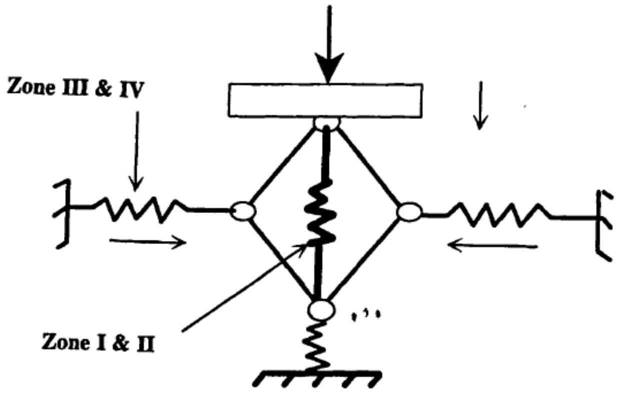


(b)

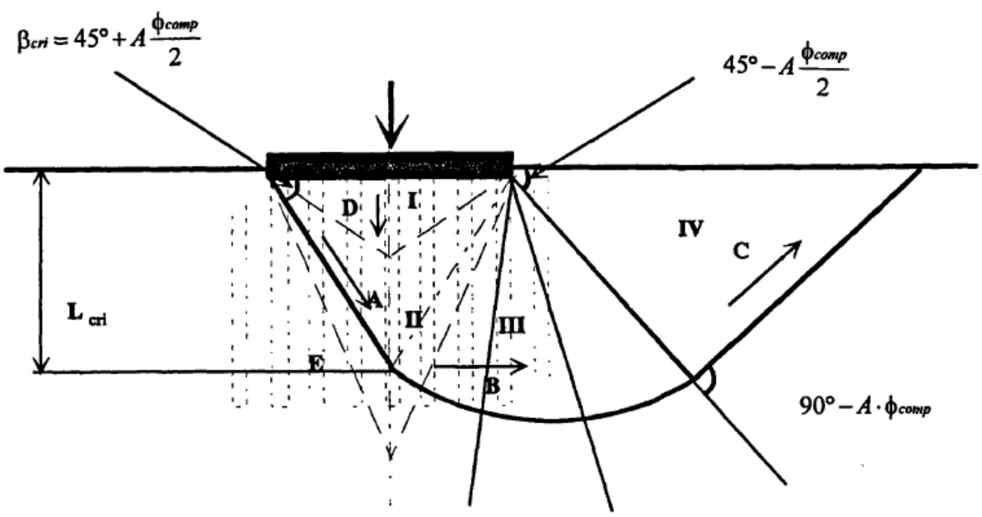


(c)

Figure 2.5: Comparison of column deformation patterns for different column length and group size, after Hu (1995).



a)



b)

Figure 2.6: (a) Analogue of stone column reinforced ground under a rigid footing load; (b) Proposed mechanism of failure of stone column reinforced foundation under a rigid footing, after Hu (1995).

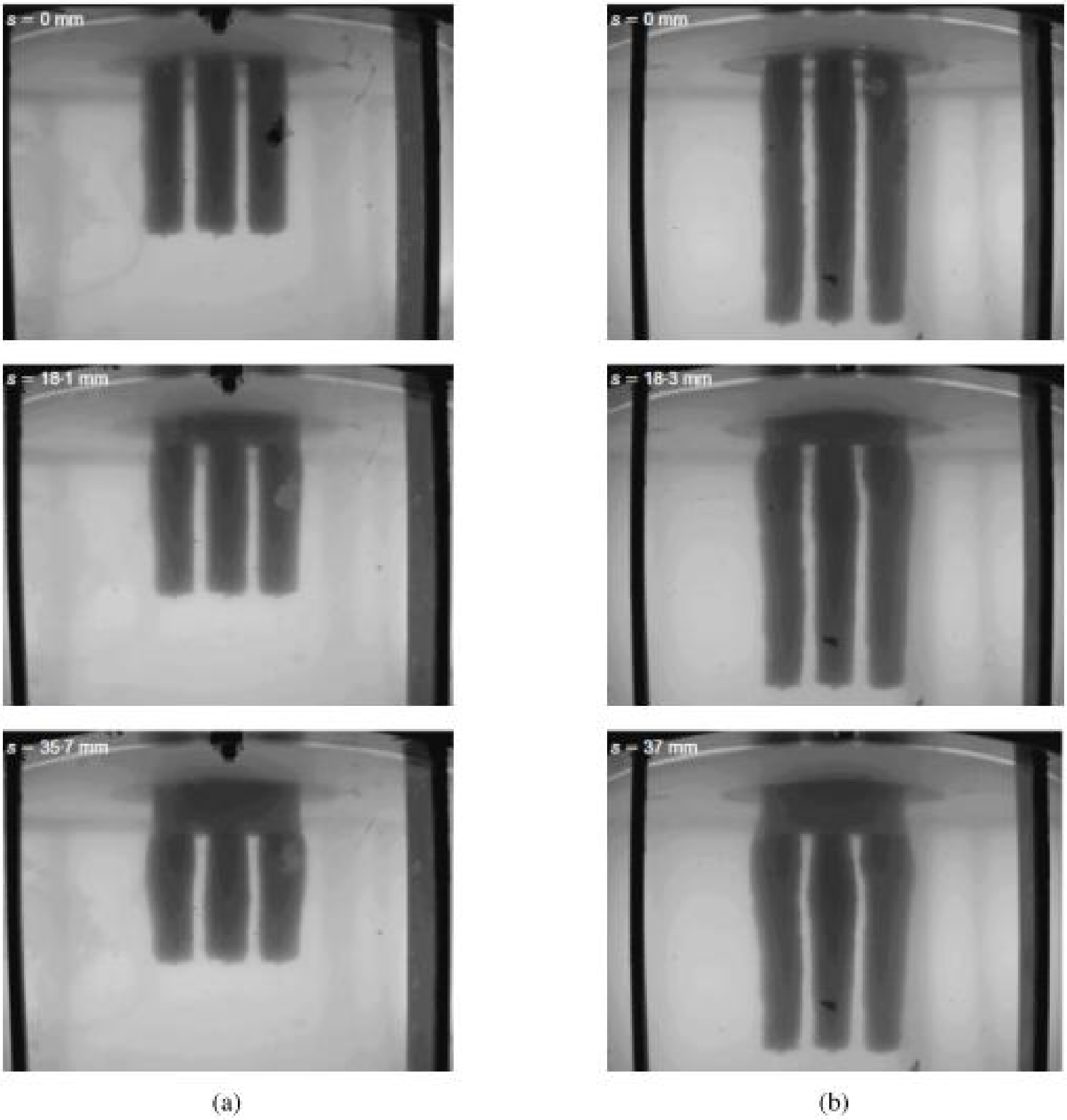


Figure 2.7: Photographs of sand columns (in transparent 'host soil') beneath a circular footing at beginning, middle and end of foundation loading process: (a) 150 mm length (L/d = 6) and (b) 250 mm length (L/d = 10). Bulging can be observed in centre columns and bending in outer columns, after Mc Kelvey (2002); Mc Kelvey et al. (2004).

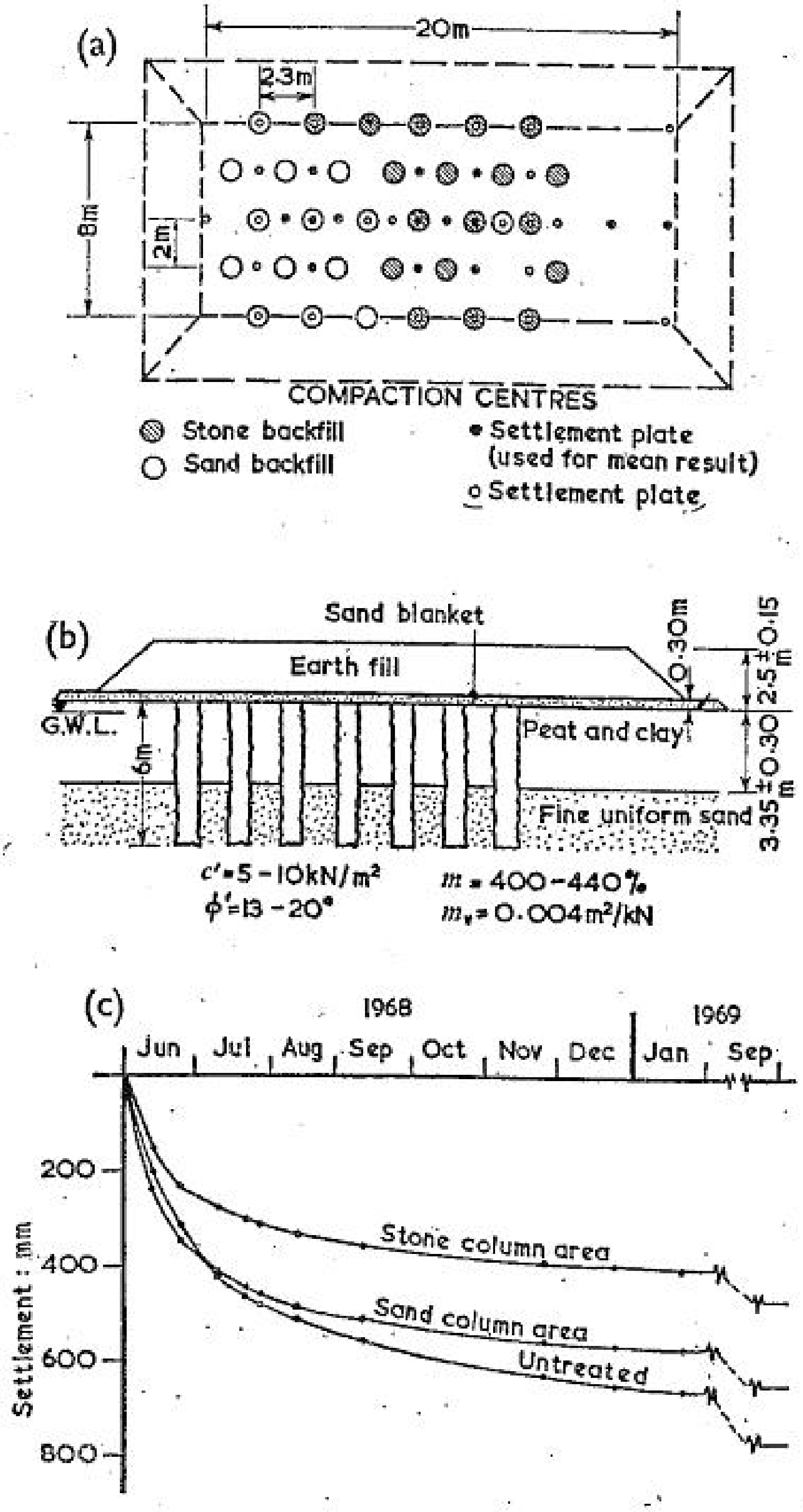
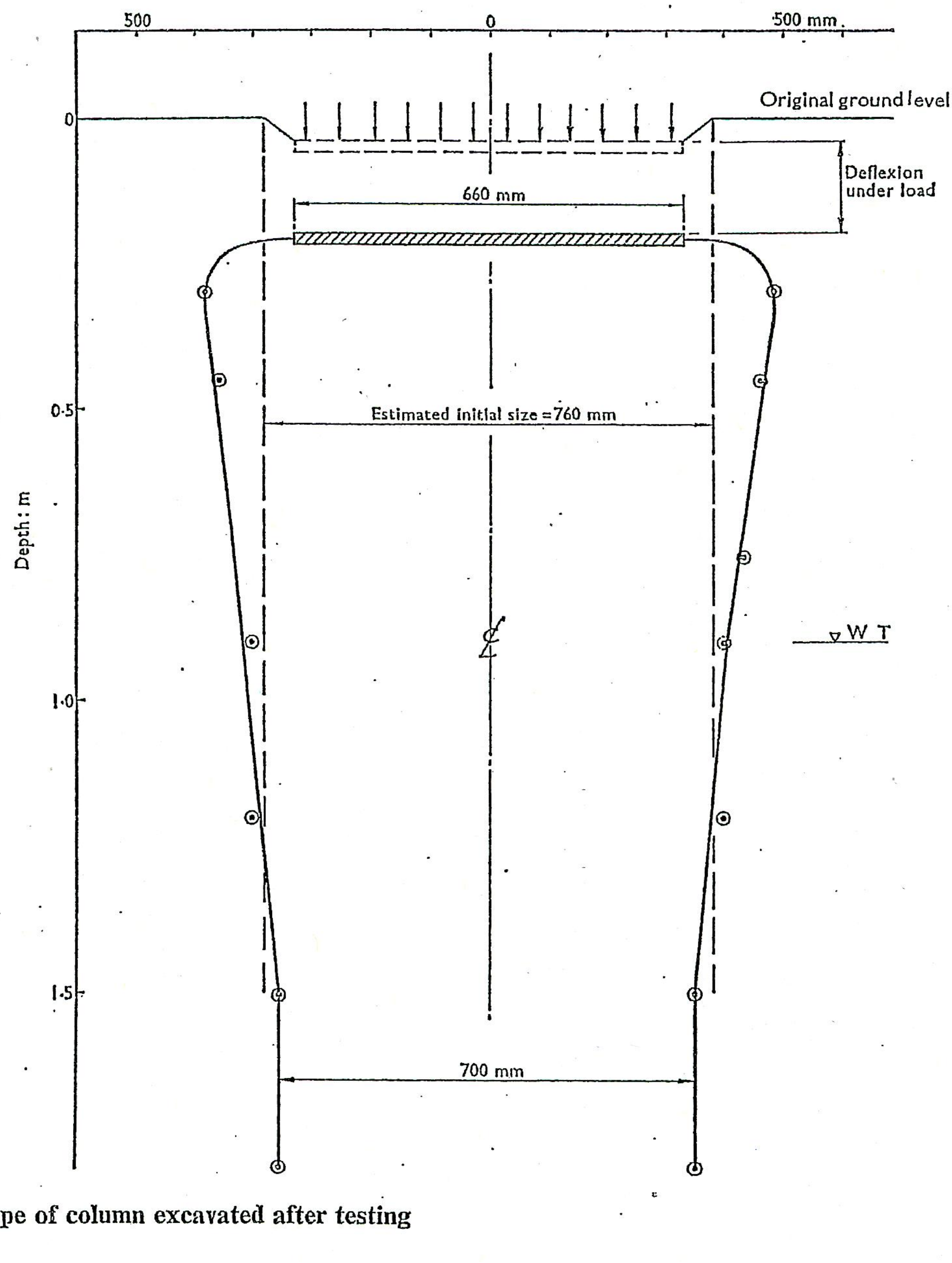


Figure 2.8: (a)-(c) Comparison of large scale field loading test results on untreated soft clay, soft clay reinforced with stone columns and soft clay reinforced with sand columns at Bremerhaven, Germany, after Greenwood (1970).



Shape of column excavated after testing

Figure 2.9: Field deformation behaviour of a single column under a (rigid) plate load test , after Hughes et al. (1976).

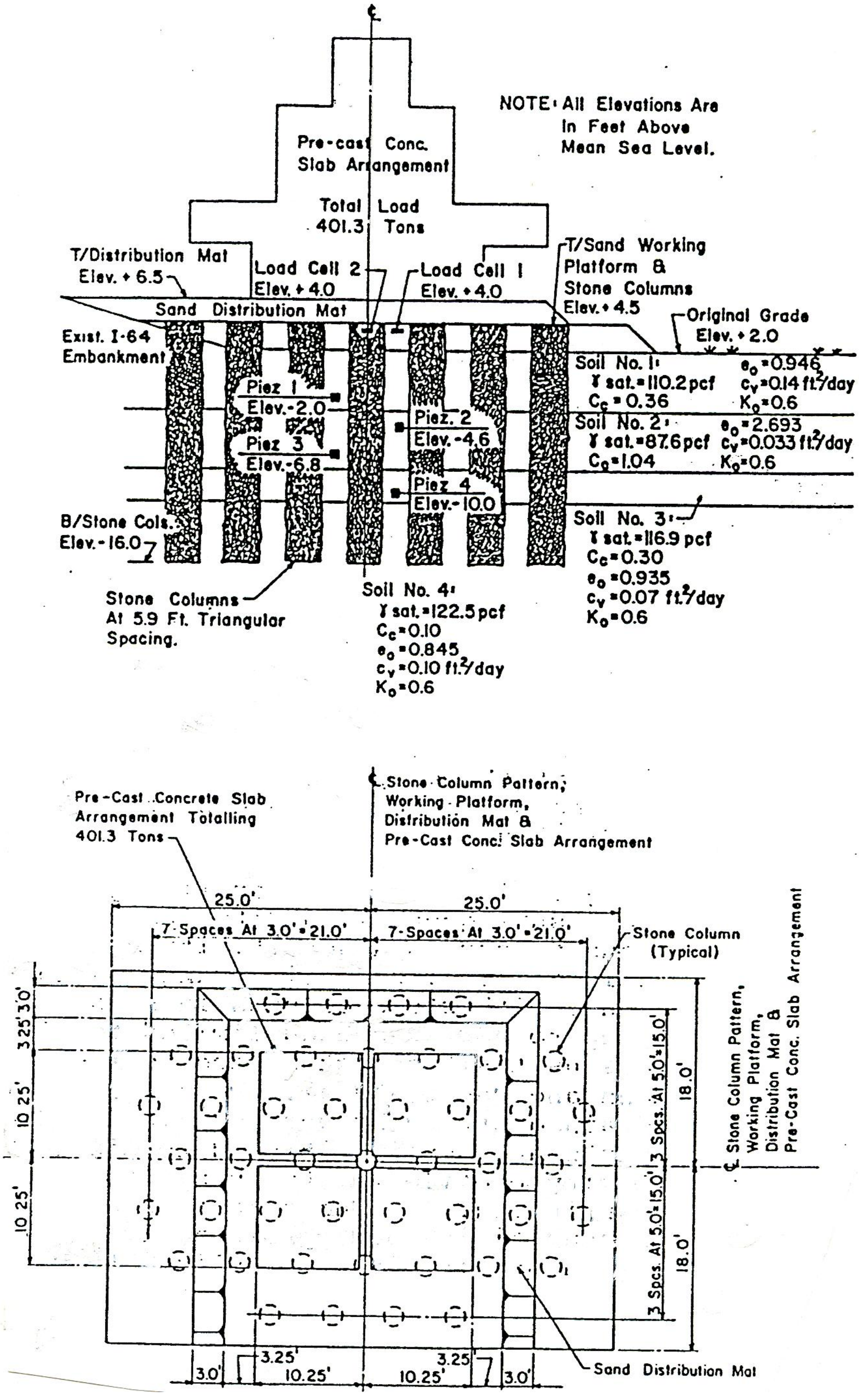


Figure 2.10: Field load test arrangement and stone column layout (after Goughnour and Bayuk, 1979a). I Inch = 25.4 mm; 1ft = 0.305 m.

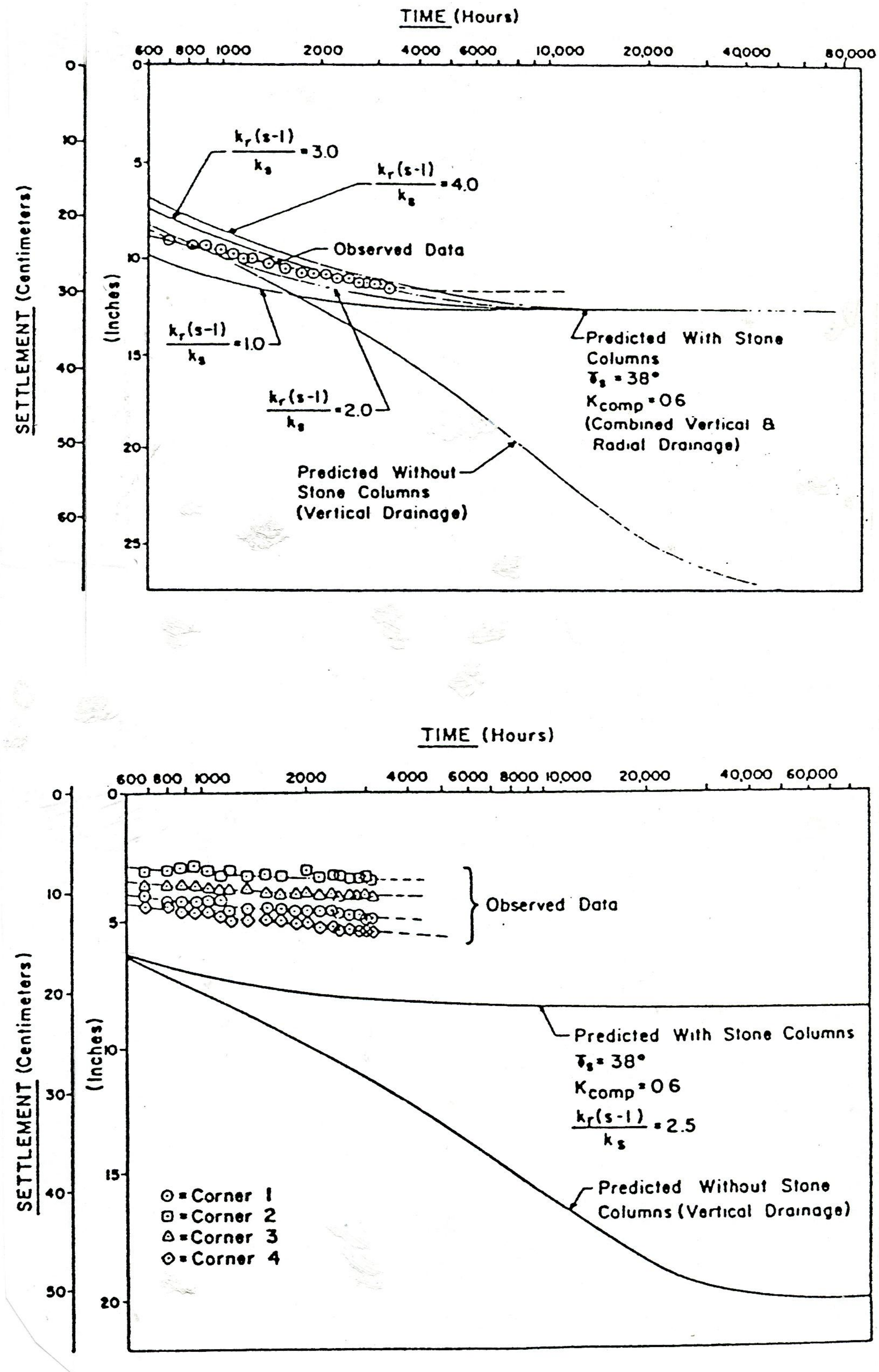


Figure 2.11: Settlement versus log time at the centre and corners of load area in field trial (after Goughnour and Bayuk, 1979a)

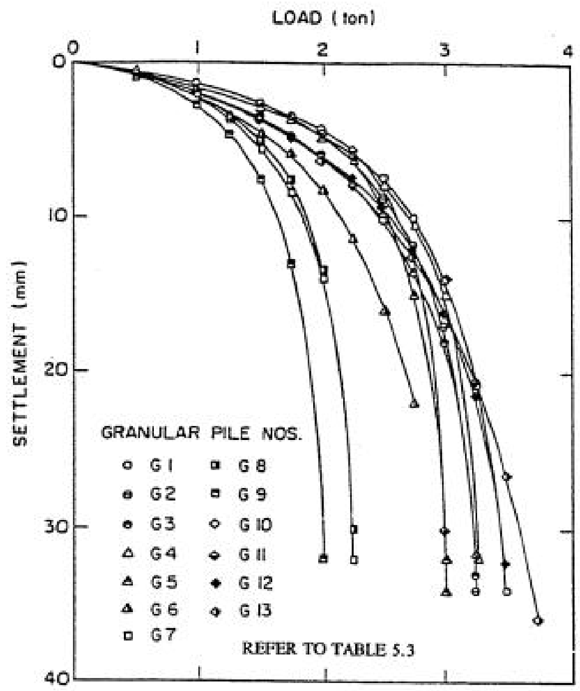


Figure 2.12: Comparison of load-settlement performance of granular columns constructed with different numbers of blows per compacted layer (after Bergado and Lam, 1987)

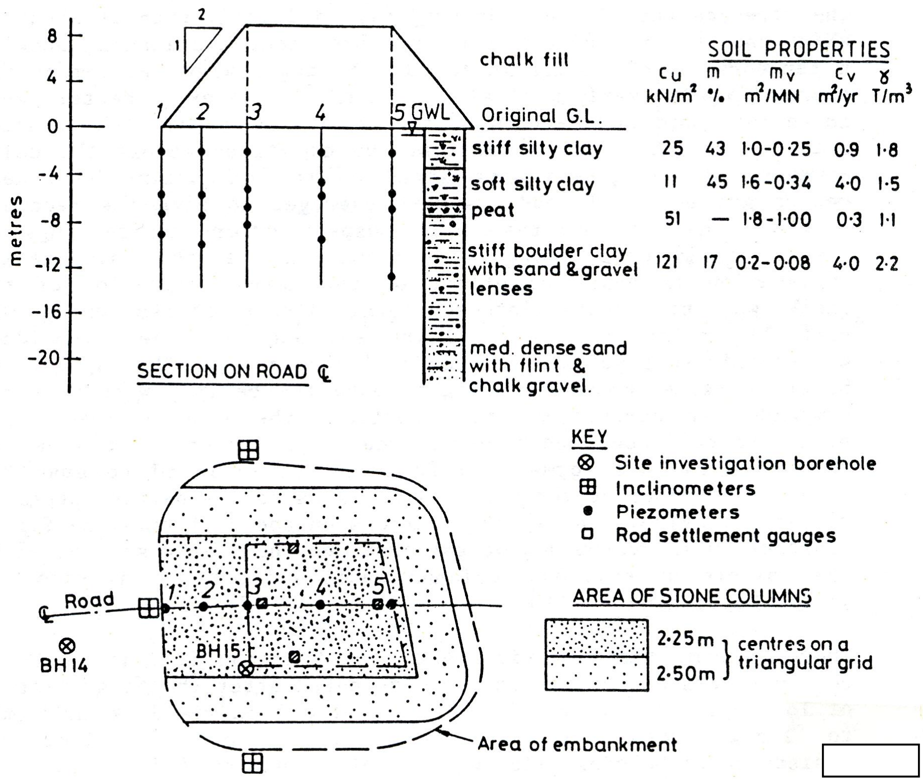


Figure 2.13: (a) Field circumstances for Humber Bridge approach stone column field trials (after Greenwood, 1991)

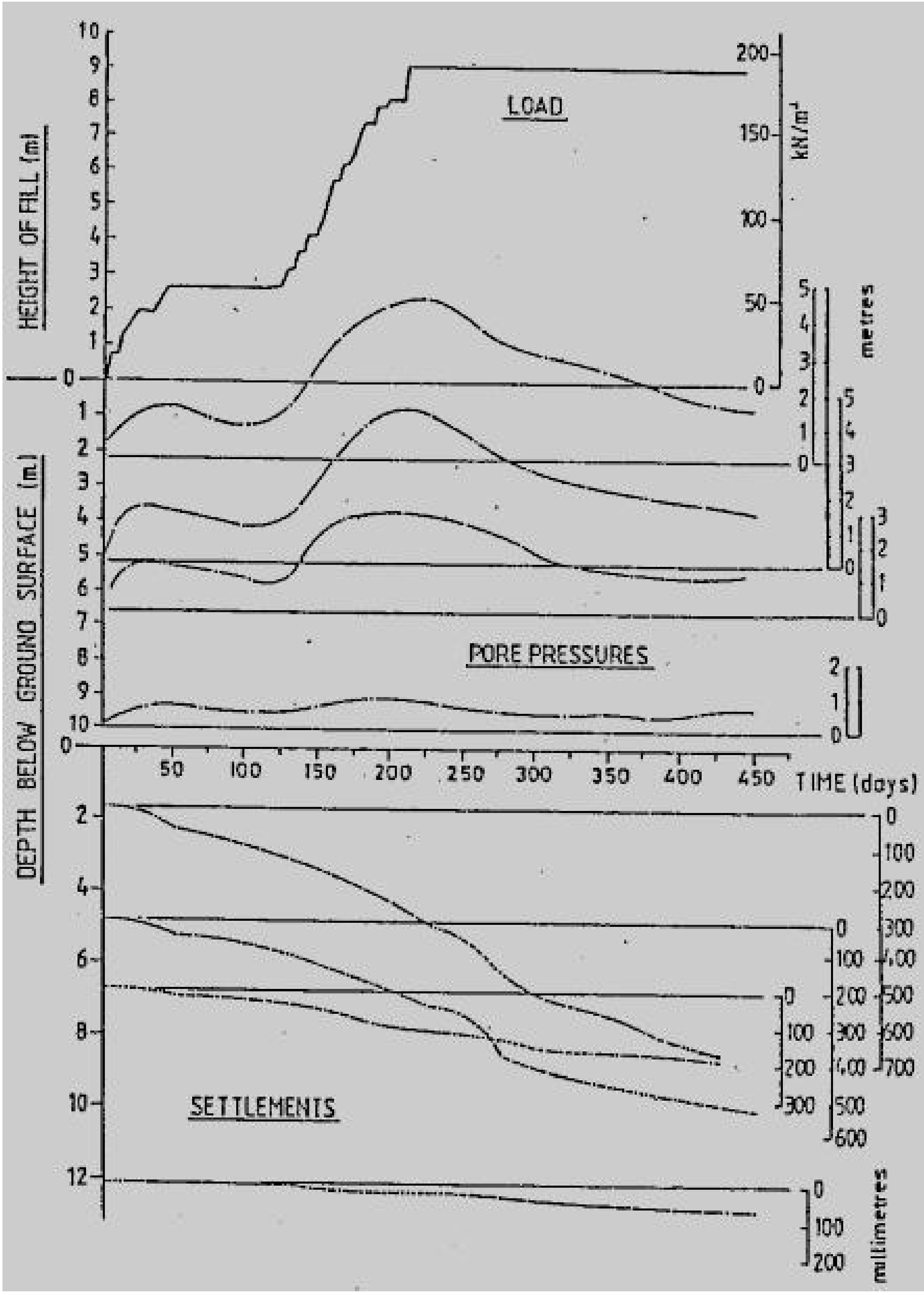


Figure 2.13: (b) Humber Bridge – measured pore pressures and settlements (after Greenwood, 1991).

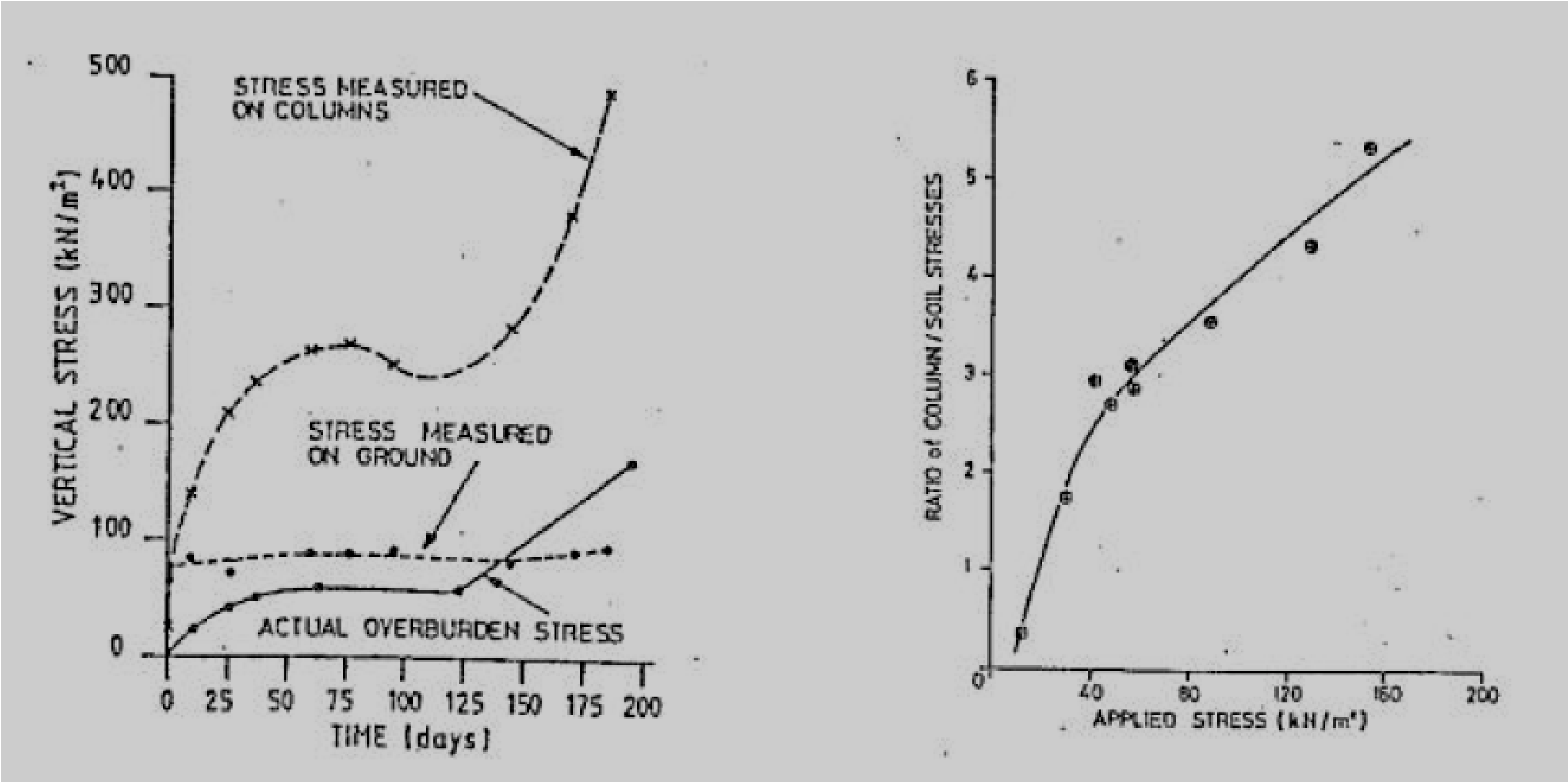


Figure 2.13: (c) Humber Bridge – measured stresses, (d) Humber Bridge – measured stress ratios (after Greenwood, 1991) .

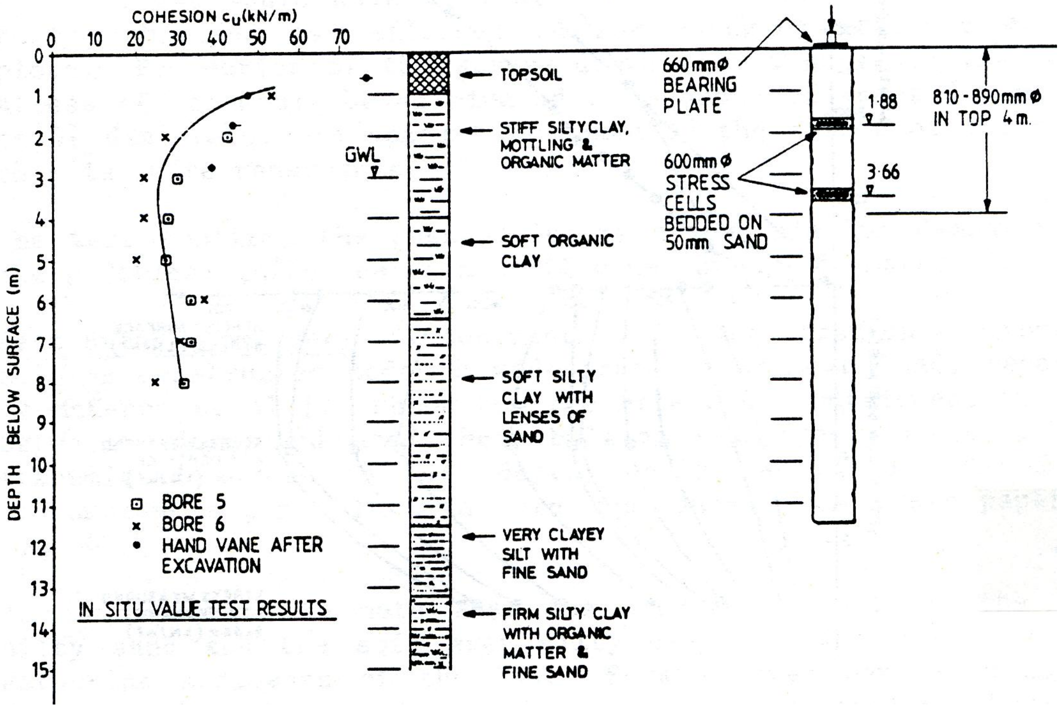


Figure 2.14: (a) Uskmouth field trial (plate) load test circumstances (after Greenwood, 1991)

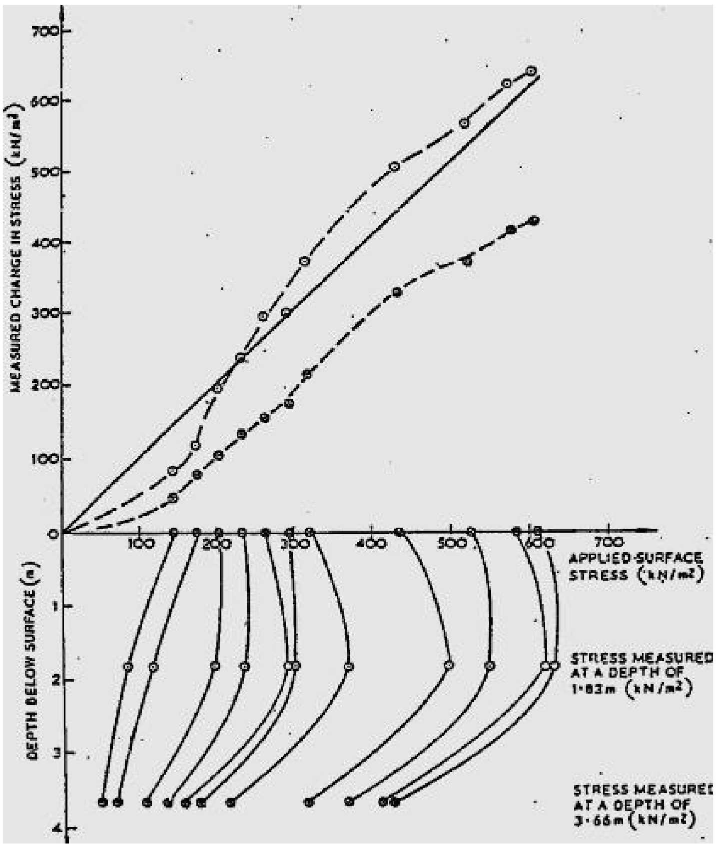
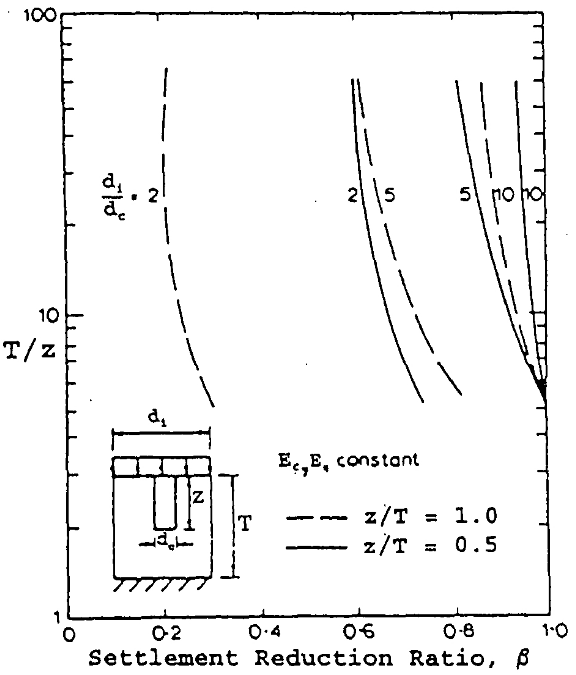
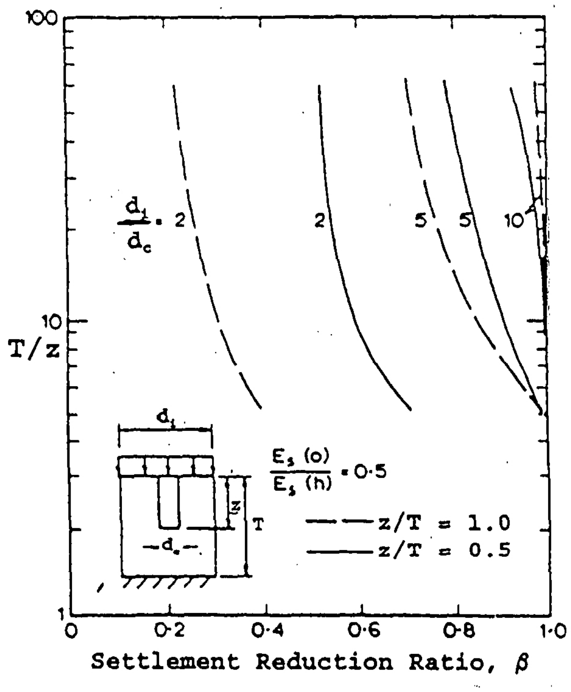


Figure 2.14: (b) Uskmouth – measured stresses in stone column (after Greenwood, 1991)

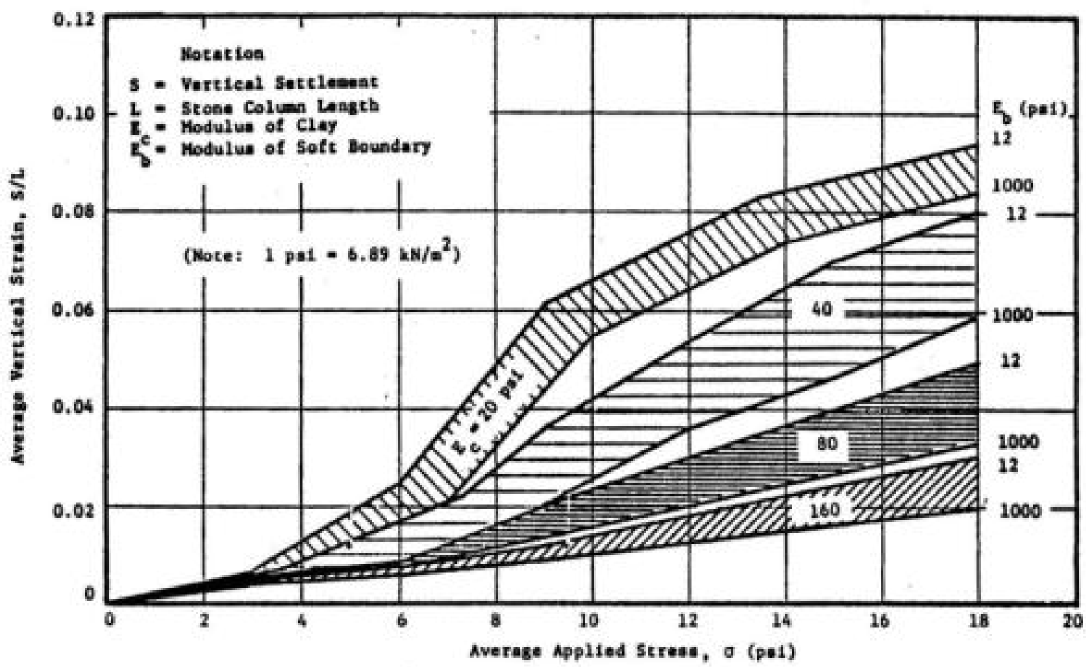


a)



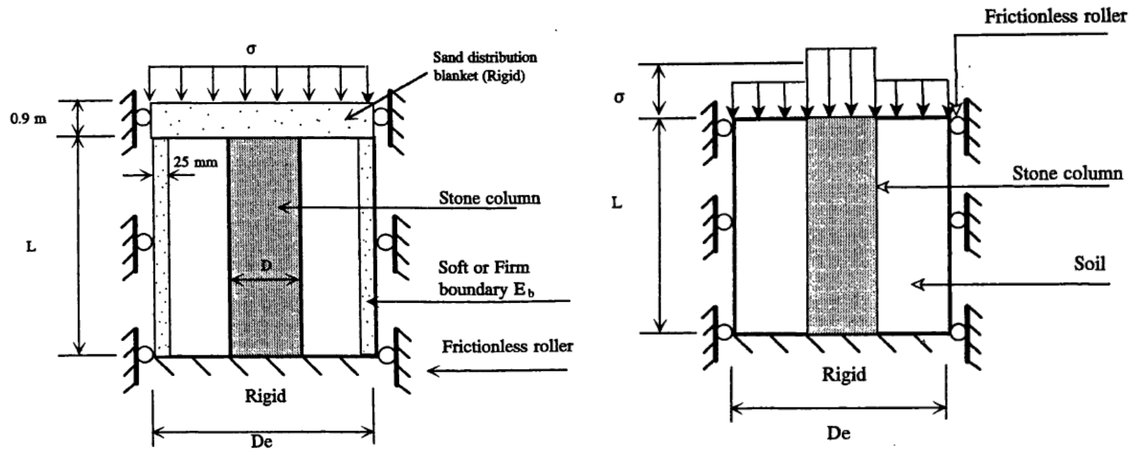
b)

Figure 2.15: Settlement reduction ratio predicted by unit cell analysis: (a) Effects of area ratio and column length. (b) Effects of linearly varying modulus in column and soil (after Balaam et al., 1977)



$a_s = 0.25$, $L/D = 20$

Figure 2.16: Load-displacement response from non-linear unit cell finite element analysis (after Barksdale and Bachus, 1983)



a) **b)**

Figure 2.17: (a) Modified unit cell approximation with soft boundaries. (b) Standard unit cell approximation provided for comparison (after Barksdale and Bachus, 1983)

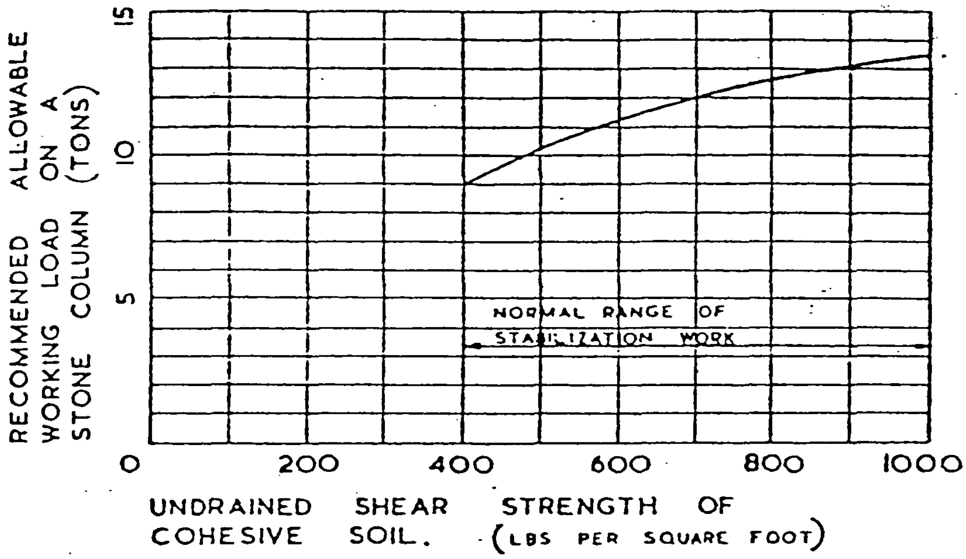


Figure 2.18: Recommended allowable working loads on stone columns formed in cohesive soils (after Thorburn and MacVicar, 1968). I inch =25.4 mm; 1ft = 0.305 m

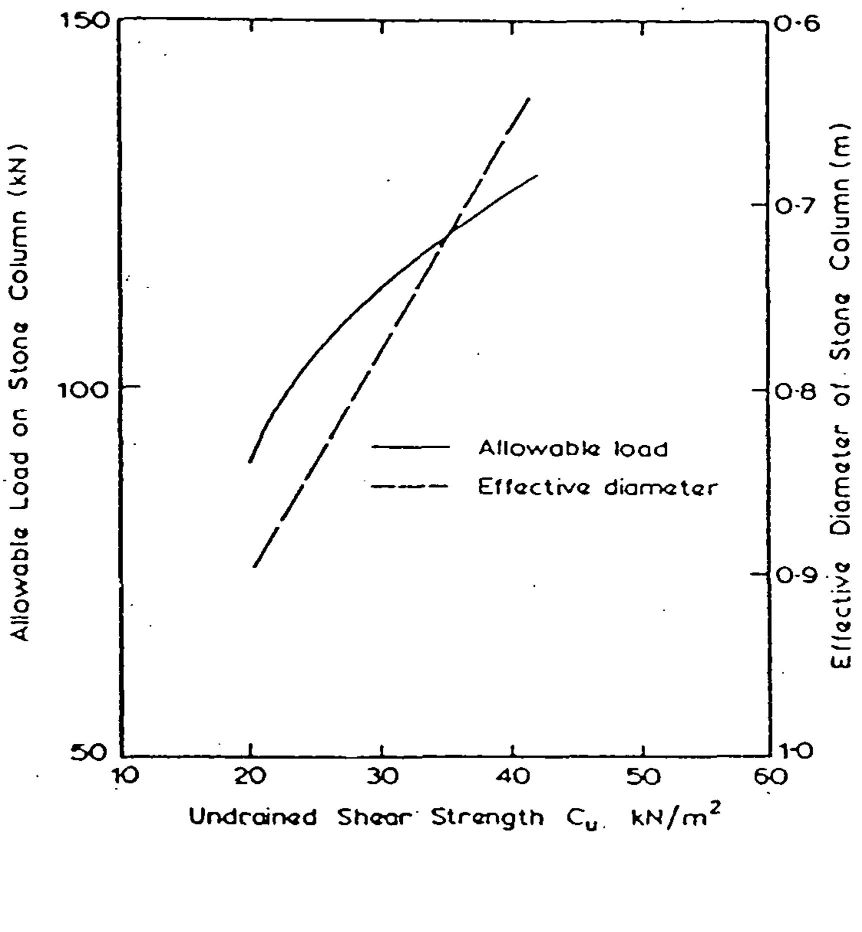


Figure 2.19: Allowable working load for stone column(s), after Thorburn, 1975.

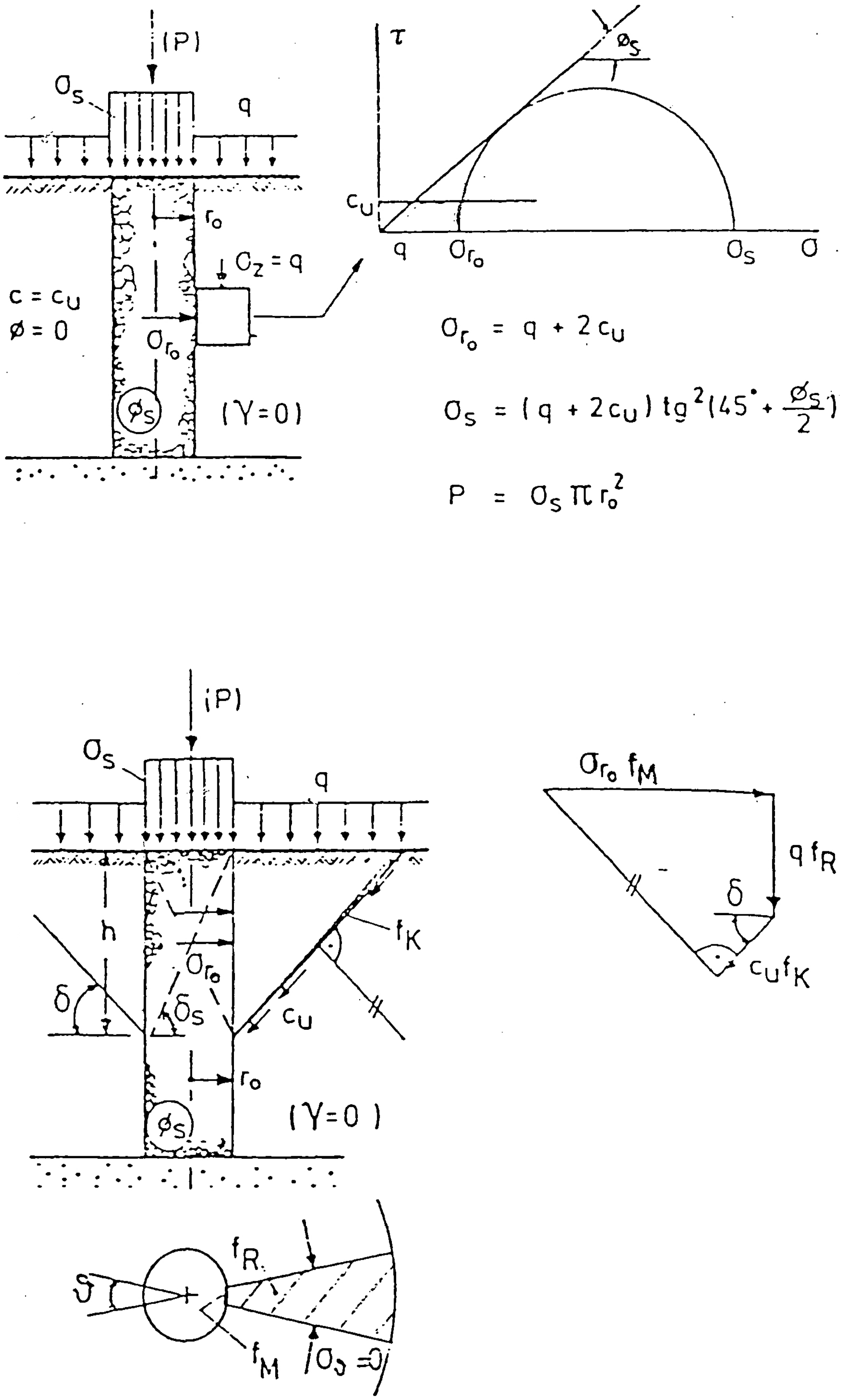


Figure 2.20: Bearing pressure approaches (after Brauns, 1978).

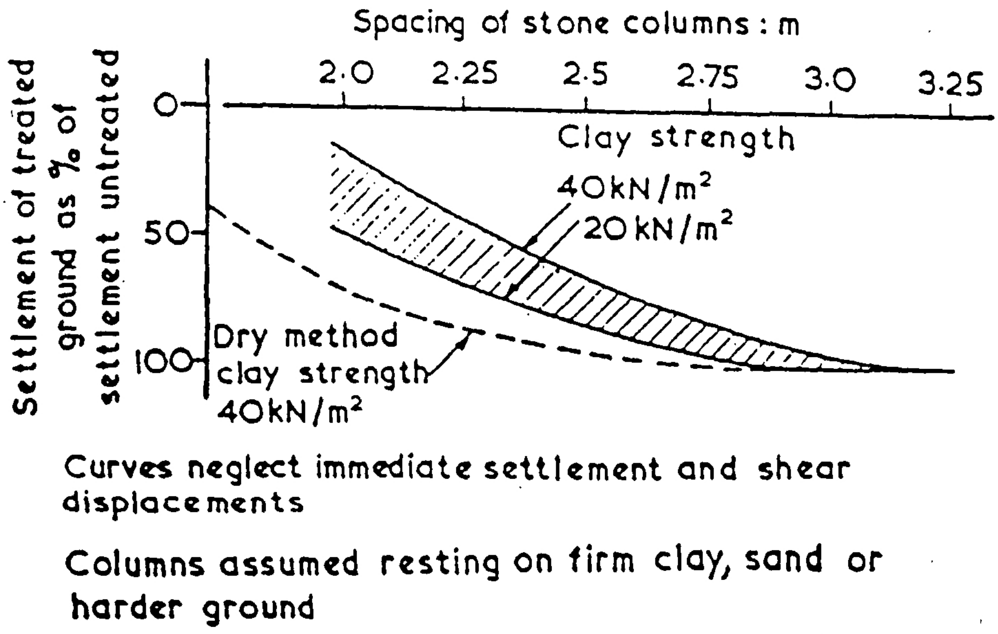


Figure 2.21: Settlement prediction diagram for stone columns in soft clay (after Greenwood, 1970).

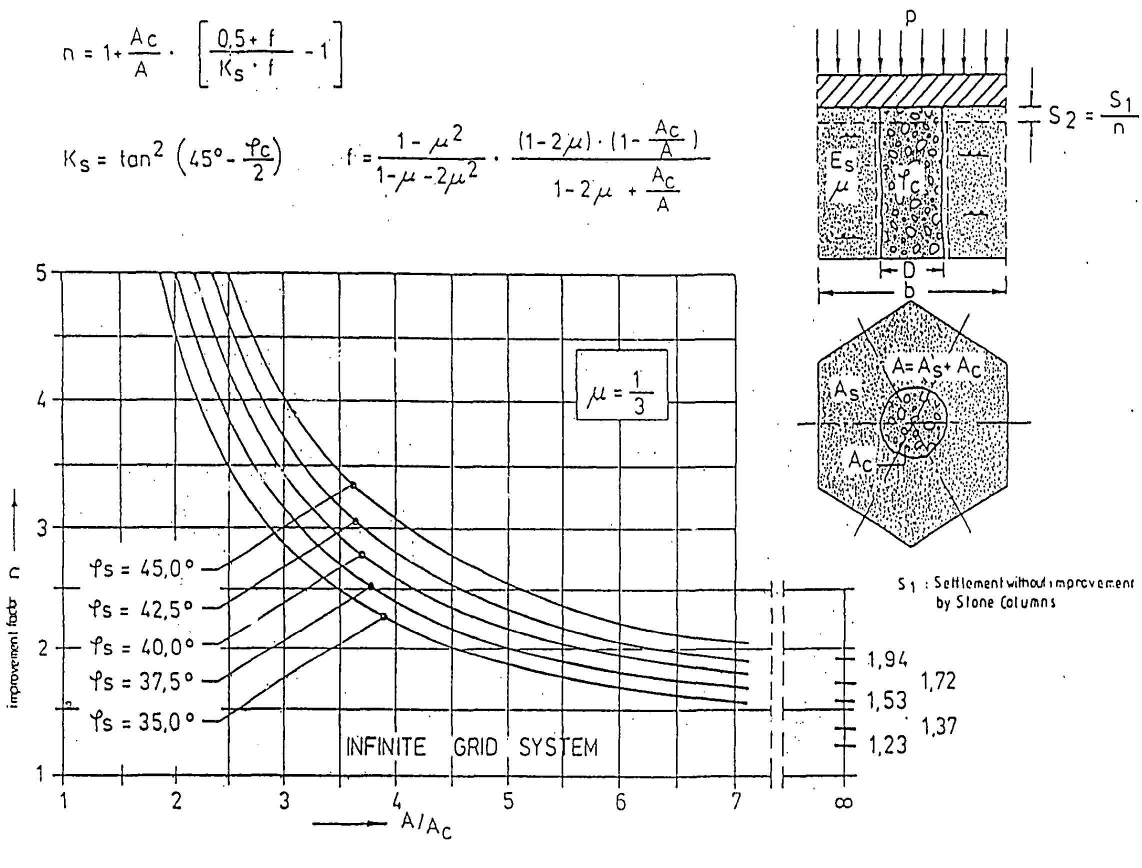
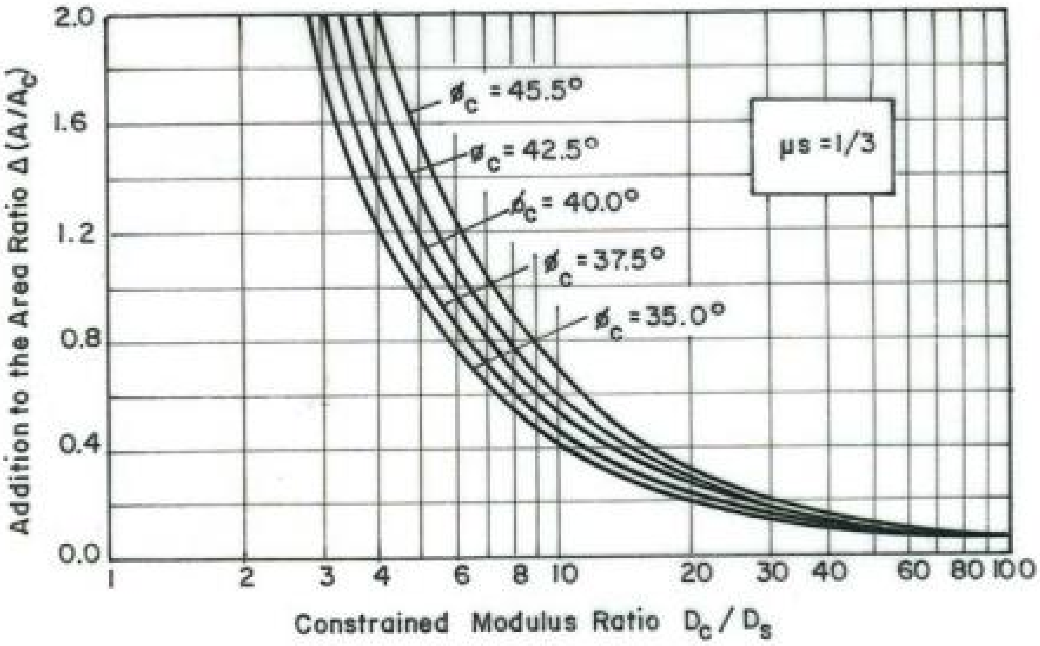
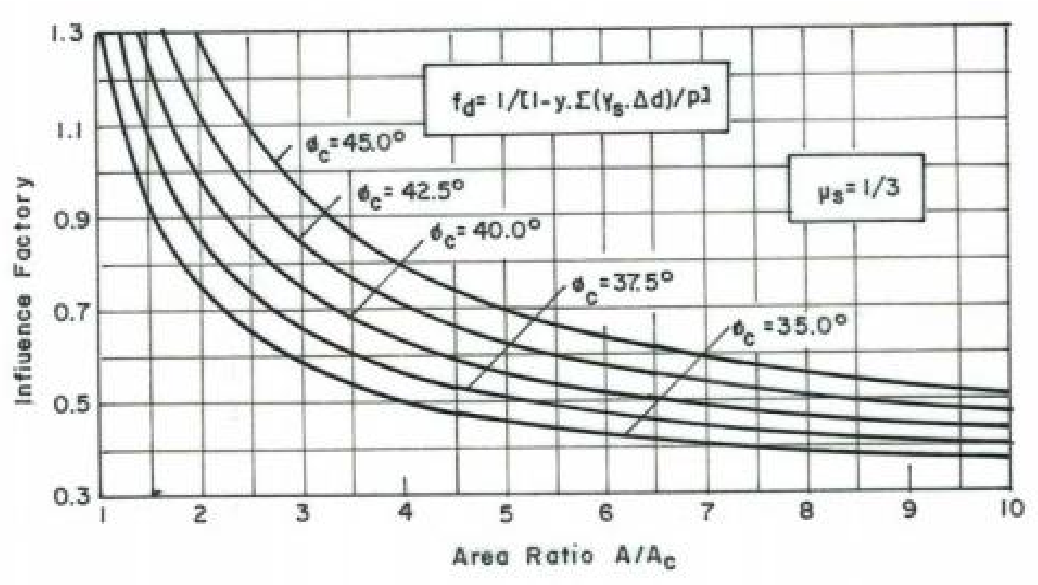


Figure 2.22: Settlement design curves and design philosophy for composite ground (soft soil reinforced with stone columns), after Priebe, 1976.



a)



b)

Figure 2.23: (a) Consideration of column compressibility (b) Determination of depth factor (after Priebe, 1995)

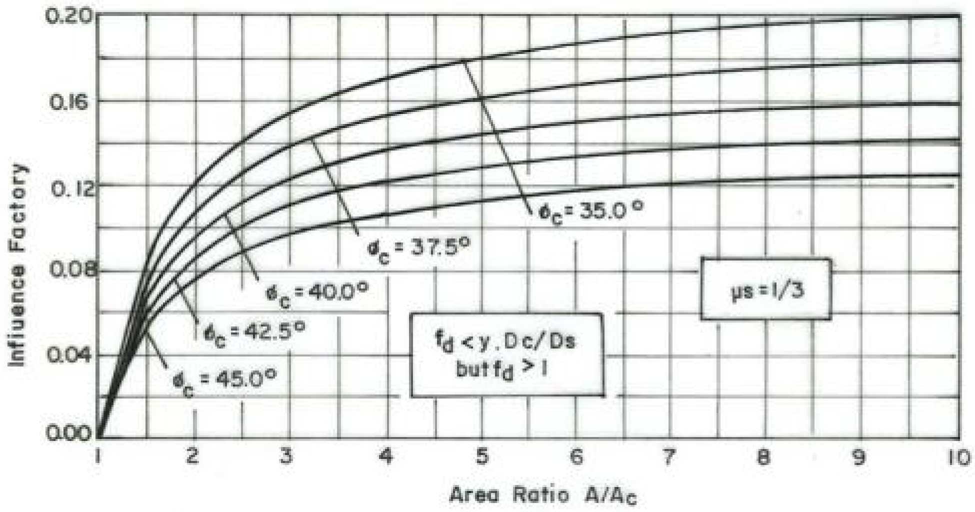


Figure 2.23: (c) Limit value of depth factor (after Priebe, 1995).

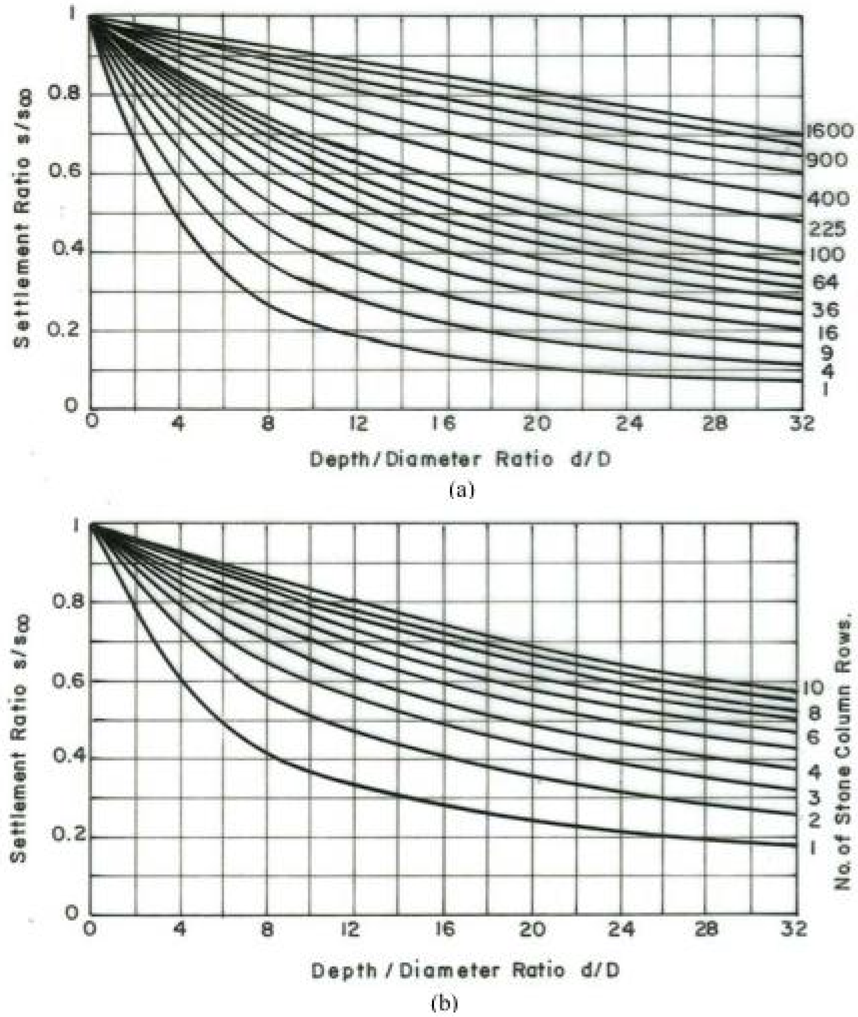


Figure 2.24: Settlement of small foundations (a) For single pad footings. (b) For strip footings (after Priebe, 1995).

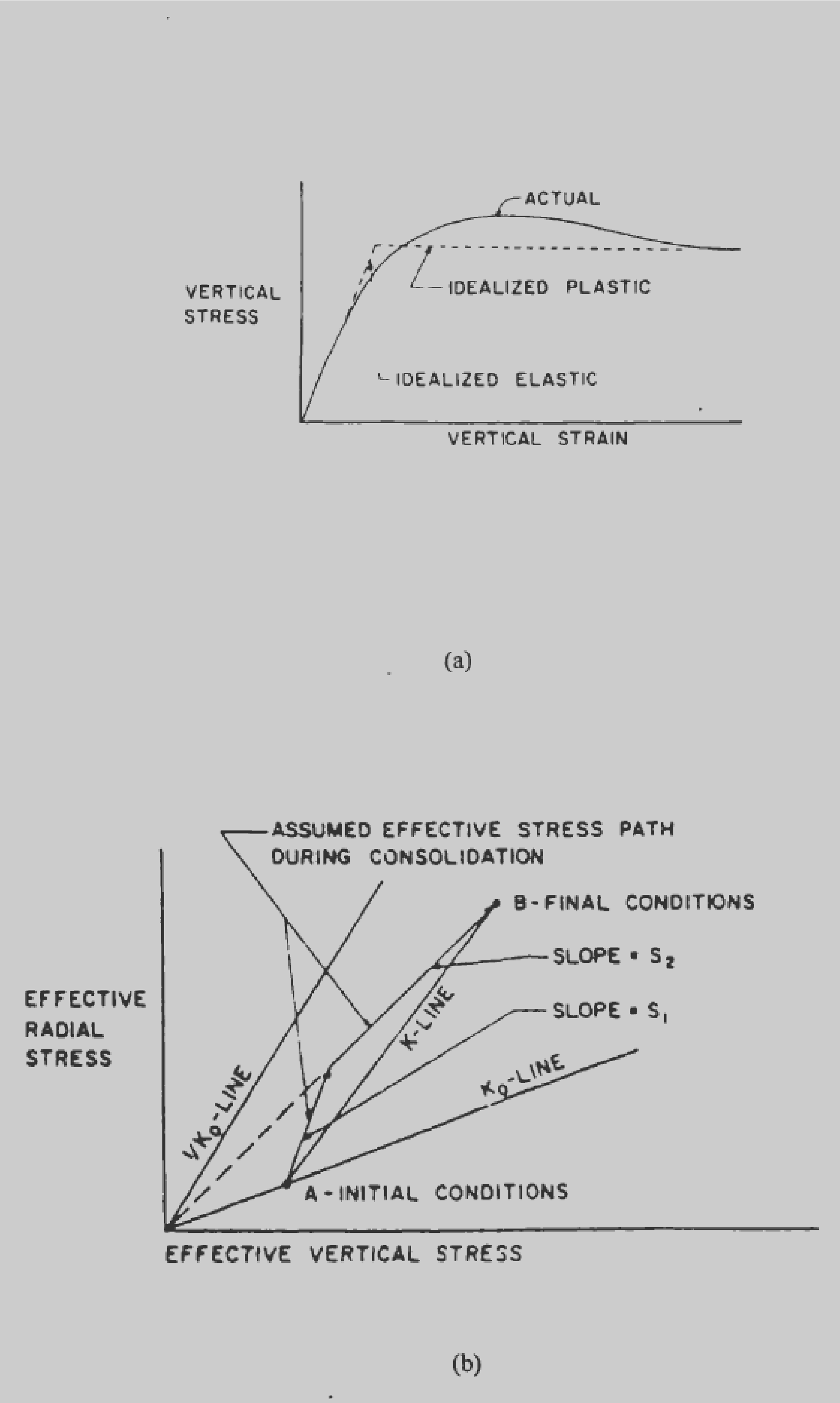


Figure 2.25: The basic diagrams used in Goughnour's (1983) method (a) The idealized stress-strain behaviour of the unit cell, (b) The assumed effective stress path in a unit cell during the loading (after Goughnour and Bayuk, 1979a).

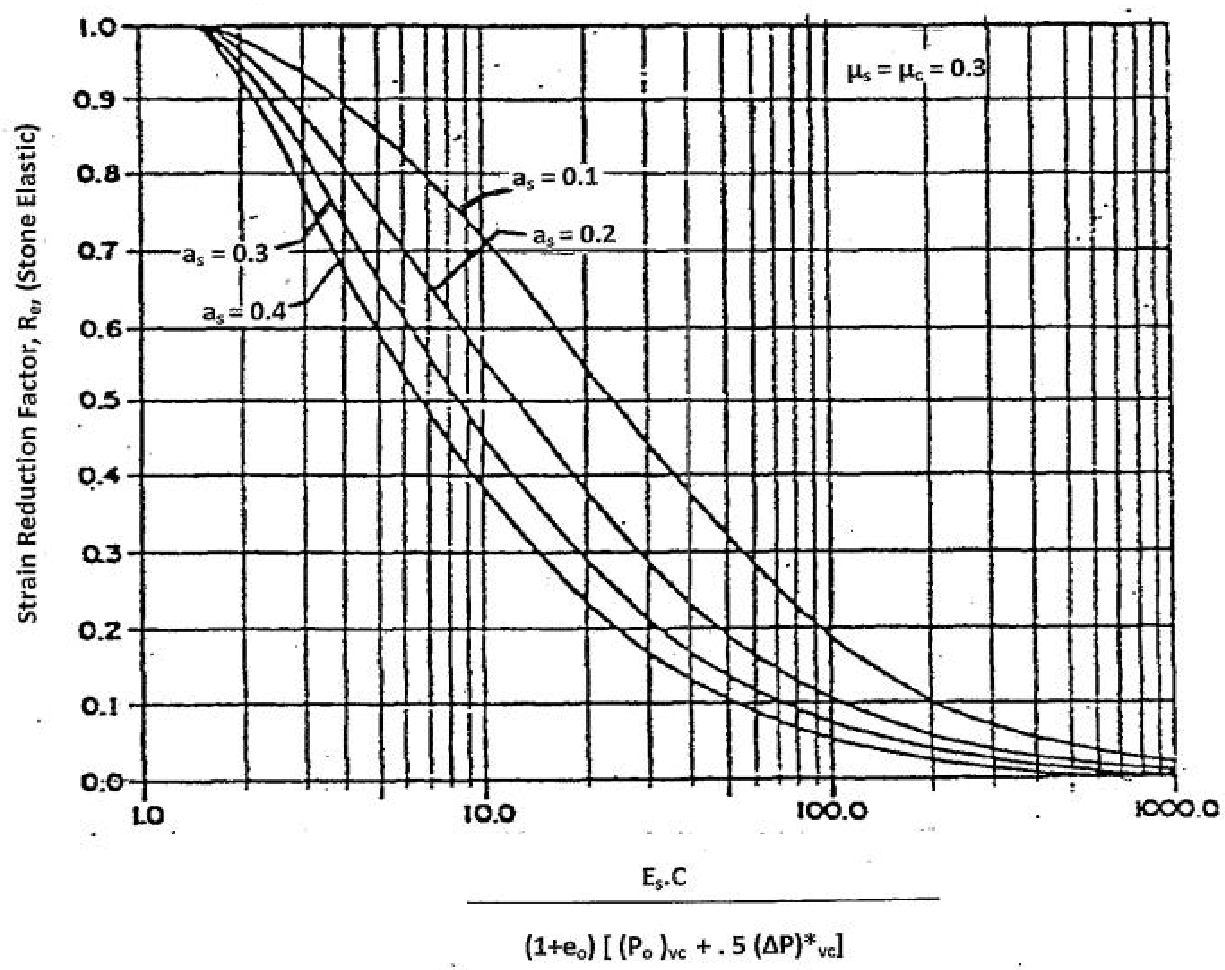


Figure 2.26: Graph for the determination of elastic strain reduction factor (R_e), (after Goughnour, 1983).

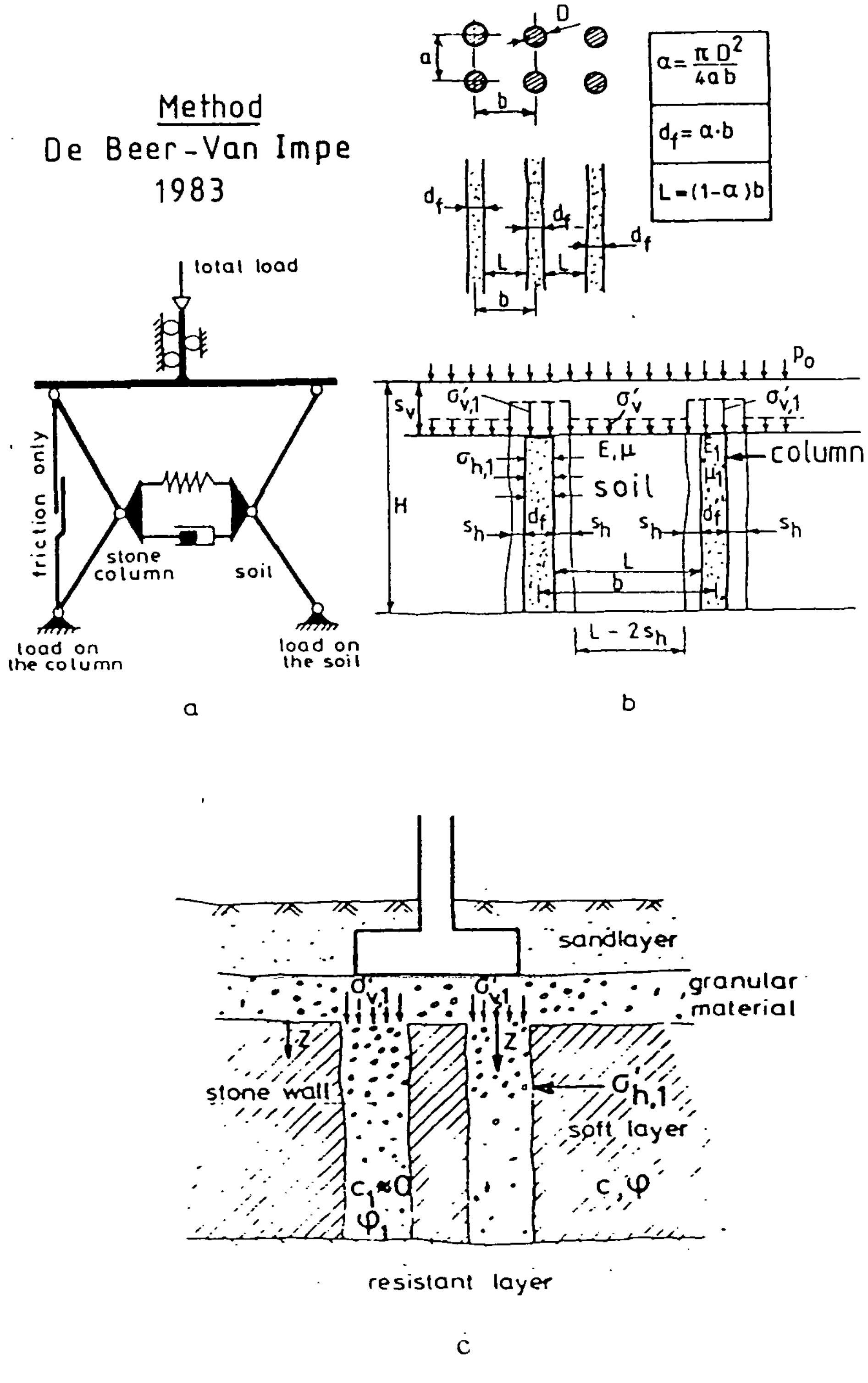


Figure 2.27: (a)-(c) Improvement of settlement behaviour of soft soil layers by means of stone columns (Granular wall method), after Van Impe and De Beer, 1983.

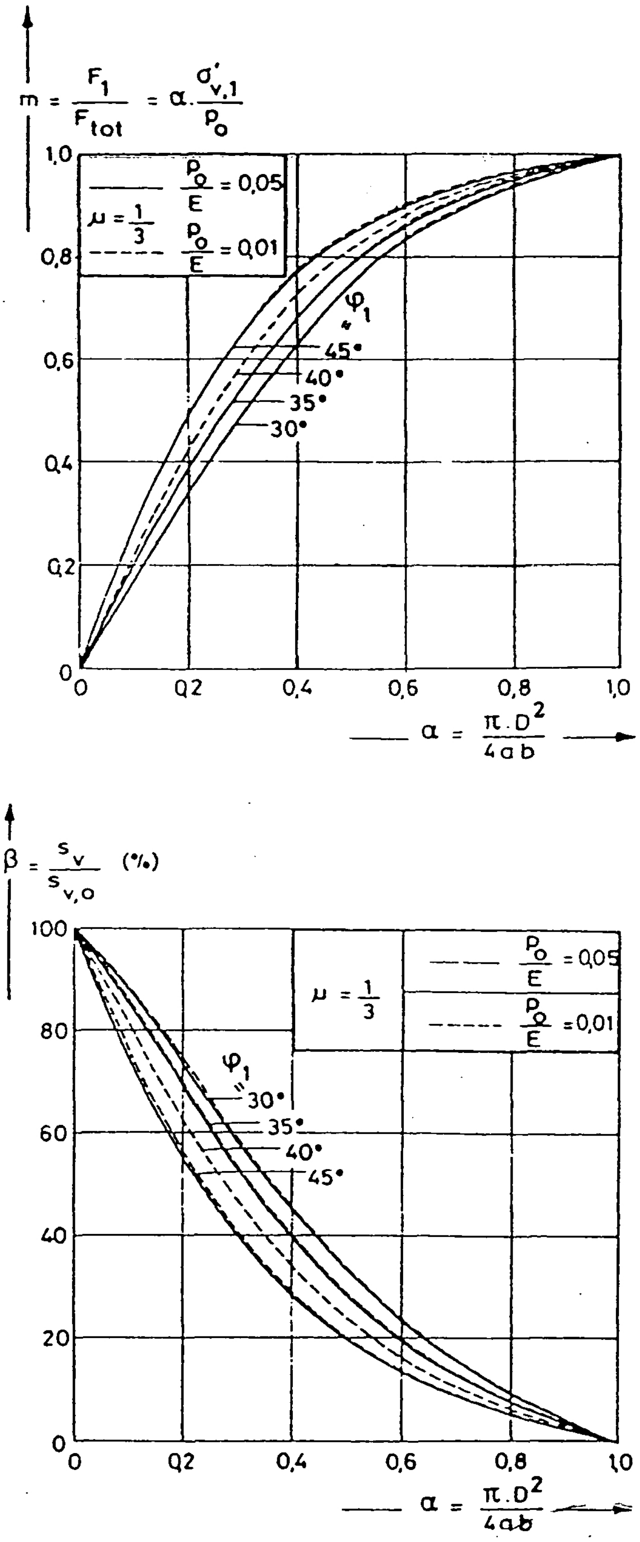


Figure 2.28: Soil Improvement design curves for stone colums (Granular wall method), after Van Impe and De Beer, 1983.

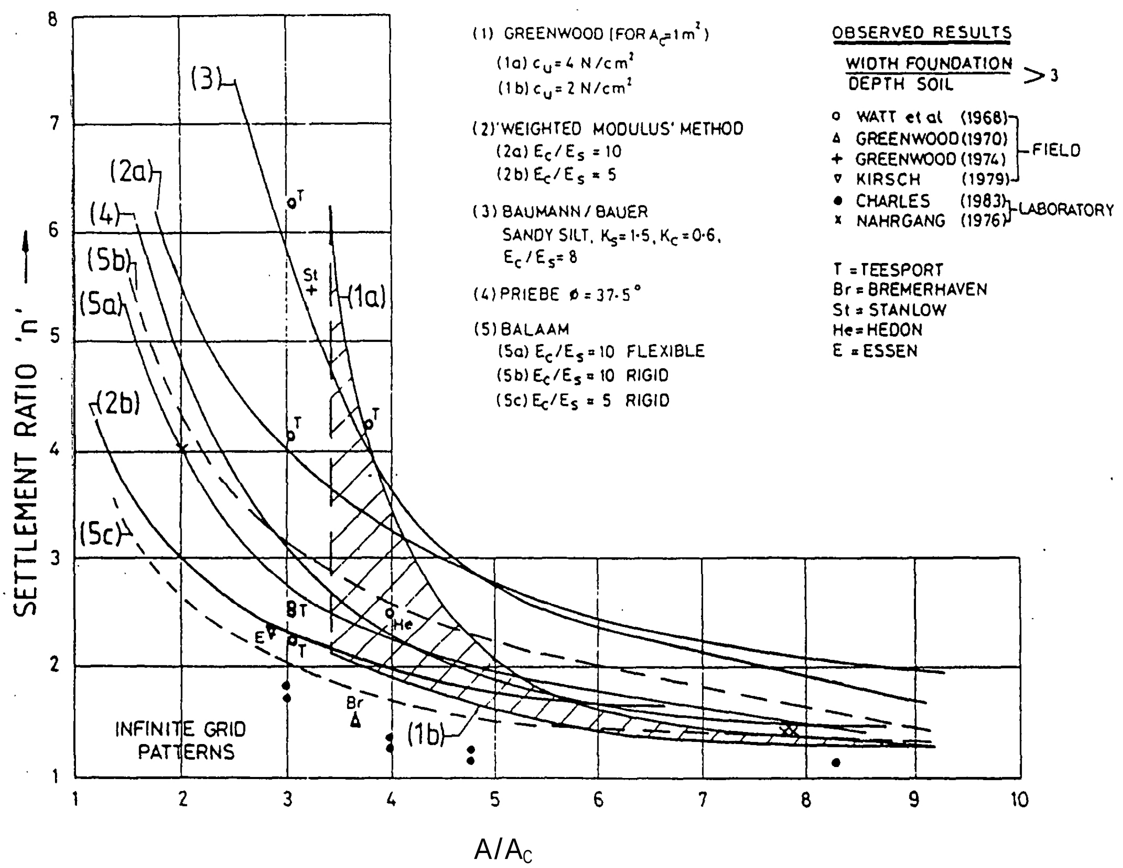


Figure 2.29: Comparison of elastic theories and field observations (after Greenwood and Kirsch, 1984). Note: A/A_C = area ratio.

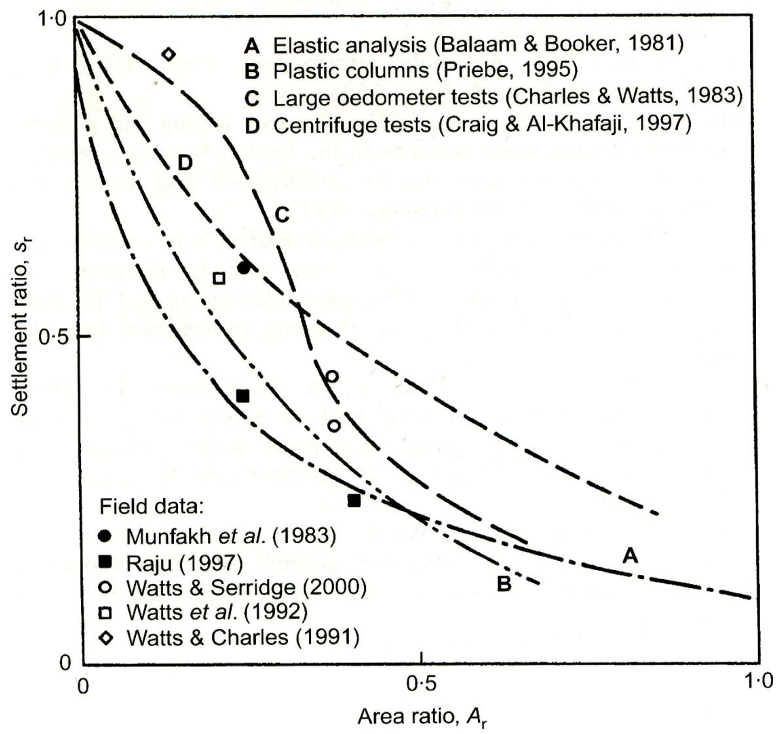


Figure 2.30: Stiffening effect of stone columns – comparison of different approaches (based upon field , laboratory and numerical modelling data (after Charles and Watts, 2002))

Baumann & Bauer Pad footing	Soil Clay	Soil Sand	**Priebe**	Soil Clay	Soil Sand	
Stone column diameter	500	600		500	600	mm
Average vertical effective stress	-	-		15	40	kPa
Initial soil friction angle phi'	0	35.0		0	37.5	Deg
Final soil friction phi'	0	37.5		0	38.8	Deg
Soil shear strength cu	50	0		50	0	kPa
Soil Young's Modulus Es	7.5	25		7.5	25	MPa
Soil Poisson's Ratio v	0.5	0.2		0.2	0.2	
Earth pressure coefficient Ka	1	0.27	Kc=Ka	0.22	0.22	
Earth pressure coefficient Kp	1	3.69		-	-	
Earth pressure coefficient Ks	1.25	0.85		-	-	
Earth pressure coefficient Kc=Ko	0.36	0.36		0.36	0.36	
Applied foundation load q	150	150		150	150	kPa
Unit area per column A	1.25	1.25		1.25	1.25	m^2
Equivalent radius a	0.63	0.63		-	-	m
Stone column friction angle	40	40		40	40	Deg
Stone column Young's Modulus Ec	40	40		40	40	MPa
Stone column spacing	1.2	1.2		1.2	1.2	m
Stone column area Ac	0.196	0.283		0.196	0.283	m^2
Area ratio Ac/A	0.16	0.23		0.16	0.23	
Ratio A/Ac	6.37	4.42		6.37	4.42	
Ratio Ec/Es	5.33	1.6		5.33	1.6	
Ratio Es/Ec	0.19	0.63		-	-	
Pc/Ps	11.57	5.39		7.18	7.67	
Pc	652.3	405.8		546.5	458.6	kPa
Ps	56.4	75.2		76.1	59.8	kPa
Basic improvement factor n_0	2.66	1.99		1.97	2.51	

Figure 2.31: Comparison of Baumann and Bauer (1974) and Priebe (1995) settlement approaches (after Slocombe, 2001).

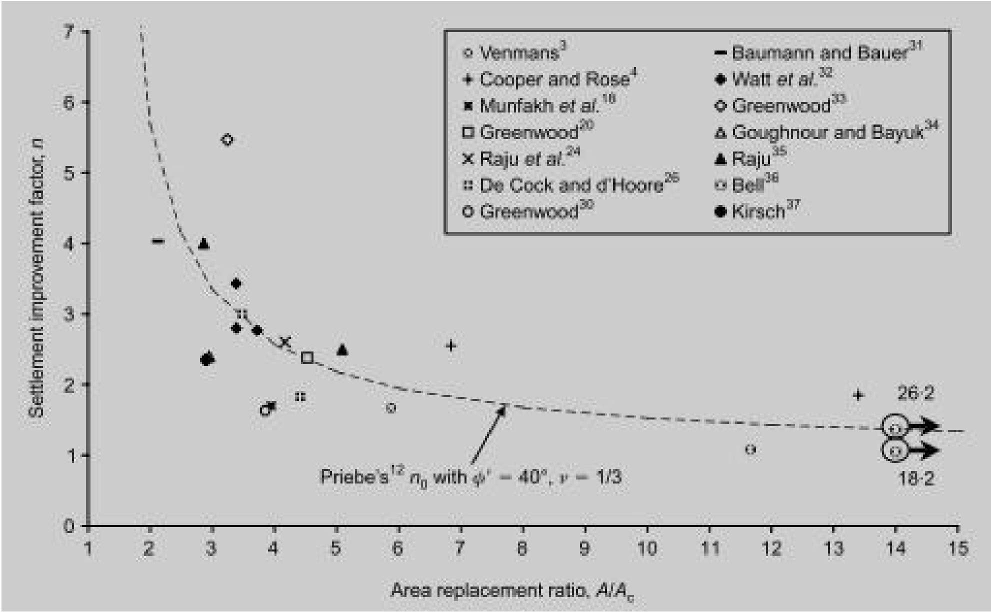


Figure 2.32: Settlement improvement factor against area replacement ratios for sites with widespread loading (after Mc Cabe et al., 2009).

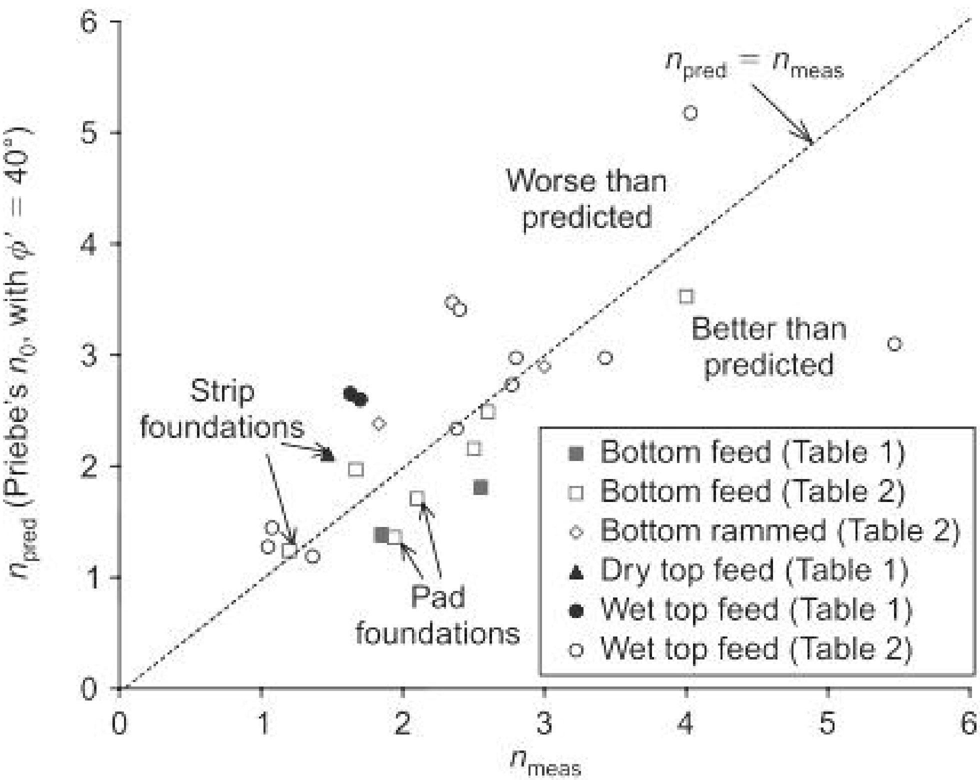


Figure 2.33 Predicted (n_{pred}) against measured (n_{meas}) settlement improvement factor for all widespread loadings and footings (after Mc Cabe et al., 2009).

Chapter 3 Documented case histories

3.1 Introduction

The importance of full scale vibro-stone column field testing cannot be underestimated. It has been emphasised that such tests are useful for improvement of current design methods (Hughes et al., 1976, Goughnour and Bayuk, 1979b, Greenwood, 1991). Field scale trials also permit assessment of ground response to both stone column installation and load application, refinement of design procedures (including validation of laboratory-based investigations) and more accurate prediction or assessment of long-term performance. Some ground improvement is potentially set at 'high risk' in difficult ground until successful field trials have been carried out, Serridge and Synac (2007). Furthermore, whilst there is a dearth of published information in the literature to illustrate the successful application of the vibro-stone column technique, both in the UK and internationally in soft clay soils, the vast majority of case studies, as evident previously from Chapters 1 and 2, relate to widespread loads associated with embankments, material stockpiles and storage tanks etc. Well documented case histories, where the settlement of low rise structures, supported by shallow narrow footings on vibro-stone column reinforced soft clay soils has been carefully monitored, or the documenting of field trials in this context, are generally lacking. It is also important to recognise that the ground improvement knowledge base has and should be based upon forensic examination of unsuccessful performance (which might be termed 'failures' in some quarters). Whilst it is evident that cases of unsatisfactory performance are not commonplace and it would therefore be misleading to impart any undue prominence to such cases, much can potentially be learnt about the ground-structure interaction and the behaviour of soils by studying the case histories of 'failures' or where problems (of poor performance) have arisen. According to Petroski (2006) design failures present perfect teaching opportunities and successful engineering includes an understanding of how things fail. Back analysis of failures potentially also gives greater insight into mechanisms which patently cannot be examined in successful load cases. It is perhaps also important to recognise that the definition of the term 'unsatisfactory performance' within the context of ground improvement has its limitations, as there can frequently be more than one cause. Moreover, in its simplest form it could be the failure to meet a performance requirement specified in the contract, whilst in practice the performance of the treated ground and associated structure may be quite adequate for purpose in terms of visual acceptability, serviceability limit states and ultimate limit states (Charles and Watts, 2002).

British Standards Institution Eurocode 7 (2004) recognises the critical importance of the quality of ground investigation information. It is clear from this document that knowledge of ground conditions and properties and the control of workmanship with regard to implementation of geotechnical processes, is recognised as usually more significant in terms of achieving specified performance requirements than the precision of calculation models or partial factors, and this also applies to the ground improvement sector. Within this sector the most common types of ground-related problems encountered relate to soil strata boundaries, i.e. geometry not as anticipated, and the geological and geotechnical characterisation of the soil profile, or what has been termed 'geo-characterisation', Serridge (2008) (see Appendix 3.1). For earthwork (embankment) projects over soft ground in the UK, where ground improvement techniques are commonly applied, there can be a significant time period between the initial site and ground investigation and the actual commencement of new earthworks projects. Boreholes can typically be quite widely spaced and may not be adequate for the more localised particular ground improvement technique(s) that are anticipated or being considered for a project. Therefore, most initial design is likely to be based on limited site investigation and it is important that the likely shortcomings of this are recognised by the designer(s) and ground engineering practitioners.

3.2 Successful Projects

Smallridge and Johnson (1990) provide a case study of a vibro-stone column project in soft ground to support a 130 m x 60 m warehouse structure (paper store) in Immingham, UK. The soil profile comprised around 2 m of soft clay fill (made ground), underlain by a further 4 m of soft alluvial clay, which in turn was underlain by firm to stiff cohesive (fine-grained) glacial till. Stone columns were constructed using the dry bottom-feed method to a depth of 6.0 m from the rig working platform level. An average column diameter of 720 mm was determined from both stone consumption records and measurement of the diameter of exposed stone columns during foundation excavation. The stone columns were installed at a frequency of one stone column for every 1.6 m^2 of strip footing area (representing an A_r equal to 25% for the recorded column diameter). Strip footing widths varied from 1.0 m to 2.5 m (Figure 3.1a) with a thickness of 600 mm and supported main structural loads, which translated to a bearing pressure requirement of 100 kN/m^2. Beneath ground floor slabs, which were ground bearing, 720 mm diameter stone columns were installed on an approximate 2.0 m square grid pattern (representing an A_r equal to 10%) to accommodate a combined live and dead load of 70 kN/m^2. The floor slab was cast in 9 m by 15 m bays with top and bottom mesh reinforcement. Post treatment settlement was estimated by the authors by first assessing the settlement under the applied loadings without treatment, using available soil geotechnical soil parameters and conventional stress distribution analysis, and then applying a settlement reduction factor (n_o) within the treated depth according to the Priebe (1976) approach prevailing at the time. A stone column (surface) area of 0.4 square metres and an angle of internal friction for the stone column aggregate of 40˚ was adopted by the authors. Settlement estimates without treatment for the slab was 215 mm reducing to 145 mm with treatment, i.e. settlement reduction factor of around 1.5. This assumed uniform settlement and it was argued that this figure probably erred on the side of caution since the floor would never be fully loaded over its whole area, because of the requirement for unloaded aisles to permit access and egress. Smallridge and Johnson (1990) also indicated that the above mentioned settlement figures applied to the centre of the building where maximum settlement was expected to occur, due to the typical 'dish-shaped' profile which is characteristic of warehouse floor slab settlements. The authors also indicated that as about 70% of the foundation load was a result of the floor load, foundations would tend to move in sympathy with the adjacent sections of floor slab. It was estimated that settlement at slab edges would be around 50% of that calculated for the slab centre and with a treatment depth of 6 m angular distortions were not expected to exceed 1 in 350 (Figure 3.1a) which was understood to have been acceptable to the client.

Settlements were also estimated using a computer based finite-element analysis with the same data input adopted as for hand calculation methods. It was found that estimated settlements beneath a fully loaded slab were of similar magnitude for both manual calculation and finite element analysis. It was indicated by the authors however that it was possible with the finite element analysis to obtain a better insight of variations in likely settlement across the building and more easily take account of the fact that the slab was not loaded over its whole area. Estimated settlements beneath a fully loaded slab were of similar magnitude for both manual calculation and the finite element analysis. The authors indicated that making allowance for unloaded aisles reduced estimated settlement(s) by approximately 17%. Estimated angular distortions varied with location and were between 1 in 300 and 1 in 460 (Figure 3.1a). With regard to actual settlement(s) recorded, the authors highlighted that there was a problem in determining precise amounts of total settlement that had occurred since the building construction had been completed, attributed to the bench mark used at the time having been disturbed. The authors indicated however that it was possible to determine relative differences in level over the bulk of the building. These data are shown in Figure 3.1b (represented by x). Also shown in this figure are actual differential settlements and angular distortions between the monitoring (measuring) points. The authors highlight however, that this information was taken from areas of the slab used as aisles, which may not have been subject to the full load of 70 kN/m^2 and therefore in these areas some hogging of the slab may have occurred resulting in lower than average settlements. These hogging moments seem to have been substantiated by their ground floor slab inspections, which revealed that a number of hairline cracks were developing running along the axes of the aisles. Monitoring of hairline cracks was therefore undertaken. From the data presented by the authors it is evident, as might be expected, that maximum settlements tend to occur in the central section of the building, with lower settlements occurring towards the corners. However, it is clear that this was not precisely followed, which is probably because of non-uniform load distribution across the building. As far as the performance of the building is concerned the authors indicated that the most important factor was the level of differential settlement that occurs, and from the data (Figure 3.1b) it can be seen that although the magnitude of the differential settlement does exceed that estimated in some areas, the actual distortions were well within the estimated maxima. Hence, it was concluded that the building was performing satisfactorily and that the ground improvement (treatment) was highly cost effective for a site where piling would have been cost prohibitive.

Greenwood (1991) reported the application of vibro-stone columns in very soft clays to accommodate a 36 m diameter asphalt-topped pad footing resting on a layer of free-draining rolled (compacted) gravel (which can effectively be regarded as a flexible base), to support a 12 m high steel oil storage tank at Canvey Island in the Thames Estuary, UK. The soil profile (Figure 3.2a) consisted of soft silty clay containing peat lenses (representing recent estuarine deposits of the River Thames), overlying medium dense silty sand. The wet top-feed method was used to install the columns on a 1.52 m triangular grid arrangement and to a depth of 10 m, and terminating in medium dense sand. The stone column treatment extended 6 m beyond the edge of the loaded area for stability reasons. Random measurements of post-installation stone column diameter near the ground surface demonstrated average column diameters of 750 mm. Pressure cells installed on the ground surface prior to tank construction measured the vertical stresses in the ground during filling and subsequent draining of the storage tank with both water and oil prior to full commissioning. The settlement of the ground was also monitored over a period of 160 days. The settlement improvement factor was stated to be 2.38. Greenwood (1991) proposed that the slow loading rate for the foundation and the magnitude of the replacement ratio (A_r) of 22 % should have ensured drained loading conditions, although unfortunately no piezometric measurements were made. The stepped shape of the load-settlement curve (Figure 3.2b) was interpreted by Greenwood (1991) as suggesting that rapid drainage was taking place under incremental loading and the laminated nature of the alluvial silty clays was likely to have enhanced radial (horizontal) drainage to the stone columns. As the foundation was loaded the stone columns initially carried a high percentage of the load (around 95%), with a resultant very high stress concentration ratio of around 25. Given the relatively close spacings of the stone columns this would suggest that the stone columns were acting as relatively 'rigid piles', at least initially. However, as the loading increased, the stress ratio (S_r) reduced quite rapidly until it reached a ratio of around 5 (representing around 20% of the applied load being carried by the stone columns at the full loading of 130 kN/m^2), which is considered to be a reflection of the flexible nature of the loaded area and indicating that as consolidation settlement increased the soil accepted a progressively larger proportion of the applied stress. Greenwood (1991) indicates that a final observed S_r of 5 suggests an isotropic stress on the clay since for φ' assumed fully mobilised and no plastic bulging in the columns, the ratio of principal stresses in the column would also be about 5. Thus stresses on the soil both vertically and radially would be approximately equal, and there would be little shear stress in the soil at this stage. This accords with the relatively small total settlements measured after 100 days. The stress ratios observed in this case history were higher than would usually be expected, but this may be due to the very soft nature of the *in situ* material (native soil). In addition, the decrease of stress ratio with applied load seems to contradict other field and laboratory studies, e.g. Hu, 1995, albeit that they did not consider widespread loads.

Johnson (1994) described a ground improvement project at Gladstone Dock in Liverpool, UK. Vibro stone columns were installed using the dry bottom-feed method beneath the proposed positions of 20 m high coal stockpiles (stacks), together with associated conveyor structures and a water treatment plant (for coal dust suppression), in order to prevent shear failure and control settlements. Ground conditions typically comprised essentially granular fill (made ground), extending to depths of between 1.5 m and 5.5 m, with the exception of the western part of the site where the near surface granular fill was underlain by cohesive (fine-grained) fill extending to depths of between 1.3 m and 4.7 m. Soft alluvial clays were present beneath the made ground, extending to a maximum depth of 8.5 m at the western extremity of the site. Over other parts of the site the alluvial soils comprised an inter-bedded sequence of sand and gravel over firm to stiff clay (glacial till), which extended to a depth of 20.5 m where sandstone bedrock was encountered. Undrained shear strength and SPT N value profiles are given in Figure 3.3a and 3.3b. The greatest thickness of soft soil occured beneath stockpile 3, for which a schematic geological cross-section is presented in Figure 3.3c. Soil and material properties are also summarised in Table 3.1. Preliminary design calculations by Johnson (1994) showed that whilst settlement of the coal stacks would be within limits required by the client, parts of the coal stacks would be unstable, if placed on unimproved soils. Therefore, the design of the treatment beneath the stacks was directed primarily at achieving a specified minimum factor of safety against ground instability. Beneath the various structures, where stability was not considered a problem, stone column design was based upon Hughes and Withers (1974) for determination of load carrying capacity and Baumann and Bauer (1974) for an initial estimate of stress distribution between column and soil and hence factor of safety against column overload. Settlement(s) were estimated, based on soil parameters without treatment and then appropriate settlement reduction factors were applied within the treated depth to allow for the 'reinforcing' effects of the stone columns in accordance with Priebe (1976; 1988). For the soil profile present at Gladstone Dock a vertical stress ratio (S_r) of up to and exceeding 7 was calculated. Whilst it is common for most design approaches to assess shear strength parameters for the stone column-soil system, which are then used in normal stability calculations, on the Gladstone Dock project average soil strength parameters (C_o' and φ'_o) appear to have been derived using the following formulae:

$$C_o = C' (1-A_r) \quad \text{------} \quad 3.1$$

$$\tan \varphi'_o = [(1-A_r).\tan \varphi'_s + S_r.A_r.\tan \varphi'_c]/[(1-A_r)(S_r-1)] \quad \text{------} \quad 3.2$$

where,

C' - Cohesion of the in situ soil
A_r - Area ratio
φ'_s - Effective angle of internal friction for soil
φ'_c - Effective angle of internal friction for stone column
S_r - Stress ratio appropriate to orientation of the failure surface

where,

$$S_r = 1+ (S_{rv} - 1) \cos \alpha \quad \text{------} \quad 3.3$$

and

S_{rv} – ratio of vertical stress in the stone column divided by vertical stress in the soil
α – Inclination of failure surface.

In the analysis of stability beneath embankments, some authors have applied S_r values of up to 5 (Goughnour et al., 1990), but more commonly stress ratios in the range 2-3 have been adopted (Munfakh et al. (1984) and Greenwood (1991)). Johnson (1994) considered that adoption of stress ratios in the range 2-3 was cautious but given the absence of substantiating geotechnical data to support higher values, an average stress ratio of 2.5 was used. Johnson (1994) recognised that other authors e.g. Goughnour et al., 1990, had indicated that stability analyses may be performed using either a total stress approach or an effective stress approach. Furthermore, it was recognised by Johnson (1994) that a total stress analysis, with low stress ratio, whilst providing a safe solution, would have involved a greater number of stone columns than may have been necessary to achieve the required Factor of Safety. However, a more satisfactory (economic) approach and which seems to have been adopted by Johnson (1994) was to allow for some drainage of the stone column-soil system as load was applied. The final design on the Gladstone Dock project therefore appears to have been based on drained conditions, but with up to 100 kN/m^2 of excess pore water pressure (pore pressure ratio (r_u) equal to 0.47). The assumption of some excess pore pressure takes account of the fact that the stone column-soil system may not be entirely free draining. The undrained soil parameters used are presented in Table 3.1. A summary of results of example calculations for stockpile 3 are presented in Table 3.2. In drained analyses, a friction angle of 27° was adopted for the cohesive made ground and alluvium. Total stress calculations in rows 1-7, (Table 3.2) show the sensitivity of the analysis to variations in stress ratio (S_r) and area ratio (A_r). It is important to note that the effective stress analysis in row 8 shows a significantly greater area per compaction or frequency of stone columns (A_o) for the given stress ratio, than does the total stress analysis at the same stress ratio. Based on a total stress analysis, with a stress ratio equal to 4 during the early stages of loading and also partially drained conditions during the later stages of loading, with r_u of 0.47 and S_r of 2.5 an area per compaction (A_o) of 6.7 m^2 was estimated by Johnson (1994) to provide a factor of safety exceeding 1.3, as required by the specification. However, because of uncertainties about appropriate values of S_r and allowing for possible variations of soil conditions outside those found, stone columns were actually installed beneath coal stockpile No. 3 with an A_o of 5.0 m^2. It is evident that a similar approach was applied to other coal stack treatment.

Raju (1997) reported the application of vibro-stone columns in extremely soft ('ultra soft'), fine-grained soils to support (coastal) highway embankments forming part of the new Shah Alam Expressway in Western Malaysia. The fine-grained soils within which the stone columns were installed were broadly subdivided into tin mine tailings (slimes), a relict of past tin mining activity in the area, and marine clays. The mine tailings were generally clayey silts with a fine sand content of about 15%. Recorded field and laboratory test data pertaining to the two soil types are given in Table 3.3, Figure 3.4a. A schematic cross section through the ground improvement works is given in Figure 3.4b. The maximum thickness of the slimes within the treatment areas was 16 m. Dutch cone tip resistances (q_c) in the material varied between 0.15 MPa and 1.0

MPa. Undrained shear strength values as low as 6 kN/m^2 were recorded near the surface of the deposit gradually increasing with depth at a rate of 2 to 3 kN/m^2 per metre. The upper very soft marine clay deposits were generally around 11 m thick with recorded dutch cone tip resistance (q_c) values of between 0.1 MPa and 0.3 MPa. Undrained shear strengths as low as 5 kN/m^2 were recorded at shallow depths, increasing at a rate of about 1 kN/m^2 per metre depth. A practical feeling for the low strength of the soils can be obtained from the fact that an embankment, approximately 1 m high, actually failed when constructed on untreated soil. The stone column layout adopted was reported to be a function of analysis and design based principally upon the Priebe (1988;1995) approach. The diameter of the columns were typically 1.0 m, but in some cases were reported as large as 1.2 m in the extremely soft soils. Although the installation method was not stated it is considered that only the wet top-feed technique could have been used to achieve the reported diameters and depths, notwithstanding the low soil strength (see Section 1.2.3). The columns were installed on a square grid pattern ranging from 1.5 m to 2.5 m centre-to-centre spacings (A_r ranging from 13-35%) dependent upon embankment height and resultant magnitude of loading. All columns were installed to the level of competent natural strata described as a dense/stiff layer by the author. The layout of the columns (variation in spacing and therefore drainage path length) was also designed to facilitate achievement of 90% primary consolidation within 6 months of completion of embankment construction. For embankment heights up to 10 m, recorded settlements were of the order of 250 mm in the mining slimes and 400 mm in the marine clay, compared to over 1.0 m where no treatment was applied. According to Raju (1997) these values imply settlement improvement factors of 4 and 2.5, for the slimes and marine clay respectively and providing a clear indication that it is possible to improve such soft soils with vibro stone columns (at least with the wet top-feed vibro stone column technique). Comparison of observed settlements with those predicted by the Priebe (1995) method show that in general the predicted values are larger than those measured on site. However, one cannot rule out the greater stiffness of the inner columns attributed to the confining action of the widespread load in reducing settlement or the benefits of a thick sand drainage blanket which could potentially have resulted in some accelerated consolidation of the underlying soft soils before settlement readings were commenced. Of significance is the settlement behaviour of the two types of very soft soil with comparable undrained shear strengths, in the presence of stone columns, which was distinctly different. The slimes consolidated very quickly, as might be expected, based upon the soil parameters in Table 3.3 - over 75% of the settlement took place during embankment construction and 90% consolidation was achieved at 90 days after completion of embankment construction and with no significant further settlement over a period of about 3 months thereafter. The marine clay however took much longer to consolidate. Over 75% of the total settlement took place after completion of the embankment construction with 90% consolidation occurring 180 days following completion of the embankment. A period of about 6 months was hence required for settlements to stabilise. This difference in behaviour was attributed to differences in sand content and therefore permeability and $C_{v,}$ (coefficient of consolidation), soil plasticity and sensitivity (Table 3.3). Since the marine clay was also more sensitive, it may have experienced a greater degree of disturbance and remoulding and smear during stone column installation, (although this should in theory have been limited by use of the wet top-feed technique), which would potentially have impacted on drainage capacity of the stone column reinforced soil. Although no parameters are given to directly substantiate improvement in undrained shear strength within the treated soil layers, this was clearly demonstrated by the fact that embankment heights up to 10 m could be constructed on the very soft soils after stone column installation (with no signs of slope failure or movement recorded, - Raju (1997) accepts that a Factor of Safety of 1.5 could only be shown theoretically), whereas a 1.0 m high embankment constructed over the untreated ground failed, as mentioned previously. Raju (1997) also highlighted that stone column construction is not routine in 'ultra soft' soils and up until the mid 1990’s, the suitability of vibro-stone columns for such very soft cohesive soils would have been questionable. With this method the operator and the supervising engineer on site need to pay very close attention to detail and automated monitoring systems which provide real time information on the installation process and ground response to stone column installation are essential. The nature and properties of the soil (including soil sensitivity) also have to be carefully assessed when estimating consolidation periods. It is also notable that Raju (1997) does not highlight the significant contribution of the 1.0 m thick sand drainage blanket on the performance of the ground improvement, as intimated previously, particularly in the context of providing an initial pre-load and assistance with drainage during the consolidation period both during and following embankment construction.

Cooper and Rose (1999) describe the successful application of vibro ground improvement works to the south roundabout embankment to the River Avon bridge of the St. Philips Causeway in Bristol, UK. The embankment was up to 7 m high and underlain by thick alluvial deposits, including soft clays with some peat, within the River Avon Floodplain. Stone columns were employed to ensure short-term stability, to accelerate and control consolidation settlement, and to reduce secondary consolidation creep effects in order to achieve stringent settlement criteria within a relatively short embankment construction and commissioning period. The particular ground conditions impacting on the design of the ground improvement scheme included: thick variable soft alluvial deposits; a buried alluvial channel infilled with soft alluvial deposits and discontinuous lenses of peat, taken as up to 2 m thick at the base of the alluvial deposits. Site Investigation locations and typical geological sections through the embankment are given in Figure 3.5a-c. In terms of geotechnical properties for the alluvial soils, a lower bound undrained shear strength of 15 kN/m^2 was used for both clay and peat, while for overall stability analysis a moderately conservative value of 20 kN/m^2 was adopted. For compressibility parameters an M_v (coefficient of volume compressibility) and C_v (coefficient of consolidation) of 0.4 m^2/MN and 1.5 m^2/yr respectively for the clayey silt and 2.0 m^2/MN and 1.5 m^2/yr respectively for the silty peat was adopted. In the absence of consolidation test data for the peat, where encountered, based upon a combination of inspection of the one trial pit that encountered peat and which was noted to have a high silt content, together with experience and reference to published values, a presumed M_v value of 2.0 m^2/MN was considered appropriately cautious by the authors. Whilst it is common for C_h to be taken as 10 x $C_{v,}$ for both of the two main soil types discussed above the C_h was assumed equal to $C_{v,}$ based upon published data and experience with similar ground conditions in the Bristol area. The stability of embankment slopes were checked for short-term and long-term conditions. The target factors of safety using a moderately conservative approach were 1.2 and 1.3 for short-term and long-term stability respectively. Local embankment stability and overall stability on the scheme as a whole were analysed using circular slip analyses. Settlements of approximately 500-650 mm were predicted on unimproved ground with significant differential settlement potential.

Settlement – stone column spacing relationships were established for the full range of embankment loadings and ground profiles anticipated. The resultant stone column centre-to-centre spacings varied from 1.8 to 2.4 m on a triangular grid arrangement, generally with the wider spacings in the low embankment regions and closer spacings in the higher embankment regions, as would be expected. Closer spacings were also required in the peat and alluvial areas to control differential settlement (see Figure 3.5d and 3.5e respectively for layout of stone columns and instrumentation). Overall the installation was predicted to reduce the maximum total settlement to around 235 mm, compared with the estimated maximum of around 600 mm on untreated ground. The action of stone columns as vertical drains to accelerate the expected settlements was a key element of the design. The 'fitness-for-purpose' performance being sought was that the settlement rate should be less than 10 mm in any three months following embankment construction. The time available under the contract required that this performance be achieved within six months of the end of construction. The settlement in the period six to nine months after completion, and the associated residual settlement, were therefore estimated for all design soil profiles and potential stone column spacings using the procedure of Kjellman (1948). This approach would appear to have produced a drain spacing sufficient to achieve a specified average degree of consolidation in a specified time. The calculation procedure was re-formulated to predict the average degree of consolidation in a specified time. The analyses indicated that primary consolidation would be up to 94% complete in key areas by the end of the critical period, though in areas with less settlement, potential wider spacings were possible, reducing this to around 80%. The longer-term settlement characteristics of the embankment would thus be mainly influenced by secondary compression settlements and these were incorporated into the design appraisal. A C_α value of 0.02 was used in design, with secondary compression being used in a full thickness peat layer but not in the alluvium.

The monitoring results of a hydraulic profile gauge (HPG8) are presented in Figure 3.5f. (Pore pressures and (relative) monitoring plate levels are given in Figure 3.5g and 3.5h respectively). This profile is useful as it illustrates the differences between the settlements of improved and (untreated) natural ground. It is useful to compare the settlement gradients between the stone column reinforced and non stone column reinforced areas in Figure 3f. The effectiveness of stone columns in providing significantly shorter drainage paths for pore pressure dissipation, compared to the untreated situation, is clearly demonstrated. It is perhaps important to note in Figure

3.5g that the excess pore water pressure is not measured as an incremental increase as the load is applied. This would suggest that the monitoring is perhaps out of phase. The buried end of the tube was located in the centre of the roundabout, and for the first 20m, passed through an area where no ground improvement was carried out, i.e. off the critical path. A pattern of greater settlement under the untreated area is evident in the displacement profiles of HPG8. The settlements at the western end of this profile gauge, where no stone columns were installed, were around 600 mm (which compares closely with pre-treatment design predictions of 540 mm of primary consolidation settlement and an estimated 60 mm of secondary compression (based on $C_\alpha = 0.04$ for an improved area). The settlement at the eastern end of the gauge tube, where the stone columns would appear to have been fully effective, is only around 220 mm. An analysis following the Priebe (1995) approach predicted 205 mm of post embankment construction consolidation settlement in this zone, with 30 mm of secondary compression.

Settlement against log-time plots indicated a C_α value of generally around 0.02-0.03, based on a 2 m thick peat layer generating the secondary compression movement. Most post construction settlement was estimated to be secondary consolidation. It was also recognised by the authors that the embankment loads on the soft alluvial soils would result in lateral loading of an adjacent piled bridge abutment foundation, due to soil consolidation and lateral 'squeezing'. This was addressed through the novel application of a transition zone of vibro concrete columns (VCC's), which effectively functioned as settlement reducing piles, between the piled bridge abutments and stone column reinforced soils to reduce the effects of soil consolidation and lateral squeezing and provide a smoother settlement profile across this transition. A load transfer platform, comprising a 1 m thick granular blanket with three layers of geogrid transferred the embankment loads onto the VCC's, (which were constructed with enlarged heads to facilitate the 'arching' mechanism via the load transfer platform onto the VCC's), which were designed for end bearing on an underlying stiff stratum. The load transfer platform was extended 5 m beyond the VCC's into the vibro stone column (VSC) zone to provide a suitable transition and to reduce differential settlement gradients.

The paper also makes a number of useful observations relating to both the design and monitoring of the vibro ground improvement. An unexpected aspect of the hydraulic gauge HPG8 profile was the very gradual increase in effectiveness of the stone columns inwards (away) from the edge of the central untreated area. Full settlement control was only achieved at a distance of some 16-20 m back from the edge of the stone column treated area. The authors intimate that it seems probable that the edge effect shown by the HPG8 profile extends right around the perimeter of the central untreated zone. The most probable explanation for this effect is cited by the authors as being the lack of ground improvement in the centre of the roundabout having induced load transfer onto the stone columns, both laterally and axially. Stresses in the stone columns would thus have increased beyond the design values and excess deflections resulted. In more usual applications of stone columns under simple linear embankments, this effect would not be considered to occur by the authors as the sloping edge of the fill would reduce the imposed stresses towards the edge of the treated zone, and no adjacent heavily loaded, but untreated area would exist.

Serridge and Synac (2007) describe the application of vibro stone columns in very soft clays to support a road embankment (up to around 3.0 m high) for a new relief road in Kings Lynn, Norfolk (UK). Preliminary trials were considered a pre-requisite on the project, owing to the fairly complex and weak nature of the soil profile. A typical geological cross-section, refined by static cone penetration tests (CPT), permitting refinement of the ground model for the site, is given in Figure 3.6a. As part of assessing ground improvement options for the site, a surcharge load test was carried out on the proposed road alignment to assess the ground response of the existing (untreated) soil profile to embankment load. The surcharge load test generated high settlements (Figure 3.6b) and in the context of overall stability the Factor of Safety was quite marginal (approaching unity). Opportunity was also provided to install a group of trial stone columns and carry out a further surcharge load test (on treated ground), utilising the sand fill that had been used for the earlier surcharge load test on untreated ground. This permitted the trials to be carried out at relatively low cost. It also provided insight into the ground response to vibro-stone column installation using the dry bottom-feed technique and the likely performance of stone columns under a surcharge load comprising approximately 3.0 m of sand fill - equating to around 60 kN/m^2, over a 5-6 week period, a longer period was not possible owing to programme constraints. The results of the trial are detailed in Figure 3.6b. The trial stone columns were installed on a 2.0 m triangular grid pattern. Stone column diameter was determined to be 700 mm from stone consumption records and exposure and measurement of stone columns prior to placement of surcharge giving A_r = 11%, and with stone columns extending through the soft alluvial soils and peat to the level of the marine sand overlying the top of the Kimmeridge Clay (Figure 3.6a). Predicted settlements (without treatment) were in line with the surcharge load test on untreated ground (Figure 3.6b). Settlements for the surcharge load test on trial stone columns is also annotated in Figure 3.6b which show significantly lesser primary consolidation settlements by a factor of 1.5-2.0, when compared to the untreated soil. Following successful trials, vibro stone columns were adopted as the main ground improvement technique for the embankment structures over soft ground. Monitoring of actual full embankment construction (up to 3.0 metres) in the main works yielded significantly lesser settlements (Figure 3.6b) than for the vibro-stone column trials, i.e. 65 mm compared to 100 mm albeit that some extrapolation of the vibro stone column field trial data had been required due to programme constraints. This supports earlier remarks and written discussion put forward by Greenwood 1976a; 1976b and Greenwood, 2004, namely that better performance of full embankment construction (widespread load) is attributed to the loading conditions strongly influencing the stiffness and strength of the columns - except for columns towards the edge of the loaded area, the columns become stiffer and stronger as load was applied. As a result stone columns in large arrays under wide loaded areas such as embankments perform better than those under small loaded areas where more columns are constrained only by ground which is not loaded. The smaller scale trial surcharge load test on stone columns at Kings Lynn therefore indicated the stone columns to be less stiff so that they deformed more under test loads compared to under full embankment construction. Pre-treatment (primary consolidation) settlements were estimated to be of the order of 130 mm and on the basis of an A_r of 11%, a stone column friction angle of 40˚ and reference to the Priebe (1995), these settlements were estimated to be reduced to around 75 mm**.** This corresponds to a settlement reduction factor of around 1.7. The reduction in drainage path length provided by the stone columns coupled with provision of a surface granular working blanket, facilitated rapid dissipation of excess pore water pressures and corresponding improvement in composite soil stiffness. This resulted in acceleration of predicted primary consolidation settlements with the result that 85-90% of the predicted primary consolidation was complete within around 3 months of completion of embankment construction. Superimposed on residual primary consolidation settlements was around 50-60 mm secondary consolidation creep, but which was anticipated to be within the normal serviceability limits of the structure.

Most of the proceeding case histories relate to the application of vibro stone columns in the context of a widespread load. However, one of very few recently executed field trials addressing stone column application beneath narrow footings is given by Mc Cabe et al. (2009), who discuss some preliminary research by Egan et al. (2008) assessing the behaviour of trial strip footings relevant to two-storey light-weight low rise structures and referred to as Contract B. Ground conditions at the site comprised a 1.5 m thick clay crust (30 kN/m^2 < C_u< 100 kN/m^2) underlain by 12 m of soft Carse Clay (average C_u = 10 kPa). Load tests were carried out on a strip footing (Figure 3.7a and b) to assess the feasibility of stone columns in these soils. A typical settlement-time graph is given in Figure 3.7b (with the first 24 hours of immediate elastic settlement removed, since this it was argued that this would occur during construction), from which it can be seen that the majority of primary consolidation settlement was complete within 8 weeks. No indication of column diameter appears to have been given.

3.3 Unsuccessful projects

McKenna et al. (1976) described an apparently 'unsuccessful' application of vibro-stone column ground improvement supporting a trial embankment at East Brent, in the Somerset Levels (UK), associated with a new alignment of the M5 motorway. This would appear to represent the first published record in the UK where it was suggested that stone columns had no apparent effect on foundation performance, which led to much discussion and debate (and indeed controversy), in particular in the first Géotechnique Symposium in Print for Ground Improvement in 1976 (Greenwood, 1976a; 1976b; Bishop, 1976; Burland, 1976; Thorburn, 1976). The field circumstances for the trial are reproduced in Figure 3.8(a).

The reported soil profile at the site comprised a deep succession of estuarine (alluvial) deposits of the River Severn, underlain by Lower Lias Clay. The upper 11 m of the estuarine sediments comprised soft silty clays, underlain by around 18 m of grey silty sand, interbedded with clay laminae and intercalated with peat lenses, particularly towards the base of the deposit. It appears that there was no sharp interface between the two estuarine deposits described. To investigate the effectiveness of stone columns in reducing settlements, field trials were undertaken. A group of thirty stone columns were installed using the wet top-feed technique on a 2.45 m triangular grid pattern under one end of a trial embankment (with the remainder of the embankment left untreated), and with the highest section approaching 8.0 m (the embankment was built up to a maximum height of 9.1 m with side slopes of 3:1). The columns were around 11.3 m long with a diameter of 0.9 m (which was confirmed by stone consumption records and column exhumation), so that A_r was 12.5%. The stone columns did not penetrate the full thickness of weak alluvium. The earth embankment foundations incorporated three groups of instrumentation consisting of rod settlement gauges, piezometers and inductive settlement gauge(s) installed in the left, central and right section of the trial embankment (see Figure 3.8a).

The settlement records made two days before the central section failed (Figures 3.8 b,c and d) show that the untreated end of the embankment settled significantly less than that with stone columns, and the untreated central section, which slid after 90 days of loading when the embankment was 7.1 metres high, had settled almost exactly the same amount as the stone column section, immediately prior to the slide. The recorded pore pressures on the day of the slip are given in Figure 3.8e. As a result the engineers suggested that the stone columns were not performing satisfactorily, i.e. the columns apparently had no effect on the amount or rate of settlement of the embankment, and were therefore not adopted in the main project works. Mc Kenna et al. (1976) postulated that the apparently unsuccessful behaviour of the columns was due to no drainage to the columns because of soil disturbance and remoulding during construction (and smearing at the column-clay interface) and also loss of clay volume in the annulus around the vibroflot which was formed during water flushing.

Greenwood, 1976a; Greenwood, 1976b and Greenwood, 1991 argue that the case study of the field trial was incorrectly interpreted and that the explanation was more complex. He provided piezometer measurements (Figure 3.9) to demonstrate that free drainage was taking place during and after stone column construction so that smearing could not have been significant. By reference to the shear strength resistance required for soil to penetrate soil pore spaces based on Raffle and Greenwood (1961), Greenwood (1976a;

1976b) was of the opinion that the strength of the clay, even softened by remoulding, would inhibit inter-penetration. Moreover, during the wet-top feed technique any silty clay sheared and softened by the lateral gyratory impacts of the vibroflot is immediately removed by the upflowing water velocity in the annulus between the vibroflot and soil. The space is made good by introducing and subsequently compacting stone aggregate (see also Chapter 1 Section 1.2). Backed by observations of excavated columns in the clay Greenwood (1976a; 1976b) also presented a magnified photograph (Figure 3.10) of the column-clay boundary, which showed only sand filled voids in the column and with no significant inter-penetration of clay. Whilst it is accepted that skin friction in a layer adjacent to the column could have been diminished marginally, assuming a rough contact, Greenwood (1976a); (1976b) and Greenwood (1991) considered it unlikely that it had regressed to its limit.

It is perhaps also worthy of note that successful application of the wet top-feed system in soft cohesive soils has been reported by several authors. Munfakh et al. (1984), for example, describe successful treatment for a trial embankment, in which only limited intrusion of fines from the treated ground was noted within the columns, mainly around the periphery. Mitchell and Huber (1985) provide similar comment in their description of the successful application of vibro stone columns to a wastewater facility in the U.S. Upon reviewing the field data further (including reference to the piezometric data – Figure 3.8e), Greenwood (1976a), (1976b) and (1991) noted that the pore pressure measurements recorded before failure of the central section of embankment (which according to Greenwood (1991) also resulted in damage to the monitoring station), show that within the stone column zone pore pressures increased more or less proportionately with depth to the base of the columns. This behaviour was considered by Greenwood (1976a;1976b and 1991) to be consistent with increasing relative movement with depth between the column and soil, suggesting 'punching'. This was attributed to shear resistance (skin friction) between column and soil in the main peat layer being destroyed by the wet top-feed process, causing the surface load of the rapidly constructed embankment to be almost fully transferred to the toe of the columns. A consequence of this was that the stone columns behaved like rigid 'friction' piles punching into a deeper (softer) soil layer, which was slightly sensitive and likely to have been remoulded, at least temporarily, to a very low undrained shear strength (less than 10 kN/m^2) by the column installation process. The widespread load from the embankment was considered to have restrained the intervening clay, preventing column bulging at any depth, so allowing stress transfer down the stone columns. The stone columns did not control the settlement because they were of inadequate length to do so, i.e. did not extend to a suitably competent stratum. By contrast piezometric measurements in the central and remote end zones without stone columns showed high pore pressures at the elevation of the peat layer between 4 and 5 metres (Figure 3.8e). The presence of the peat is barely reflected at the stone column end where pore water pressure dissipation appears to have taken place. The central section which slipped was found to have failed on or just below the peat. The untreated end settled (presumably by shearing displacement) above the peat layer, whereas the stone column end showed uniform settlement throughout the depth of the deposits. It is clear that the stone columns were not particularly effective in controlling settlements. In addition to the reasons for the poor performance suggested by Greenwood (1991) above, other contributory factors may have been the low area replacement ratio (A_r = 12.5%) and the actual layout of the stone columns.

Hu (1995) notes that it is likely that installing stone columns beneath the central region of the embankment would not be of great assistance in reducing the lateral displacement near the toe of the embankment and beyond and following Tavenas et al. (1979) the overall settlement beneath the central region would be unlikely to be reduced significantly. This is supported by work by Almeida (1984) using centrifuge modelling. By installing a group of columns of low area replacement ratio (A_r of 10%) beneath the embankment edge region only, Alemeida (1984) found that the settlement in the central section (unreinforced) attributed to the embankment load is reduced by about 30%. It is important to recognise however, that all the potential contributory factors to the poor performance at East Brent will not be completely understood, principally because of lack of site investigation and geotechnical characterisation of the soil profile at the location of the trials.

Greenwood (1991) reported a case history of ground improvement failure beneath an 18 metre diameter liquid natural gas (LNG) sphere, constructed on a rigid concrete pedestal foundation, which effectively provided a rigid surface raft (Figure 3.11a), near Mumbai in India. The soil profile (Figure 3.11a) comprised between 10 m and 12 m of very soft marine clay (with an undrained shear strength of 10 kN/m^2; a liquid limit of 110, plasticity index of 65 and a moisture content of 70-80%, indicating a clay with extremely high plasticity (CE) and liquid limit higher than moisture content – indicative of a very sensitive soil deposit), overlying rockhead. Stone columns were installed using the wet top-feed method on a 1.2 m square grid pattern and through the full depth of soft (sensitive) alluvial soils to the level of the rockhead. The stone columns were nominally 0.9 m in diameter (estimated from stone consumption records), giving a soil replacement (A_r) of around 45%. Recognising there would be load sharing between the stone columns and soil, the Supervising Engineer for the project requested load tests on concrete footings constructed over single columns and spanning two columns and the intervening ground. Results of the three load tests are given in Figure 3.11b. Load appears to have been applied fairly rapidly as settlement stabilised at each increment and each test apparently only took a few days. Recorded settlements for stresses up to a maximum of 1.5 times the intended design stress (265 kN/m^2) for the structural foundation were in the range 15-50 mm and deemed satisfactory. It should perhaps be noted that the 2.0 m x 1.5 m test pad underlain by two stone columns recorded a maximum settlement of 50 mm, more than twice that recorded on the two other test pads supported by one stone column (Figure 3.11b). This should have perhaps raised some concerns whilst interpreting the data. Whilst the presence of two stone columns would have provided significantly shorter drainage path lengths compared to where one stone column was present, the plot nevertheless appeared to be approaching a failure condition before the design load was reached. Notwithstanding this (presumably based upon the maximum recorded settlements not exceeding 50 mm in these short duration tests), a decision was made to proceed with construction and fully load test (hydro-test) the approximately 3,000 tonnes capacity tanks with water prior to commissioning. The first of the sphere foundations was tested by pumping water into the sphere and allowing it to stand at a number of incremental levels. Within 110 hours a total of 1,700 tonnes of water had been added to the (structure) dead weight of 1,300 tonnes and recorded foundation tilt had reached 91 mm with an average settlement of 300 mm. The observed tilt progressed further, resulting within a period of a few minutes, in total failure and accompanying ground heave and cracking of the surface crust over a distance of about 3.0 m (Figures 3.11d and e). The heave was observed to continue slowly over a period of a few days before stabilising. Examination of Figures 3.11d and 3.11e is indicative of a rotational failure, perhaps triggered by an eccentric loading. A plot of the water load test is re-produced in Figure 3.11c. Back analysis by Greenwood

(1991) suggested that pore pressure dissipation under the applied loading was at most around 15%, despite the close spacing and estimated large diameter of the columns, the implication being excessive soil smear and remoulding within the soil. This gave a resultant reduction in radial constraint from the native soil and in turn loss of strength of the column by a factor of 2.4 times. Immediately prior to failure, the ratio of stresses on the columns and soil was calculated as corresponding to around 10 because of loss of soil strength. Greenwood (1991) concluded that the small scale load tests were of limited value (and potentially misleading), where a widespread load (and by inference – a deep stress bulb) is applied over a soft clay profile strengthened by stone columns. The increase in vertical stress distribution beneath small scale steel plates or concrete test pads dissipates very quickly with depth and provides misleading results in the context of large loaded areas which will stress the soil to some depth.

Wilde and Crook (1992) described the settlement of a steel portal frame factory unit with dimensions of 90 m by 20 m in Warrington, UK, comprising simple pad and strip footings and a ground bearing floor slab. The initial site and ground investigation showed the site to be underlain by a sequence of soft fine-grained alluvial soils, varying in thickness from 5 m to 10 m. Prior to implementation of vibro stone column techniques the site was brought up to the required development plateau levels by addition of up to a maximum 1.5 m depth of upfill, which was coincident with the maximum 10 m thickness of the alluvial deposit beneath the building footprint. The stone columns were located in closely spaced groups beneath pad footings, (as would be normal practice) at up to 2.0 m centres beneath intervening strip footings and on a general grid pattern beneath ground bearing floor slab areas. However, the columns did not fully penetrate the weak alluvial soils. Post construction monitoring of the portal frame structure showed that over a 6 year period 120 mm of total settlement occurred with a maximum differential settlement of 100 mm along the length of the structure (Figure 3.12). It was estimated that there was probably another 50 mm of settlement during the construction period, but this was not recorded. Levelling of the floor slab revealed similar movements to those suffered by the foundations. It is evident that a significant component of the recorded settlement (around 80%) was attributable to the surcharge effect of the upfill (analogous to negative skin friction effects), associated with the raising of site levels rather than the relatively small weight of the industrial unit. This brief case history clearly demonstrates the need for careful evaluation of vibro stone column design in soft fine-grained (clay) soils, including full understanding of site levels and the impacts of any changes prior to ground improvement, particularly where any significant raising of site levels is proposed, i.e. site regrade needs to be critically appraised. Unusually, this structure appears to have performed quite adequately for its purpose, but is unlikely to have been the case if the structure had been a terraced row of brick masonry residential units, which would have been much more sensitive to total and differential settlement.

Hu (1995) suggested that inadequate site investigation information might have been a cause of the East Brent 'failure' described previously. Hu (1995) stated that the claim that the stone columns were ineffective was made by Mc Kenna et al. (1976) on the basis of settlement measurements made at various locations along the length of the embankment, namely, the left-end, central (both unreinforced) and right end (partly reinforced) of the trial embankment (Figure 3.8a). According to the site investigation report by Mc Kenna (1968), twelve boreholes were sunk mainly in the region around the central section of the embankment and the general profile of the ground section shown adopted may have been based on a previous site investigation near the embankment site (Loc.No. 4765, Soil Mechanics), together with what would appear to be certain geological assumptions, particularly at the left hand end (Figure 3.8a). Following failure of the central section of the embankment, the slip surface was encountered immediately beneath the principal peat layer, corresponding to a shallower depth than that predicted by McKenna (1968) by using a conventional total stress analysis, which indicated that the thickness of the underlying principal peat layer and soft silty clay are likely to have had a significant influence on the settlements. The marked differences in settlement between the left and central sections (Figure 3.8a) seems to suggest that the thickness of the peat and soft clay layers beneath the left end of the bank are less than in the central and right section (Figure 3.8a) if the available site investigation information is interrogated in more detail. This speculation does appear to be supported by pore pressure profiles, which show that the depth of the maximum excess pore water pressures at the left end is around 3 to 4 m less in the centre and at the right end, (Figure 3.8a) providing some indication and evidence of the depth of the compressible layer(s) in that location. Therefore, the extent of the apparent ineffectiveness of the stone columns under the right end of the embankment (Figure 3.8a) deduced principally from the settlement observations, may be a chance occurrence associated with unforeseen ground conditions or insufficient information concerning the geotechnical properties of the soft clay layer in the ground, as eluded to earlier.

Bell (2004) described an example of a range of quality (control) issues, possibly arising from insufficient attention to construction detail, on what he considers should have been a routine vibro-stone column ground improvement project in a mixed (heterogeneous) soil profile in the UK. A typical soil profile for the site is shown in Figure 3.13. The intended objective of the vibro stone column treatment had been to provide adequate bearing capacity for a range of foundations and to control settlement, with the expectation that stone columns would have been constructed continuously to varying depths of penetration through the firm soils into the better underlying granular material below, dependent upon the stress depth influence of the foundations. Following in-situ testing (plate load testing on stone columns) which raised some concerns, an investigation of the installed vibro stone column ground treatment was carried out. This in-situ testing and the exposing, excavating and logging of several columns along their vertical axis demonstrated that many columns had been very poorly constructed. In the worst cases, site records suggested that treatment had been carried out to depths of up to 4 m. Whilst it is cited that there was no way of establishing whether the vibrator had penetrated to such depths, the exposed column A in Figure 3.13 would appear not to have been constructed satisfactorily beyond about 2.5 m in depth. Even within this depth range it was apparent that the stone column was discontinuous. The exposed column B appears to have been a better column, but it is clear that the nominal diameter was reducing (tapering) with depth. In the lower sections it is less than the nominal diameter typically expected for stone columns (i.e. between 450 and 600 mm) and attributed by Bell (2004) to the lack of building up the column in discrete lifts, each of which is compacted to predetermined limits such as target hydraulic pressure (or ammeter reading), dependent upon whether the vibroflot is hydraulically or electrically driven (section 1.2.1). Accurate records of the amount of stone consumed by each stone column, i.e. appropriate quality control procedures, would also have revealed the deficiencies of the construction during installation. A complete re-treatment of the site is understood to have eventually been carried out, and an example column C from this repeated work is also shown in Figure 3.13. A different standard of construction technique was clearly employed, although the work was conducted with identical vibroflot equipment to the original. The column is continuous, is constructed fully to the correct depth, and has a minimum diameter of about 400 mm, which was deemed adequate for the design. Some variation in diameter is to be expected for properly compacted columns, as strata or layers with different lateral resistances (stiffnesses) are encountered. In particular, this case history perhaps demonstrates the importance of having experienced operators, preferably with in cab monitoring on the installation rig so that real-time installation parameters can be monitored, particularly where any direct engineering supervision is lacking.

Other aspects of unsatisfactory performance described by Charles and Watts (2002), and from review of the literature, are defined as related to unrealistic expectations of what can be achieved by vibro ground improvement (treatment). Unsatisfactory performance of the treated ground can result from inadequacies in:

- assessment of required structural performance
- diagnosis of ground problem
- choice of treatment
- design of treatment
- execution of treatment
- appreciation for potential long-term deterioration of treated ground.

It is perhaps appropriate here to introduce the results of load tests undertaken on 2.74 m square concrete footings constructed over both untreated and treated Carse Clay soil at Grangemouth, Scotland, described by Thorburn (1975) in the First Géotechnique Symposium in Print on Ground Improvement, entitled - Ground treatment by deep compaction. The dry top-feed (displacement) technique had been inappropriately attempted in the soft Carse Clay deposits. Significant, albeit potentially temporary remoulding of the clay soil accompanied by significant contamination of the stone column material with clay, due to bore instability issues within the saturated soft clays, led to poorer performance under load than that experienced for the untreated ground (Figure 3.14), clearly demonstrating the inappropriateness of the dry top-feed technique to the prevailing ground conditions. The investigations were carried out prior to the advent of the dry bottom-feed technique (see Chapter 1 Section 1.2.3) and where common practice at the time would have been to adopt the wet top-feed technique (see

Chapter 1 Section 1.2.3) to maintain bore stability during stone column construction in these soil conditions.

	BULK DENSITY	ANGLE OF INTERNAL FRICTION
Coal	9.5 kN/m^3	35°
Made ground & Alluvium	19 kN/m^3	0°
Glacial till	20.5 kN/m^3	0°
Sandstone	20.5 kN/m^3	37°

Table 3.1: Gladstone Dock, Liverpool: Soil and material properties (after Johnson, 1994)

c kN/m^2	ϕ'	Sr	Ao	Ac m^2	Ar	FOS m^2	Ru
20	0	4	6.7	0.33	0.049	1.34	N/A
20	0	3	3.8	0.28	0.074	1.32	N/A
20	0	3	4.1	0.30	0.073	1.32	N/A
20	0	3	4.3	0.33	0.077	1.34	N/A
20	0	2.5	3.3	0.28	0.085	1.30	N/A
20	0	2.5	3.3	0.30	0.091	1.33	N/A
20	0	2.5	3.5	0.33	0.094	1.35	N/A
0	28.5	2.5	6.7	0.28	0.042	1.50	0.47

Table 3.2: Gladstone Dock, Liverpool: Output from Numerical analysis (after Johnson, 1994)

Site	**Soil Type**	**W [%]**	**W_I [%]**	**W_P [%]**	**PI [%]**	**Clay [%]**	**Silt [%]**	**Sand [%]**	**S_t [-]**	**C_v [m^2/yr]**
Kinrara	Mining Slime	60	60	30	30	40	45	15	2-3	4.0
Kebun	Marine Clay	100	100	40	60	50	45	5	4-5	1.0

Table 3.3: Mining slime and marine clay properties, Shah Alam Expressway, Western Malaysia (after Raju, 1997). S_t = undrained shear strength.

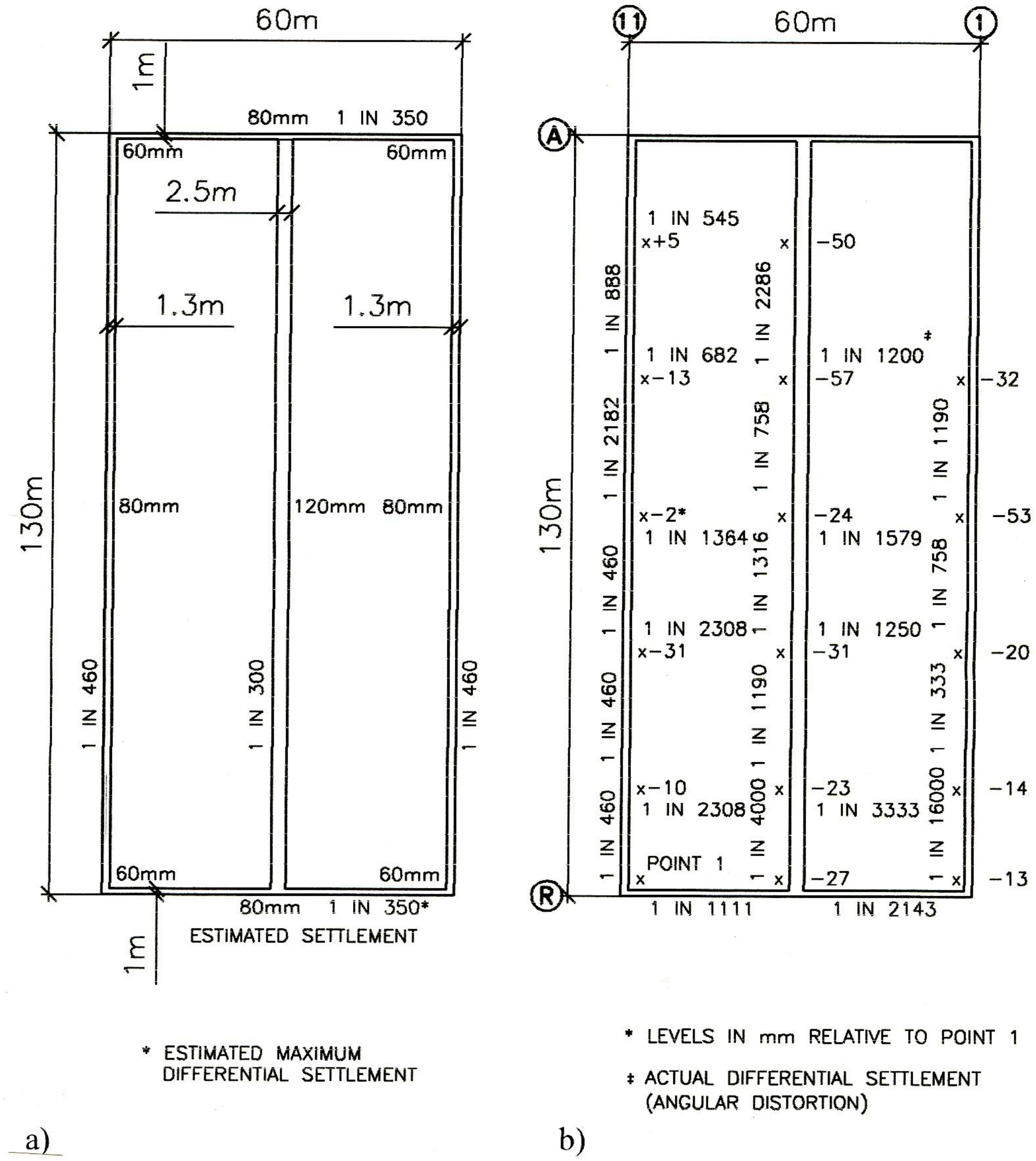
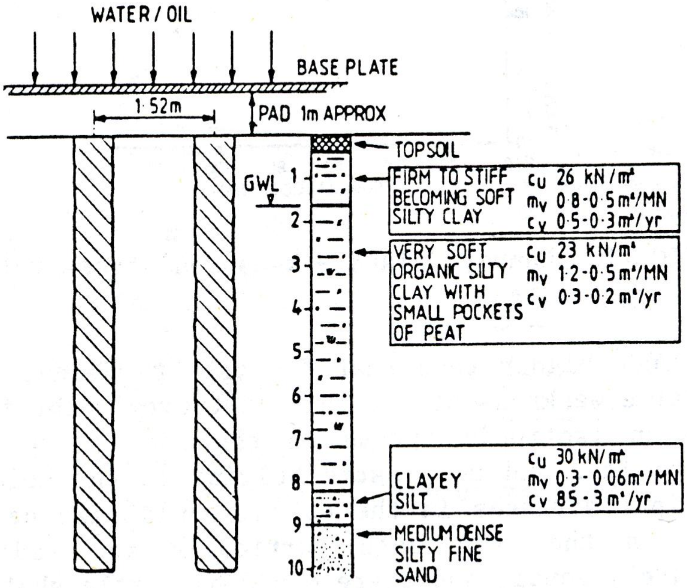
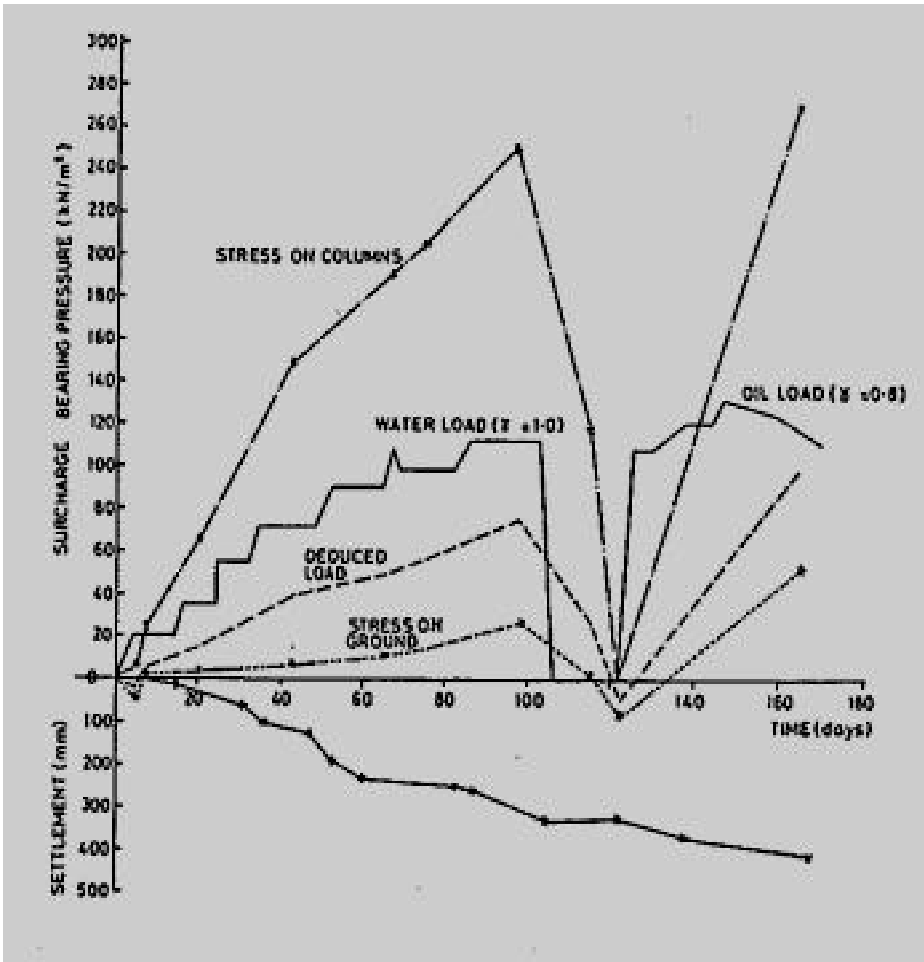


Figure 3.1: (a) Estimated differential settlements; (b) Actual differential settlements at Immingham, UK (after Smallridge and Johnson, 1990). x = relative differences in level.



(a)



(b)

Figure 3.2: Canvey Island (a) Soil properties. (b) Measured stresses and settlement (after Greenwood, 1991).

(a) (b)

(c)

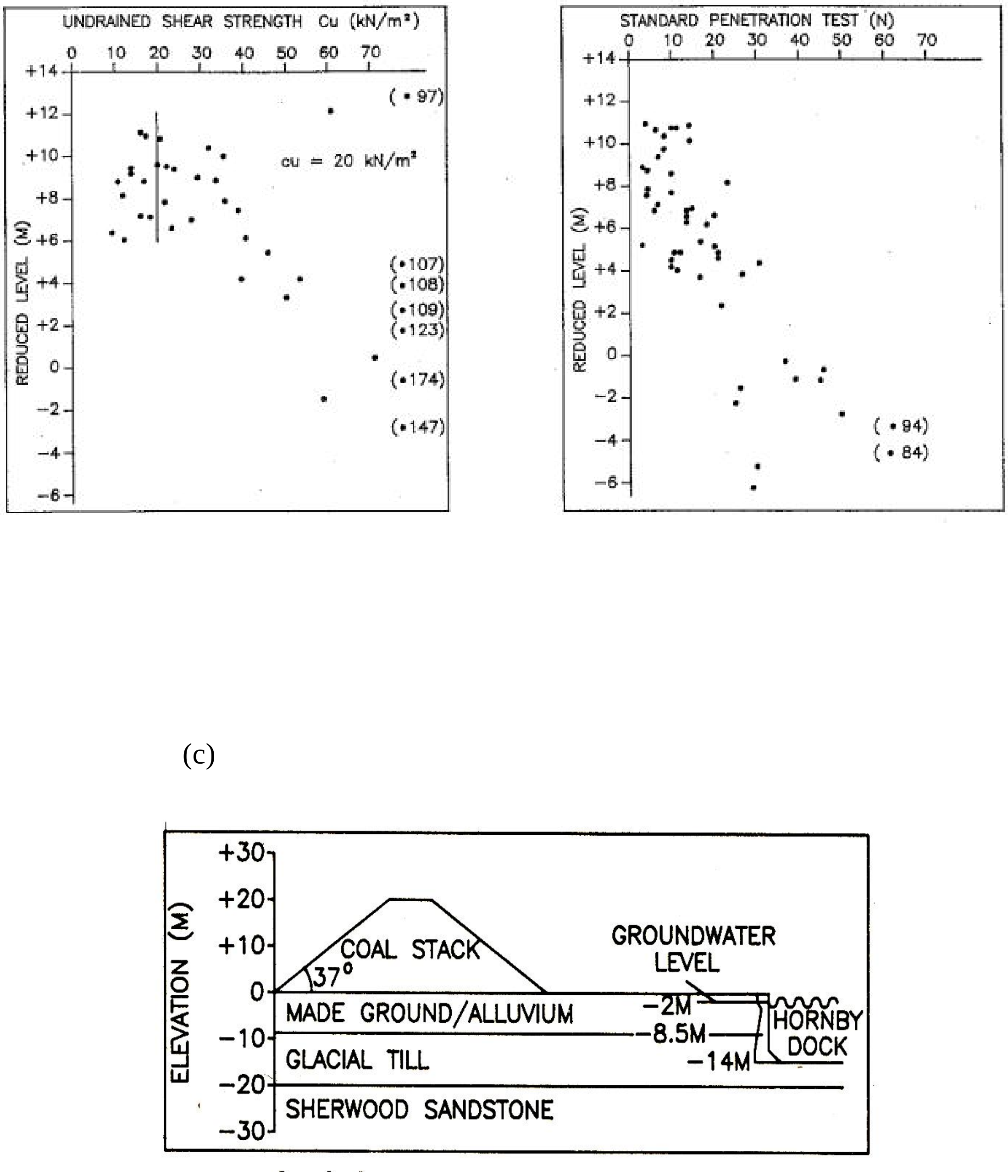
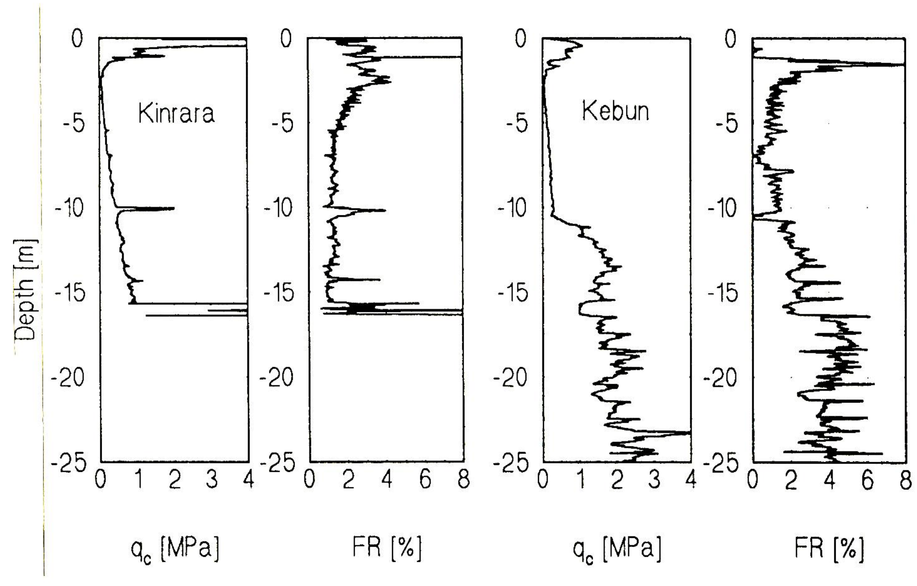
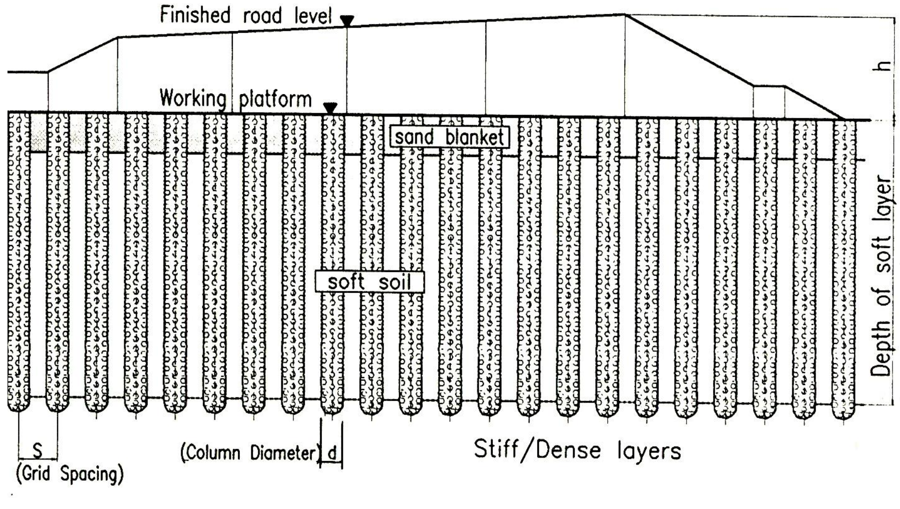


Figure 3.3: (a) Undrained shear strength (b) SPT N profile (c) Schematic section showing coal stack 3 and underlying geology – Gladstone Dock, Liverpool (after Johnson, 1994).



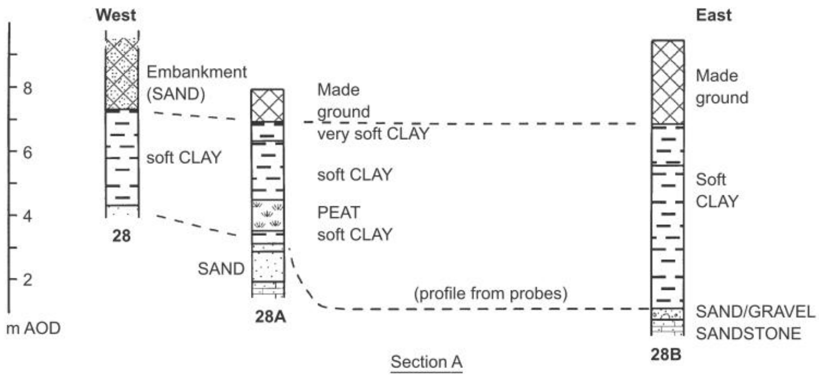
(a)



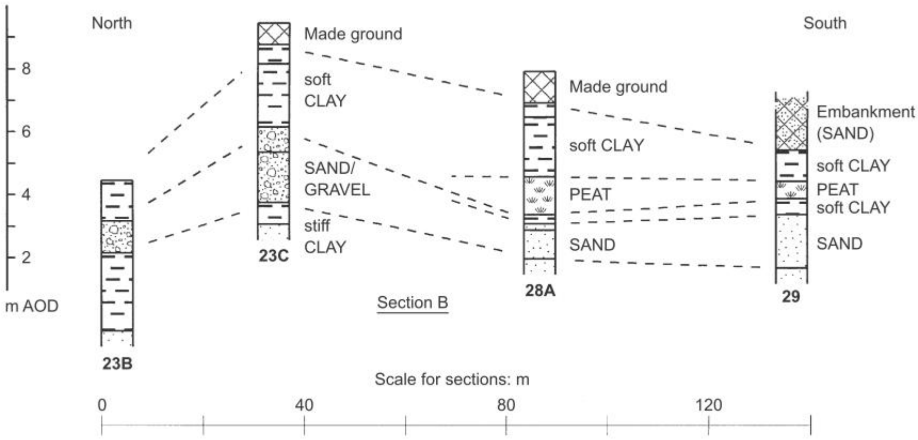
(b)

Figure 3.4: (a) CPT Profiles and (b) Layout of stone columns, Shah Alam Expressway, Western Malaysia (after Raju, 1997).

a)



b)



c)

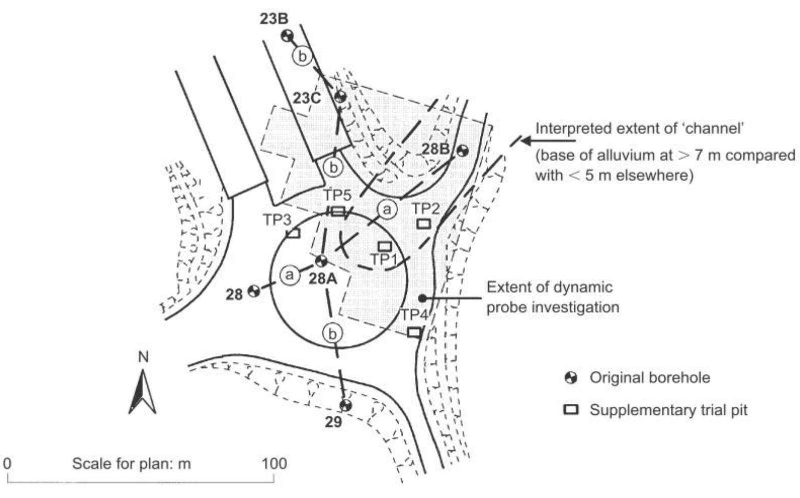
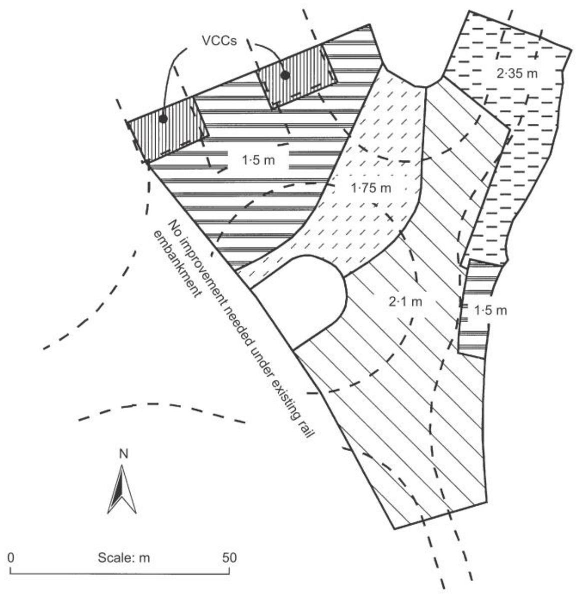


Figure 3.5: Bristol, St. Philips Causeway (a) and (b) Geological sections, (c) Location of site investigation (after Cooper and Rose, 1999).

(d)



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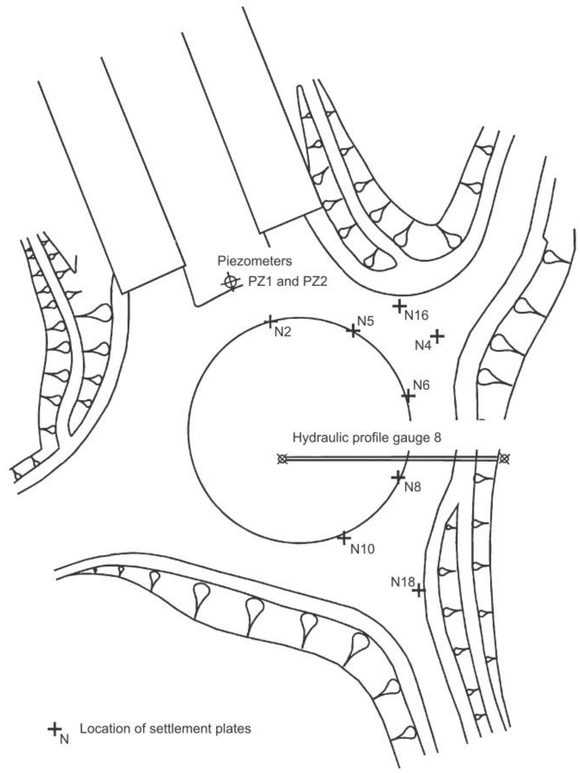
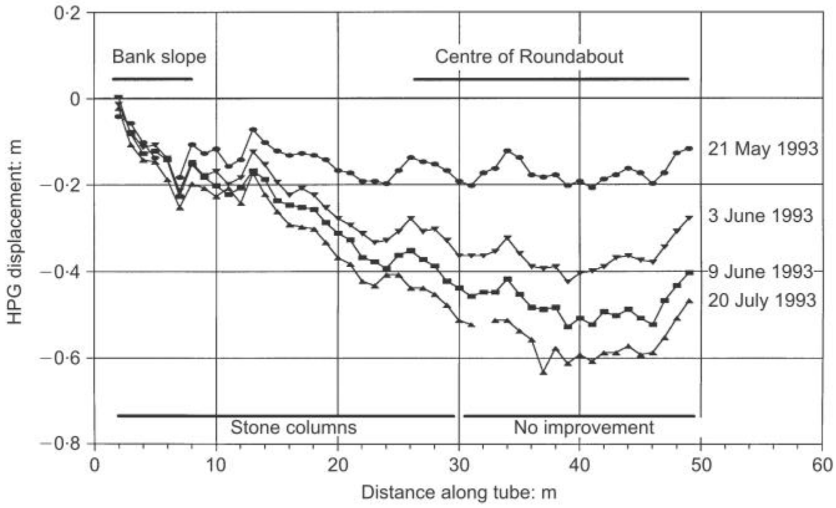
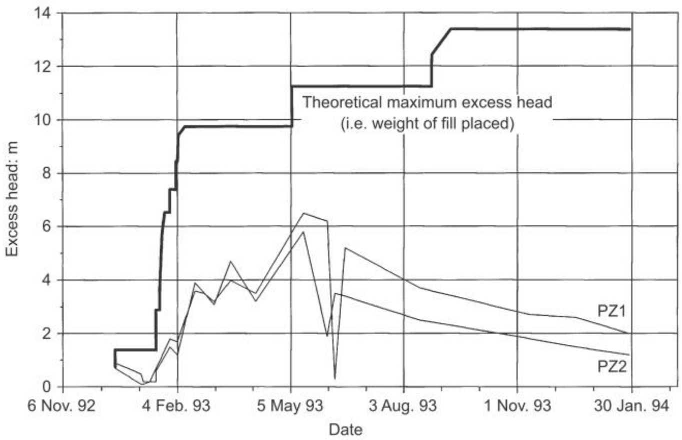


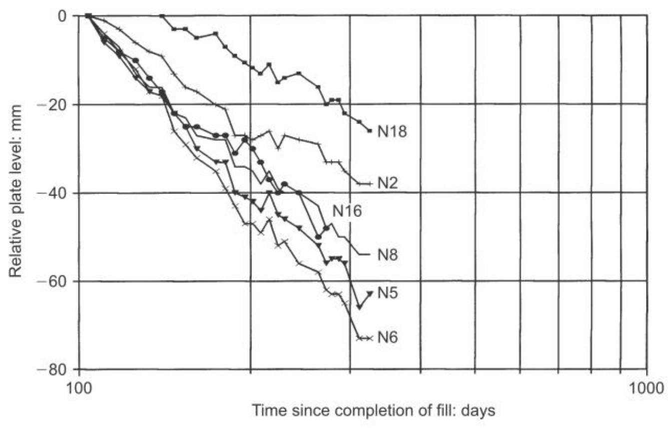
Figure 3.5: Bristol, St, Philips Causeway, (d) Stone column layout and (e) Instrumentation locations (after Cooper and Rose, 1999).



(f)

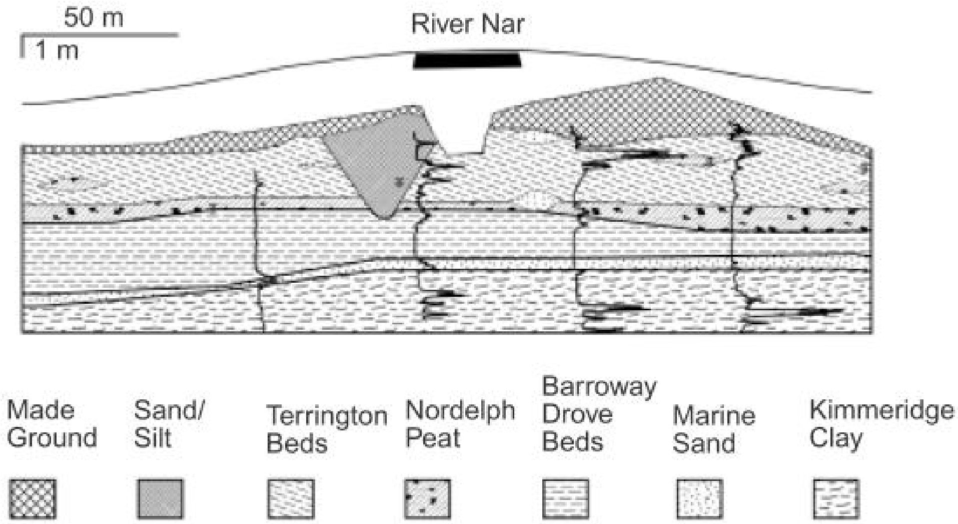


(g)

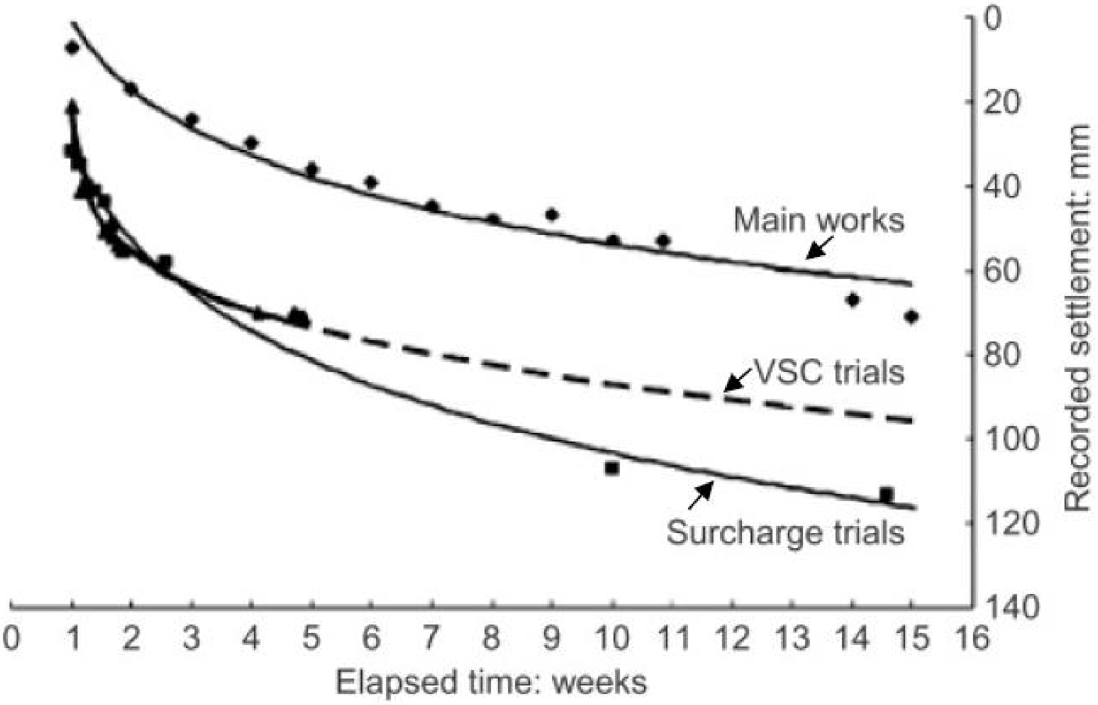


(h)

Figure 3.5: Bristol, St. Philips Causeway (f) Hydraulic Profile Gauge 8 displacements, (g) Excess pore pressures, (h) Settlement of monitoring plates (after Cooper and Rose, 1999).



(a)



(b)

Figure 3.6: Norfolk, Kings Lynn relief road (a) Geological ground model and (b) Embankment settlement data (vibro stone columns (VSC)), after Serridge and Synac, 2007.

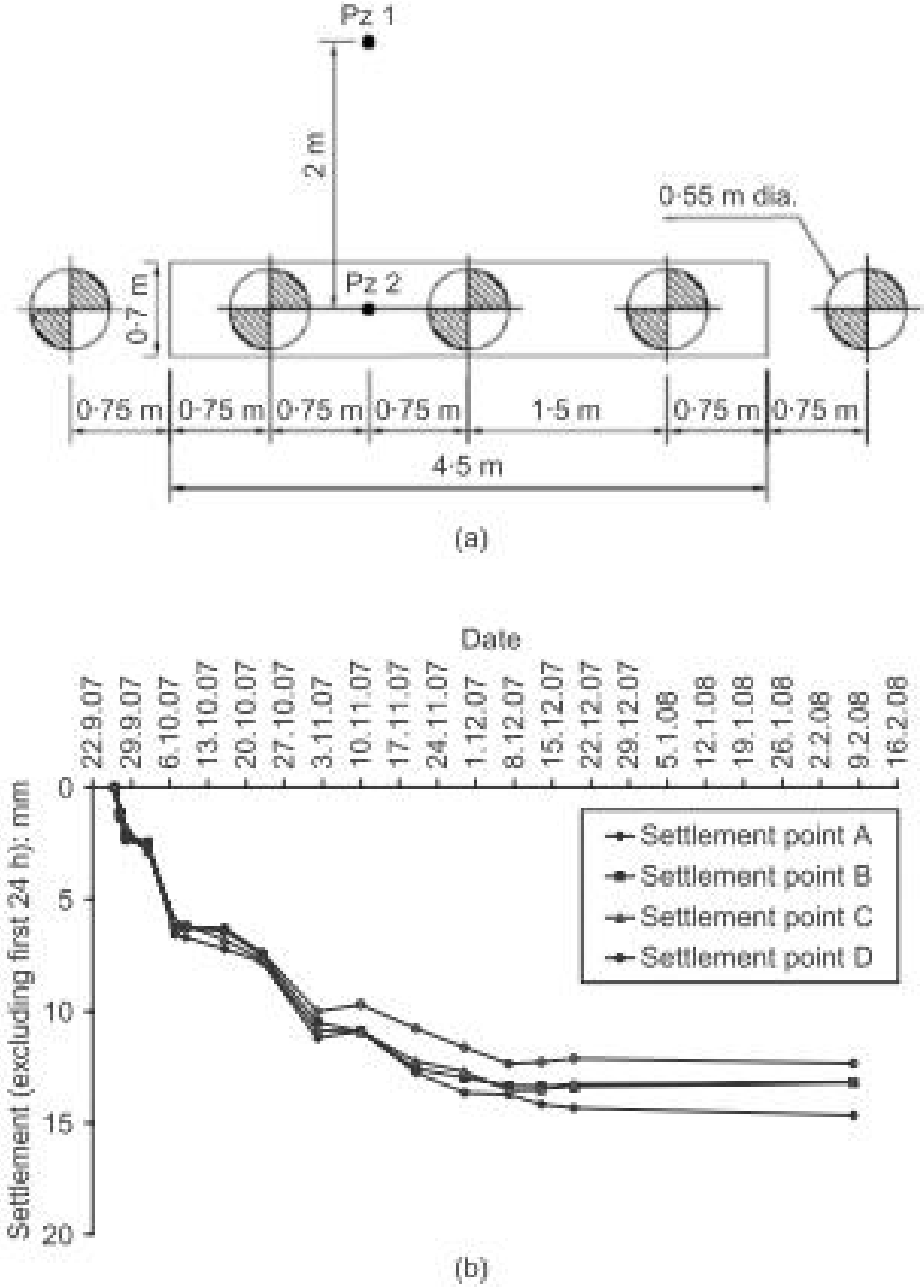


Figure 3.7: (a) Trial strip footing (P_Z = Piezometer); (b) Settlement-time behaviour of trial strip footing over stone columns in Carse Clay (after Egan et al., 2008).

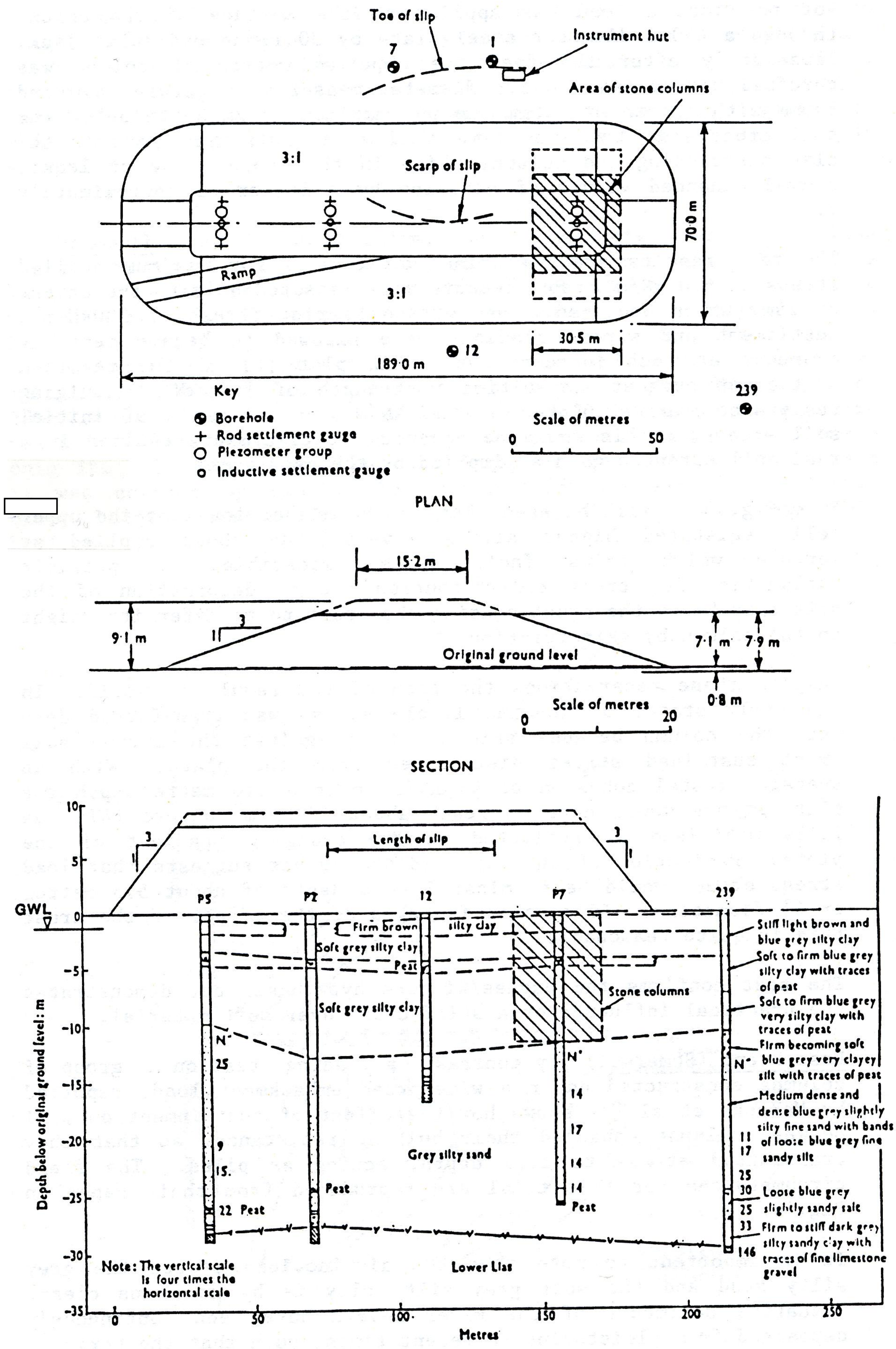
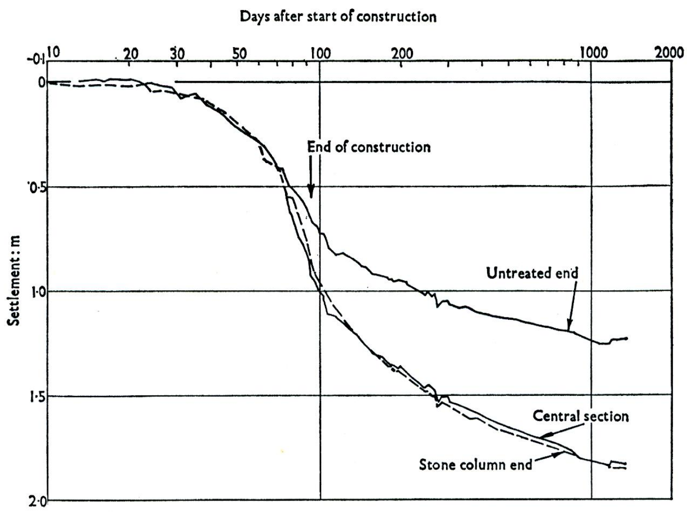


Figure 3.8: East Brent trial embankment (a) Field circumstances (after Mc Kenna et al., 1976).



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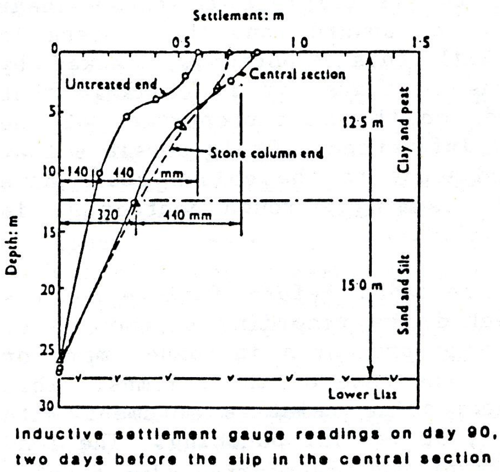

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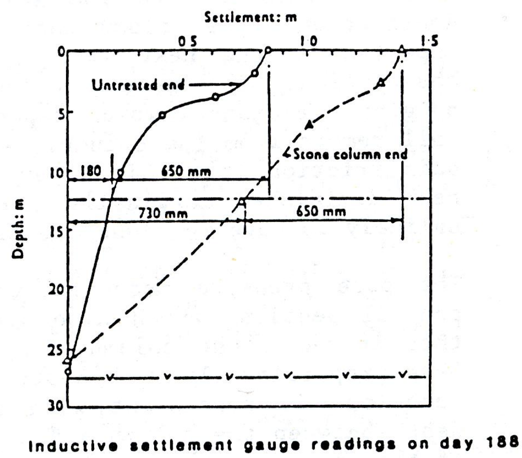

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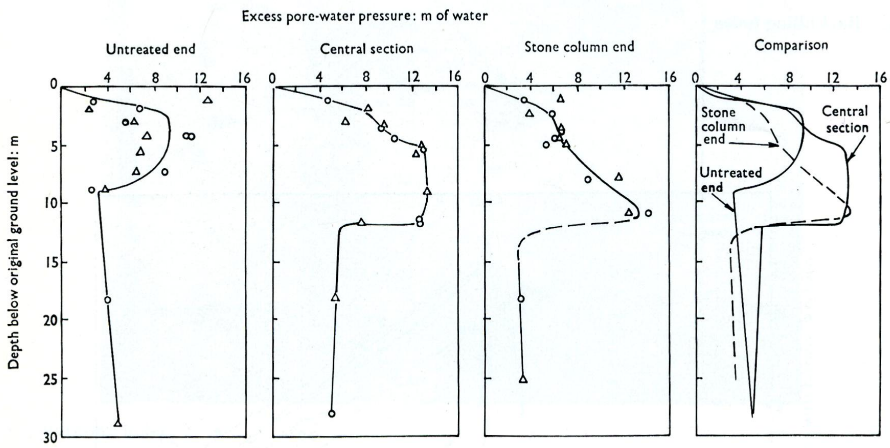
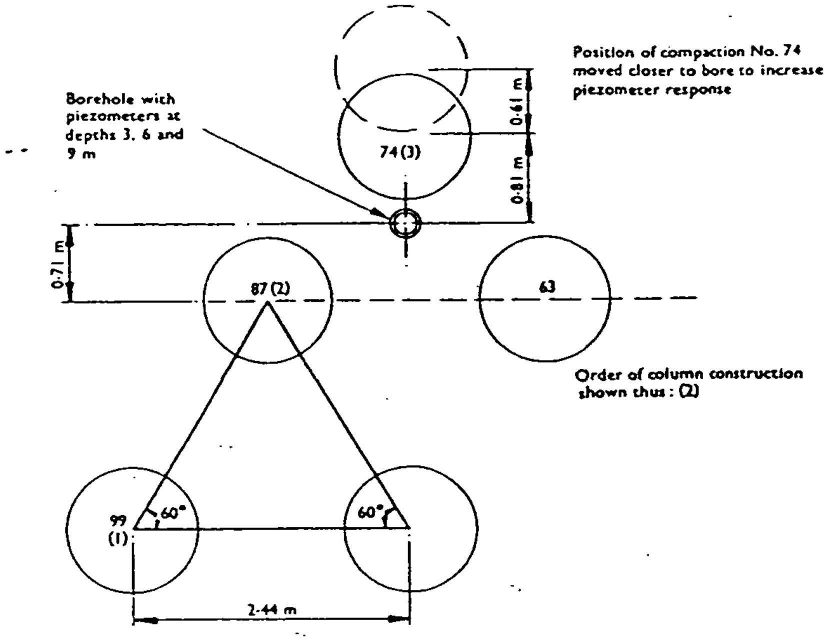


(e)

Figure 3.8: East Brent trial embankment (b) Settlement of the three centre-line rod settlement gauges against time, (c) Inductive settlement gauge readings on day 90, two days before slip, (d) Inductive settlement gauge readings on day 188, East Brent trial embankment, (e) Pore pressures on day of slip (after Mc Kenna et al., 1976).



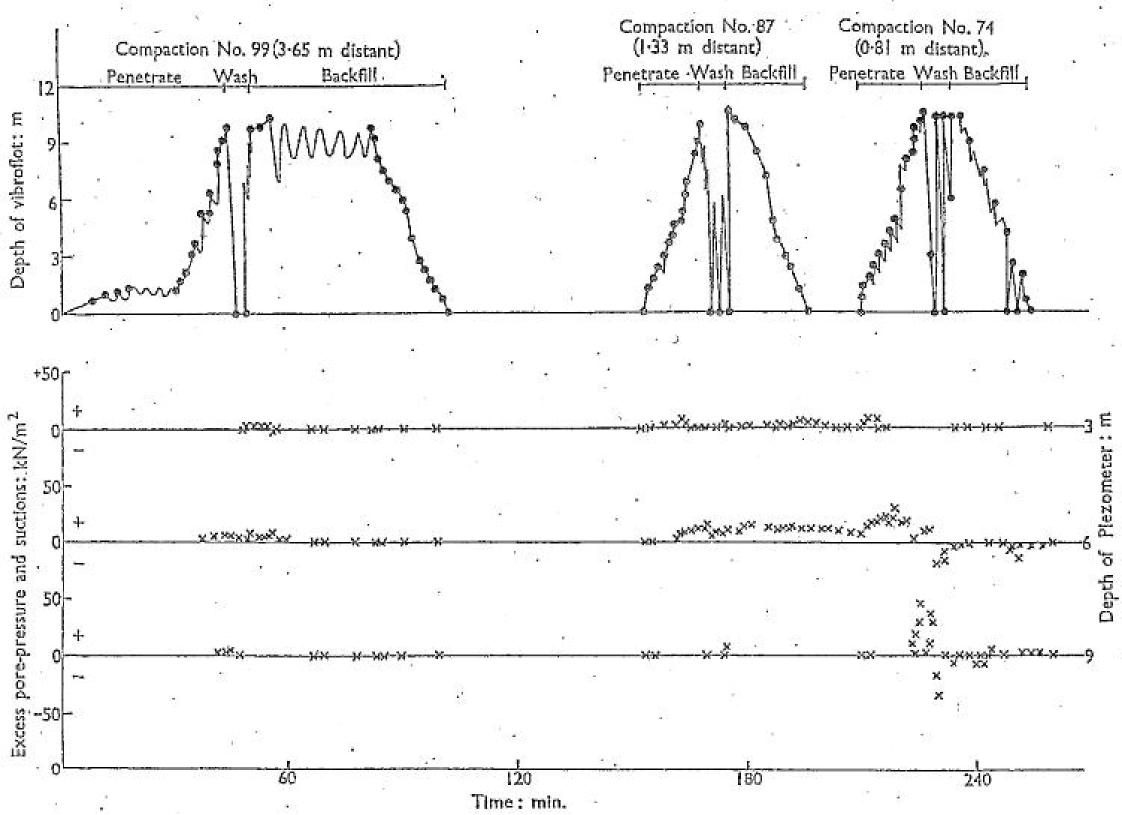
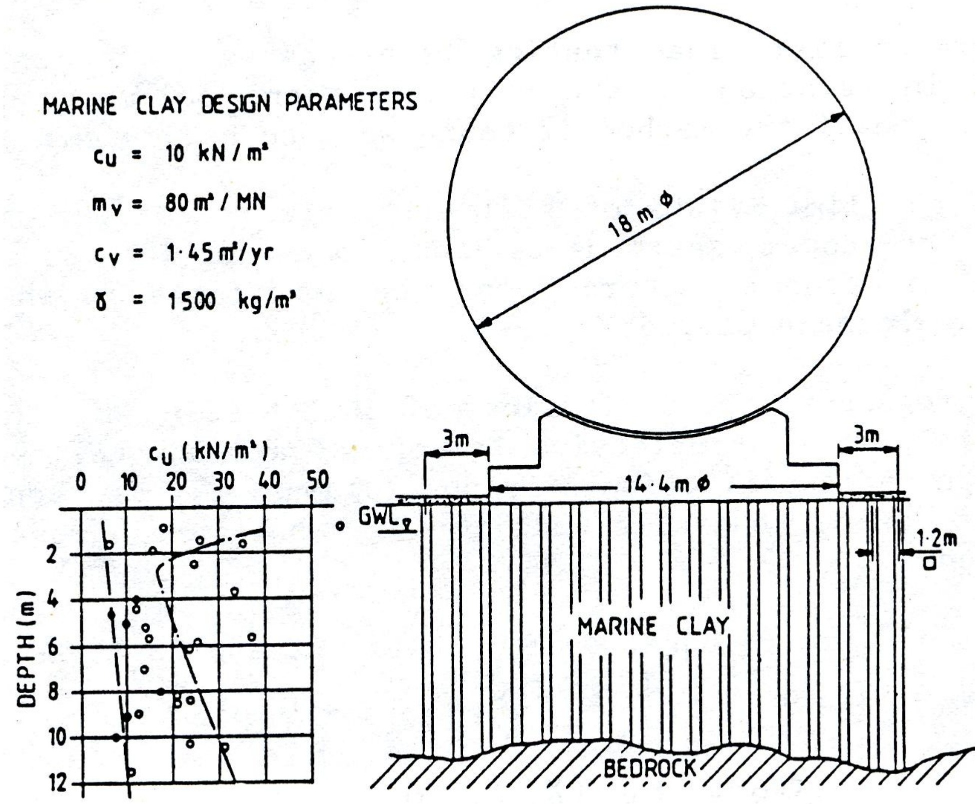


Figure 3.9: East Brent trial embankment. Plan of instrumentation and piezometric observations during stone column construction (after Greenwood, 1976b).

Figure 3.10: East Brent trial Embankment – Magnified image of exposed stone column (after Greenwood, 1976b).

a)



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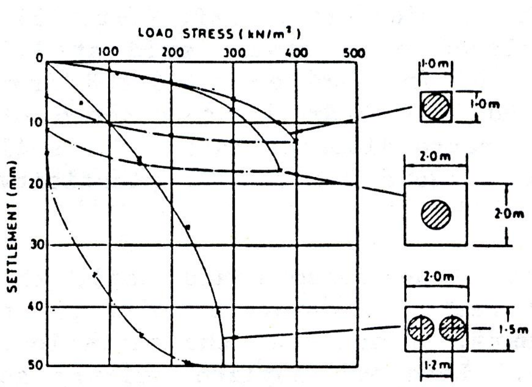

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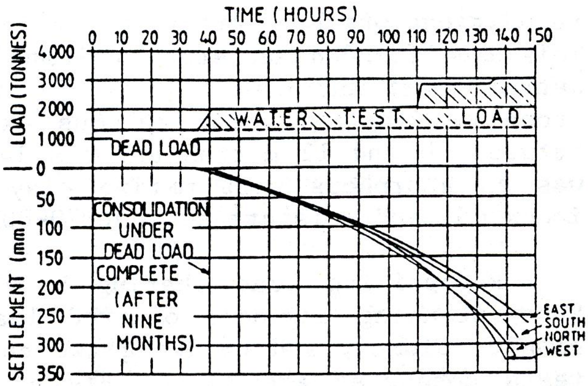
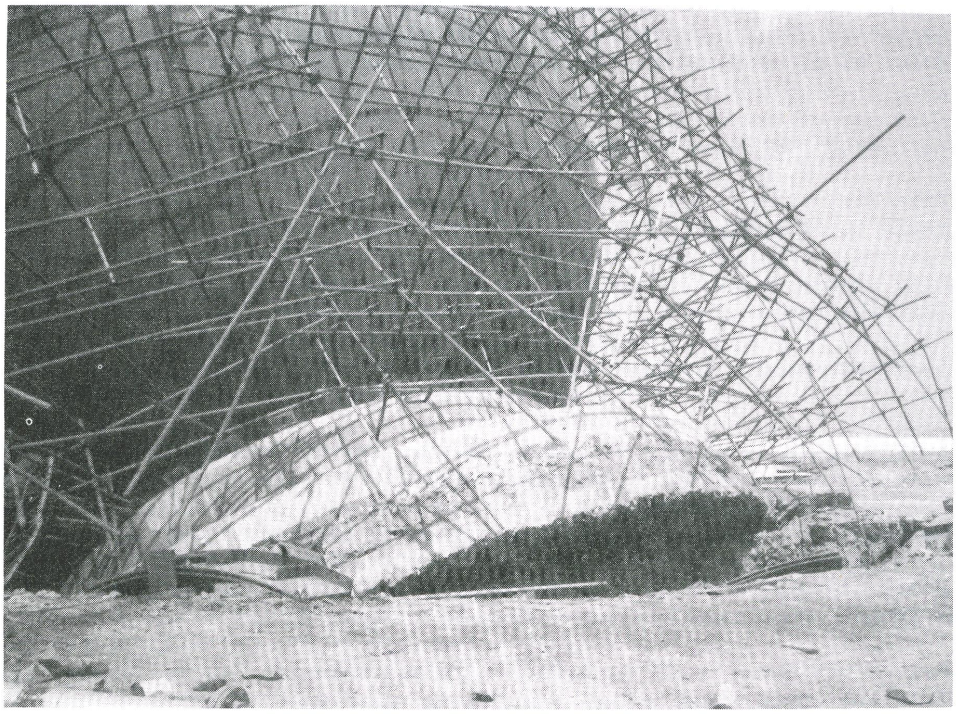


Figure 3.11: India, LNG sphere. (a) Field circumstances (b) Small scale load test result and (c) full scale water testing loading records (after Greenwood, 1991).



d)

e)

Figure 3.11: India, LNG sphere (d) and (e) Progressive foundation failure (after Greenwood, 1991).

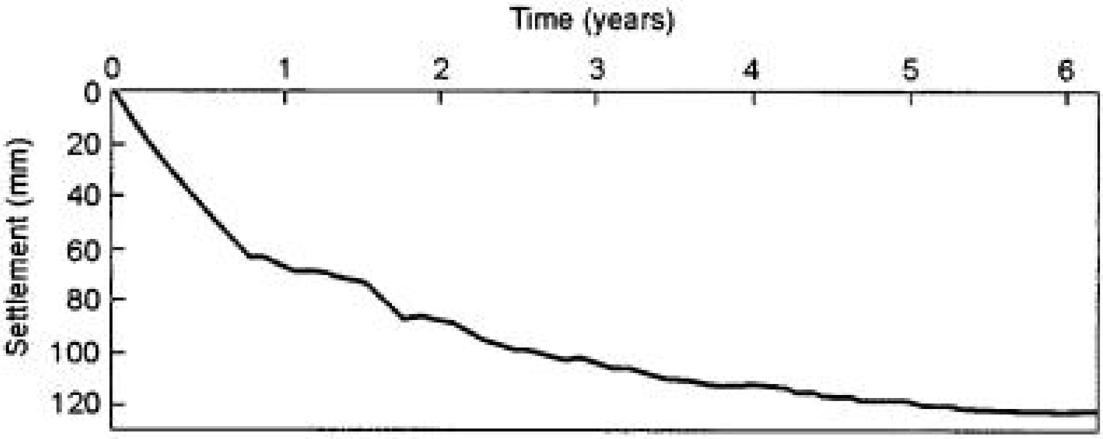


Figure 3.12: Settlement of factory on vibro stone columns (after Wilde and Crook, 1992)

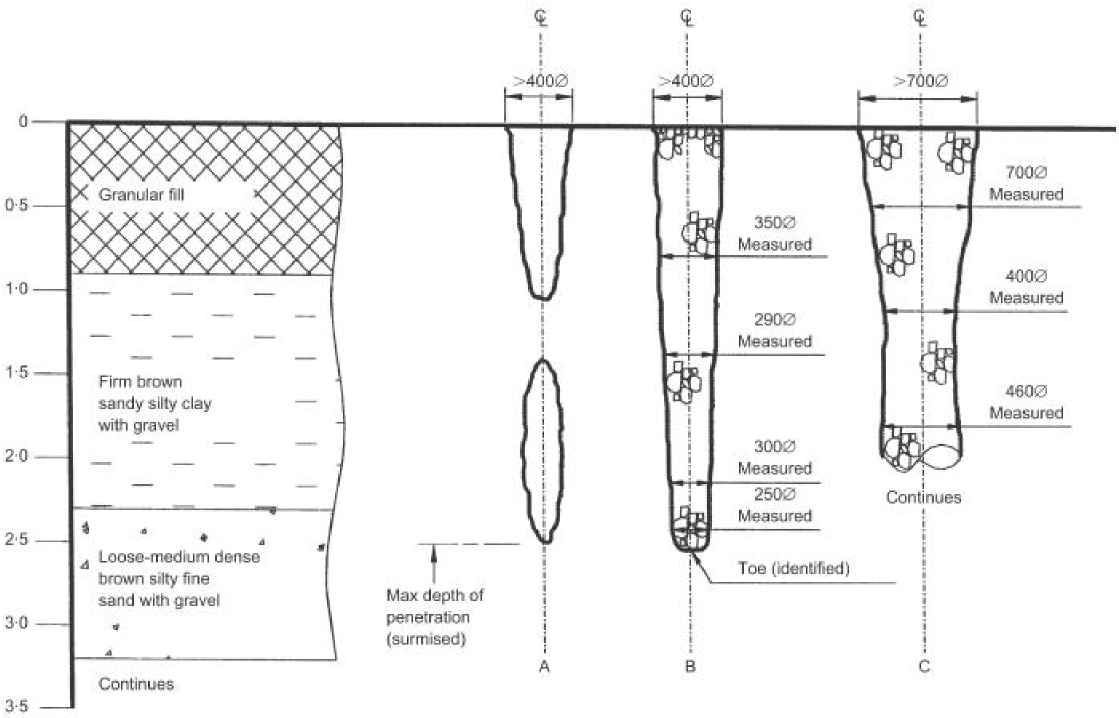


Figure 3.13: Poor quality control during stone column installation (after Bell, 2004).

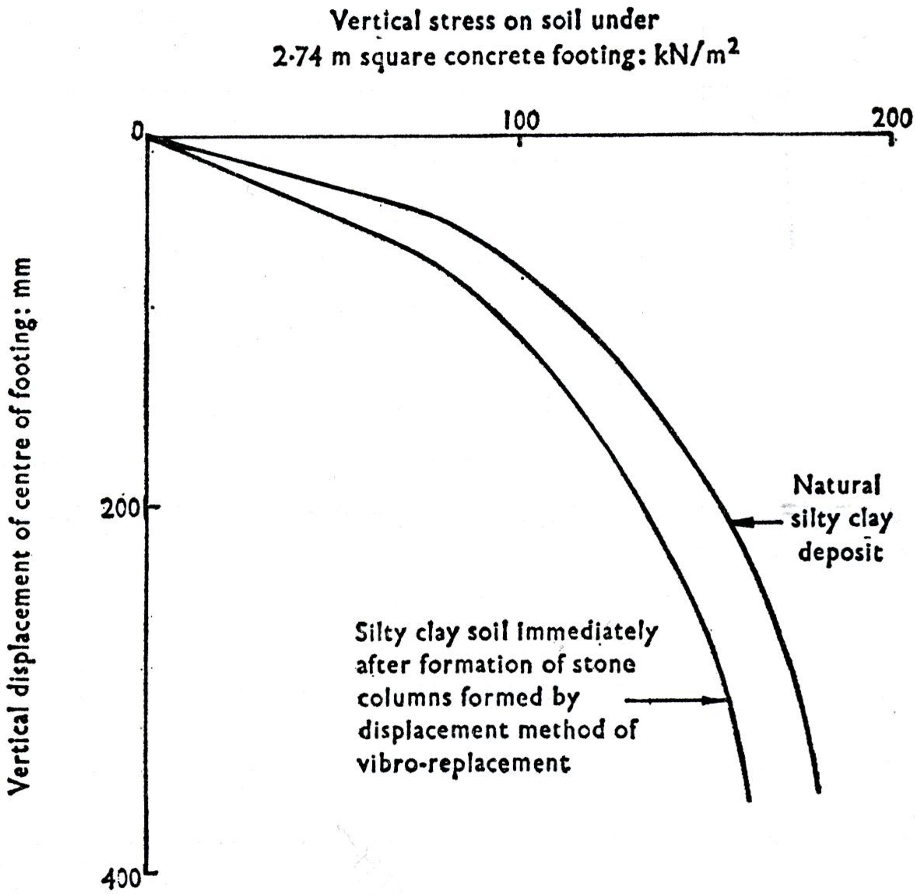


Figure 3.14: Result of plate load test on untreated and treated soft natural clay soil, Grangemouth District (after Thorburn, 1975).

Chapter 4 Research programme

4.1 Introduction

Increasing economic and demographic pressures are forcing many new building developments to take place on land that previously was considered marginal, on low-lying estuarine or coastal land. On such sites ground conditions are typically soft and of poor quality for foundation construction and damaging total, and more significantly-differential movements, may result from compression of the ground due to building loads or even in extreme cases, bearing (capacity) failure. Attempts have been made in the past to construct shallow, narrow footings (foundations) on deep deposits of soft compressible soils, in some cases attempting to utilise extremely thin surface soil 'crusts' above these very weak soils, without adoption of ground improvement (or piling, prior to the development of modern ground improvement techniques). This has inevitably resulted in poor structural performance as evidenced in Figure 4.1(a) which shows visible tilt in late 1930's housing constructed in Southport, UK, over the deep Downholland Silt (clayey silt) Formation, without adoption of deep foundations. Figure 4.1(b) also shows a similar house type in the same locality which has suffered the same pattern of differential settlement (tilt), juxtaposed with a modern housing development on deep foundation piles (free from visible differential settlement). These comments are significant in the context of the UK as much future development (particularly low-rise housing, together with light industrial and retail units), is anticipated over soft alluvial ground such as the Thames Eastern Corridor, the Severn Estuary and the Forth Estuary.

Vibro stone column ground improvement techniques are being increasingly considered for the development of marginal land with deep soft soil deposits, to increase shear strength (and therefore bearing capacity) and reduce the compressibility of soft natural soil deposits, but where some settlement can be tolerated, and can provide an economic and sustainable alternative foundation solution to the traditional and potentially cost prohibitive approach of (deep foundation) piling. However, there is little if any research data available for this application, particularly for low-rise, light-weight structures supported by shallow, narrow footings.

4.2 Design of field trials

4.2.1 Introduction

Historically, the wet top-feed stone column technique has been used in soft clay soils, particularly below ground water level (as previously discussed), but environmental concerns regarding disposal of effluent, advances in vibro technology and economic constraints, have resulted in the wet top-feed technique being largely superceded by the dry bottom-feed technique. Despite this transition from one technique to another and the increasingly more widespread use of vibro stone columns in soft soils, stone column construction using the dry bottom-feed method (and their subsequent performance), in deep soft soil deposits is not well understood and has not been investigated in any detail at field scale. The limited published field data relating to vibro stone columns in soft clay soils have concentrated on the wet top-feed technique. There are no documented large scale field trials using the dry bottom-feed technique in deep soft (sensitive) clay profiles, apart from field trials described by Serridge (2001), who investigated more widespread partial depth stone column group behaviour in deep soft ground (Bothkennar Clay) supporting a raft foundation, and more recently work by Castro (2007), again focussing on more widespread loads. Whilst there is some limited case history information from actual ground improvement contracts, for example – Smallridge and Johnson (1990), Johnson (1994), Cooper and Rose (1999), Serridge and Synac (2007), as discussed in Chapter 3, these case histories again focus on widespread loads beneath ground bearing floor slabs or embankments over soft ground, with no data relating to narrow, shallow footings over soft ground, apart from some relatively recent settlement data provided by Egan et al. (2008) (Ch.3,Section 3.2). It is clear, on the basis that stone columns rely, at least in part, for their support on the passive resistance afforded by the surrounding soil, that the design and successful application of vibro stone column ground improvement in soft ground depends on the use of appropriate engineering parameters for the surrounding soil, as well as the stone column material and also an awareness of how the soil (and various mechanical properties) may be modified during the intallation process and which cannot be satisfactorily achieved in laboratory investigations and numerical simulation of the field situation. This research therefore evaluates an instrumented field trial at the Bothkennar soft clay research site in Scotland, where partial depth (partially penetrating) vibro stone columns were installed using the dry bottom-feed (vibrodisplacement) method beneath shallow, narrow trial footings in an attempt to address this lack of knowledge and understanding.

Bothkennar lies in the Forth Valley of Central Scotland, approximately mid-way between Edinburgh and Glasgow, and borders on the southern bank (floodplain) of the River Forth Estuary on former intertidal mudflats, approximately 1 km south of the Kincardine bridge (Figure 4.2). The site selected for the field trials at Bothkennar was a facility for large or full scale soft clay research, with the site having been used extensively for research into *in-situ* testing and also 'undisturbed' sampling techniques, Hight et al.,1992, together with the full-scale performance of piles, shallow foundations on untreated ground (Jardine et al., 1995), and more widespread load on partial depth vibro stone columns (Serridge, 2001). The site was owned and managed by the UK government through the Engineering and Physical Science Research Council (EPSRC) and subsequent to the field trials was to be taken over by the RSPB (Scotland) to be used as a wildlife sanctuary. The principal advantages of the Bothkennar soft clay research site which influenced its selection were as follows:

- Access was good, particularly for heavy plant.
- The site was protected from flooding.
- The soil profile was fairly uniform and relatively 'uncomplicated'.
- Comprehensive 'state-of-the-art' ground investigation and geotechnical data existed for the site. A wealth of research has been previously undertaken and was detailed in an Institution of Civil Engineers Géotechnique Symposium-in-Print (Vol. 42, No.2: 1992).
- The Bothkennar clay was expected to exhibit typical (normally consolidated) natural clay features such as anisotropy and inter-particle bonding; viscocity.
- The soft clay profile was devoid of any peat layers.
- There was no history of past or current mining activity beneath the site.
- It had a purpose built single-storey building on site with office, field laboratory and communication links.
- It was sufficiently isolated to obviate any risk of vandalism (particularly to instrumentation).

The soil profile at the location of the field trials associated with this research at Bothkennar comprised a 'recent' (geologically), normally to lightly over-consolidated clay profile with a thin surface desiccated 'crust', up to around 1.5 m thick, underlain by a deep deposit of soft clay (Bothkennar Clay). Good geotechnical characterisation is essential for any field trials and as intimated above, comprehensive geotechnical data existed for the Bothkennar site. The deposit comprises approximately 18 m of soft becoming soft to firm grey to black micaceous clayey silt, underlain by about 3 m of dense well-graded sand, gravel and cobbles. Below about 14 m the soft clay becomes considerably more laminated and may be associated with a change in depositional environment. The clay also becomes firmer below this depth. The surface 'crust' which contains shell fragments in the lower parts, can be identified by its distinctive reddish-brown colour and higher undrained shear strength, compared to the underlying grey-black clayey silt. Key profiles associated with these data are shown in Figure 4.3. It is evident, however, that there is limited published information on the geotechnical properties of the desiccated 'crust' at Bothkennar, with most attention having focussed on the underlying soft Bothkennar Clay. The 'crust' was anticipated to play an important role in the objectives of the field (research) trials and hence a small programme of ground investigation was therefore proposed and undertaken to address these shortcomings. This incorporated the excavation of a shallow trial pit with a backhoe excavator, the carrying out of in-situ hand shear vane tests, and also recovery of 'undisturbed' samples for subsequent laboratory one-dimensional (1-D) consolidation testing and determination of moisture (water) content and plasticity indices, in order to address the requirements for accurate soil description, undrained shear strength and consolidation (compressibility) parameters for bearing capacity and settlement predictions within this soil layer.

Soil conditions at Bothkennar historically might have been considered marginal for dry bottom-feed stone column treatment, particularly for low-rise structural applications, due to low undrained shear strength and high compressibility characteristics, but more significantly - soil sensitivity[1] (see Figure 4.4).

Footnote: [1]Soil Sensitivity (S) is usually defined as the ratio of the undisturbed shear strength to the remoulded shear strength.

This research has therefore provided the opportunity to assess any implications or limitations associated with use of the dry bottom-feed technique in a deep soft sensitive clay soil. The average reported field vane strengths vary from about 20 kN/m^2 at 2 m depth, i.e. below the crust, rising steadily with depth to 60 kN/m^2 (Figure 4.3), which would be acceptable for column construction using the dry bottom-feed method. However, the sensitivity of the soft clay, whilst varying with depth is reported in the range 5-10 for field vane measurements (Nash et al.,1992a) and this implies the possibility of significant reductions in strength due to soil disturbance and shearing during the installation of stone columns. In addition, the depth of the soft clay deposits (typically up to 20 m) at the Bothkennar site dictate that only partial depth treatment was practical.

Opportunity was available at the Bothkennar soft clay research site for field trial monitoring over a period of around 12 months. As highlighted previously, whilst larger column groups have received some attention, small groups or single rows of columns beneath narrow footings have received limited if any attention and were therefore investigated in the field trials associated with this research. There was opportunity to study the installation of stone columns under difficult conditions, particularly ground responses, and to closely examine the subsequent performance of the treated ground under foundation loading, i.e. shallow, narrow footings constructed (both within and at the base of the 'crust') over varying depths and spacings of stone columns.

To be of real value the stone column field trials were designed and tested under realistic loading conditions. Appropriate engineering parameters, which are discussed below, were also required in the design procedure. Apart from satisfactory geotechnical characterisation of the soils for vibro stone column design (Serridge, 2008), significant parameters identified from examination of previous work for stone column design in soft soils (see Chapter 2), relate to the undrained shear strength and compressibility of the in-situ (host) soil, angle of internal friction of the stone column material (aggregate), stone column diameter, compressibility (stiffness) of the compacted column, bulk density characteristics of both column and soil and at rest earth pressure coefficients (K_o). Some authors, e.g. Priebe, 1995, account for a certain change in the stress state during installation, by using higher values of the earth pressure at rest than for the natural soil. Arguably in the context of the Bothkennar field trials, as previously discussed, soil sensitivity is also critical. Generally it was assumed for design predictions that the surrounding clay soil maintains its original strength and stiffness parameters whilst the improvement is dominated by the highly compacted stone column material, i.e their 'reinforcing' role. This is supported to some extent by the literature review undertaken in Chapter 2, where it is evident that the design of vibro stone columns in soft clay soils beneath footings does not take into account the improvement of the surrounding soil apart from a certain increase in the stress state typically using earth pressure at rest assumptions. Any improvement of the in-situ soil would therefore act as a hidden safety factor in the system.

4.2.2 Soft clay undrained shear strength

It was suggested, from consultation with Specialist Ground Improvement Contractors in the UK, that the lower bound value of undrained shear strength which exists within a clay soil profile within the stress depth influence of the foundation (and notably where the thickness of such a soil layer is greater than the anticipated column diameter), should be adopted in the vibro stone column design and which should incorporate a minimum factor of safety of 1.5 against column bulging, which is common industry practice and related to the undrained shear strength of the surrounding soil. For the stone column-soil composite in clay soils, allowable bearing capacity is routinely based on undrained shear strength, although a factor of safety of 3 is commonly adopted in order to ensure that the loading is on the sensibly linear component of the stress-strain curve for the soil and that settlements are maintained within normally accepted tolerable limits. With time, the clays will strengthen under the higher loadings as any excess pore water pressures dissipate (which will be facilitated by the presence of stone columns). Hence the worst case is at the time of initial loading and for gradually applied or static loading, bearing capacity should progressively increase. Field vane tests and/or In-situ hand shear vane tests typically provide the most direct means of obtaining the undrained shear strength, although recourse is often made to the results of laboratory tests on recovered 'undisturbed' samples. On the basis of these comments reference was made to the available geotechnical data for the site (in particular values obtained from the average of recorded peak (shear strength) values in the upper soft Bothkennar Clay soil profile below the surface crust (see Figure 4.3)) and supplemented, prior to commencement of the trials, by hand shear vane tests undertaken within the anticipated zone of stone column bulging (below founding depth). The hand shear vane tests were undertaken from the side of the pit using a pilcon hand vane tester with extension rods to obtain the in-situ undrained shear strength data.

4.2.3 Stone column aggregate

A maximum 40 mm (range 10-40 mm) aggregate particle size is typically employed for the dry bottom-feed system in order to avoid blockages in the 150-175 mm diameter stone delivery tube attached to the side of the bottom feed vibroflot (see Figure 1.6b and 1.9a and b). Based upon the authors knowledge and from discussions with Specialist Contractors, rounded or sub-rounded aggregates of comparatively uniform grading, generally 20/40 mm sizes, pass most easily through the stone delivery tube, reducing concerns over the risk of arching and therefore blockage of aggregate in the tube occurring. The pre-requisites for the 40 mm aggregate used in the field trial(s) were as follows:

- Must be locally available to minimise environmental impact of aggregate transport by road and also cost. Unfortunately, there were no local sources of recycled aggregate which met the grading requirements for the dry bottom-feed system, in part due to the remoteness of the site from urban areas, necessitating use of locally sourced primary aggregate and which proved more sustainable.

- To resist the impact forces of the vibroflot during stone column construction and remain stable in the soil and groundwater conditions present at the site, - at the time of the field trials the aggregate was required to be hard and inert, with an Aggregate Crushing Value (ACV) of less than 30% and a minimum 10% fines value (soaked) of 100 kN, and which are in line with the requirements of BRE BR391 Specifying stone columns (2000). These physical property parameters have been superceded more recently by the Los Angeles Abrasion (LAA) value (see BS EN 1097-2 (1998)).

- Have an (acceptable) particle shape which was rounded, angular or irregular. Flaky or elongated particles would be unacceptable due to risk of blockages and fragmentation of the aggregate under the impact loads of the vibroflot during stone column construction.

- The aggregate was required to be free from impurities, i.e. no organic materials and with a fines (clay/silt/dust) content not exceeding 5%.

- Have a minimum angle of shearing resistance of 40˚ to ensure good mechanical interlock.

At the time of the trials determination of the above physical properties was established by recourse to the procedures outlined in BS 812 (1990). In terms of the angle of shearing resistance of the stone column aggregate, it is rare for both this parameter and also the stone column deformation modulus to be measured directly for vibro stone column projects. For the purposes of the field trial(s), however, they were considered significant design parameters (as intimated in Section 4.2.1 above). Therefore, a representative sample of the stone aggregate to be used in the trials was taken to the Building Research Establishment (BRE) laboratories at Garston, Hertfordshire for testing in their large triaxial cell apparatus, to determine angle of shearing resistance (and bulk density). The apparatus was chosen because of its successful use by BRE in investigating the behaviour of granular fills over a number of years.

4.2.4 Determination of stone column diameter

Stone column diameter (and by implication - cross-sectional area), is considered to be the single most important stone column design parameter (particularly in respect of load capacity and hence spacing for a given load application, and also for estimating settlement reduction), but is rarely measured in the field. Since the stone column diameter achieved is very much dependent upon both ground conditions and method of installation (see Chapter 1), it was considered that it should be evaluated directly by exhumation of selected 'test' stone columns installed at either end of the field trial location (a 6.2 m long stone column was to be installed at each end of the footprint of the field trials and these columns were subsequently excavated to a depth of just less than 4 m below ground level, with the diameter of each stone column measured at regular intervals within this depth range), to permit comparison with pre-trial assumptions of stone column diameter.

As intimated previously, such exhumation to measure stone column diameter directly is rarely carried out in practice and the best approximation of the effective diameter, without recourse to column exhumation, is related to stone aggregate consumption records (and their back analysis) and to a lesser extent based upon experience and empirical data for column construction in different soil types (or in similar ground conditions to those being considered), by the Specialist Contractors to verify design assumptions. The latter appears to be relied upon for initial estimates of stone column diameter for design purposes. It was therefore also considered important that methods employed to determine stone consumption indirectly should also be evaluated and compared with the data obtained by direct methods as described above, in order to establish the relative accuracy and reliability of such an approach. Investigation of this was incorporated into the research by measuring the volume of backfill used to construct each stone column in the trials, according to the number of calibrated bucket loads required for construction of a typical column (of known depth), with data logged in spreadsheet format. By assigning a value of density for the stone column aggregate in its loose and dense state, it was possible to estimate the effective column diameter. Typical values of unit weight which were considered appropriate, were based upon published data, i.e. 16 kN/m^3 for loose tipped stone aggregate to around 18 kN/m^3 for aggregate in its compacted state, Goughnour and Bayuk (1979a). Table 4.1 of the (former) British Standard: BS8002 (1994), suggests a similar range of values. It was considered that some allowance should be made for waste, 10% appeared reasonable (based upon discussion with Specialist Contractors), when assessing stone consumption data. Apart from the 'end-bulb' construction adopted in the field trials (which was a commonly adopted procedure at the time), it was assumed that each batch of stone aggregate was deposited equally over a specified length of column for most of the remainder of the column, with allowance for some reduction through the upper stiff crust.

4.2.5 Stiffness parameters

Engelhardt and Kirsch (1977) suggested a drained stiffness (deformation modulus) (E_c) of 58 MPa as being a representative value for stone columns. Mitchell (1981) has suggested that for analysis purposes, modulus values of the order of 40 MPa are appropriate, whilst Balaam and Poulos (1983) stated that data from back analysis of plate loading tests suggested that an appropriate value of (E_c) lies between 40 and 70 MPa, which clearly represents quite a wide range. By way of clarification the stone column deformation modulus value at the Bothkennar trial site was to be determined directly from short duration 600 mm diameter plate load tests carried out on representative trial stone columns approximately one week after their installation. For a circular loading plate, the following equation, as is typical, was adopted:

$$E_c = \frac{I_q . Q_b (1-\upsilon^2)}{\Delta_s} \quad \text{-- 4.1}$$

where: I_q = Influence factor (= $\pi/4$ for circular plate)

Q_b = average plate bearing pressure

υ = Poisson's ratio

Δ_s = average settlement under plate at Q_b

Since the plate load test is a short duration test a Poisson's ratio of 0.30 and 0.50 was adopted for granular soils and fine-grained soils respectively, to allow for the fact that in fine-grained soils the plate (load) test will effectively be undertaken in undrained conditions. Based upon discussions with Specialist Contractors in the UK a minimum stone column deformation modulus of 40 MPa was considered safe and was anticipated to be confirmed by the plate load tests and incorporated into the design predictions (bearing capacity and settlement).

4.2.6 Design approach for field trials

Decisions on the layout of the stone column field trials, including chosen stone column spacings, lengths and founding depth for footings, were made on the basis of the following:

As previously discussed, stone columns in soft clays act as 'reinforcing' elements to provide a stone column-soil composite with enhanced bearing capacity and settlement characteristics. BRE BR 391 Specifying vibro stone columns (2000) implies that a suitable design approach should be adopted, related to the type of ground to be treated and foundation type to be used, to evaluate column load capacity and settlement characteristics and hence define the limits of column spacing and depth compatible with the magnitude of load application and for compliance with stated tolerances for post-construction movements. Whilst design charts (e.g. Greenwood (1970), see Chapter 2 section 2.5.3) are suitable for preliminary design purposes, it was considered that site specific design calculations should be made for the Bothkennar field trials based upon the particular site specific circumstances. This was in line with normal practice within the ground improvement industry in the UK, based upon reference to the literature (Chapter 2); BRE Specifying vibro stone columns (2000); discussions with Specialist Contractors and also taking account of the parameter determination (requirement) outlined in Section 4.2.2 to 4.2.5 above. As previously discussed (Chapter 2) a number of analytical approaches for stone column design have been proposed for different applications. For bearing capacity most are refinements of early work by Baumann and Bauer (1974) and Hughes and Withers (1974). Whilst Baumann and Bauer (1974) is also used for aspects of the settlement calculations valuable contributions including design charts have been put forward by Priebe (1976;1995) to address settlement reduction associated with the 'reinforcing' effect of the stone columns. As discussed in Chapter 2 (Section 2.5.4), the Priebe (1995) approach is more reliable in clay soils than the Baumann and Bauer (1974) approach. Appropriate variations and permutations of these methods are used for most vibro stone column applications and this was confirmed again by reference to the literature and again by discussions with Specialist Contractors.

A sequential (iterative) design approach was proposed for the field trial predictions, taking account of the above comments and addressing: load capacity of columns (including factor of safety against bulging failure of an individual stone column); minimum column length and settlements associated with the stone column reinforced ground, but also considering degree of confidence in the ground conditions. The design approach therefore incorporated:

- Assessment of the ultimate load carrying capacity of an individual stone column and hence factor of safety against bulging failure (column over-load) - *Item 1.*

- The minimum stone column length required to safely support the proposed loading conditions using both peak and remoulded undrained shear strengths, with due regard for settlements - *Item 2.*

- Prediction of column load; stress distribution between column and soil - *Item 3*

- Stress distribution beneath the footings and prediction of pre-treatment settlement under the proposed applied loads and stresses - *Item 4*

- Prediction of settlement for the stone column reinforced soil profile (including assessment of settlement below the treated depth) under the applied loads - *Item 5.*

Item 1: For assessment of the ultimate load carrying capacity of stone columns and hence factor of safety against bulging failure (column over-load), where the stone columns are distributed under small (narrow) footings, in such a way that they exist close to a footing perimeter, it is normal practice and sufficiently accurate to treat them as isolated stone columns. The most commonly used method for an initial calculation of the ultimate (and in turn) safe capacity of a stone column is that of Hughes and Withers (1974):

$$\sigma_v' = (1+\sin\varphi'_c)/(1-\sin\varphi'_c)(\gamma_b h_c + 4C_u + p) \text{--} 4.2$$

where σ_v' is the ultimate vertical effective stress in the soil (kN/m^2), φ'_c is the friction angle of the stone column material (up to 45° typically adopted in UK), γ_b is the unit weight of the soil (kN/m^3), h_c is the critical depth (m), C_u is the undrained shear strength of the soil (kN/m^2) and p is the surcharge (kN/m^2). The critical depth h_c is often taken as the depth from ground level to the base of the foundation plus one-two stone column diameter(s).

The safe capacity of the stone column is then given by:

$Q_c = \sigma_v' A_c F$ --- 4.3

where: Q_c is the safe capacity of the stone column (kN), A_c is the area of the stone column (m^2) and F is the factor of safety. A minimum factor of safety of between 1.5 and 2.0 against bulging failure of an individual stone column is typically adopted and was confirmed by discussions with Specialist Contractors. The critical depth (h) i.e. depth at which column bulging occurs, was taken as twice the column diameter below formation, (in accordance with Hughes and Withers (1974) observations) in this instance 1.5 m, i.e. within the upper soft clay below the crust. Ultimate stone column bearing capacity was to be determined using this approach and adopting soil parameters from both Figure 4.3, the BRE Bothkennar soils database, and the results of hand shear vane tests carried out as part of the current research. A typical design calculation extract resulting from this analysis is presented in Figure 4.5.

Item 2: Prediction of minimum stone column length required to safely support the proposed loading conditions (considering both peak and remoulded shear strengths), for the trials was based on Hughes and Withers (1974) for calculating the stone column length required to prevent end bearing failure at the toe occurring before bulging failure near the top of the column. Assuming C_u is constant over the depth (length) of the column, the expression given in equation 2.2 was used to calculate the depth at which vertical stress (σ_{vz}) in the column will be zero (Figure 4.6(a)). Stone column length was considered to have an important impact on the performance of vibro stone columns in soft clays soils. Using the Hughes and Withers (1974) formula: equation - (4.2) defined above and based upon the following parameters: an average (peak) undrained shear strength of 20 kN/m^2, (determined from reference to Figure 4.3 and the undertaking of some representative hand shear vane tests), a stone column diameter of 0.75 m, (determined from test column excavation at either end of the field trial area), a unit weight for the stone column of 17 kN/m^2 (determined as part of the large triaxial testing at the BRE laboratory) and an ultimate column capacity of 543 kN/m^2 (determined from equation (4.2) above), a design treatment depth of 5.5 m was calculated (see Figure 4.6a). A 5.7 m column length (erring on the side of caution, i.e. 5.5 m + 0.2 m) below founding depth was typically adopted for the field trials. However, it was decided that beneath two of the trial footings, column lengths of 3.7 m and 7.7 m respectively (below founding depth) would be adopted in order to investigate the impact of column length on foundation performance. The 7.7 m column length (representing an approximate 50% increase on the 5.7 m column length calculated above) was a reflection of the minimum column length determined on the basis of using remoulded undrained shear strengths (see Figure 4.6(b)), acknowledging the fact that there could potentially be significant soil disturbance during column installation – due to the implied sensitivity of the soft Bothkennar Clay. The 3.7 m column length was intentionally made shorter than the minimum design length calculated, in order to investigate the assertions made in the Hughes and Withers (1974) hypothesis regarding minimum column length. This has influenced decisions on the details of the stone column arrangements and depths in the field trials which are summarised in Table 4.1.

Item 3: Where stone columns are installed they are generally an order of magnitude stiffer than the surrounding soils and by principles of load share the stone columns will attract a greater proportion of the load (P_c) compared to the surrounding soil (P_s) and which was defined by Baumann and Bauer (1974) as follows:

$$\frac{P_c}{P_s} = \frac{1 + 2\left(\frac{Es}{Ec}\right).K_s.ln.\left(\frac{a}{r_o}\right)}{2\left(\frac{E_s}{E_c}\right)K_c.ln\left(\frac{a}{r_o}\right)} \qquad \text{----------------------------} \quad 4.4$$

and

$$P_o.A_o = P_c.A_c + P_s.A_s \qquad \text{----------------------------- 4.5}$$

where:

P_o = imposed load from foundation; P_c = stress on stone column; P_s = stress on soil; A_o = unit area per stone column; A_s = cross-sectional area of stone column; A_c = cross section area of treated soil, E_c = modulus of deformation for stone column; E_s = modulus of deformation for soil; K_s = Earth pressure coefficient for column; K_c = Earth pressure coefficient for soil; r_o = stone column radius; $a = (Ao/\pi)^{0.5}$

By inputting appropriate values for the various parameters into equations (4.4) and (4.5) values of P_c and P_s may be determined. From the calculated values of P_c (load carried by stone column) the factor of safety against bulging failure of an individual stone column can be determined from Q_{ult}/P_c where Q_{ult} is the ultimate carrying capacity of the stone column. The calculations used to determine the values of Q_{ult} for the various trial foundations are presented in Appendix 4.1 and a summary of the values given in Table 4.2. For comparison purposes and by way of a sensitivity analysis, Q_{ult} values were also calculated using a more updated spreadsheet and which gave similar values (see Appendix 4.2).

Item 4: The stress distribution beneath the foundations and estimates of pre-treatment settlements based upon pre-existing soil properties is an important part of the design process. For the Bothkennar field trials it was considered important to understand the magnitude of settlement(s) one was dealing with without vibro stone column treatment and to demonstrate a requirement for ground improvement. There are several stress distribution models available (mainly developed from Boussinesq, 1885), which can be applied to a rectangular strip footing over clay soil e.g. Janbu et al. (1956), Giroud (1971) and Butterfield and Banerjee (1971). In order to select an appropriate methodology for analysing an appropriate stress distribution model, the various approaches mentioned above were applied to the proposed trial footings at Bothkennar. It was found that the stress distributions were very similar and on this basis the Janbu et al. (1956) method (Figure 4.7) applicable to a uniformly loaded rectangular foundation in an elastic clay of finite thickness was adopted. Although the trial foundations to be constructed at Bothkennar were essentially to be rigid, the approach nevertheless appeared to fit the Bothkennar profile (with its surface crust), reasonably well. Discussions with Specialist Ground Improvement Contractors again confirmed this approach was reasonable. Effective stress and coefficient of volume compressibility (m_v) parameters derived from the BRE data-base are given in Figure 4.8(a) and 4.8(b). These parameters have provided the basis for pre-treatment settlement predictions for each of the trial footings and for each of the two main load increments. The results of the pre-treatment settlement predictions (which should be read in conjunction with Figures 4.7 and 4.8), are provided in Figures 4.9(a) to 4.9(h) inclusive. Review of these pre-treatment settlement predictions demonstrate values in the range 20-23 mm for the first load increments (average 33 kN/m^2) and 43.5-46.5 mm for the second load increment (average 70 kN/m^2). Assuming a conventional nominally reinforced narrow footing, a settlement figure of up to 46.5 mm was considered to exceed acceptable tolerances for low-rise brick masonry structures supported on narrow strip footings for example. It is considered that a 35 kN/m^2 loading for a long period is potentially questionable without some form of ground improvement, but dependent upon application and settlements tolerances. Furthermore, such a low bearing capacity is likely to be inadequate for a number of low-rise structures on strip footings, i.e. higher loads will be generated unless a raft foundation is considered, but the raft would have a deeper stress bulb. Moreover, it was considered that if the full 70 kN/m^2 load had been applied in one increment, a failure or instability condition may have potentially been approached.

Item 5: The majority of stone column designs in the UK use the Priebe (1995) method (discussed in Chapter 2, section 2.5.3) for assessment of settlement associated with the 'reinforcing' effect of stone columns where the (A/A_c) as a function of stone column friction angle can be used to obtain the basic reduction factor n_o (that is applied to the untreated settlement within the treatment depth (see Figure 2.22)). To this reduced settlement must be added the settlement contribution from the untreated soil layers associated with the imposed foundation stresses.

Based upon the proposed stone column layouts and depths for the field trials (see Table 4.1), estimates of post treatment settlements have been made, i.e. allowing for the reinforcing effect of the stone columns, using the approach of Priebe (1995). Although the ratio of area of soil: area of stone columns (A/A_c) calculated for the trial footings yielded a basic improvement factor (n_o) of between 3 and 4, experience has shown that such values are optimistic when dealing with narrow footings compared to widespread loads (Greenwood, 1991; Serridge and Synac, 2007), except near the edges of widespread loads, e.g. embankments (Cooper and Rose, 1999), because of the lack of confining support from additional stone columns beyond the edge of the footings. This is particularly true for soft sensitive clays which may be subject to some (temporary) remoulding during stone column installation and a maximum settlement reduction of 2.1 within the soil layers treated was therefore adopted for the field trial(s). The anticipated range of instrumentation to be used in the field trials is given in Table 4.3 and the full range of loading increments to be applied to the trial footings, given in Table 4.4. The predictions of post treatment settlements under the trial footings for the two main load increments adopted for the field trials are summarised in Table 4.5a (1st load increment) and Table 4.5b (2nd load increment). Settlements ranged from 10-11.5 mm for the first load increment and 21.5-23.6 mm for the second load increment, based upon adoption of the maximum settlement reduction factor of 2.1 within the treated depth range described above, corresponding to settlement reductions of around 50% and falling within post construction total settlement limits normally acceptable for low-rise structures on narrow strip footings, i.e. within normal serviceability limits. It should be noted that whilst column compressibility is not considered in the Priebe n_o approach, the fact that the settlement reduction factor would be expected to increase with depth attributed to increasing over-burden pressure and lateral restraint with depth, was not allowed for, provided a hidden safety factor. The approach of using n_o (Priebe basic improvement factor) has been used elsewhere - see Chapter 2 and Mc Cabe et al. (2009).

Secondary consolidation is not considered significant and according to Priebe (1995) there are further benefical effects of column rigidity, depth of overburden and group effects to consider to further refine the post-treatment settlement prediction (n_1 and n_2 factors), but these have not been considered here, in order to facilitate comparison with published historical data, which is typically based upon n_o (see Chapter 2, section 2.5.4 and Chapter 3, Section 3.2) as intimated above.

4.2.7 Stone column spacing

As intimated previously, one of the most important applications for vibro stone columns beneath narrow footings over soft ground is for low-rise housing. NHBC Chapter 4.6 Vibratory Ground Improvement techniques (2011) (including earlier versions of the document), restricts maximum column spacings beneath footings to 2.0m, and in soft to very soft soils, whether for low-rise housing or low-rise industrial or commercial applications it is not uncommon to restrict maximum stone column spacings to around 1.5 m centres beneath narrow footings, in order to safely support the specified bearing capacities and control settlements. On this basis it was proposed that both 1.5 m and 2.0 m stone column spacings (trial footing 1 and 2 respectively - Table 4.1) would be investigated in the field trials.

4.2.8 Founding depth and footing shape

Whilst most trial footings were to be founded at a minimum depth of 0.5 m within the (maximum 1.5 m thick) crust at Bothkennar, in line with general construction practice for shallow footings, it was recognised that in all instances there may not be a surface crust present on soft clay sites. It was therefore decided to found one of the (trial) footings (trial footing 6 – Table 4.1) at the base of the crust (1.2 m depth at location of field trials), in order to evaluate the impact of the absence of the crust on the performance of the installed vibro-stone columns. Additionally, the impact of footing shape was investigated at one of the trial footing locations (trial footing 7, Table 4.1) by use of a square (1.5 m x 1.5 m) pad footing, as might be adopted for a light portal frame industrial unit, in order to permit comparison to a (rectangular) strip footing(s) as might typically be applied to low-rise housing, at the remainder of the trial footing location(s), supported by stone column reinforced soil, Table 4.1. It was also intended to compare all trial footings on stone column reinforced soil with the performance of a trial footing of similar dimensions to the trial strip footings, founded at 0.5 m depth within the crust on untreated ground. This is again reflected in the trial details given in Table 4.1.

4.2.9 Selection of Instrumentation

A range of instrumentation was to be employed during the trials to monitor ground response during stone column installation and also both during and subsequent to trial footing construction (and subsequent incremental loading), over the stone column reinforced Bothkennar Clay soil. The instrumentation employed (Table 4.3) comprised a mixture of standard methods and more specialised techniques, adapted for the particular purposes of the field trials. Whilst budget constraints for the field trials dictated the scope of instrumentation adopted, instrumentation type and location was designed to broadly compliment those on a previous trial at Bothkennar beneath raft foundations on both untreated and treated ground (Watts et al., 2001; Serridge, 2001) and taking due cognisance of the fact that some instruments were shown to yield little valuable data, or were judged unsuitable for the purpose for these raft foundation investigations e.g. the inclinometer gauge. A conventional servo-accelerometer torpedo type inclinometer gauge had been previously installed to measure lateral ground displacement immediately outside the area of a raft without any ground improvement support at Bothkennar (Chown and Crilly, 2000), but the technique proved to be of limited value as lateral movements had been very small and not easily detectable.

The overall requirement was that the instrumentation had to survive and give accurate data during the installation of the vibro stone columns and also to subsequently measure ground response (changes to soil properties were anticipated) and performance during construction and subsequent loading of the trial (concrete) footings over the installed stone columns. Access to the site was available for sufficient time to permit monitoring over a minimum of 5 months for each of two main load increments anticipated for the trials. This allowed settlement and creep to be monitored for significantly longer than conventional dummy foundation tests or zone load tests used to monitor vibro stone column reinforced ground performance in practice (see Chapter 1, Section 1.8). The selection together with the positioning and installation of the instrumentation can be considered in two stages: pre-stone column installation and post stone column installation.

The majority of instrumentation was installed prior to stone column installation (miniature push-in earth pressure cells; pneumatic piezometers and an electrolevel inclinometer gauge), but with flatjack pressure cells installed after stone column installation and prior to foundation construction and with levelling studs (for precise levelling requirements), installed during foundation construction as discussed below:

Pre-installation:

This included the installation of instrumentation to facilitate measurement of lateral displacement; ground stresses and pore pressures associated with the stone column installation, incorporating an inclinometer, earth pressure cells and piezometers.

Inclinometer - Given the soft saturated nature of the clay-silt deposits at Bothkennar, combined with the displacement anticipated with the dry bottom-feed technique, together with the fact that column installation was expected (at least) initially to take place under undrained conditions, significant ground displacement was anticipated. Hence in order to measure lateral ground displacement during stone column construction (and susbsequent foundation loading), and taking account of previous comments regarding conventional servo-accelerometer torpedo type inclinometer gauges, a special inclinometer system, originally developed by the Building Research Establishment (BRE) and comprising individual electro-level measuring units mounted in rigid, articulated aluminium box sections, was utilised to provide more sensitivity in order to accommodate the localised displacement anticipated during the installation of predominantly single rows of columns. The system was also very flexible and had been used in a wide variety of applications by the BRE for both ground and structural monitoring, so hence had a proven record. It should be noted that whilst the degree of resolution of the electro-levels is greater than the torpedo systems referred to previously, the range is generally smaller. The special inclinometer system was to be installed at a distance of 0.5 m from the centre of a selected stone column installation point, to gain maximum information on soil displacement, whilst avoiding damage to the equipment.

Earth pressure measurements - At Bothkennar, where a surface crust is underlain by deep soft clay soil, it was considered important to be able to measure vertical as well as horizontal (lateral) stress. The magnitude of stresses within the ground can have an important influence on the engineering behaviour of engineering structures from a bearing capacity, settlement and stability standpoint. Furthermore, the horizontal stress will be a function not only of depth and unit weight of the soil, but also of the stress-strain relation and stress history of the soil. Reliable determination of horizontal stress usually requires *in-situ* measurement. Historically methods of measurements have included the use of hydraulic fracture tests, self-boring instruments and push-in pressure measuring devices. Push-in spade-shaped earth pressure cells have proved to be simple and reliable to use and reasonably consistent measurements of in-situ stress can be obtained, Tedd et al. (1989). There is, however, a tendency for them to over-read even when the excess pore pressures set up during installation have dissipated. BRE developed an instrumentation system to measure vertical and horizontal stress, Watts and Charles (1988), and which has a proven history of reliability (including at Bothkennar) and comprises miniature earth pressure cells which can be pushed horizontally from 150 mm and 200 mm diameter vertical boreholes using a special placing device.

The miniature earth pressure cells, Watts and Charles (1988) are designed principally for soft clays and were to be installed prior to stone column installation and subsequent foundation construction and load application in the field trials. The principle advantages of the miniature pressure cell system which influenced or determined their selection for the Bothkennar trial(s) are summarised below:

- Capable of measuring both vertical and horizontal in-situ earth pressure in soft clay soils.

- Have been calibrated against known soil stresses in laboratory and field trials (including at Bothkennar).

- Over-read due to soil disturbance is relatively small, a maximum of $0.5C_u$ being indicated when measuring horizontal total earth pressure and less when measuring vertical stress.

- A number of pressure cells can be installed from a single small diameter vertical borehole to measure stress in any orientation and at depth.

- Miniature cells installed by BRE over the period 1986-91 (43 sites throughout the UK), and at other sites internationally, continue to operate satisfactorily and have demonstrated reliability and long term stability in both constant and changing stress conditions. Cells installed at Bothkennar in 1989 (as part of a separate research programme associated with the original site geotechnical characterisation of the Bothkennar Clay), continue to function satisfactorily.

The only real disadvantage of the cells is that they cannot be readily retrieved once installed. It was considered that the advantages (or potential benefits to be gained), outweighed this and should therefore generally not preclude their use, particularly on the Bothkennar site, even for short term measurements, because of the potential data acquisition that could be achieved. In terms of the application of the push-in earth pressure cell equipment to the field trials at Bothkennar, important parameters required for interpreting the results of the trials were the distribution of vertical and horizontal stress with depth and the Poisson’s ratio for the soil as described previously. The miniature cell provided a simple method of measuring the vertical and horizontal stresses at selected depths beneath the trial stone column installation area. Where the proposed loaded area is small, as was the case for the trial (strip) footings over the stone column reinforced soil at Bothkennar, the cells could be pushed to the required location from a borehole outside the loaded area. Stress changes were required to be continuously monitored during initial vibroflot penetration and subsequent column construction, to investigate the stress development (and subsequent dissipation), during the construction process at each trial stone column installation point.

Pneumatic Piezometers – Pneumatic piezometers were designed to be installed in the clay 'crust' and immediately underlying soft clay close to selected trial column positions, to facilitate observation of and changes in pore water pressures both during and after stone column construction, in order to assess rate and magnitude of increase and subsequent rate of dissipation of pore pressures during and after stone column installation and also during and subsequent to application of load to trial footings constructed over the installed stone columns. This included evaluation and measurement of pore pressure changes associated with any lateral stress increases during and subsequent to column installation and foundation loading within the 'critical zone' (zone of anticipated stone column bulging under applied load). It was also recognised that the influence on the stress state becomes significant when movement is restrained e.g. by a surface 'crust'. Piezometers were installed at similar locations to the total earth pressure cells around and between selected (stone) columns. From reference to work by Gäb et al. (2007) and Gäb et al. (2008) and discussions with Specialist Ground Improvement Contractors, the rise of the stress level is reduced within a distance of less than around four times the column diameter due to remoulding and liquefaction effects, so instrumentation needed to be located within this general zone, ideally as close to stone column positions as practical, but without risk of damage during stone column installation.

The main advantages of pneumatic piezometers are:

- Accurate measurement of pore water pressures in fully saturated soil.
- Low volume change, therefore fast response.
- Short response time even in low permeability soils such as clay.
- Level of tubing in relation to readout is not critical.
- The pneumatic tubing is strong and flexible.
- Mechanical simplicity, relatively inexpensive, reliable and robust with over 50 years of application.

The working range for the pneumatic (gauge) readout was 0 to 20 metres head of water. It is important to recognise that the instrument is designed to operate in saturated soils but will record short term negative pressues when fitted with a high air entry filter. However, because there is no facility for de-airing of the tip, this diaphragm type of piezometer is unsuitable for measuring long term negative pressures in partially saturated soils. The small volume change resulting from the diaphragm deflection during the reading process can influence measurements when the tip is installed within a highly impermeable material.

Post-installation:

Flatjack pressure cells – In order to measure stress distribution between stone columns and intervening soil (and therefore stress ratio (S_r)) under the trial footings and how this varies with time and magnitude of loading, to facilitate better understanding of the response of the stone column reinforced soil to load application, it was decided to install flatjack (pancake-type) 300 mm diameter pneumatic pressure cells, subsequent to stone column installation and prior to footing construction. Cells were therefore to be installed in the top section of a selected stone column at founding level and at the same level in the intervening at most of the trial footing locations. The pressure cells have a long record of successful application within the BRE within field-based research projects and with proven reliability. The cells could also be recovered for re-use on completion of the field trials.

Precise levelling – When monitoring settlement of an element or structure to establish what is happening, rates of movement must be determined together with any changes in these rates to ensure a (bearing capacity) failure condition is not being approached and to establish time periods over which primary consolidation settlements are progressing. Such rates might typically equate to only a few mm per month. An accuracy of measurement better than +/- 0.5 mm was therefore desirable. This accuracy is generally not possible with normal site surveying equipment – a precise level and staff, purpose made levelling stations and stable datum was therefore considered necessary for the Bothkennar field trials.

4.2.10 Trial details and data aquisition

On the basis of previous comments there are three aspects of stone columns that required investigation in these field research trials: (1) the ground response to installation of partially penetrating (partial depth) dry bottom-feed stone columns, through visual observation and monitoring of an installed suite of instrumentation previously described in Section 4.2.9 above, (2) changes to the soil stress field and pore pressure during the re-equilibration period following stone column installation, and (3) the interaction between the foundation and the stone column reinforced soil.

During the (Bothkennar) field trials it was proposed to investigate different arrangements (layouts) of stone columns, i.e. spacings and lengths, together with founding depth within the thin surface crust, below narrow footings subject to incremental loading and permit comparison with similar untreated footing sizes. The 12 month site access period available, as previously described, permitted hold periods of around 5 months for each of two separate (main) load increments proposed, to reflect the range of loading conditions typically associated with low-rise structures on narrow footings.

Details of the location and layout of the trials (trial footings, trial columns and test columns: columns 21-36;45-46), are given in Figure 4.10a and 4.10b. The suite of trial footings, together with their dimensions and arrangement of stone columns (based upon the above comments and results of analysis in Sections 4.2.6.) is given in Table 4.1 and also Figures 4.11a-e, which provide sections through each of the field trial footing locations, together with locations of instrumentation. The field trial design was influenced by approaches representative of current (design) practice and carried out in a specific way to reflect typical footing arrangements below low rise structures, notably housing and light industrial units. Data acquisition (see also Section 1.8 - Monitoring, testing and quality control) carried out during the field trials is described below:

- Observation of the column installation process (including any ground heave), together with manual monitoring of the volume of stone aggregate used at each stage of stone column construction, to permit indirect estimates of stone column diameter and also to permit comparison with direct methods (based upon column exhumation).

- Monitoring of a suite of instrumentation installed prior to stone column installation (Figure 4.11a-e and Table 4.3) This was to include pneumatic piezometers, an electro-level inclinometer and earth pressures cells, as previously described. Pressure cells were located at depths of 0.5 m and 1.1 m below anticipated founding level (i.e. zone of anticipated column bulging), to permit measurement of lateral (horizontal) stresses (adjacent to selected stone column positions). Measurement of pore pressures (adjacent to selected stone column positions), again located at 0.5m and 1.1m below founding level to facilitate evaluation of ground response during and after column installation within the anticipated zone of stone column bulging was also proposed.

- Measurement of 'toe' pressure (300 mm) below the base (toe) of selected stone column positions to facilitate investigation of load transfer mechanisms down the granular column (length) both during stone column installation and subsequent application of load.

Soil-structure interaction, in respect of response of the partially penetrating stone columns to incremental loading of trial footings constructed over the installed columns was then to be subsequently evaluated. This included:

- Installation of a second suite of instrumentation (Figure 4.11a-e and Table 4.3) (principally flatjack pressure cells) and monitoring of these over a period of around 12 months in parallel with the first suite of instrumentation (and the setting up of a precise levelling system). The flatjack pressure cells would permit measurement of the proportion of the total applied vertical stress attracted by the instrumented stone columns and intervening soil and how this varied during the different loading stages, including with time. This behaviour was closely monitored at the commencement, mid-point and towards the end of each of the load increment cycles (with the exception of trial footing 6 where it was not possible to measure stresses in the intervening soil due to issues with instrumentation, i.e. damage to the instrumentation at that location) and from which the stress ratio (S_r) can be calculated at these 3 stages (commencement; mid-point and end) described above.

- Monitoring of Phase 1 instrumentation for measurement of 'toe' pressure 300 mm below the base of selected stone column positions to facilitate investigation of load transfer mechanisms down the column during application of foundation load;

- Measurement of pore pressures adjacent to selected stone column positions again located at 0.5 m and 1.1 m below founding level to facilitate evaluation of ground response during foundation loading within the anticipated zone of stone column bulging.

- Pressure cells were located at depths of 0.5 m and 1.1 m below anticipated founding level (i.e. zone of anticipated column bulging) to permit measurement of lateral (horizontal) stresses (adjacent to selected stone column positions) during incremental loading of the foundations. The main purpose was to investigate any bulging.

In common with typical foundation design for many low-rise construction projects in the UK, concrete strip foundations were (to be) used in the field trial(s). Steel mesh reinforcement (Figure 4.12 a and b) was to be provided in both top and bottom elements of the trial strip footings (in line with NHBC Chapter 4.6 (2011) guidance). Strip dimensions and details are given in Table 4.1. The trial strip footings were constructed shortly after the ground treatment, in common with general construction practice.

4.3 Numerical analysis

The design of vibro stone columns in soft clay is typically achieved using empirical or semi-empirical methods (see Chapter 2). These design approaches typically require a good deal of experience with stone columns and tends to be conservative. In situations where the limits of the design methods are reached or where complex ground conditions introduce uncertainties into these empirical design calculation approaches, numerical computations can provide the design engineer with additional insight into the problem. It is important to recognise that numerical analyses require comparative studies either by field observations or trials (in this case we have the Bothkennar field trials), or conventional design methods to prove their usefulness. Furthermore, it is important to emphasise that numerical modelling facilitates sensitivity analyses, parametric studies, rapid comparative studies of effect of changes on stone column arrangement etc.. (e.g. different stone column arrangements adopted in the Bothkennar field trials).

Nevertheless there is the issue of how does one represent the system and relevant parameters. Review of the literature (Chapter 2) where the finite element method has been used to model ground improved with stone columns mostly relate to wide-area loading, using either a unit cell approach e.g. Domingues et al. (2007) or a 2-D axisymmetric approximation e.g. Elshazly et al. (2008). Whilst some 3-D modelling of wide-area loading has been carried out, e.g. Gäb et al. (2008), no research on 3-D modelling of narrow strip footings supported by partial depth vibro stone columns in soft clay (as at Bothkennar) has been published, apart from some preliminary work by Killeen and Mc Cabe (2010) on groups of stone columns beneath (square) pad footings. As part of this research some preliminary 3-D modelling has therefore been undertaken to model the Bothkennar field trials where narrow footings are supported by varying arrangements of partial depth vibro stone columns in a deep soft clay deposit. It is important to highlight, as intimated previously, that the vertical stress beneath footings decays much more sharply with depth than the stress beneath wide loaded areas, which makes partial depth treatment in deep soft clays permissible (assuming there is no raising of site levels which would surcharge the soil profile – see Chapter 3, section 3.3, - Wilde and Crook, 1992). Analytical theory is much less well developed for this application, with reliance falling heavily on empirical methods. The Plaxis 3-D Foundation (Version 2.2) finite element geotechnical software package was used, principally because of its availability, proven record of use and reliability (accompanied by good product support), user-friendliness; accuracy; widespread use in geotechnical engineering and some previous use in vibro stone column applications for widespread loads (e.g. Kirsch, 2008) and beneath pad footings, as mentioned previously, e.g. Killeen and McCabe (2010).

The performance and accuracy of Plaxis 3-D Foundation has been carefully tested by carrying out analyses of problems with known theoretical solutions. A selection of these benchmark analyses is described in Chapter 2 to 6 of the user manual for the software, for example (Plaxis, 2008). Plaxis 3-D Foundation has also been used to carry out predictions and back-analysis calculations of the performance of full-scale structures as additional checks on performance and accuracy. Moreover, Plaxis has been used extensively for the prediction and back-analysis of full-scale projects. This type of calculation may be used as a further check on the performance of Plaxis provided that good quality soils data and measurements of structural performance are available. Four validation examples can also be found in the last chapters of the User Manual. The use of Plaxis 3-D Foundation is preferable (to 2-D) when modelling pad and strip footings, where the 3-D nature of a problem needs to be captured. However, it is known that Plaxis 3-D has greater limitations than Plaxis 2-D in terms of modelling larger strains. With this in mind, a simple Plaxis 3-D model of the different footing arrangements for the Bothkennar field trials was developed to investigate boundary effects and settlements.

Trial footing	Dimensions (L x B)	Founding depth	Ground treatment		
			Number of columns	Column spacing	Column length (below foundation)
1	6.0m x 0.75m	0.5m	4	1.5m	5.7m
2	6.0m x 0.75m	0.5m	3	2.0m	5.7m
3	3.0m x 0.75m	0.5m	2	1.5m	3.7m
4	3.0m x 0.75m	0.5m	2	1.5m	5.7m
5	3.0m x 0.75m	0.5m	2	1.5m	7.7m
6	3.0m x 0.75m	1.2m	2	1.5m	5.7m
7	1.5m x 1.5m	0.5m	2	1.2m	5.7m
8	3.0m x 0.75m	0.5m	No treatment	-	-

Table 4.1: Summary of Bothkennar trial footing arrangements.

Trial footing	**2nd Load increment (kN/m^2)**	**Area per compaction (unit area per stone column) (Ao)**	**Ultimate bearing carrying capacity of columns (Qult) kN/m^2**	**Factor of safety against bulging failure of stone column**
1	72.00	1.13 m^2	548.79	3.10
2	67.10	1.50 m^2	548.79	2.65
3	67.80	1.13 m^2	548.79	3.29
4	71.10	1.13 m^2	548.79	3.14
5	67.80	1.13 m^2	548.79	3.29
6	69.80	1.13 m^2	548.79	3.20
7	67.90	1.13 m^2	548.79	3.33

Table 4.2: Summary of calculated ultimate carrying capacity (Qult) and factor of safety against bulging failure for stone columns beneath trial footings 1-7.

INSTALLATION TYPE	MEASURED PARAMETER	INSTALLATION LOCATIONS	INSTALLATION DEPTH RANGE	DRILL/ EXCAVATION METHOD	INSTALLATION METHOD	MONITORING TECHNIQUE
Pneumatic piezometers	Pore water pressure	Alongside columns under trial footings 4 and 6. Between columns and inside/outside treatment on raft.	0.7m – 5.0m below treated ground level	Nominal 100mm borehole using small CFA rig + hand auger	Tip placed within sand cell and backfilled with weak cement/bentonite grout.	Glötzl pneumatic readout
BRE Miniature push-in earth pressure cells	In-situ vertical and horizontal total earth pressure	Between and alongside columns and beneath the toe of columns	0.7m – 8.5m below treated ground level	Shell and auger 200mm boreholes + small CFA and hand auger	BRE special placing device. Cells pushed up to 600mm from vertical borehole.	Glötzl pneumatic readout
Flatjack type 300mm diameter pressure cells	Vertical total stress close to the top of columns and in soil between columns	Selected columns and between columns under pads 1-6	Approx.50-100mm below foundation level	Hand excavation	Column stone removed, cells placed in sand (graded up to fine gravel) pockets and stone recompacted over cells. Similar technique in natural soil using sand only.	Glötzl pneumatic readout
Electro-level inclinometer gauge consisting of five rigid sections with gauge in each	Horizontal displacement of soil alongside column	Alongside column, trial footing 4.	Surface to 2.5m	Small CFA and hand auger	Complete gauge assembled and lowered down vertical borehole which was then grouted	Electronic measurement of gauges + survey measurement of top movement

Table 4.3: Summary of range of instrumentation to be installed for the Bothkennar field trials.

Trial footing	Founding depth (metres)	1st Load increment kN/m^2	2nd Load increment kN/m^2	3rd Load increment kN/m^2	4th Load increment kN/m^2
1	0.50	35.5	72.0		
2	0.50	32.9	67.1		
3	0.50	33.1	67.8		
4 (**)	0.50	34.9	71.1	108.1	125.1
5	0.50	32.1	67.0		
6 (**)	1.20	34.2	69.6	105.5	122.4
7	0.50	32.7	67.0		
8 (*)	0.50	34.3	71.6		

(*) Untreated footing (**) footing loaded to failure

Table 4.4: Summary of loading increments to be applied to trial footings at Bothkennar.

Trial footing	First load (kN/m^2)	A_o (m^2)	Stone column diameter (mm)	A_c (m^2)	A/A_c	Friction angle of stone column aggregate (φ')	Priebe basic improvement factor (n_0)	Pre-treatment settlement (mm)	Post-treatment settlement within treated depth (mm)	Post-treatment settlement below treated depth (mm)	Sum of post-treatment settlement within and below treated depth (mm)
1	33.5	1.13	0.75	0.44	2.56	42.5	2.1	21.7	20.90/2.1= 09.95	0.80	10.75
2	32.9	1.50	0.75	0.44	3.41	42.5	2.1	21.3	20.51/2.1= 09.76	0.79	10.55
3	33.1	1.13	0.75	0.44	2.56	42.5	2.1	21.4	19.99/2.1= 09.51	1.41	10.92
4	34.9	1.13	0.75	0.44	2.56	42.5	2.1	22.6	21.76/2.1= 10.36	0.84	11.20
5	32.1	1.13	0.75	0.44	2.56	42.5	2.1	20.8	20.60/2.1= 09.81	0.20	10.01
6	34.2	1.13	0.75	0.44	2.56	42.5	2.1	23.1	22.29/2.1= 10.61	0.81	11.42
7	32.7	1.13	0.75	0.44	2.56	42.5	2.1	21.2	20.39/2.1= 09.71	0.81	10.52

Table 4.5: (a) Estimated post treatment settlements of trial footings under first load increment.

Trial footing	Second load (kN/m^2)	A_o (m^2)	Stone column diameter (mm)	A_c (m^2)	A/A_c	Friction angle of stone column aggregate (φ')	Priebe basic improvement factor (n_0)	Pre-treatment settlement (mm)	Post-treatment settlement within treated depth (mm)	Post-treatment settlement below treated depth (mm)	Sum of post-treatment settlement within and below treated depth (mm)
1	72.0	1.13	0.75	0.44	2.56	42.5	2.1	46.6	44.03/2.1= 20.99	2.57	23.55
2	67.1	1.50	0.75	0.44	3.41	42.5	2.1	43.5	41.01/2.1= 19.52	2.26	21.78
3	67.8	1.13	0.75	0.44	2.56	42.5	2.1	43.9	40.82/2.1= 19.43	3.08	22.51
4	71.1	1.13	0.75	0.44	2.56	42.5	2.1	46.1	43.48/2.1= 20.70	2.62	23.32
5	67.8	1.13	0.75	0.44	2.56	42.5	2.1	43.9	42.85/2.1= 20.40	1.05	21.45
6	69.6	1.13	0.75	0.44	2.56	42.5	2.1	45.1	42.56/2.1= 20.27	2.54	22.81
7	67.0	1.13	0.75	0.44	2.56	42.5	2.1	43.5	40.97/2.1= 19.50	2.43	21.93

Table 4.5: (b) Estimated post treatment settlements of trial footings under second load increment.

a)

b)

Figure 4.1: (a) Tilt – 1930's detached house; (b) Tilt – 1930's detached house juxtaposed against modern housing development.

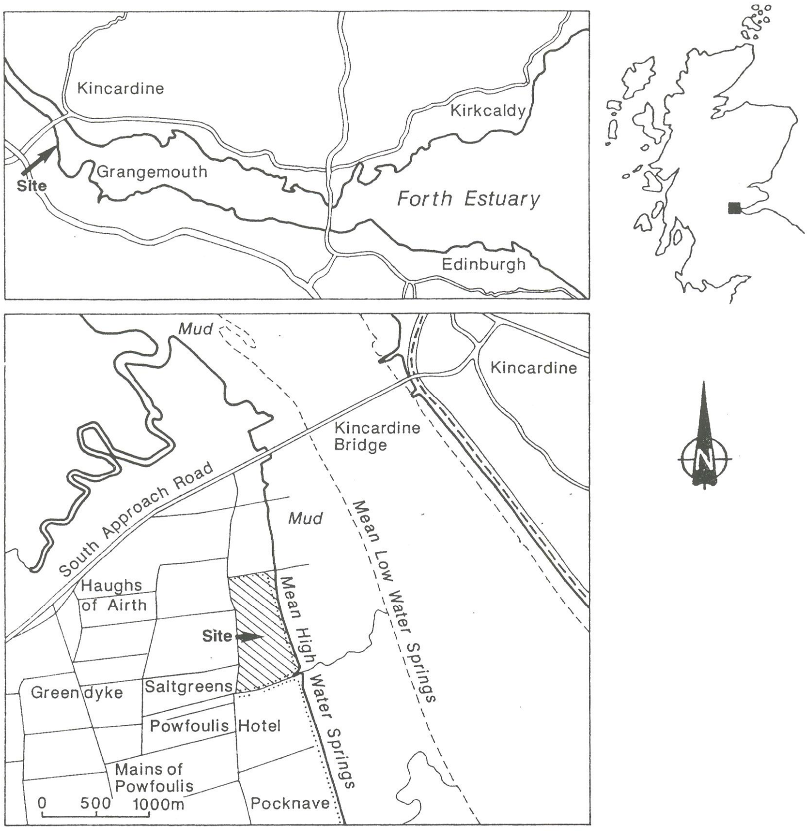


Figure 4.2: Location of the Bothkennar soft clay research site (to left of picture with its boundary flood defence bund visible) and view of the mud flats looking upstream on the River Forth towards Kincardine Bridge.

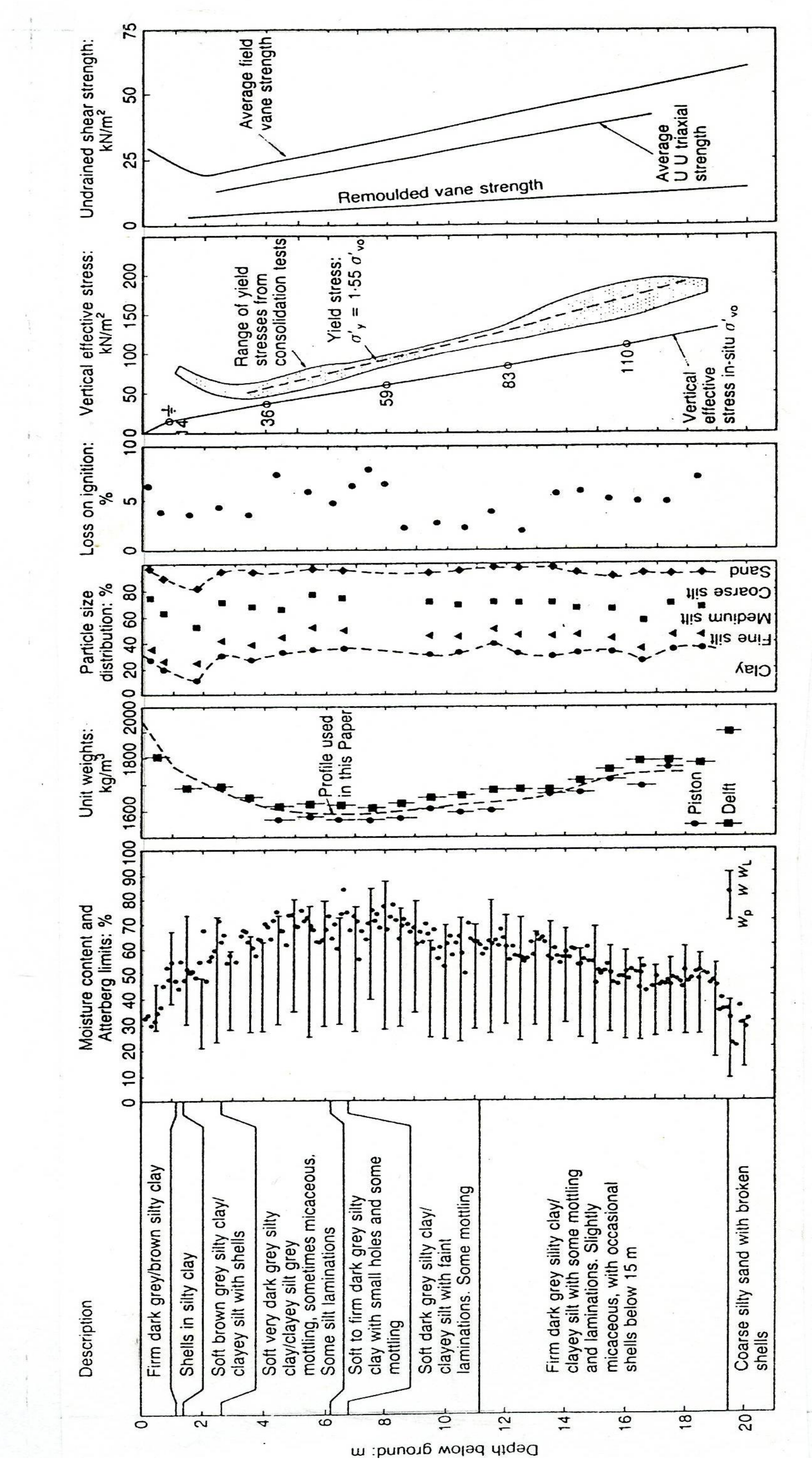


Figure 4.3: Key geotechnical profile for the Bothkennar soft clay research site (after Nash et al., 1992a)

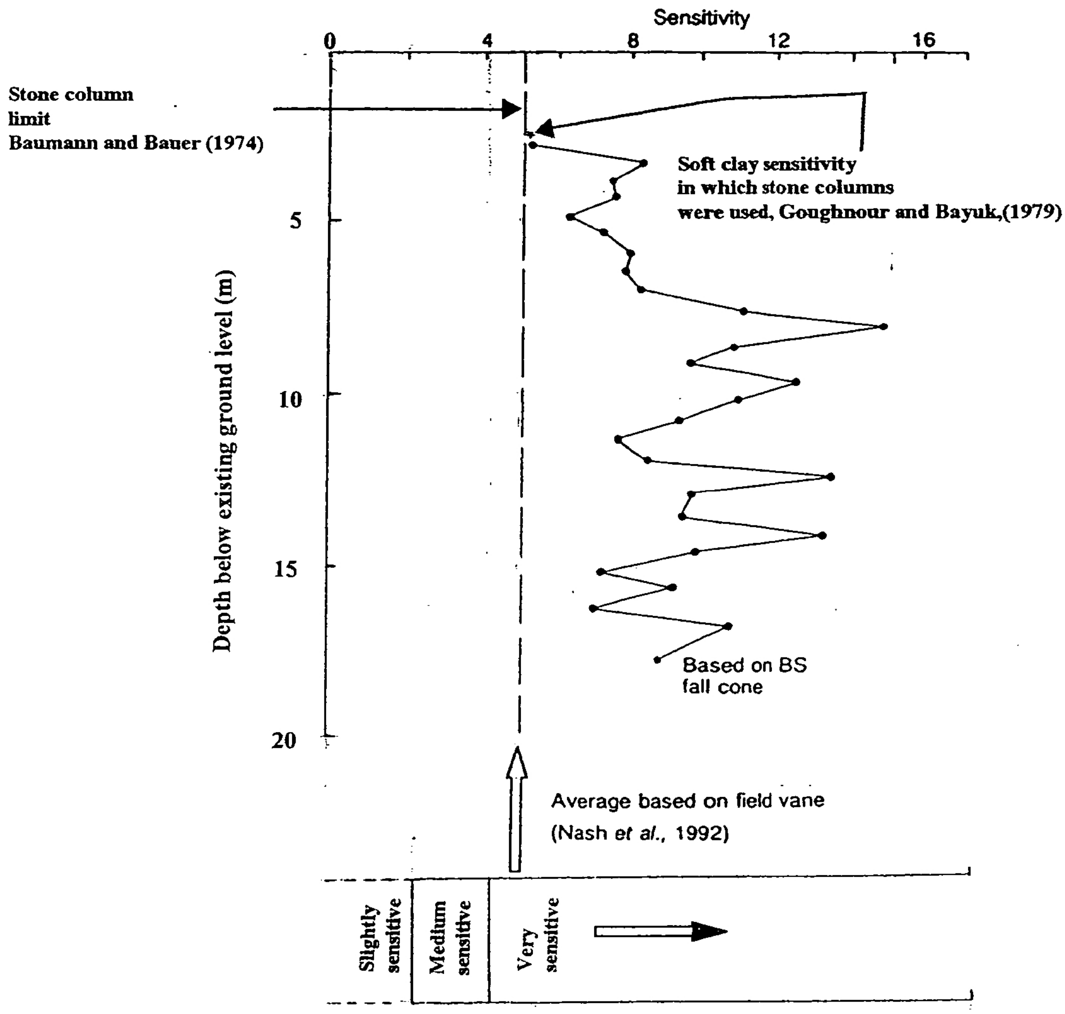


Figure 4.4: Bothkennar soil sensitivity profile (after Nash et al., 1992a) annotated with historical limits for vibro stone columns.

DETERMINATION OF ULTIMATE CARRYING CAPACITY OF STONE COLUMN

From Hughes and Withers (1974)

$$Q_{ult} = \frac{(1+\sin\phi')}{(1-\sin\phi')}.(\gamma.h + 4c_u + p)$$

Where :

Q_{ult} = Ultimate bearing capacity of stone column

φ´ = friction angle of stone column aggregate

γ = Unit weight of soil

h = depth at which stress is considered (critical depth). Zone of bulging (2D)

C_u = undrained shear strength of soil

p = surcharge (taken as zero)

For:

φ´ = 42.5°

γ = 17 kN/m^3

h = 1.5 m (based upon a column diameter of 0.75 m)

c = 20 kN/m^2 (peak undrained shear strength within bulging zone)

Therefore:

Q_{ult} = 543 kN/m^2

For a stone column diameter of 0.75 m, the column cross-sectional area is 0.44 m^2. Therefore the ultimate load on the column = 543 x 0.44 = 239 kN. Allowing for an FOS of 2 against bulging failure of an individual stone column, the working column load = 239/2 = 120 kN.

Figure 4.5: Bearing capacity calculation (individual stone column) for field trials. (See also Equation 4.2 Chapter 4).

Based on Hughes and Withers (1974) an expression which can be used to calculate the change in stress with depth within a stone column is:

$$\sigma_{vz} = \sigma_v + M(\gamma_c.D - 4c)$$

where,

σ_v - ultimate capacity of stone column

M - ratio L_c/D, where L_c is critical length to bulging and D is diameter of stone column.

γ_c - bulk density of stone column material

c - shear strength of soil

Therefore, for

$D = 0.75$ m

$\sigma_v = 543$ kN/m^2

$\gamma_c = 17$ kN/m^3

$L_c = 1.5$

Depth m	M (L_c/D)	$M.\gamma_c.D$	C_u (kN/m^2)	M.4c	σ_{vz} (kN/m^2)
1.0	1.33	17	20.0	106.67	453
2.0	2.67	34	20.0	213.33	364
3.0	4.00	51	20.0	320.00	274
4.0	5.33	68	20.0	426.67	184
5.0	6.67	85	20.0	533.33	95
6.0	8.00	102	22.0	704.00	-59
7.0	9.33	119	24.0	896.00	-234
8.0	10.67	136	24.0	1024.00	-345
9.0	12.00	153	24.0	1152.00	-456
10.0	13.33	170	24.0	1280.00	-567

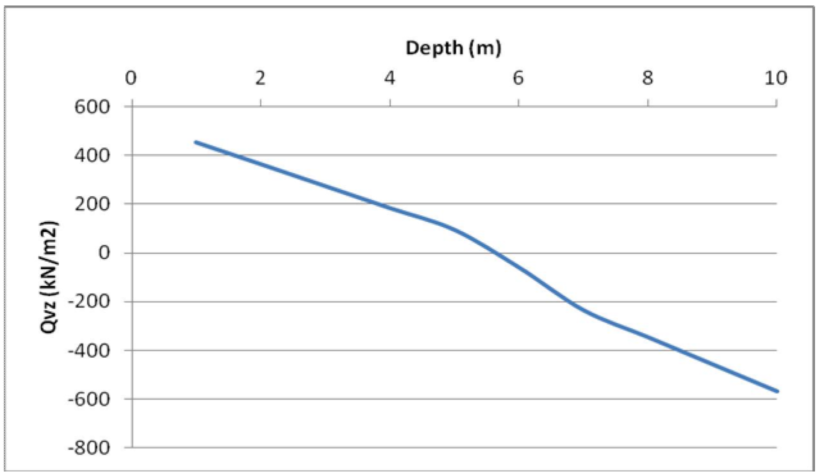


Figure 4.6: (a) Calculation of minimum stone column length (after Hughes and Withers, 1974) based upon recorded peak undrained shear strength in Bothkennar Clay.

Based on Hughes and Withers (1974) an expression which can be used to calculate the change in stress with depth within a stone column is:

$$\sigma_{vz} = \sigma_v + M(\gamma_c.D - 4c)$$

where,

σ_v - ultimate capacity of stone column

M - ratio L_c/D, where L_c is critical length to bulging and D is diameter of stone column.

γ_c - bulk density of stone column material

c - shear strength of soil

Therefore, for

D = 0.75 m

σ_v = 543 kN/m^2

γ_c = 17 kN/m^3

L_c = 1.5

Depth m	M (L_c/D)	M.γ_c.D	C_u (kN/m^2)	M.4c	σ_{vz} (kN/m^2)
1.0	1.33	17	8.0	42.67	517
2.0	2.67	34	7.5	80.00	497
3.0	4.00	51	8.5	136.00	458
4.0	5.33	68	10.0	213.33	398
5.0	6.67	85	11.0	293.33	335
6.0	8.00	102	11.5	368.00	277
7.0	9.33	119	11.9	443.52	218
8.0	10.67	136	12.0	512.00	167
9.0	12.00	153	12.3	590.40	106
10.0	13.33	170	12.5	666.67	46
11.0	14.67	187	12.7	745.07	-15
12.0	16.00	204	12.7	812.80	-66

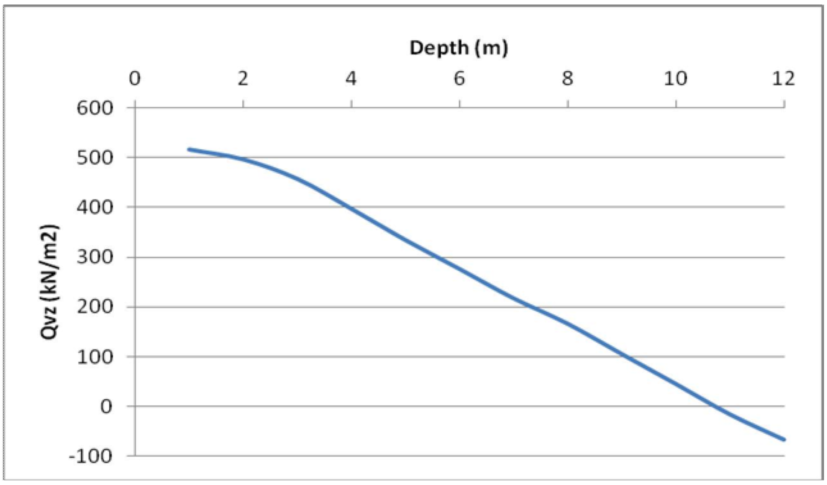


Figure 4.6: (b) Calculation of minimum stone column length (after Hughes and Withers, 1974) based upon remoulded shear strength in Bothkennar Clay.

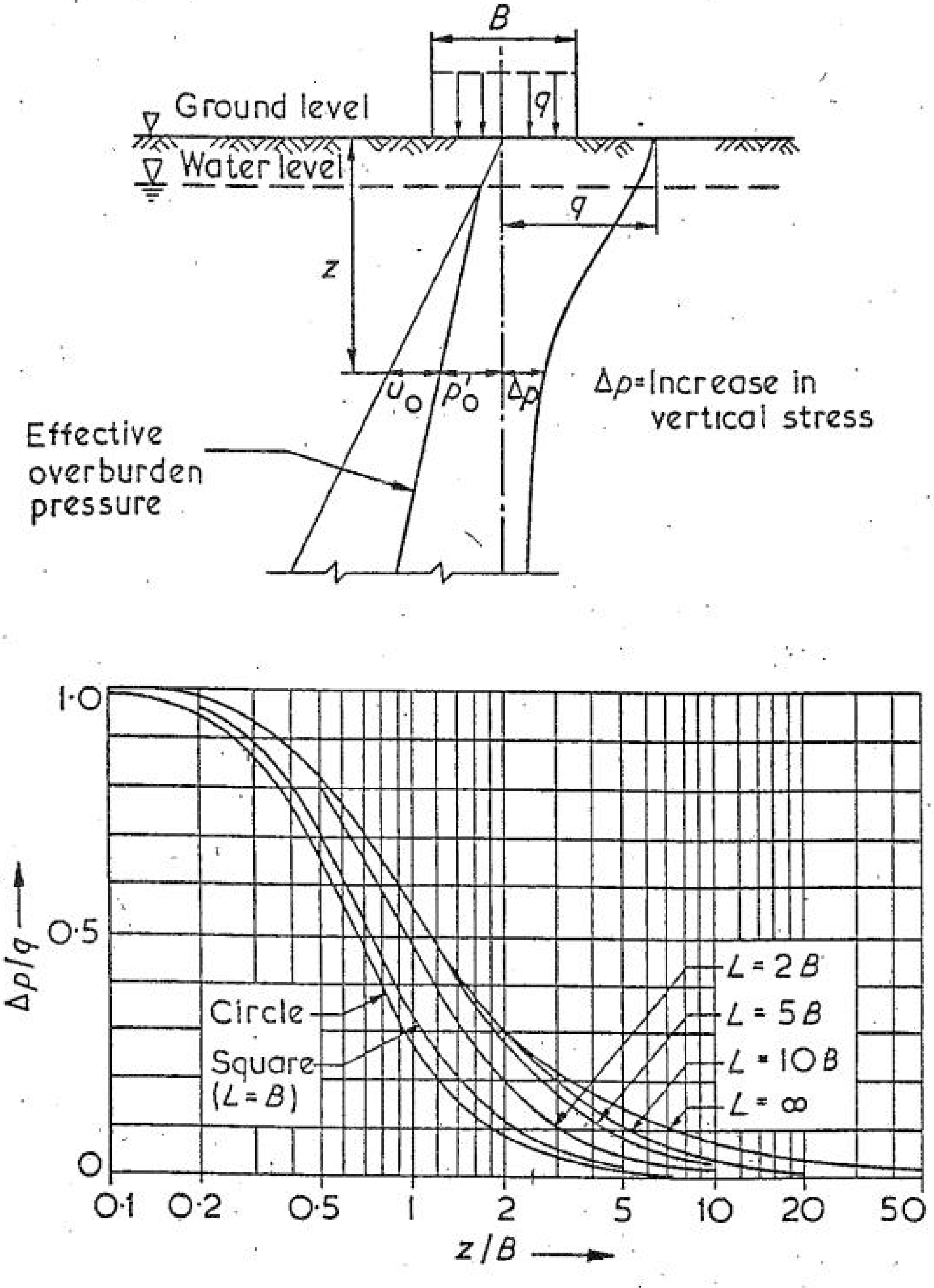


Figure 4.7: Determination of increase in vertical stress under the centre of uniformly loaded (flexible) footings (after Janbu et al., 1956)

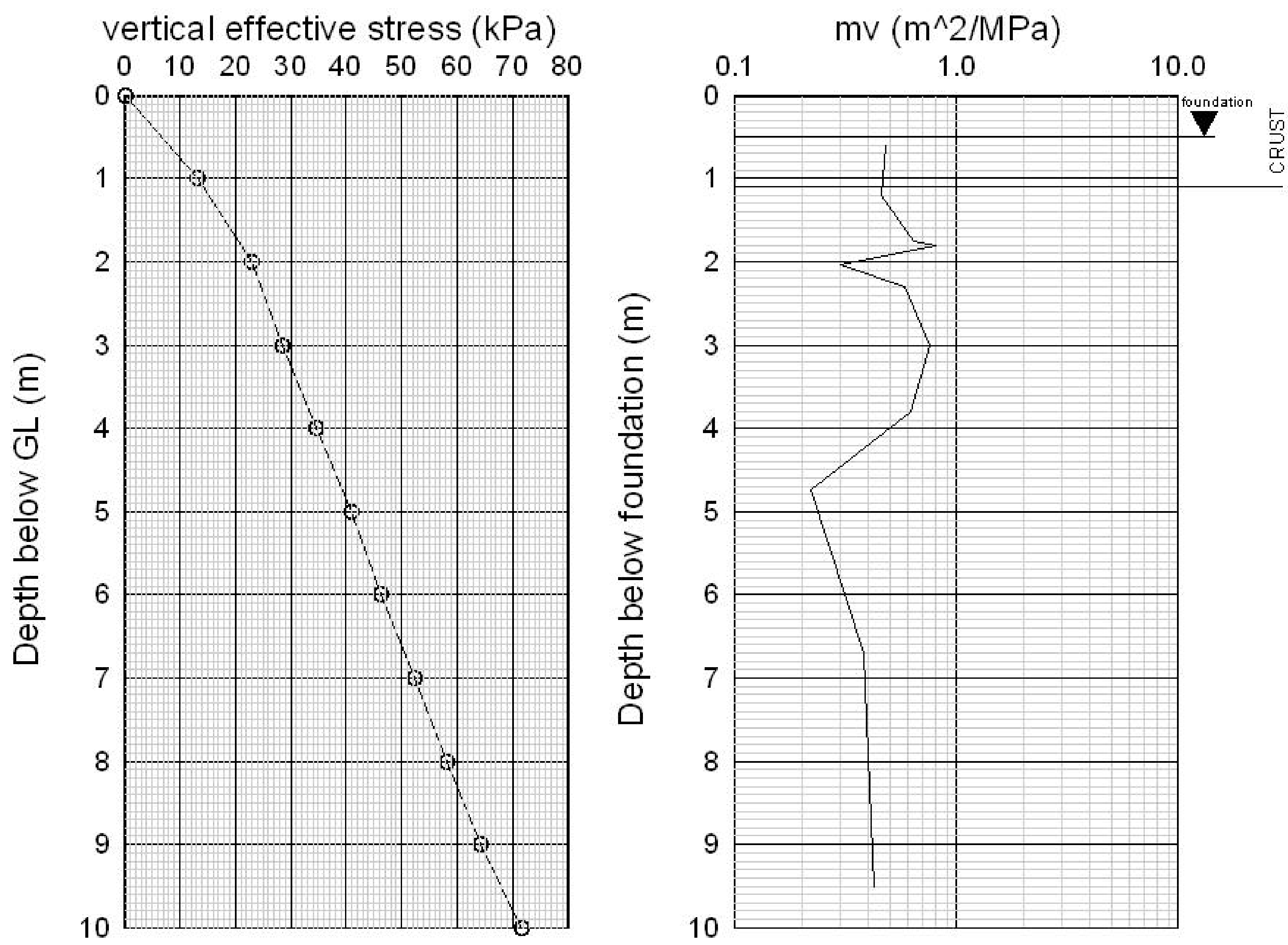
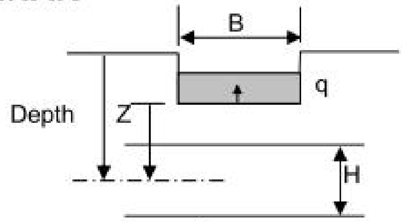


Figure 4.8: (a) Vertical effective stress profile to a depth of 10 m below ground level at Bothkennar. (b) Coefficient of volume compressibility (M_V) within upper 7 m of soil profile at Bothkennar (from BRE Bothkennar database).

FOUNDATION SETTLEMENT AT BOTHKENNAR

TRIAL FOOTING 1 - 1st load increment

Formation depth	0.5 m
Foundation width (B)	0.75 m
Applied foundation pressure, q	33.5 (kN/m^2)

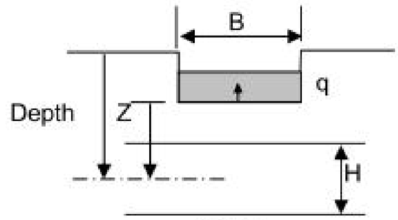


Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	Δp/q	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	11.31
2	1	23	1.5	2.00	0.28	0.5	4.69
3	1	28.5	2.5	3.33	0.14	0.6	2.81
4	1	34.5	3.5	4.67	0.09	0.45	1.36
5	1	41	4.5	6.00	0.05	0.19	0.32
6	1	46.3	5.5	7.33	0.04	0.25	0.34
7	1	52.2	6.5	8.67	0.03	0.35	0.35
8	1	58.7	7.5	10.00	0.025	0.35	0.29
9	1	64.1	8.5	11.33	0.02	0.35	0.23

Total settlement **21.7** (mm)

TRIAL FOOTING 1 - 2nd load increment

Formation depth	0.5 m
Foundation width (B)	0.75 m
Applied foundation pressure, q	72 (kN/m^2)

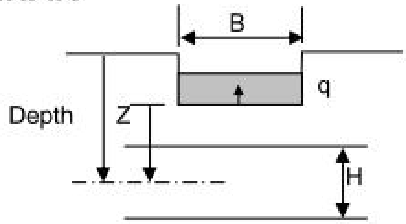


Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	Δp/q	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	24.30
2	1	23	1.5	2.00	0.28	0.5	10.08
3	1	28.5	2.5	3.33	0.14	0.6	6.05
4	1	34.5	3.5	4.67	0.09	0.45	2.92
5	1	41	4.5	6.00	0.05	0.19	0.68
6	1	46.3	5.5	7.33	0.04	0.25	0.72
7	1	52.2	6.5	8.67	0.03	0.35	0.76
8	1	58.7	7.5	10.00	0.025	0.35	0.63
9	1	64.1	8.5	11.33	0.02	0.35	0.50

Total settlement **46.6** (mm)

Figure 4.9: (a) Stress distribution and pre-treatment settlement prediction under the two main load increments for trial footing 1.

FOUNDATION SETTLEMENT AT BOTHKENNAR

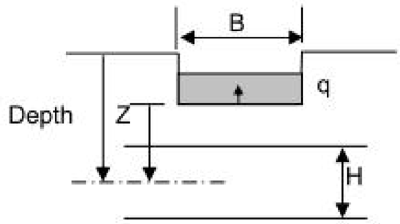


TRIAL FOOTING 2 - 1st load increment

Formation depth	0.5 m
Foundation width (B)	0.75 m
Applied foundation pressure, q	32.9 (kN/m^2)

Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	Δp/q	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	11.10
2	1	23	1.5	2.00	0.28	0.5	4.61
3	1	28.5	2.5	3.33	0.14	0.6	2.76
4	1	34.5	3.5	4.67	0.09	0.45	1.33
5	1	41	4.5	6.00	0.05	0.19	0.31
6	1	46.3	5.5	7.33	0.04	0.25	0.33
7	1	52.2	6.5	8.67	0.03	0.35	0.35
8	1	58.7	7.5	10.00	0.025	0.35	0.29
9	1	64.1	8.5	11.33	0.02	0.35	0.23

Total settlement **21.3** (mm)



TRIAL FOOTING 2 - 2nd load increment

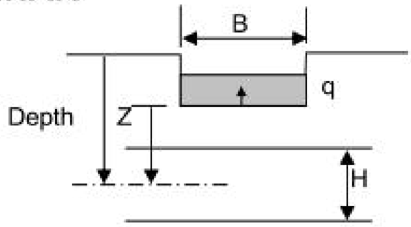
Formation depth	0.5 m
Foundation width (B)	0.75 m
Applied foundation pressure, q	67.1 (kN/m^2)

Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	Δp/q	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	22.65
2	1	23	1.5	2.00	0.28	0.5	9.39
3	1	28.5	2.5	3.33	0.14	0.6	5.64
4	1	34.5	3.5	4.67	0.09	0.45	2.72
5	1	41	4.5	6.00	0.05	0.19	0.64
6	1	46.3	5.5	7.33	0.04	0.25	0.67
7	1	52.2	6.5	8.67	0.03	0.35	0.70
8	1	58.7	7.5	10.00	0.025	0.35	0.59
9	1	64.1	8.5	11.33	0.02	0.35	0.47

Total settlement **43.5** (mm)

Figure 4.9: (b) Stress distribution and pre-treatment settlement prediction under the two main load increments for trial footing 2.

FOUNDATION SETTLEMENT AT BOTHKENNAR

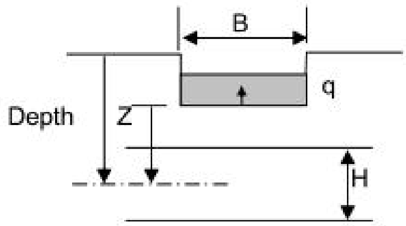


TRIAL FOOTING 3 - 1st load increment

Formation depth 0.5 m
Foundation width (B) 0.75 m
Applied foundation pressure, q 33.1 (kN/m^2)

Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	$\Delta p/q$	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	11.17
2	1	23	1.5	2.00	0.28	0.5	4.63
3	1	28.5	2.5	3.33	0.14	0.6	2.78
4	1	34.5	3.5	4.67	0.09	0.45	1.34
5	1	41	4.5	6.00	0.05	0.19	0.31
6	1	46.3	5.5	7.33	0.04	0.25	0.33
7	1	52.2	6.5	8.67	0.03	0.35	0.35
8	1	58.7	7.5	10.00	0.025	0.35	0.29
9	1	64.1	8.5	11.33	0.02	0.35	0.23

Total settlement **21.4** (mm)



TRIAL FOOTING 3 - 2nd load increment

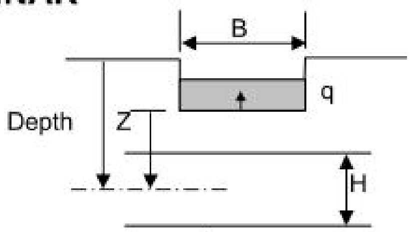
Formation depth 0.5 m
Foundation width (B) 0.75 m
Applied foundation pressure, q 67.8 (kN/m^2)

Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	$\Delta p/q$	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	22.88
2	1	23	1.5	2.00	0.28	0.5	9.49
3	1	28.5	2.5	3.33	0.14	0.6	5.70
4	1	34.5	3.5	4.67	0.09	0.45	2.75
5	1	41	4.5	6.00	0.05	0.19	0.64
6	1	46.3	5.5	7.33	0.04	0.25	0.68
7	1	52.2	6.5	8.67	0.03	0.35	0.71
8	1	58.7	7.5	10.00	0.025	0.35	0.59
9	1	64.1	8.5	11.33	0.02	0.35	0.47

Total settlement **43.9** (mm)

Figure 4.9: (c) Stress distribution and pre-treatment settlement prediction under the two main load increments for trial footing 3.

FOUNDATION SETTLEMENT AT BOTHKENNAR

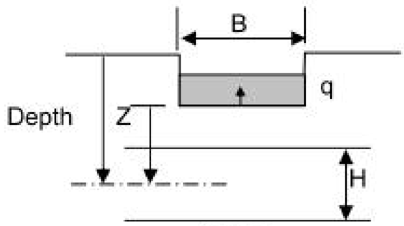


TRIAL FOOTING 4 - 1st load increment

Formation depth	0.5	m
Foundation width (B)	0.75	m
Applied foundation pressure, q	34.9	(kN/m^2)

Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	∆p/q	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	11.78
2	1	23	1.5	2.00	0.28	0.5	4.89
3	1	28.5	2.5	3.33	0.14	0.6	2.93
4	1	34.5	3.5	4.67	0.09	0.45	1.41
5	1	41	4.5	6.00	0.05	0.19	0.33
6	1	46.3	5.5	7.33	0.04	0.25	0.35
7	1	52.2	6.5	8.67	0.03	0.35	0.37
8	1	58.7	7.5	10.00	0.025	0.35	0.31
9	1	64.1	8.5	11.33	0.02	0.35	0.24

Total settlement **22.6** (mm)



TRIAL FOOTING 4 - 2nd load increment

Formation depth	0.5	m
Foundation width (B)	0.75	m
Applied foundation pressure, q	71.1	(kN/m^2)

Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	∆p/q	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	24.00
2	1	23	1.5	2.00	0.28	0.5	9.95
3	1	28.5	2.5	3.33	0.14	0.6	5.97
4	1	34.5	3.5	4.67	0.09	0.45	2.88
5	1	41	4.5	6.00	0.05	0.19	0.68
6	1	46.3	5.5	7.33	0.04	0.25	0.71
7	1	52.2	6.5	8.67	0.03	0.35	0.75
8	1	58.7	7.5	10.00	0.025	0.35	0.62
9	1	64.1	8.5	11.33	0.02	0.35	0.50

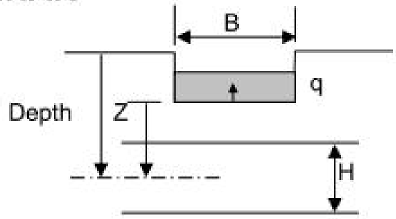
Total settlement **46.1** (mm)

Figure 4.9: (d) Stress distribution and pre-treatment settlement prediction under the two main load increments for trial footing 4.

FOUNDATION SETTLEMENT AT BOTHKENNAR

TRIAL FOOTING 5 - 1st load increment

Formation depth	0.5 m
Foundation width (B)	0.75 m
Applied foundation pressure, q	32.1 (kN/m^2)

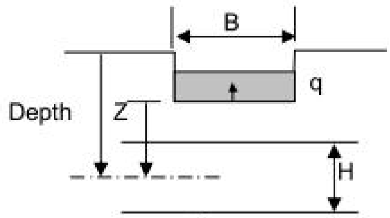


Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	∆p/q	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	10.83
2	1	23	1.5	2.00	0.28	0.5	4.49
3	1	28.5	2.5	3.33	0.14	0.6	2.70
4	1	34.5	3.5	4.67	0.09	0.45	1.30
5	1	41	4.5	6.00	0.05	0.19	0.30
6	1	46.3	5.5	7.33	0.04	0.25	0.32
7	1	52.2	6.5	8.67	0.03	0.35	0.34
8	1	58.7	7.5	10.00	0.025	0.35	0.28
9	1	64.1	8.5	11.33	0.02	0.35	0.22

Total settlement **20.8** (mm)

TRIAL FOOTING 5 - 2nd load increment

Formation depth	0.5 m
Foundation width (B)	0.75 m
Applied foundation pressure, q	67.8 (kN/m^2)



Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	∆p/q	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	22.88
2	1	23	1.5	2.00	0.28	0.5	9.49
3	1	28.5	2.5	3.33	0.14	0.6	5.70
4	1	34.5	3.5	4.67	0.09	0.45	2.75
5	1	41	4.5	6.00	0.05	0.19	0.64
6	1	46.3	5.5	7.33	0.04	0.25	0.68
7	1	52.2	6.5	8.67	0.03	0.35	0.71
8	1	58.7	7.5	10.00	0.025	0.35	0.59
9	1	64.1	8.5	11.33	0.02	0.35	0.47

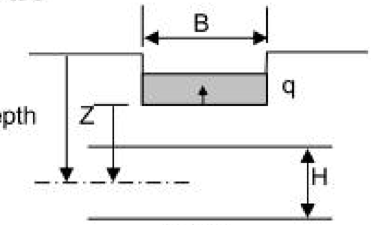
Total settlement **43.9** (mm)

Figure 4.9: (e) Stress distribution and pre-treatment settlement prediction under the two main load increments for trial footing 5.

FOUNDATION SETTLEMENT AT BOTHKENNAR

TRIAL FOOTING 6 - 1st load increment

Formation depth	0.5 m
Foundation width (B)	0.75 m
Applied foundation pressure, q	34.2 (kN/m^2)

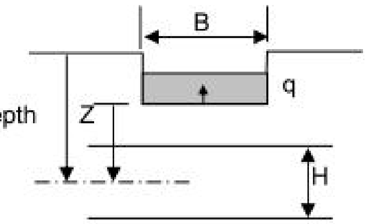


Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	Δp/q	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	11.54
2	1	23	1.5	2.00	0.28	0.5	4.79
3	1	28.5	2.5	3.33	0.14	0.6	2.87
4	1	34.5	3.5	4.67	0.09	0.45	1.39
5	1	41	4.5	6.00	0.05	0.19	0.32
6	1	46.3	5.5	7.33	0.04	0.25	0.34
7	1	52.2	6.5	8.67	0.03	0.35	0.36
8	1	58.7	7.5	10.00	0.025	0.35	0.30
9	1	64.1	8.5	11.33	0.02	0.35	0.24

Total settlement **22.2** (mm)

TRIAL FOOTING 6 - 2nd load increment

Formation depth	0.5 m
Foundation width (B)	0.75 m
Applied foundation pressure, q	69.6 (kN/m^2)



Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	Δp/q	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	23.49
2	1	23	1.5	2.00	0.28	0.5	9.74
3	1	28.5	2.5	3.33	0.14	0.6	5.85
4	1	34.5	3.5	4.67	0.09	0.45	2.82
5	1	41	4.5	6.00	0.05	0.19	0.66
6	1	46.3	5.5	7.33	0.04	0.25	0.70
7	1	52.2	6.5	8.67	0.03	0.35	0.73
8	1	58.7	7.5	10.00	0.025	0.35	0.61
9	1	64.1	8.5	11.33	0.02	0.35	0.49

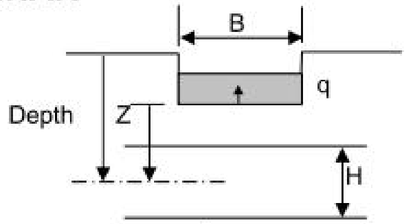
Total settlement **45.1** (mm)

Figure 4.9: (f) Stress distribution and pre-treatment settlement prediction under the two main load increments for trial footing 6.

FOUNDATION SETTLEMENT AT BOTHKENNAR

TRIAL FOOTING 7 - 1st load increment

Formation depth	0.5 m
Foundation width (B)	0.75 m
Applied foundation pressure, q	32.7 (kN/m^2)

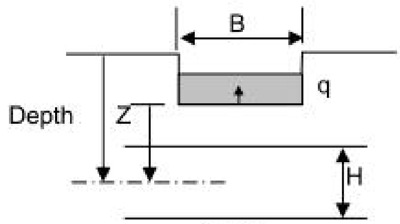


Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	$\Delta p/q$	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	11.04
2	1	23	1.5	2.00	0.28	0.5	4.58
3	1	28.5	2.5	3.33	0.14	0.6	2.75
4	1	34.5	3.5	4.67	0.09	0.45	1.32
5	1	41	4.5	6.00	0.05	0.19	0.31
6	1	46.3	5.5	7.33	0.04	0.25	0.33
7	1	52.2	6.5	8.67	0.03	0.35	0.34
8	1	58.7	7.5	10.00	0.025	0.35	0.29
9	1	64.1	8.5	11.33	0.02	0.35	0.23

Total settlement **21.2** (mm)

TRIAL FOOTING 7 - 2nd load increment

Formation depth	0.5 m
Foundation width (B)	0.75 m
Applied foundation pressure, q	67 (kN/m^2)



Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	$\Delta p/q$	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	22.61
2	1	23	1.5	2.00	0.28	0.5	9.38
3	1	28.5	2.5	3.33	0.14	0.6	5.63
4	1	34.5	3.5	4.67	0.09	0.45	2.71
5	1	41	4.5	6.00	0.05	0.19	0.64
6	1	46.3	5.5	7.33	0.04	0.25	0.67
7	1	52.2	6.5	8.67	0.03	0.35	0.70
8	1	58.7	7.5	10.00	0.025	0.35	0.59
9	1	64.1	8.5	11.33	0.02	0.35	0.47

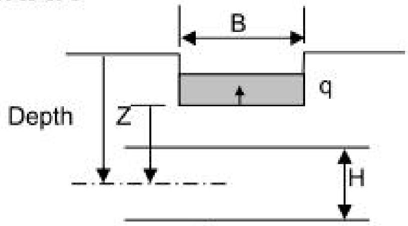
Total settlement **43.4** (mm)

Figure 4.9: (g) Stress distribution and pre-treatment settlement prediction under the two main load increments for trial footing 7.

FOUNDATION SETTLEMENT AT BOTHKENNAR

TRIAL FOOTING 8 - 1st load increment

Formation depth 0.5 m
Foundation width (B) 0.75 m
Applied foundation pressure, q 34.3 (kN/m^2)

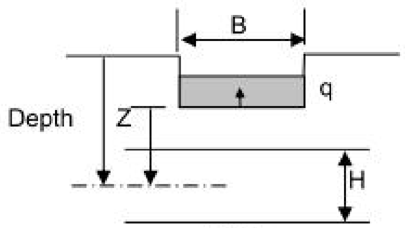


Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	Δp/q	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	11.58
2	1	23	1.5	2.00	0.28	0.5	4.80
3	1	28.5	2.5	3.33	0.14	0.6	2.88
4	1	34.5	3.5	4.67	0.09	0.45	1.39
5	1	41	4.5	6.00	0.05	0.19	0.33
6	1	46.3	5.5	7.33	0.04	0.25	0.34
7	1	52.2	6.5	8.67	0.03	0.35	0.36
8	1	58.7	7.5	10.00	0.025	0.35	0.30
9	1	64.1	8.5	11.33	0.02	0.35	0.24

Total settlement **22.2** (mm)

TRIAL FOOTING 8 - 2nd load increment

Formation depth 0.5 m
Foundation width (B) 0.75 m
Applied foundation pressure, q 71.6 (kN/m^2)



Depth (m)	Layer, H (m)	In-situ effective stress (kN/m^2)	Z (m)	Z/B	Δp/q	mv (m^2/MPa)	Layer settlement (mm)
1	1	13	0.5	0.67	0.75	0.45	24.17
2	1	23	1.5	2.00	0.28	0.5	10.02
3	1	28.5	2.5	3.33	0.14	0.6	6.01
4	1	34.5	3.5	4.67	0.09	0.45	2.90
5	1	41	4.5	6.00	0.05	0.19	0.68
6	1	46.3	5.5	7.33	0.04	0.25	0.72
7	1	52.2	6.5	8.67	0.03	0.35	0.75
8	1	58.7	7.5	10.00	0.025	0.35	0.63
9	1	64.1	8.5	11.33	0.02	0.35	0.50

Total settlement **46.4** (mm)

Figure 4.9: (h) Stress distribution and pre-treatment settlement prediction under the two main load increments for trial footing 8.

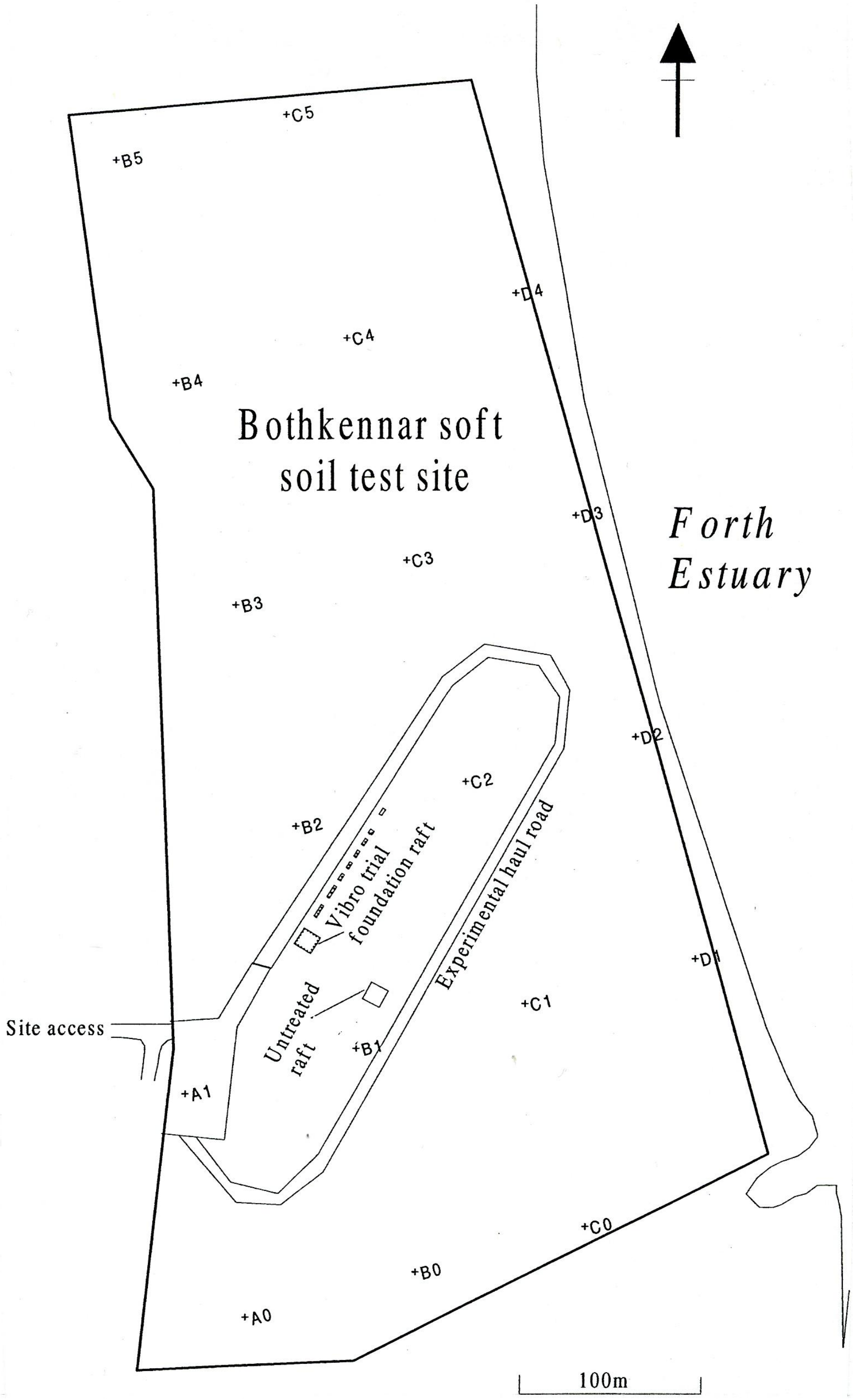


Figure 4.10: (a) Location of the trial footings in the context of the Bothkennar site.

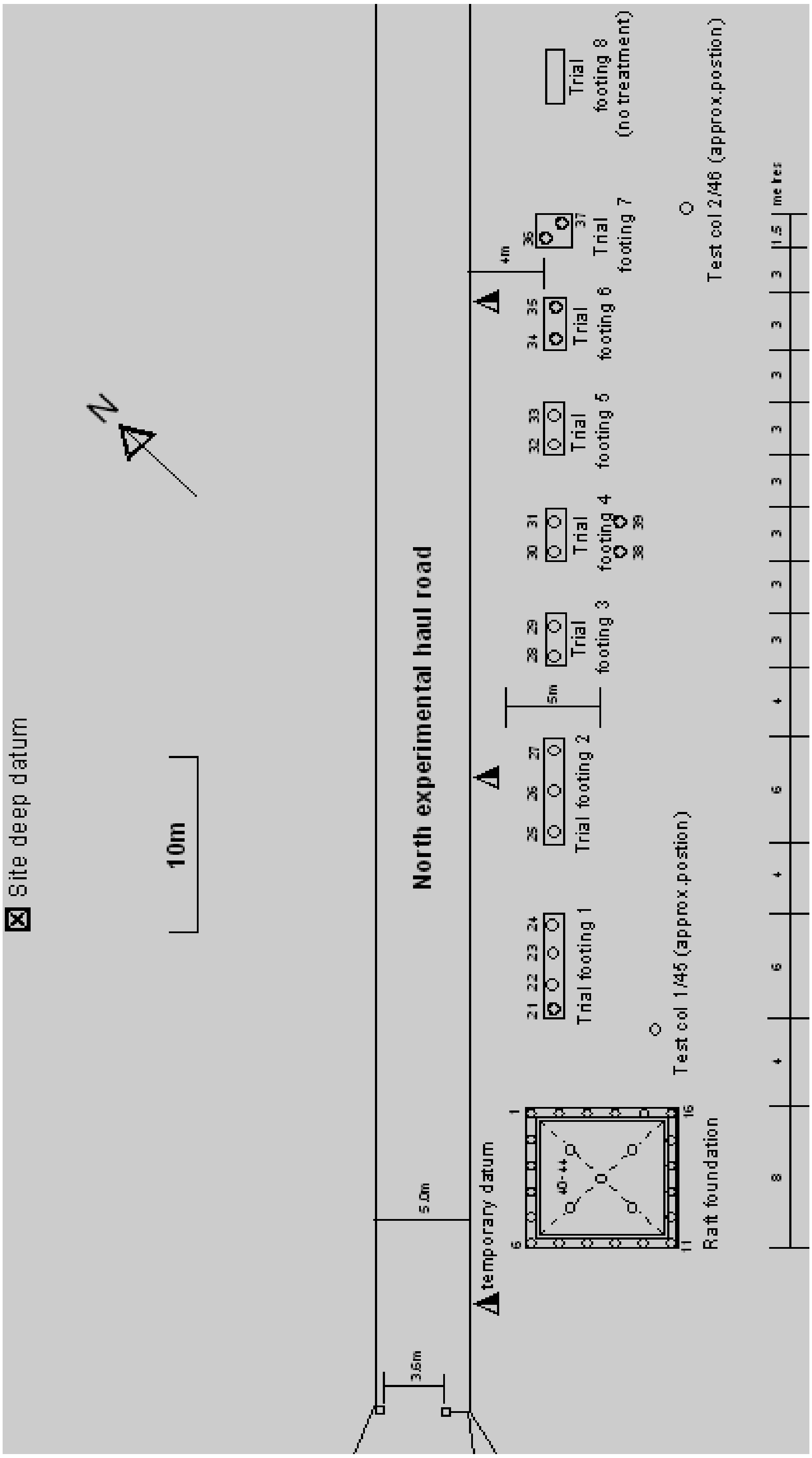


Figure 4.10: (b) Location of the trial footings in the context of the Bothkennar soft clay research site.

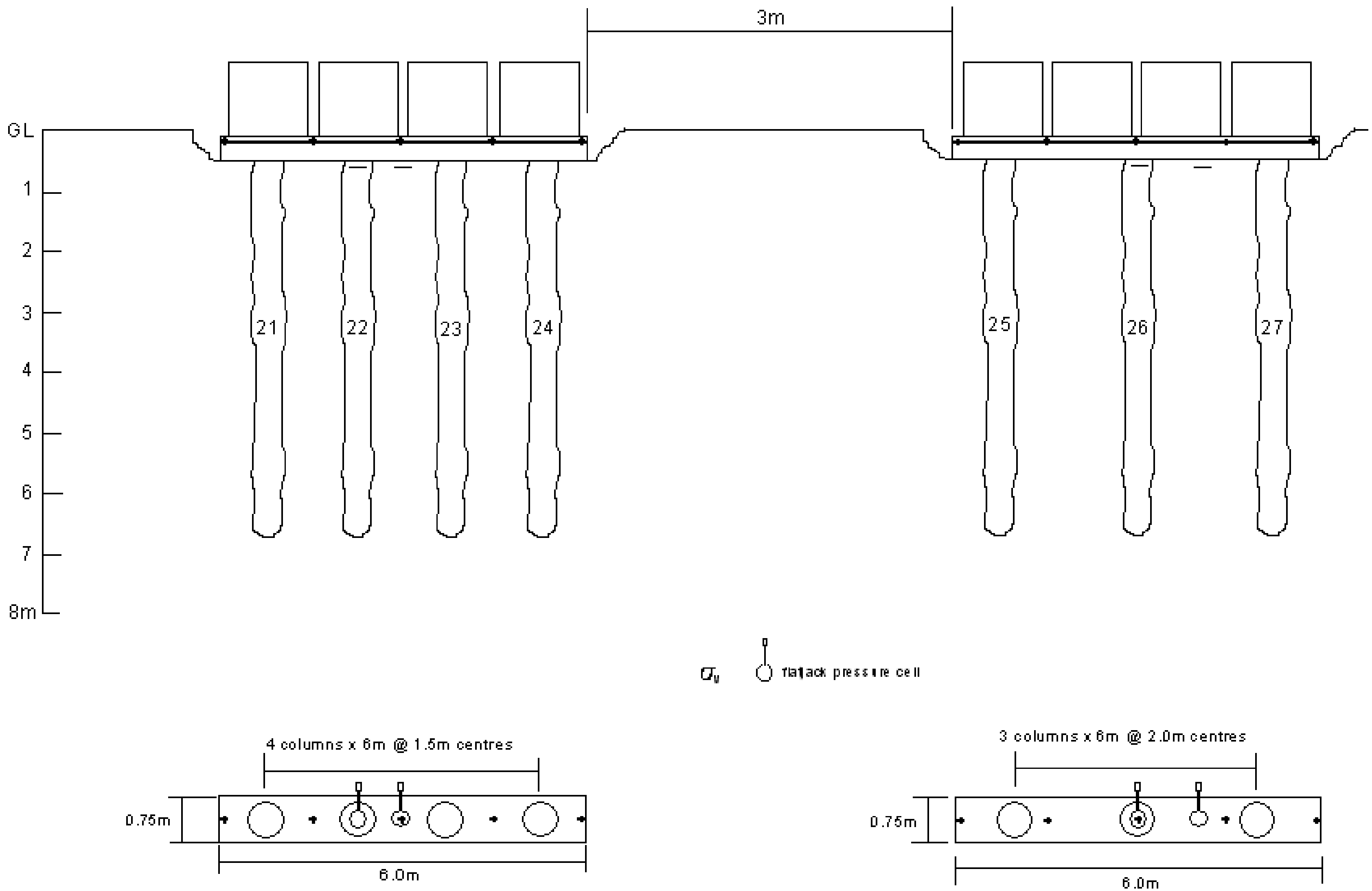


Figure 4.11: (a) Investigation of stone column spacing beneath trial footings 1 and 2 (including instrumentation locations - settlement monitoring stations represented by + symbol).

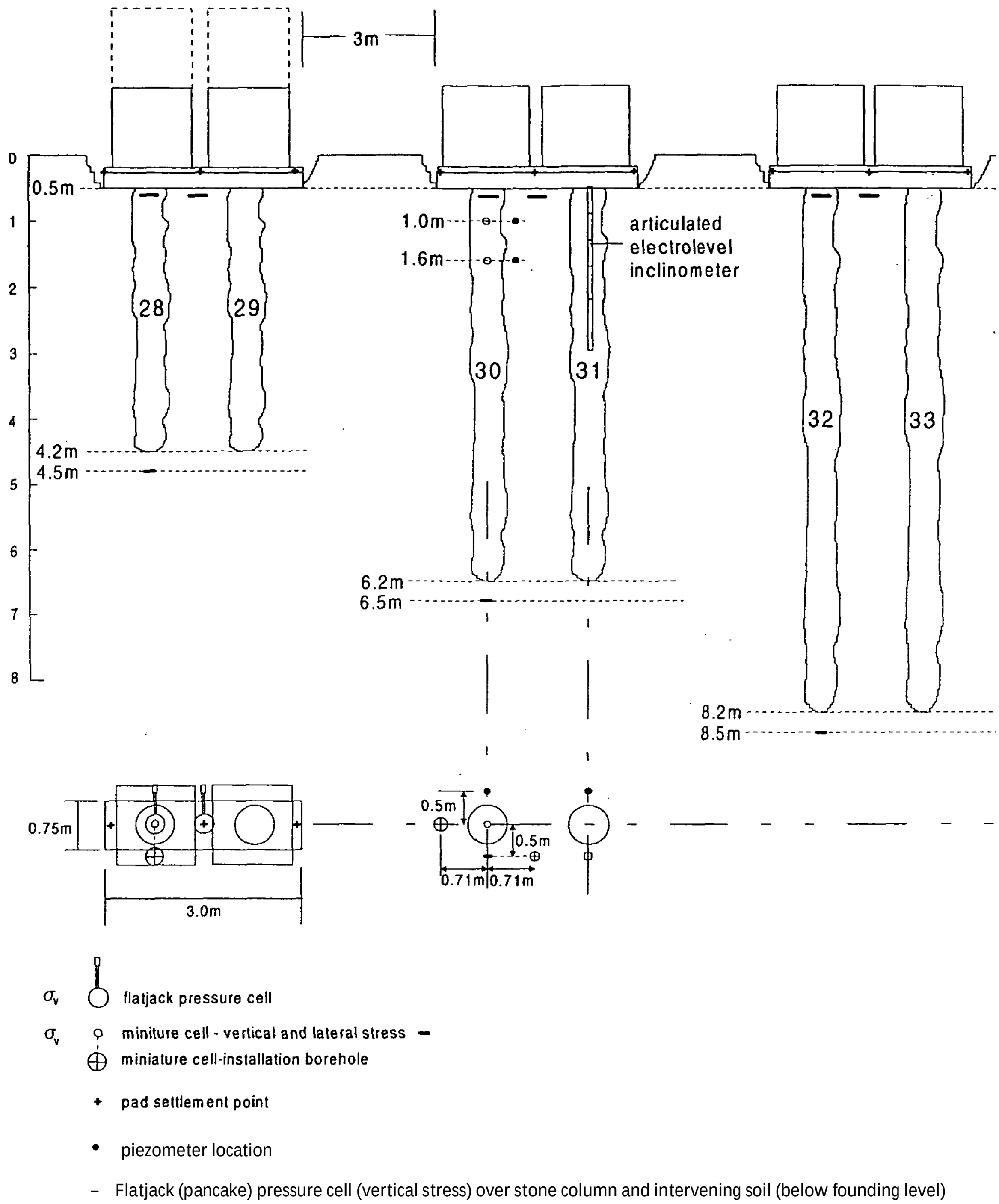
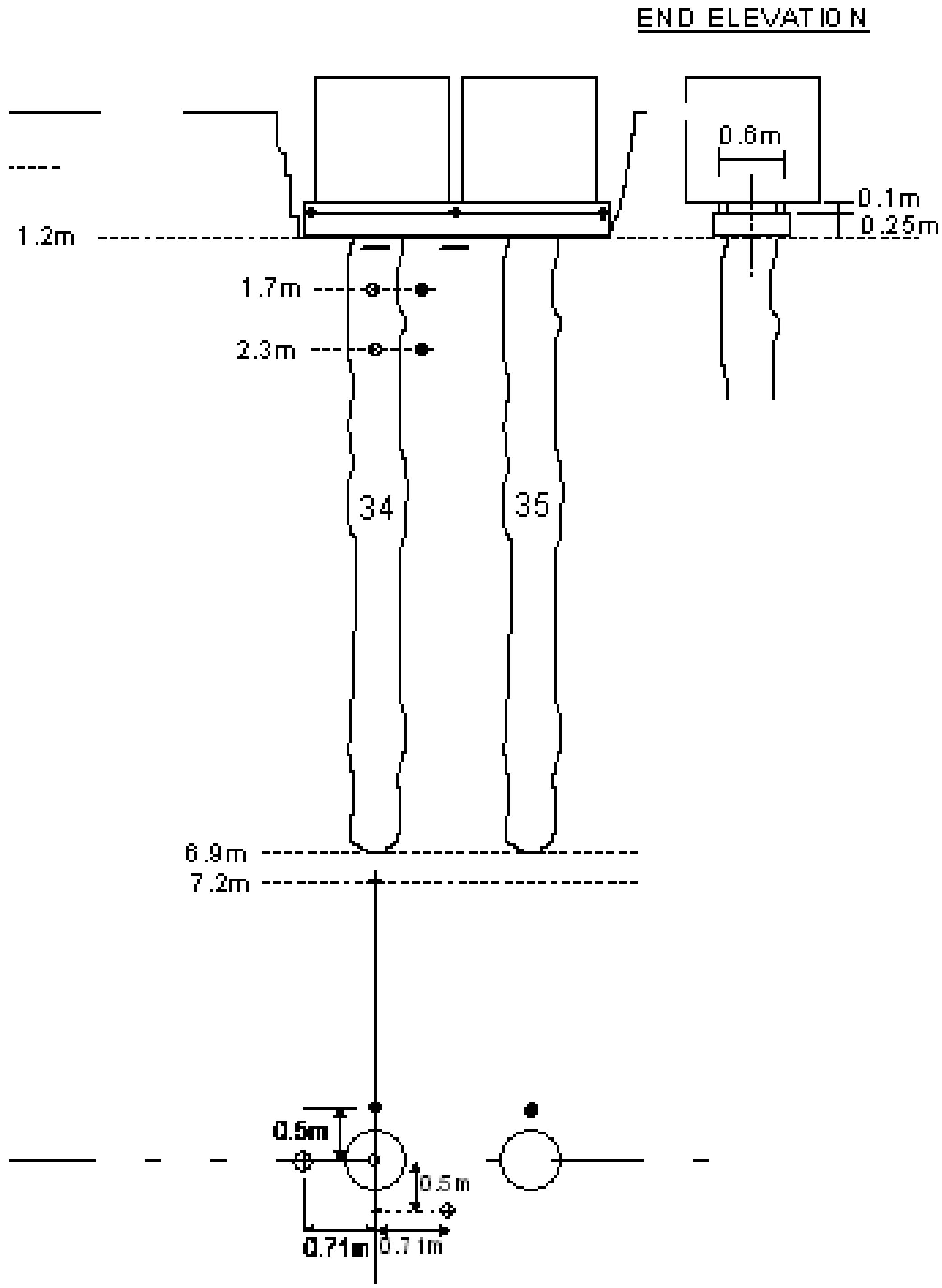


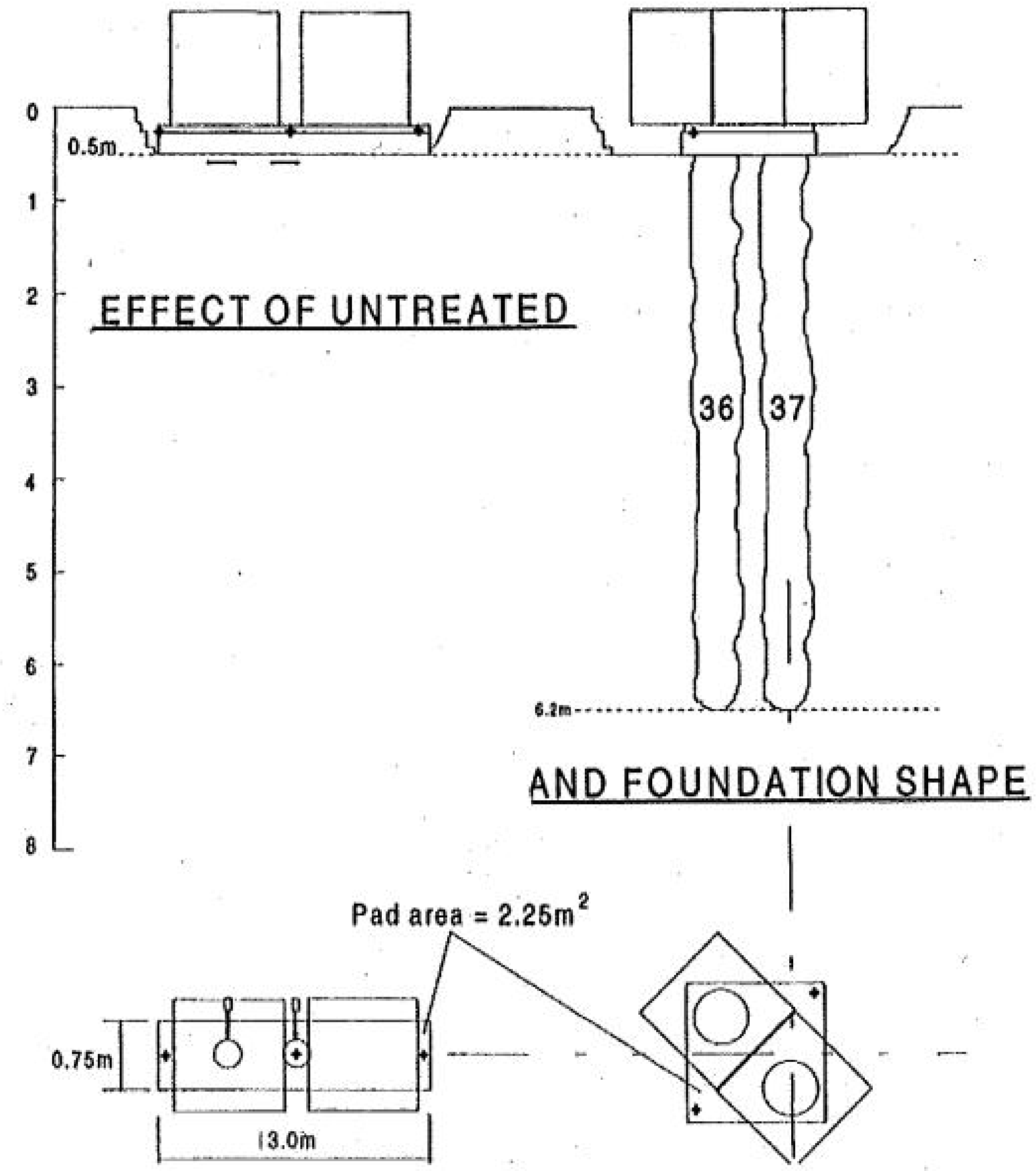
Figure 4.11: (b) Investigation of variation in stone column length (treatment depth) beneath trial footings 3-5 (instrumentation locations annotated).



○ = Miniature pressure cell (vertical and lateral stress measurements)

• = Piezometer location ; — Flatjack (pancake) pressure cell (vertical stress)

Figure 4.11: (c) Investigation of 'crust' effect – trial footing 6 (including instrumentation locations).



d) e)

- Flatjack (pancake) pressure cell (vertical stress) ; + levelling points

Figure 4.11: (d) Investigation of absence of treatment - trial footing 8 and (e) footing shape - trial footing 7 (including instrumentation locations).

(a)

(b)

Figure 4.12: Bothkennar trial footing construction (a) 0.5 m depth in crust (trial footings 1-5 and 8) and (b) 1.2 m depth in crust (trial footing 6)

Chapter 5 Field equipment, soils, materials and test procedures.

5.1 Stone column installation equipment

A Bauer HBM4 dry bottom-feed stone column installation rig was employed for the trials (Figure 5.1) and is a large multi-purpose rig weighing approximately 70 tonnes – (700 kN), comparable to medium to large piling plant when used for vibro stone column applications. Although designed to impart a relatively low-ground bearing pressure via its tracks, it would normally only operate from a temporary platform (piling mat) of granular material, comprising clean crushed rock or recycled demolition rubble such as crushed concrete or brick. However, apart from the cost implications, this would also have posed some practical difficulties for some site activities (notably instrumentation and monitoring), associated with the field trials. By carrying out the stone column installation at the end of the summer period, and undertaking routine bearing capacity calculations using the rig bearing pressures provided, the desiccated 'crust' was found to be sufficiently competent to support the installation rig safely.

5.2 Stone column aggregate

Based upon the requirements outlined in Section 4.2.3, the aggregate selected for the trials was a sub-rounded to rounded 40 mm single size gravel aggregate supplied from the Avondale Quarry, of Rufford Top Dress aggregate suppliers at Polmont in Falkirk, Stirlingshire. It was a hard and inert material with a significant hard sandstone and quartzite component. A stockpile of the material on site is illustrated in Figure 5.2 together with a grading analysis for the aggregate.

The apparatus used to determine the angle of shearing resistance of the stone column aggregate used at Bothkennar was the large triaxial test equipment housed in the Geotechnics laboratory at the Building Research Establishment in Garston. The testing facility utilises commercially available cells and loading frames manufactured by Engineering Laboratory Equipment (ELE) Ltd. The Laboratory has two cells and loading frames for 228 mm diameter samples having a nominal height of 500-550 mm.

The cells have axial load capabilities of 10t and 50t, with two loading frames available to compliment the two cell capacities. Figure A5.1.1a in Appendix 5.1 shows the aggregate sample within a 10t triaxial cell in the 50t frame, together with data logging equipment. The test data was fully logged and data downloaded to spreadsheets for calculation. The sample was tested under drained conditions with measurement of change in cell water volume preferred to the measurement of air and water draining out of the sample for the calculation of sample volume change. This is due to complications and inaccuracies associated with mixed air-water flow and volume measurement. Cell pressure is developed using a precision regulated air supply to an air-water interchange vessel. The pressure feed then proceeds through an automatic volume change measuring device designed specifically for the application and which electronically measures the volume flow through the system and is controlled automatically to facilitate long term testing. The volume of water and air draining from the sample is measured using a purpose-built combined air trap-measuring cylinder. Pore pressure and cell pressure are measured by Bell and Howell pressure transducers while the applied axial load was measured by both an internal cell mounted on the deviator ram and also an external proving ring. Several membranes are required outside the inner 'sacrificial' one. At the level of sample strength dealt with in the tests, membrane correction is small and can be largely neglected. The initial and final state of the test specimen is presented in Figure 5.3.

The results of the triaxial tests are presented in Figure A5.1.1b of Appendix 5.1. It should be noted that the derived average value, φ' (peak) = 46.6°, was higher than values reported by Leslie (1963) (φ' = 42°) and by Mitchell (1981) (φ' = 41°). Both Leslie (1963) and Mitchell (1981) used triaxial apparatus to obtain the angle of shearing resistance values commonly quoted for medium dense to dense gravels. This is not unexpected given that all techniques of vibro stone column construction aim to leave the gravel well compacted in-situ. Furthermore, Thomson (1987) recommended a value of 42° and 45° respectively for high quality well compacted rounded and angular aggregates. Mc Cabe et al. (2009) indicate that a design friction angle (φ' = 40°) is a conservative assumption in the case of the dry bottom-feed system. Interestingly, Herle et al. (2008) advocate the use of higher φ' values, i.e. in excess of 50°, than are commonly adopted in UK design, but which were based on shear box tests carried out on stone aggregate at high relative density levels however. It is clear that various test procedures to determine angle of shearing resistance in stone column material have their pros and cons. In the absence of more sophisticated test procedures at the time the current methods, particularly the triaxial tests, appear to produce results which are within an acceptable range.

During this research it was evident that the shear strength parameters of aggregates are also commonly determined using a large (300 mm x 300 mm) direct shear box and conducting drained (effective stress) tests. Head (1994) suggests that the apparatus is suitable for materials containing particle sizes up to 50 mm. It should be emphasised that both the triaxial and the direct shear box tests have their limitations. In the triaxial tests the principal axes of stress and strain are fixed and correspond to the axes of the apparatus. In the direct shear box tests there is no information about the stresses on other planes other than the horizontal plane of the sample; in particular the vertical plane. Clearly further development and research is required on these aspects, but is not practical or feasible within the scope of the current research and one has to work with current equipment and state-of-the-art.

In order to err on the side of caution, and recognising the fact that laboratory compacted aggregate has different boundary conditions to the field situation, the recorded angle of shear resistance (φ') of 46.6° for the stone column aggregate to be utilised in the Bothkennar field trials was down-graded to a value of 42.5°, as this is a commonly used (maximum) value in soft ground, certainly in the UK. Furthermore the value is in close agreement with the laboratory determined angle of shearing resistance values reported by Mitchell (1981) and Thomson (1987) referred to previously.

A bulk density value of $18kN/m^3$ was adopted for the stone column aggregate to be used in the (Bothkennar) field trial(s) derived from a laboratory based large compaction test on a bulk sample of the aggregate and which was in line with reported values in BS8002 (1994).

5.3 Bothkennar soft clay (research) test site

5.3.1 Site description and geology

The Bothkennar soft clay research site is a low-lying level field of some 11Ha in size and with an elevation of between 2.45 m and 3.1 m AOD. The tidal range in the Forth is currently 5.0 m, with mean high spring tide being +2.86m OD. Consequently the site is bounded on three sides by flood protection bunds. The soil profile essentially comprises up to 20 m of soft saturated fine-grained (clay/silt) soil deposits. Because of its research applications the Bothkennar soft clay research site has been subject to a number of comprehensive site investigations and the soft clay deposit is well-documented geotechnically (e.g. Institution of Civil Engineers Géotechnique Symposium in Print, 1992; Nash et al., 1992a; Hight et al., 1992).

The engineering geology of the Forth Valley has been investigated extensively by a number of researchers; Sissons (1966;1969), the British Geological Survey (BGS) have published a number of reports and engineering geological maps- Gostelow and Lambert (1979), Browne et al. (1984), Gostelow and Browne (1986). The Quaternary geology of the Forth Valley including the Bothkennar area have been further investigated by Hawkins et al. (1989), Nash et al. (1992a) among others, comprising a well documented stratigraphy, with Late glacial deposits – a succession of glacial till (boulder clay), laminated clay and gravel present between rockhead and the Post glacial deposits, Figure 5.4. The Bothkennar site lies within the outcrop of the Holocene raised estuarine deposits, locally termed 'Carse Clays', which occur widely at the head of the Forth Estuary. Gostelow and Browne (1986) indicated that the Post glacial Carse Clays of the Forth Valley can be divided into three main units (Figure 5.4) – an upper desiccated horizon ('crust') overlying a weathered Carse Clay horizon, in turn underlain by unweathered Carse Clay. The Post Glacial Holocene sequence overlies the Late-glacial Bothkennar Gravel Formation (BGF), which is present throughout this area (Sissons 1969; Browne et al., 1984), and at the Bothkennar soft clay research site occurs as a gravelly sand at an elevation of around – 13 m to – 19 m. OD. General stratigraphy is also summarised in Figure 5.5a-d.

The upper Carse Clay deposits are the principal stratigraphic unit being investigated in this research programme at Bothkennar and will be subsequently referred to as the 'soft clay' or the 'Bothkennar Clay'. The upper desiccated layer will be referred to as the 'crust'. The unweathered Bothkennar Clay is terminated upwards by an erosion surface which is believed to represent a former intertidal surface. Upon this surface is a discontinuous shell bed, radio carbon dated at around 3000 ^{14}C yrs BP. Above this lies a thin unit of clayey silt ('crust') containing lenses of detrital shell material and disseminated shell debris which extended from around 2 m AOD to the modern day ground surface of around 3 m AOD.

5.3.2 Properties of the 'crust'

Although the crust is less than 1.5 m thick, it was intended to use this as a founding layer in a significant component of the proposed field trials. As previously intimated, an important pre-requisite was to obtain a reasonable suite of geotechnical data for the crust, but within the budget constraints of the field trials, and taking account of the fact that very limited geotechnical data existed for the crust prior to this research. Barras and Paul (2000) state that the crust at Bothkennar has developed during the past 200 years by three sets of processes. The sediments of the Saltgreens Member (Figure 5.5d) accumulated over a three year period (*c.* 1784-87) by artificially induced settling. They consolidated under their own weight and also induced an additional, slower consolidation of the underlying sediments of the Claret Formation (Upper Carse Clay). During this stage Barras and Paul (2000) considered that these soils probably remained fully saturated, except perhaps in their uppermost (few) 10-50 mm and so their water content was controlled by self-weight compression. Following this stage, the introduction of artificial drainage created an upper, partially saturated zone in which effective stress was thus increased by soil suction, probably principally by plant evapo-transpiration during periods of drought. This resulted in an additional volume reduction and a moderate (150-200 kN/m^2) degree of over-consolidation. Concurrently with these stages, the profile became desalinated, acidified and oxidized to an extent controlled by depth, freshwater inflow and aeration. Barras and Paul (2000) concluded that the physical development of the crust was rapid and is now largely completed, whereas its chemical development is not yet completed.

The shallow trial pit investigation previously referred to (Section 4.1) was extended to a depth of around 1.5 m to expose the full depth of the crust (approximately 1.1-1.2 m in thickness within the trial area). From inspection of an exposed face in the crust the soil was described as a firm or firm to stiff brown very silty clay with pockets of weakly cemented rust-brown silt. In some parts of the exposed face the soil could be described as firm or firm to stiff brown very clayey silt with pockets of weakly cemented rust-brown silt. It was apparent therefore that the crust was transitional between a clay and a silt. Representative hand shear vane tests were undertaken within the crust and show the undrained shear strength to vary substantially - from about 120 kN/m^2 at 0.2 m depth to 40 kN/m^2 towards the base of the crust. Two undisturbed samples of the crust were recovered from a depth of 0.6 m for one-dimensional (1-D) consolidation testing. The results are detailed in Appendix A5.2 (Figure A5.2.1 a and b and Figure A5.2.2 a and b). The Coefficient of volume compressibility (M_v); the Coefficient of consolidation (C_v) and the voids ratio indicate the soil to be at the upper end of the medium compressibility range. The curved nature of the semi-log plots – voids ratio versus log of pressure (kPa) within the range 0-160 kPa (Appendix A5.2), are indicative of a lightly over-consolidated clay, the resulting compressibility being much less than the underlying softer normally consolidated Bothkennar clay (Claret Beds). The liquid and plastic limits (and therefore plasticity index) were determined for a recovered sample of the crust (also recovered from a depth of 0.6 m). The test results are detailed in Appendix A5.2: Figure A5.2.3 – the Liquid and Plastic Limits for the crust are 49% and 28% respectively and with its Plasticity Index and Liquidity Index being 21% and 0.24 respectively. On the Plasticity chart the soil lies just below the A-line which forms the boundary between clay and silt of intermediate plasticity and the soil was therefore designated MI (silt of intermediate plasticity). Moisture content was generally lower than for the underlying soft clay and was close to the plastic limit near the surface.

The foregoing geotechnical data for the crust add to the existing database for Bothkennar which, when combined with the extensive suit of values available for the geotechnical properties of the underlying soft (Bothkennar) clay, proves particularly useful for predictions of pre- and post- (vibro stone column) treatment load-settlement behaviour, together with stone column bearing capacity. Key geotechnical parameters for the crust, based upon the above comments and investigations are summarised in Table 5.1.

5.3.3 Properties of the soft (Bothkennar) Clay

As indicated previously, soil conditions at Bothkennar could be considered as marginal for stone column installation, hence it was important to identify the more significant soil properties which would influence stone column behaviour during the field trials, in the context of both ground response to column installation and also load-settlement performance of the stone column reinforced ground.

Groundwater level is normally close to the surface at Bothkennar and during the trial period ground water level varied from - 0.3 m to ground level. A summary of geotechnical parameters from the BRE database for the Bothkennar soft clay site is given in Figure 4.3. A more detailed description of the more pertinent soil properties and which are considered to be relevant to the field trials are given below:

Soil classification test data

A firm to stiff silty clay crust about 1.1-1.2 m thick is immediately underlain by a thin (approximately 300 mm thick) band of shells in a soft clay matrix (Figure 5.6a). Below the shelly band is soft dark grey very clayey silt/very silty clay. Particle size distribution (PSD) curves (Figure 5.6b) agree with visual descriptions; within the depth range under consideration in the field trials, there is typically around 30-40% clay content, 50-60% silt content and hence a small sand component.

Soft clays are usually normally consolidated to slightly over-consolidated, with high voids ratio and low dry unit weight. According to Das (1990), typical voids ratios lie in the range 0.9-1.4 and with dry unit weights being in the range 11.5-14.5 kN/m^3 and which are typical of the Bothkennar Clay. Data on variation in bulk density within the Bothkennar Clay soil profile obtained by Hight et al. (1992) and Lloyd (1989) are shown in Figure 5.7, together with data on water content profiles. The bulk density reduces with depth below the crust to reach a minimum of 1.57 Mg/m^3 (which equates to a very low dry density (< 1 Mg/m^3)) at about 5 m depth and then increases with depth beyond this point, with the rate of increase apparently higher below 12.5 m. Typical values of p bulk and G_s indicate a voids ratio of around 2, i.e. very loose (and indicative of a potentially sensitive soil, see section 4.2.1).

The Bothkennar clay has an organic component as shown in Figure 5.8a (with specific gravity profile also provided in Figure 5.8b) and with reported variations present in the soil profile shown, attributable to the differing methods of analysis used.

Loss on ignition tests have yielded higher organic content values (typically between 3 and 5%) than values obtained by chemical analysis. Paul et al. (1992) noted a lack of plant or fibrous material in the Bothkennar Clay and concluded that the recorded organic component is in the form of a residue of marine organisms, which have attached themselves to the soil clay particles.

Liquid and plastic limits of around 40% with moisture content close to the liquid limit have been observed within the upper 10 m of the soil profile. Initial recorded plasticity indices are around 40% (Figure 5.9). Hawkins et al. (1989) have shown that the Atterberg limits of the Bothkennar Clay are strongly affected by whether or not drying occurs and/or the type of drying that occurs before measurements are made in the laboratory. As might be anticipated, Paul et al. (1992) established that this effect is a consequence of the organic content of the Bothkennar Clay, with the temperatures associated with the oven drying being clearly sufficient to burn off the organic component. These authors showed that the Bothkennar Clay has anomalously high and variable plasticity, compared to other normally consolidated soft clay profiles in the UK (e.g. Thames Estuary), which results from its organic content and which gives rise to misleading values of apparent activity and poor correlation of parameters with plasticity index. After removal of the organic content, the plasticity index lies between 18% and 22% which is typical for an inert silty soil and can generally be classified as silt of intermediate to high plasticity; material retained in its natural state i.e. with retention of organics, is classified as clay or silt of high to very high plasticity. The clay mineralogy does not appear to have a significant impact on the above properties however.

Variation in stiffness of the Bothkennar Clay with depth is given in Figure 5.10a. The undrained shear strength (Figure 5.10b and c) of the Bothkennar Clay increases in direct proportion to effective overburden pressure (σ'_{vo}) from 20 kN/m^2 immediately below the crust, to 50-55 kN/m^2 (firm) above the gravel beds (BGF), encountered at depths ranging from 14 m to 22 m below existing ground level. Within the depth range anticipated for the field trials, i.e. upper 8.5 m, the undrained shear strengths range from around 20 to 35 kN/m^2. This appears fairly typical for soft clays in the UK and also shows some degree of conformity to Skempton's C_u/p' equations. However, it is important to note that the undrained shear strength of the Bothkennar Clay is higher than would be expected for comparable deposits, particularly for a normally consolidated to lightly over-consolidated clay with such a high plasticity index, i.e. P.I. = 40% for the soil in its natural state. It is probable that the high undrained strength is linked to high effective strength parameters and it has been suggested that these may arise from bonding of the clay due to ageing effects, together with a significant 'angular' silt component contributing to a very high effective angle of shearing resistance (about 34˚) with recorded high angles of shearing resistance (φ') measured in both triaxial compression and extension of 37˚ and 42˚ respectively. A high angle of shearing resistance was also measured in residual conditions - φ'_r of 30˚ in a ring shear apparatus. Although the soil Bothkennar Clay is termed a clay, it is perhaps more analogous to the 'slimes' one associates with tailing dams.

Soft clays are typically characterised by low permeability, i.e. equal to or less than 10^{-7} m/s. Within the Bothkennar clay there is an apparent well defined variation of vertical permeability (k_{vo}) with depth. The horizontal permeability (k_{ho}) shows a similar trend but subtle variations in the fabric of the clay with depth become more significant. The profiles of vertical and horizontal permeability at in situ void ratios are presented in Figures 5.11a and b. It is noticeable that there is a scatter of results at particular depths, depending upon whether field or laboratory methods were utilised to obtain the data, which may be a reflection of sample disturbance issues for soil samples recovered for laboratory testing.

Soft clays are also known for their high compressibility and therefore large settlements are to be expected when load is applied to these soil types. Whilst Compression index (C_c) is arguably a better parameter to consider for weak normally consolidated clays, since in theory it should be constant over a large stress range, the coefficient of volume compressibility (m_v) (or the reciprocal of this, i.e. soil stiffness modulus (E_s)), tends to be a more commonly used parameter in stone column design, for assessment of pre-treatment consolidation settlement in soft clays, before making allowance for the 'reinforcing' effects of the stone columns. Values of m_v for the Bothkennar Clay have been measured in consolidation tests on 'undisturbed' samples (Nash et al.,1992a) and are summarised in Figure 5.11c. This shows a decrease of compressibility with depth from around 1.0-1.1 m^2/MN (which classifies as a soil of high compressibility, Tomlinson (1995), Head (1994)) at around 2.0 m depth, to between 0.25 and 0.50 m^2/MN at around 15 m depth. Considerable variation of the compressibility around yield has been demonstrated and is discussed more fully by Nash et al. (1992b).

Bothkennar Clay is known to be susceptible to creep and to be anisotropic, sensitive and lightly cemented, i.e. it possesses a 'structure' (Hight et al., 1992 and Clayton et al., 1992). As intimated previously in section 5.3.3. high effective strength parameters may arise from bonding of the clay due to ageing effects, together with a significant 'angular' silt component contributing to a very high effective angle of shearing resistance. The structural component of resistance in the Bothkennar Clay has been shown to be reduced by shear strains or by volume strains (i.e. disturbance forces). Destructuring by shear at constant water content results in shrinking of the soils' initial bonding surface, which is manifest as a reduction in vertical yield stress in oedometer tests and a reduction in peak strength. Progressive destructuring has been demonstrated in experiments described by Clayton et al. (1992), for example. Clayton et al. (1992) conducted triaxial tests on natural Bothkennar clay from a depth range of 6.5-8.5 m, incorporating three distinct geological facies. Testing was conducted using axial and radial strain gauges. Results from tests on Laval samples showed that the breakdown of bonding is progressive, with the stress-strain response suggesting a stick-slip phenomenon where the soil structure undergoes a series of collapses, as particle bonds are destroyed, followed by stiffer behaviour. They also showed that the outer yield surface (structure surface) of the soil collapses towards the stable state boundary surface for the reconstituted material. In addition, results indicated that upon plastic straining the virgin compression line asymptotically approaches the intrinsic compression line, as expected for a structured material. The authors tentatively suggested that plastic volumetric strains were more influential than plastic shear strains in the destructuration of this soil. Hight et al. (1992) also gave an indication of the influence of sampling techniques on soil structure, (Figure 5.12a and Figure 5.12b). Figures 5.12a and b clearly shows the disturbance caused by different sampling methods employed by different researchers at Bothkennar in attempts to obtain 'undisturbed' samples of the Bothkennar Clay for subsequent laboratory analysis, including compression testing. It was noted that sampling techniques thought to cause greater disturbance caused a reduction in the initial stiffness and the peak undrained shear strength and cause a general shrinking of the yield surface. The Sherbrooke samples appeared to retain more structure than equivalent Laval samples, although both of these samples retain much more structure than conventional tube samplers.

Parameters that are sensitive to disturbance include (in order of decreasing sensitivity): vertical yield stress; peak strength; pre-yield and post-yield compressibility; small-strain stiffness. The fact that the Bothkennar Clay is structured and that the structural component of resistance is easily disturbed due to shear and volumetric strains has a number of implications including soil disturbance and re-moulding during installation of partial depth vibro stone columns, particularly in the zone where basal 'end bulbs' are constructed in the Bothkennar Clay.

Normally consolidated clay very often exhibits sensitivity values of 1.0 to 4.0 and in most cases up to 8.0 (Nash et al., 1992a). Heavily over-consolidated clays and most glacial tills (boulder clays), are insensitive, so that S equals 1.0 (unity). The sensitivities measured in-situ at Bothkennar using the field vane and in the laboratory using the fall cone are plotted in Figure 5.13c together with in-situ void index, Figure 5.13a and the liquidity index, Figure 5.13b. The sensitivity measured with the fall cone varies with depth with a general increase between depths of 7 m and 14 m. Within this depth range, sensitivity values typically range from 7 to 15 (very high – very sensitive). Between 2.5 m and 6.0 m depth, sensitivity values typically range between 5 and 8 which is still high (high to sensitive). These values confirm that the Bothkennar Clay is sensitive and with the presence of some structuring or fabric which is subject to disturbance (resulting from volume strains). The above comments are compatible with the comments made earlier in section 5.3.3 regarding high voids ratio and comments on degree of 'looseness'. In the context of the field trials, this also implies the possibility of significant reductions in strength due to soil disturbance (remoulding) and shearing during the installation of columns, as already eluded to. Reference should also be made to Figure 4.4 which shows a similar diagram annotated with upper limits of soil sensitivity for stone columns suggested by Baumann and Bauer (1974) and Goughnour and Bayuk (1979a).

5.3.4 In-situ conditions

The piezometric profile at the (Bothkennar) site is approximately hydrostatic with a measured piezometric level throughout the clay and in the Bothkennar Gravel Formation of 0.5-1.0 m below ground level, with small (seasonal) fluctuations.

Soft clays have a high water content (Bjerrum, 1967). Das (1990) suggested a value ranging between 30 and 50%, generally at or close to the liquid limit. At Bothkennar there is a well defined trend in the variation of water content with depth (Figure 5.7a and 5.7b). Water (moisture) content increases from approximately 30-55% at 1.2 m depth to maximum values of around 65-80% at 7 m which is very high and indicative of a high voids ratio and a sensitive clay (see section 5.3.3). Below 7 m depth moisture content reduces, at a rate that accelerates below 14 m, but then reducing to around 45% just above the Bothkennar Gravel Formation (BGF) at 20 m.

A number of measurements have been made of in-situ stiffness of the Bothkennar Clay using self-boring pressuremeters and geophysical methods – some results are presented in Figure 5.10a. Values of the shear moduli determined over a strain range of 0.34% demonstrate, as might be expected, that stiffness increases approximately linearly with depth. This is typical of other normally consolidated soil profiles found in the UK and is mirrored by the increase in undrained shear strength with depth (Figure 5.10 b and c).

The in-situ vertical stress σ_{vo} (Figure 5.14a;b) has been calculated on the basis of the variation in bulk density (Figure 5.7c) and assumption of hydrostatic conditions with the water table 0.75 m below existing ground level (Hight et al., 1992). Estimates of in-situ horizontal stress (σ_{ho}) for the Bothkennar Clay were made on the basis of lift-off pressures in self-boring pressuremeter tests, and measurements from spade cells, pushed in below the base of boreholes by BRE. The apparent variation in σ_{ho} with depth is considered to have been influenced by soil composition, depositional history and nature of instrumentation and method of measurement (Lloyd, 1989). The total stress distributions shown in Figure 5.14a have been combined to produce the variations with depth of in-situ vertical and horizontal effective stresses and of K_o (equal to $\sigma_{ho'}/\sigma_{vo}$), shown in Figures 5.14b and 5.14c. The general trend is that K_o is high in the crust (corresponding to some degree of over-consolidation) clearly demonstrated previously and is less than 1.0 below it, decreasing slightly with depth, indicative of a more normally consolidated to lightly over-consolidated clay profile. It is worthy of note, however, that agreement between different in-situ tests is poor and it is not possible to draw more definitive conclusions about K_o other than to say that it probably lies between 0.6 and 0.9 for most of the soft clay soil profile.

The geological history of Bothkennar suggests a maximum unloading due to erosion of approximately 15 kN/m^2 (see section 5.3.2). This would have given rise to an over-consolidation ratio (OCR) which reduces throughout the depth of the deposit from 1.25 at circa 5 m to 1.15 at 15 m. Fluctuations in groundwater level have occurred, with a possible maximum lowering of 3.5 m (in the past), but it seems unlikely that this episode of groundwater lowering would have coincided with the erosion: it thus follows that maximum OCR's slightly higher than those quoted and reducing with depth, could occur. One dimensional consolidation tests, Nash et al. (1992b) and investigations of yielding, Smith et al. (1992) indicate an apparent OCR of 1.4 to 16 over the full depth of the soft clay. Leroueil et al. (1992) indicate the Bothkennar Clay has an OCR of approximately 1.5. The fact that the apparent OCR exceeds that attributable to stress history is further evidence that the clay is structured (in other words it has a fabric) or exhibiting an ageing effect. Since the changes in effective stress may be relatively recent, it is possible and based on previous comments, that the effects of stress history have been imposed on the structured clay. On the basis of the foregoing comments, the Bothkennar Clay is classified as normally to lightly over-consolidated.

The yield stress ratio (ratio of yield stress to current vertical effective overburden stress Burland, 1990), is plotted in Figure 5.15b alongside the profile of vertical effective stress (Figure 5.15a) determined for a current ground water level of 0.75m depth. This figure also includes yield stress data from a series of incremental load consolidation tests reported by Nash et al. (1992b) - since several tests were carried out at each level the average results are shown together with the range. There is noticeably more scatter in the latter test results (perhaps reflecting some of the inherent variability of the Bothkennar Clay), which can be interpreted as making definition of a characteristic OCR for the site difficult. To assist comparison with other similar deposits, the values of OCR found from the tests with small load increments have been plotted against the plasticity index, Nash et al. (1992a) in Figure 5.16a. This figure also shows the relationship given by Bjerrum (1973) and suggests that in this respect Bothkennar Clay is not dissimilar to other post glacial clays. In addition, correlation of in-situ vane strength with plasticity index and OCR (after Chandler, 1988) has been presented in Figure 5.16b and c respectively.

Smith et al. (1992) conducted oedometer tests on 'undisturbed' Bothkennar clay samples recovered from 5 to 6 m depth and with the objective of gaining some insight into bonding and destructuring. In comparing the compression curves to the corresponding compression curve for the reconstituted clay (the intrinsic compression line), the yield stress for the natural sample was found to be 1.5 times greater than that of the reconstituted samples at the same void ratios.

Nash et al. (1992b) observed creep behaviour of Bothkennar clay during incrementally loaded oedometer tests. They noted that creep effects were most prevalent immediately after yield, and suggested that this was associated with the structural breakdown during yield. Although the tests in the current research were not specifically designed to examine creep effects, it may be expected that some secondary compression will have occurred and will have some bearing on the field trial results. Nash et al. (1992b) also demonstrated that the yield stress observed in oedometer tests was strongly dependent on the applied strain rate, with higher yield stresses resulting from faster strain rates.

5.4 Implementation of field instrumentation

A summary of the instrumentation installed for the field trials is detailed in Table 4.3. Prior to stone column installation readout leads were ducted in plastic piping just below ground level, Figures 5.17a & 5.17b.

Push in pressure cells

Two different types of push-in instrumentation have been used (historically): the spade-shaped total earth pressure cell, Massarch (1975) and the flat dilatometer, Marchetti (1975). Generally spade-shaped cells are installed by jacking them a short distance beyond the bottom of a vertical borehole to measure horizontal stress. When spade shaped cells are used, sufficient time is allowed to elapse after installation for excess pore pressures caused by installation to dissipate. The measurement that is then made should be reasonably close to the undisturbed in-situ stress and has to be interpreted using an empirical correlation to give undisturbed horizontal stress (see Figure 5.18).

The miniature push-in cell, Figure 5.19 used in the trials was designed to be jacked horizontally into the soft clay from a vertical 150 mm borehole. It comprised a 2.4 mm thick oil filled-envelope attached to a wedge shaped slim body with twin nylon tubes connecting the cell (envelope) to the portable pneumatic readout at ground level. The miniature cells were monitored using a pneumatic unit which supplied gas at a constant flow rate. Readout units which operate by supplying gas to the cell balancing valve, then detect the return flow and cut off the supply pressure and measure the valve close-up pressure were not suitable for reading the miniature cells. The over-pressurisation of the cell which results from this technique has been found to give inaccurate readings.

Before installation in the field each cell was calibrated under hydraulic pressure in a small pressure vessel. The cell had a measuring area 44 mm in diameter, an overall length of 115 mm and a maximum body thickness of 20 mm in the direction of stress measurement. The oil filled envelope was connected directly to a pneumatic transducer incorporated in the flat body of the cell. Applied external pressure was transmitted to the oil in the envelope and measured through the transducer. Cells could be pushed horizontally into undisturbed soil to measure either vertical or horizontal stress. The placing device was cylindrical incorporating a double acting hydraulic jack that could be operated from ground level, Figure 5.19. The miniature earth pressure cell was loaded into a breech across the diameter of the machine. A breech ram, activated by the double acting jack via a pair of linear cam plates, advanced the miniature cell out of a port in the side of the machine. A vertical magazine located directly above the breech held a number of steel spacers (Figure 5.19). As the ram returned to its rearmost position a single spacer dropped under gravity into the breech and was in turn pushed out of the device, advancing the miniature cell a further 86 mm from the borehole. This operation was repeated until the cell was 600 mm (4 x 150 mm borehole diameter(s)) from the side of the borehole. The last spacer in the magazine has a latching mechanism so that it could be retrieved thus enabling the machine to be raised back to the surface.

The placing device can be operated either in unlined boreholes or from within liners of greater than 150 mm diameter. Different pieces can be attached to the placing device ensuring a close fit in casing up to 250 mm diameter. At Bothkennar, unlined holes were used to install the shallower depth cells (designed to monitor horizontal earth pressure), with lined holes used for installation of the deeper cells including those beneath the toe of stone columns. The borehole diameters were typically 200 mm. The lowest (deepest) earth pressure cell was installed first, with subsequent installations carried out at progressively shallower depths. A zero reading on the miniature pressure cell was taken with the installation equipment down the borehole at the depth which the cell was to be installed, after allowing time for the cell temperature to reach equilibrium with the surrounding soil.

When the total earth pressure cells are pushed into cohesive soil excess pore pressures are generated. With the cell advanced to its final position pressure measurements were taken at increasing intervals of time and the results were plotted to a logarithmic time scale to observe the dissipation of the excess pressures generated by jacking the cell into the soil and to determine the stable equilibrium pressure, and proved to be reasonably close to the undisturbed in-situ stress. Some general deductions could be made about soil conditions from the installation of miniature cells; the hydraulic pressure required to jack cells into the clay reflecting the stiffness of the clay, the dissipation time for excess pore pressure was related to the permeability characteristics of the soil.

In terms of the operating range, there are no fixed rules, but calibration was typically up to 4 bar for the field trials, although field operating pressures were somewhat lower than this - around 2 bar maximum. Accuracy was evaluated in terms of C_u, based on laboratory and comprehensive field trials (Watts and Charles, 1988; Watts and Charles, 1991). Over-read of 0.24 C_u for vertical stress and $0.5C_u$ for horizontal stress were calculated in laboratory trials (Watts – Pers Comm, 2011).Vertical over-read in the Bothkennar trials was 0.12 C_u. (see also Figure 5.18).

A total of eight miniature push-in pressure cells were installed prior to stone column installation (with the special placing device employed to push cells horizontally up to four borehole diameters) orientated to measure (horizontal or vertical) total earth pressure during and subsequent to the installation of stone columns. Four of the cells were installed to measure lateral (horizontal) earth pressures alongside the upper 2.5 m of stone columns 30 and 34. A further four cells were installed at approximately 300 mm below the design depth (toe) of stone columns 28,30,32 and 34 to measure vertical pressure beneath the base of the stone columns. Toe depths of the columns corresponded to 4.2m; 6.2 m; 8.2 m and 6.9 m respectively from the installation level (working platform level for the rig).

Inclinometer

The purpose built electro-level inclinometer system (Figure 5.20) was prepared off-site (comprising individual electro-level measuring units mounted in devised rigid, articulated aluminimum box sections) and then lowered down a pre-drilled borehole, bored by a small diameter mechanised continuous flight auger (CFA) drilling rig. When the inclinometer was suspended in position the borehole was grouted back to the ground surface. The instrumentation was designed to measure lateral displacement within the upper 2.5 m below ground level. Displacement was related to the top of the gauge which was monitored by optical means. The special inclinometer system was installed at a distance of 0.5 m from the centre of a proposed stone column installation point, i.e. stone column No. 31 at the location of trial footing 4. The five rigid and linked elements of the gauges contained an individual electro-level gauge calibrated for a working range of +/- 3˚, which were calibrated in a special laboratory rig prior to assembly and transportation to site. The gauge was monitored using a dedicated hand-held readout device.

Piezometers

Pneumatically operated piezometers manufactured by Soil Instruments Ltd., Figure 5.21 were installed in the clay crust and underlying soft Bothkennar clay. A total of four pneumatically monitored piezometers (P7 to P10) were installed adjacent to column No. 30 (trial footing 4) and column No. 34 (trial footing 6) at 0.5 m and 1.1 m below proposed founding level for these trial footings (see Table 4.1; 4.3 Chapter 4) i.e., in the clay crust and soft clay close to column positions. All piezometers were installed in boreholes up to 5 m deep, which were bored by a small diameter mechanized CFA rig or hand augered. Each tip, which was de-aired off site, was lowered into a prepared sand cell in the water filled boreholes. The tips were covered with more sand and the boreholes grouted back to the ground surface with fluid grout.

Flatjack pressure cells

Pneumatic total earth pressure cells were used to investigate the distribution of contact pressure beneath selected trial footings. Each cell comprises an oil-filled envelope, which is connected by a short length of steel tubing to a pneumatic transducer. The transducer is rigidly attached to the cell and permanently embedded in the soil with it. Two nylon tubes connect the transducer to the ground surface where they can be attached to a portable pneumatic readout unit. To take readings dry nitrogen is supplied from the readout unit to one side of a flexible diaphragm valve incorporated in the transducer. When the supply pressure is sufficient to balance the cell pressure on the reverse side of the diaphragm, the valve opens allowing flow along the return line to a detector in the readout unit. This closes the gas supply and the pressure at which the valve closes is recorded on the unit.

The 300 mm diameter flatjack pressure cells described in Section 4.2.9 were installed over selected stone column positions at founding depth and at the same level in the soil between stone columns, just beneath the interface of the foundation blinding concrete prior to foundation construction. The cells have two active faces and were placed on a prepared surface in the clay soil (Figure 5.22) and the excavated soil was replaced and firmed down over the top face (prior to foundation construction). The same types of cells were used to measure total vertical pressure in the top section of some of the stone columns. Approximately 100 mm of stone was carefully excavated from the top of the column and thin layers of fine gravel and then sand were used to provide a relatively smooth bed for the cell. Once in place the sand, fine gravel and stone aggregate were re-compacted in reverse order to complete the column back to formation level. Correct operation of each cell was checked by placing a dead weight on the soil surface or top of the column and the cell reaction monitored on the pneumatic readout unit.

Levelling system

In the light of comments in Chapter 4 (Section 4.2.9) a precise levelling system was selected to measure surface settlement, Figure 5.23a. The system had been used successfully to monitor settlement of a number of structures, Wood and Perrin (1984), Wood (1989) and Cheney (1989) and it had been found to produce reliable results. The levelling positions used to monitor settlement beneath the footings comprised levelling studs (Figure 5.23b). Each stud was a 75 mm long stainless steel bolt capped with a stainless steel dome nut and cast into the wet concrete during the construction of the trial footings, leaving the dome nut proud of the surface. All the surveying readings were related to one of the established deep datum(s) found in the gravels approximately 20 m below the soft clays at Bothkennar. The datum proved to be reliably stable on the Bothkennar site. Before each levelling session the level and tripod was left to acclimatise for a minimum of 15 minutes or longer, dependent upon prevailing weather conditions. Generally overcast conditions were preferred. When there was direct sunlight appropriate protection of the equipment was undertaken throughout the levelling operation. Up to date tabular records of the results of the surveys were kept as well as original results in the field survey book.

5.5 Stone column installation

The stone column positions (Figure 4.10b, Chapter 4) were set out using steel pins as markers to ensure accurate positioning of the vibroflot by the rig operator. The stone columns were installed using leader mounted dry bottom-feed equipment (Figure 5.1). Following initial penetration of the vibroflot to the depths designed for the trials a large volume of stone aggregate was introduced into the bottom section (toe) of the stone columns (around 50% of the aggregate introduced into each stone column), to facilitate construction of an enlarged toe in the soft clay to provide a suitably compacted base ('end bulb'), to reflect current practice at the time, and from which the remainder of the partial depth stone column could be constructed to the top of working platform level. Once an adequate resistance had been built up at the toe (typically in excess of 20-22 MPa) the remainder of the stone column construction was carried out using the standard dry bottom-feed installation procedure (Chapter 1, section 1.2.3). This involved raising the vibroflot by around 0.5 m in the bore and releasing stone aggregate, assisted by air pressure down the tremie pipe and then re-penetrating to compact the introduced aggregate. The whole operation was then repeated in stages until a compact stone column was formed to the surface. Furthermore, at an early stage in the field trials it was sometimes found beneficial to line the bore with aggregate during initial penetration of the vibroflot to the design depths. This procedure increased bore stability and reduced friction between the vibroflot and the soft clay - during this operation the vibroflot was raised up and down (surged) several times over a vertical distance of approximately 0.50-1.0 m, in stages, whilst releasing stone aggregate. This procedure appeared to facilitate more efficient stone column construction in the soft sensitive clay, but inevitably increased stone consumption.

5.6 Trial footing construction and loading

The trial footings (foundations) were arranged in a line close to an existing experimental haul road (geogrid reinforced hardcore) (see Figure 4.10a and b), to allow safe mobile crane access and set-up whilst loading the trial foundations (Figure 5.24). Once concrete cube tests had confirmed that the required 28-day strength had been achieved for the foundation concrete, imported concrete kentledge blocks were used to load the trial footings and provide a uniformly distributed load (Figure 5.24). The average weight of the individual concrete kentledge blocks (which were approximately 1.2 m cubes) was around 4 tonnes (40 kN), with a more precise reading recorded by the mobile crane as it lifted and positioned each kentledge block in place. A layer of bricks was placed at each foundation position prior to placement of kentledge (Figures 5.25) for safety and stability reasons and to ensure good contact pressure. Load was to be applied in two main approximately equal increments to all trial footings, corresponding to an average bearing pressure of around 33 kN/m^2 (range 32.9 kN/m^2 to 34.9 kN/m^2) for the first load increment (first row of blocks) and around 70 kN/m^2 (range 67.1 kN/m^2 to 72 kN/m^2) for the second load increment (second row of blocks). It was intended that each of the two main load increments would be in place for a minimum period of 5 months. Figure 5.26 shows the first and second load increments in place for the trial footings. At the end of the second load increment period, two further load increments (additional concrete blocks) corresponding respectively to average cumulative bearing pressures of around 107 kN/m^2 and 125 kN/m^2, were to be applied in quick succession to two of the trial footings – footings 4 and 6 (see Table 4.4) and also the untreated trial footing 8, to investigate the approach of bearing capacity failure. These combined additional third and fourth load increments were in place for a further 2 months, after which all kentledge was removed from all trial footings.

The field data was to be recorded in site log books and transferred to spreadsheets and processed accordingly using the on-site facilities present during the trials. The interpreted data and parameters will be compared with published data and also data for the untreated footing 8, together with previous work by Jardine et al. (1995), on untreated footings at Bothkennar.

5.7 Finite element analysis

Numerical (modelling) analysis of the field trial data was proposed using the Plaxis 3D geotechnical software package. Development of Plaxis numerical modelling commenced in 1967 at the Technical University of Delft for the analysis of a river embankment on soft soil in the lowlands of Holland. In subsequent years, Plaxis has been extended to cover most areas of geotechnical engineering. To deal with accuracy in geotechnical problems the Plaxis program contains various types of elements and nodes (Plaxis Manual, 2008). In the input program of Plaxis the geometry is given by entering different soil layers, structural parts and external loads etc.. Plaxis 9 (3-D) supports different models to simulate the behaviour of the soil and a choice of various material models was available: Linear Elastic model; Hardening Soil model; Hardening Soil model with Small Strain Stiffness; Soft Soil model; Mohr Coulomb model. For the Bothkennar field trials the Hardening Soil (HS) model was considered most appropriate for analysing the stone column-soil composite. In contrast to an elastic perfectly- plastic model, the yield surface of a hardening plasticity model is not fixed in principal stress space, but it can expand due to plastic straining. Some parameters of the Hardening Soil model coincide with those of the non-hardening Mohr-Coulomb model. These parameters are friction angle, (φ'), cohesion, (c), and dilatancy angle, (ψ). In addition to these parameters, the Hardening Soil model requires basic stiffness parameters E_{50}^{ref}secant stiffness in standard triaxial test, E_{Oed}^{ref} tangent stiffness for primary oedometer loading, E_{ur}^{ref} unloading-reloading stiffness and m power for stress-level dependency. Selection of the material or soil type is made at the input stage for each trial footing. The soils and materials are assigned relevant material properties, such as stiffness and density, which are in turn assigned to elements together with assignment of appropriate boundary conditions. When the model is complete, a mesh is generated and both initial stresses (and pore water pressures, as required) are initiated before moving to the calculation program.

The Plaxis 3-D Foundation program permits an automatic generation of unstructured 2D finite element meshes based on the top (plan) view. The 2D mesh generator is a special modified version of the triangular generator, which was developed by Sepra in the Netherlands. A 3-D mesh is automatically generated, taking account of the soil stratigraphy and structure levels as defined in the site investigation boreholes and work planes. There are options for global and local mesh reinfinement. Global mesh refinement can affect the horizontal element distribution as well as the vertical element distribution. Quadratic 15-node wedge elements are available to model the deformations and stresses in the soil. Due to non-horizontal/uniform soil stratigraphy, these elements may degenerate once to 13-node volume elements or twice to 10-node tetrahedral elements.

The performance and accuracy of Plaxis 3-D Foundation has been carefully tested by carrying out analyses of problems with known theoretical solutions. A selection of these benchmark analyses is described in Chapter 2 to 6 of the Plaxis user manual (Plaxis Manual, 2008). Plaxis 3-D Foundation has also been used to carry out predictions and back-analysis calculations of the performance of full-scale structures as additional checks on performance and accuracy. Moreover, Plaxis has been used extensively for the prediction and back-analysis of full-scale projects throughout Europe (Plaxis, 2008). This type of calculation may be used as a further check on the performance of Plaxis provided that good quality soil data and measurements of structural performance are available. Some such projects are published in the Plaxis Bulletin, on the internet site: http://www.plaxis.nl and are available at Plaxis. Four validation examples can also be found in the last chapters of the user manual (Plaxis, 2008). It is perhaps also important to recognise that the development of Plaxis and Plaxis 3-D Foundation would not be possible without world-wide research at universities and research institutes and it is clear that the Plaxis development team is in contact with a large network of researchers (and users) in the field of geo-mechanics and numerical methods, to ensure that the high technical standards of Plaxis is maintained. Direct support is obtained from a series of research centres. This provided reassurance and confidence in the selection of the program for this research (accompanied by some training in its use and application(s)).

Soil Parameter	Description/Value
Soil Description	Stiff becoming firm brown very silty CLAY with pockets of weakly cemented rust-brown silt
Soil bulk unit weight (γ_b)	17 kN/m^3
Soil moisture content	29% (Average)
Undrained shear strength	120 kN/m^2 at 0.2 m reducing to 40 kN/m^2 at 1.0 m depth.
Plasticity indices	Liquid Limit (LL) = 49 Plastic Limit (PL) = 28 Plasticity Index = 21 Liquidity Index = 0.24
Coefficient of Compressibility (M_v)	0.45-0.50 m^2/MN
Coefficient of Consolidation (C_v)	3m^2/yr

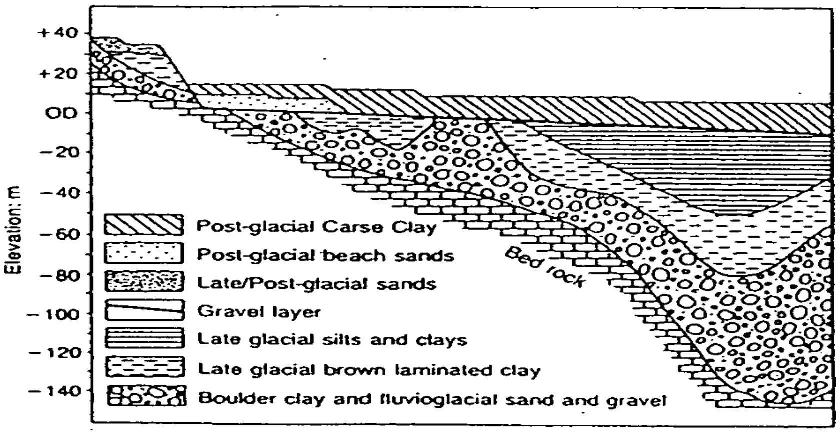
Table 5.1: Geotechnical data for Bothkennar 'crust'

Figure 5.1: Installation of stone columns using the dry bottom-feed technique during the Bothkennar field trials.

Sieve Size (mm)	Weight (g)	% Passing	British Standard (mm)
37.5	286	88.2	85-100
20.0	2119	0.5	0-25
10.0	8	0.2	0-5

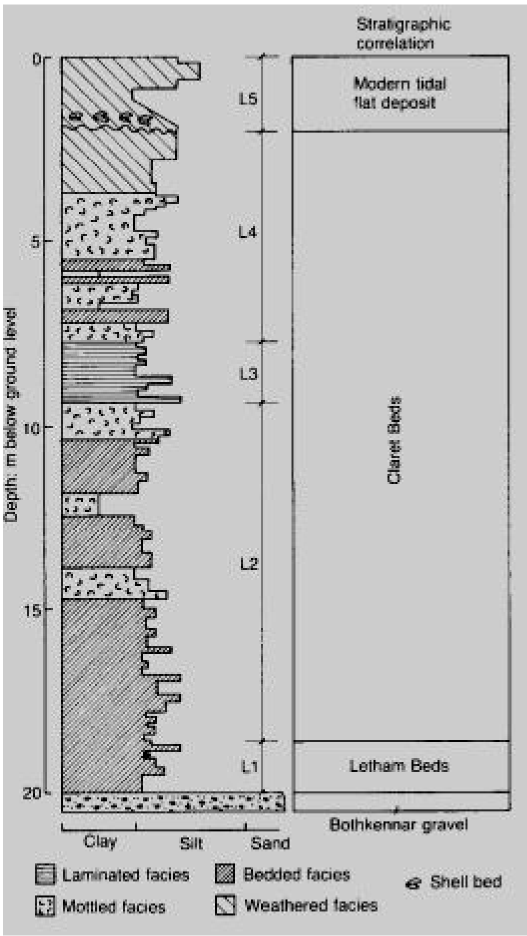
Figure 5.2: Stockpile of stone column aggregate for Bothkennar trials and grading certificate for quality control purposes.

Figure 5.3: Sample of stone column aggregate (selected for use in Bothkennar field trials) before and after testing in BRE large diameter triaxial cell apparatus. (300 mm long steel rule provided for scale)



Geotechnical Units/Groups	Soil Description of Groups	Geological Stratigraphic Classification		
Ai	Top 0.5 m of clayey topsoil or brick/cinder made ground overlying firm or stiff dark yellowish brown clayey SILT or silty CLAY with moderate brown mottling. Shell bands and pods are common.	POST GLACIAL	Desiccated layer	CARSE CLAY
Aii	Very soft-firm dark yellowish brown to olive grey silty CLAY. In north of area this becomes closely laminated with fine/medium SAND.		Weathered Carse Clay	
B	Very soft-soft olive black silty CLAY with laminations of silty fine SAND, silty animal burrows, small shell fragments and occasional carbonaceous pods are also present.		Unweathered Carse Clay	
C	Soft olive grey clayey SILT with laminations or lenses of fine SAND or medium/fine SAND with laminations of silty CLAY or fine/coarse GRAVEL.			
D	Compact sandy coarse GRAVEL with COBBLES and BOULDERS sometimes with a clay matrix. Shells are present in south east of area.	LATE GLACIAL	Gravel	
E	Soft-firm greyish brown or dusky yellowish brown silty CLAY closely laminated with silty fine SAND, to loose to dense medium SAND with clayey laminations.		Laminated Clay	
F	Soft greyish brown silty CLAY with occasional laminations or dustings of silt or silty fine SAND. Sand laminations become more numerous near the base of this layer.			
G	Firm/stiff to hard dusky yellowish brown silty sandy gravelly CLAY (BOULDER CLAY). This often passes down into compact SAND especially in the southern part of the area.		Glacial Deposits	

Figure 5.4: Nature of infill deposits in upper bedrock valley of River Forth (after Gostelow and Browne, 1986) and geological classification of the Carse Clay (after Gostelow and Browne, 1981, 1986).



(a)

Stratigraphic classification	Unit name
Recent alluvium	
Post-glacial (Carse) deposits	Grangemouth Beds
clays, silts, sands and peat	Claret Beds
	Letham Beds
Late glacial deposits	
gravel	Bothkennar Gravel
silts, clays, laminated clays	Loanhead Beds
Glacial till	
Bedrock	

(b)

Thickness: m	Stratum	Stratigraphy
1·5	Firm, dark brown clayey **silt** with frequency fine rootlets and layers of *Cardium edule* present in the lower part	Crust
16·0	Soft becoming firm, dark grey/black micaceous silty **clay**/clayey **silt**, partly thinly laminated and mottled	Grangemouth, Claret Beds
1·5	Fine to coarse grey/brown **sand** with shell fragments	Letham Beds
2·0	Dense to very dense well-graded **sand, gravel** and **cobbles**	Bothkennar Gravel
2·0	Firm dark grey silty **clay**	Loanhead Beds
20·0	Stiff dark grey slightly clayey sandy **silt** with frequent subrounded gravel	Till
—	Slightly to faintly weathered, white, medium-grained, moderately strong **sandstone**	Carboniferous probably including the Glenfuir coal seam

(c)

Figure 5.5: Stratigraphy at the Bothkennar research site (a) after Paul et al. (1992). (b) and (c) after Nash et al. (1992a).

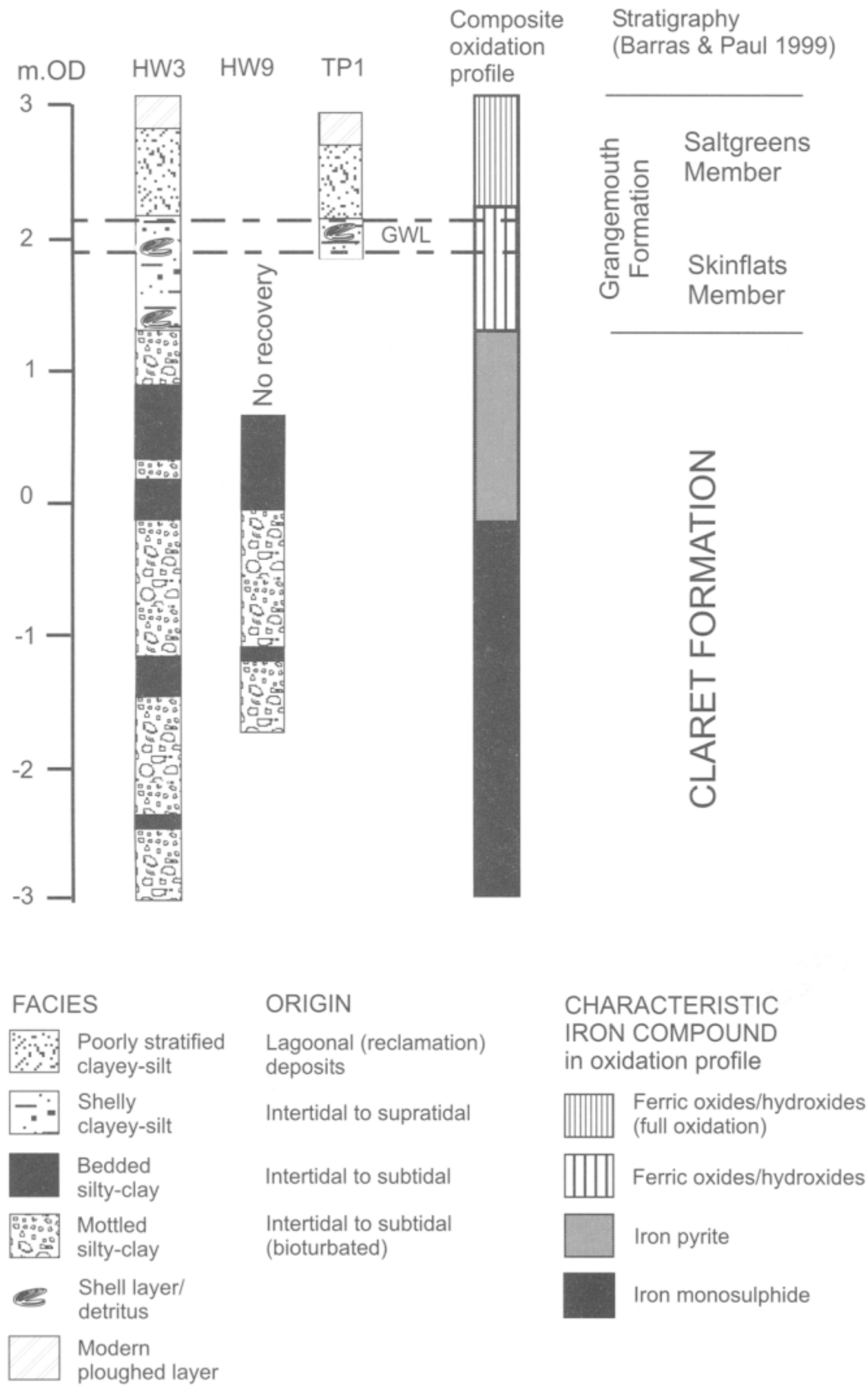
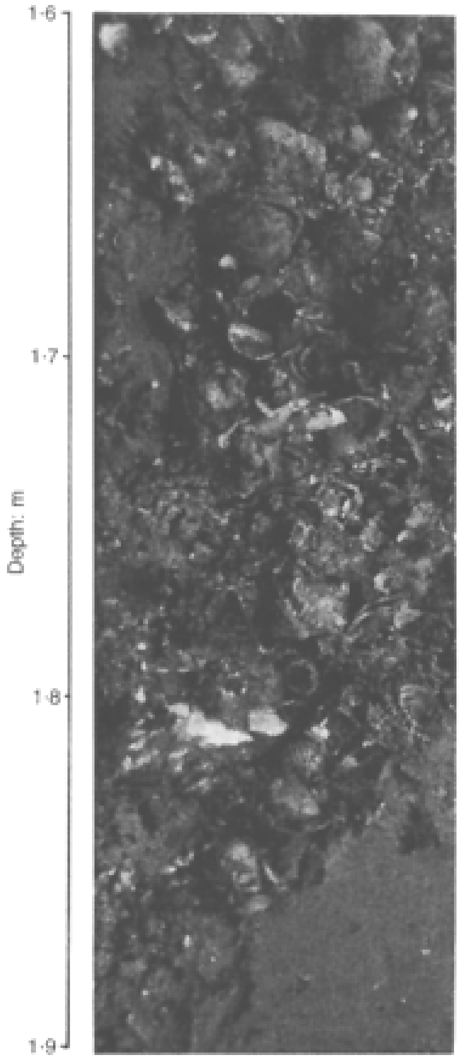


Figure 5.5 (d) Stratigraphy at the Bothkennar research site (after Barras and Paul, 2000)



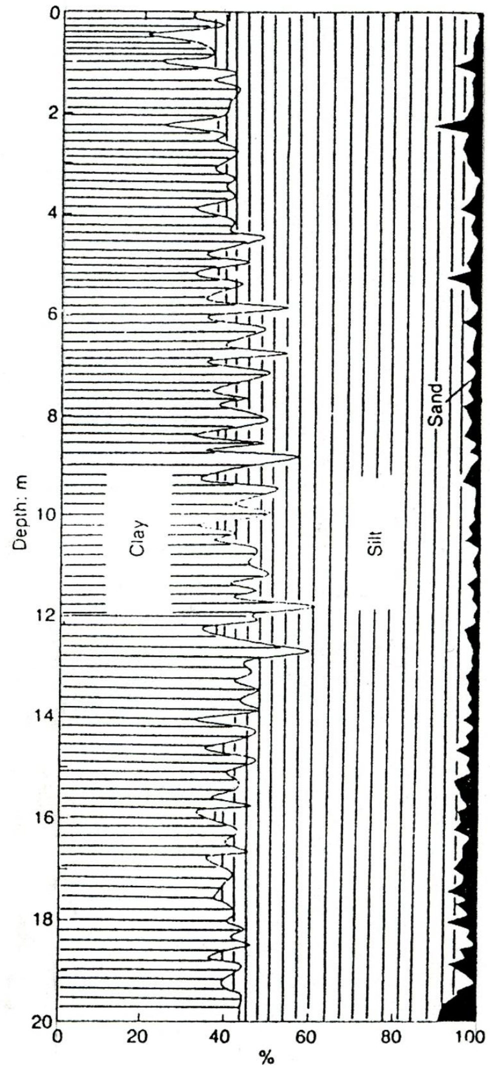


Figure 5.6: (a) 300 mm thick shelly layer (1.60-1.90 m depth) at Bothkennar. (b) Profile of particle size distribution within the upper 20 m of the soil profile at Bothkennar (after Paul et al., 1992).

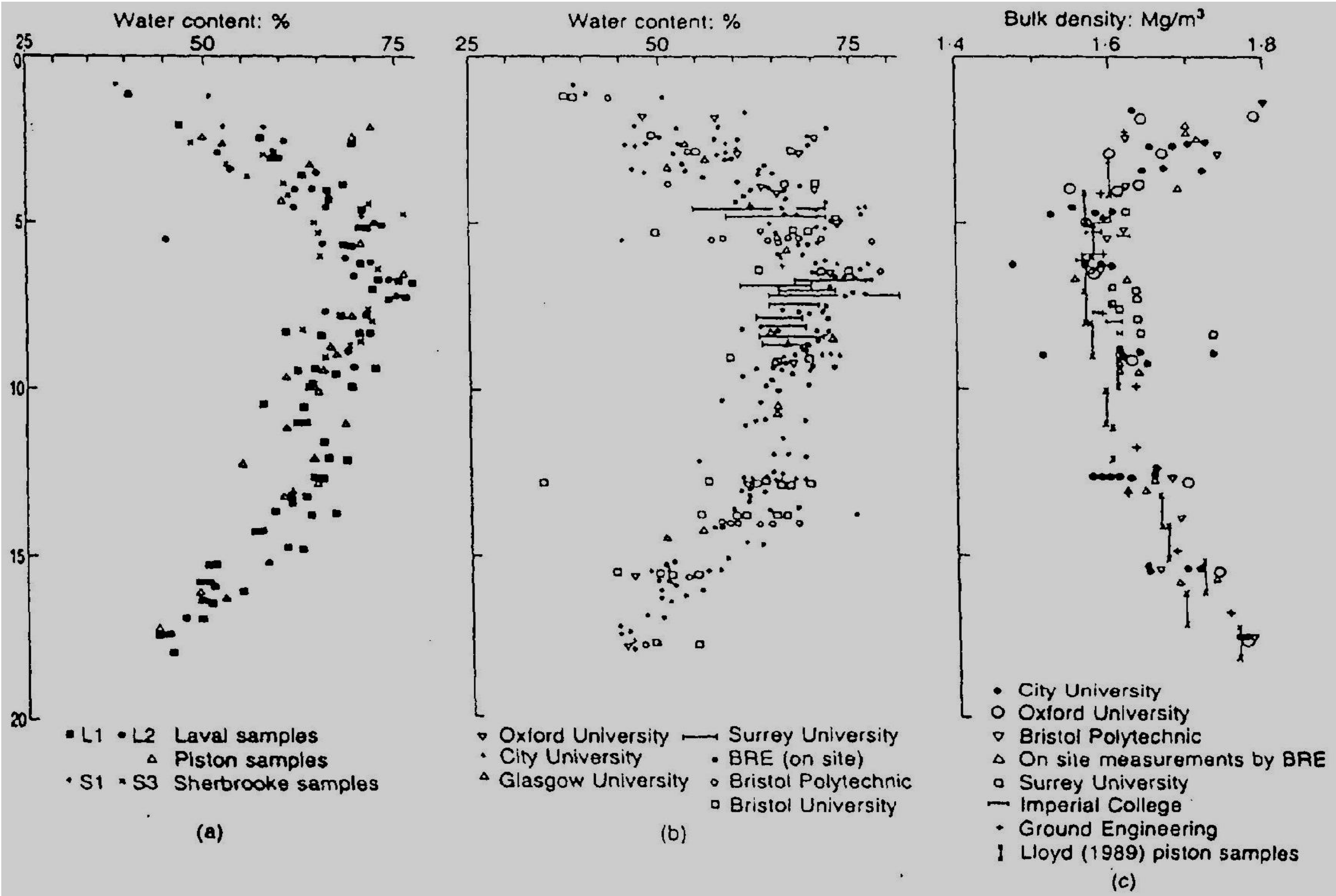


Figure 5.7: Variation in water content and bulk density with depth: (a) On-site measurement of water content profile. (b) On-site and off-site memeasurements of water content profile. (c) Bulk density profile (after Hight et al., 1992).

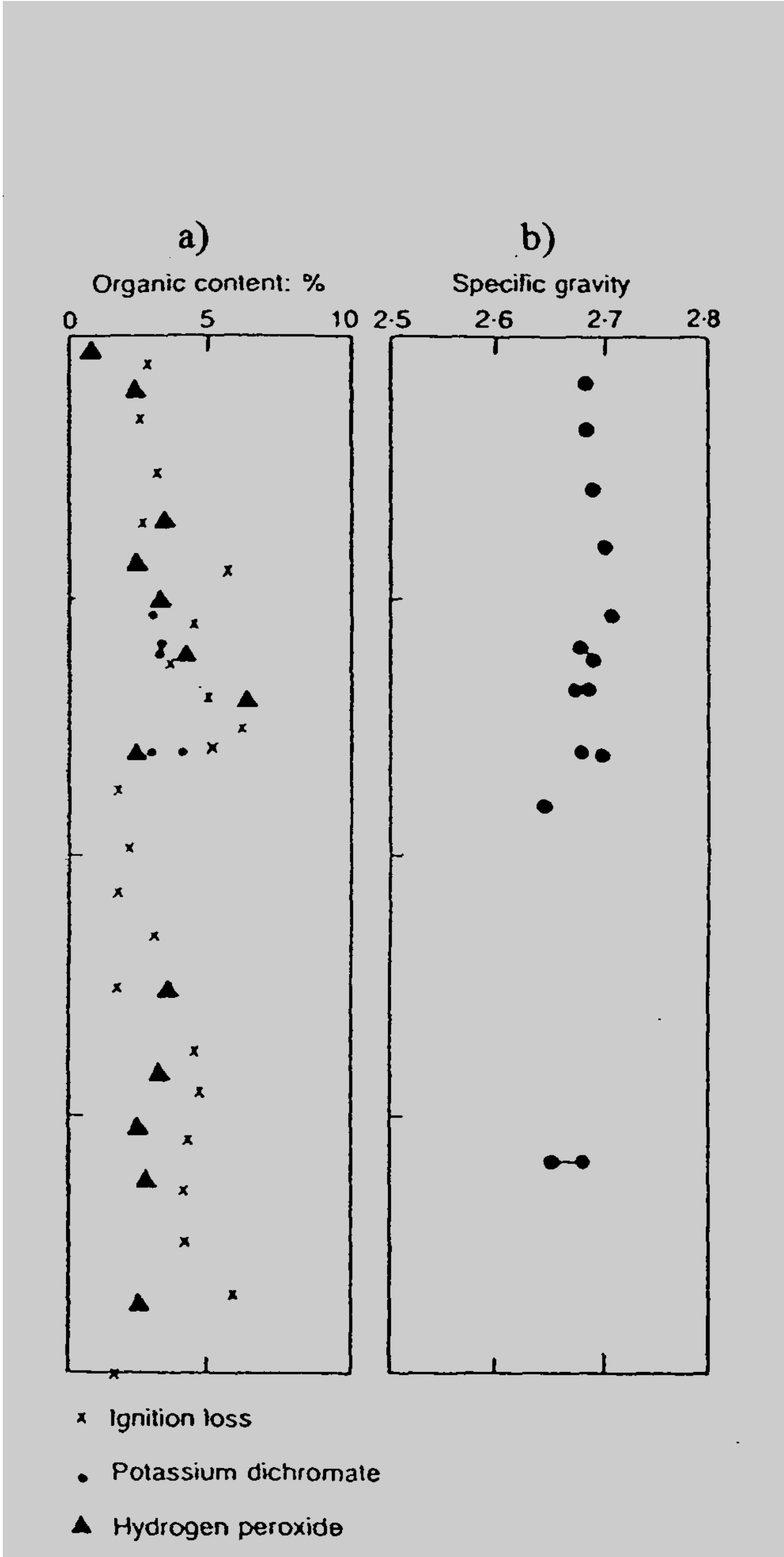


Figure 5.8: (a) Organic content within the upper 20 m of the Bothkennar Clay profile. (b) Soil specific gravity within upper 20 m of the soil profile at Bothkennar (after Hight et al., 1992)

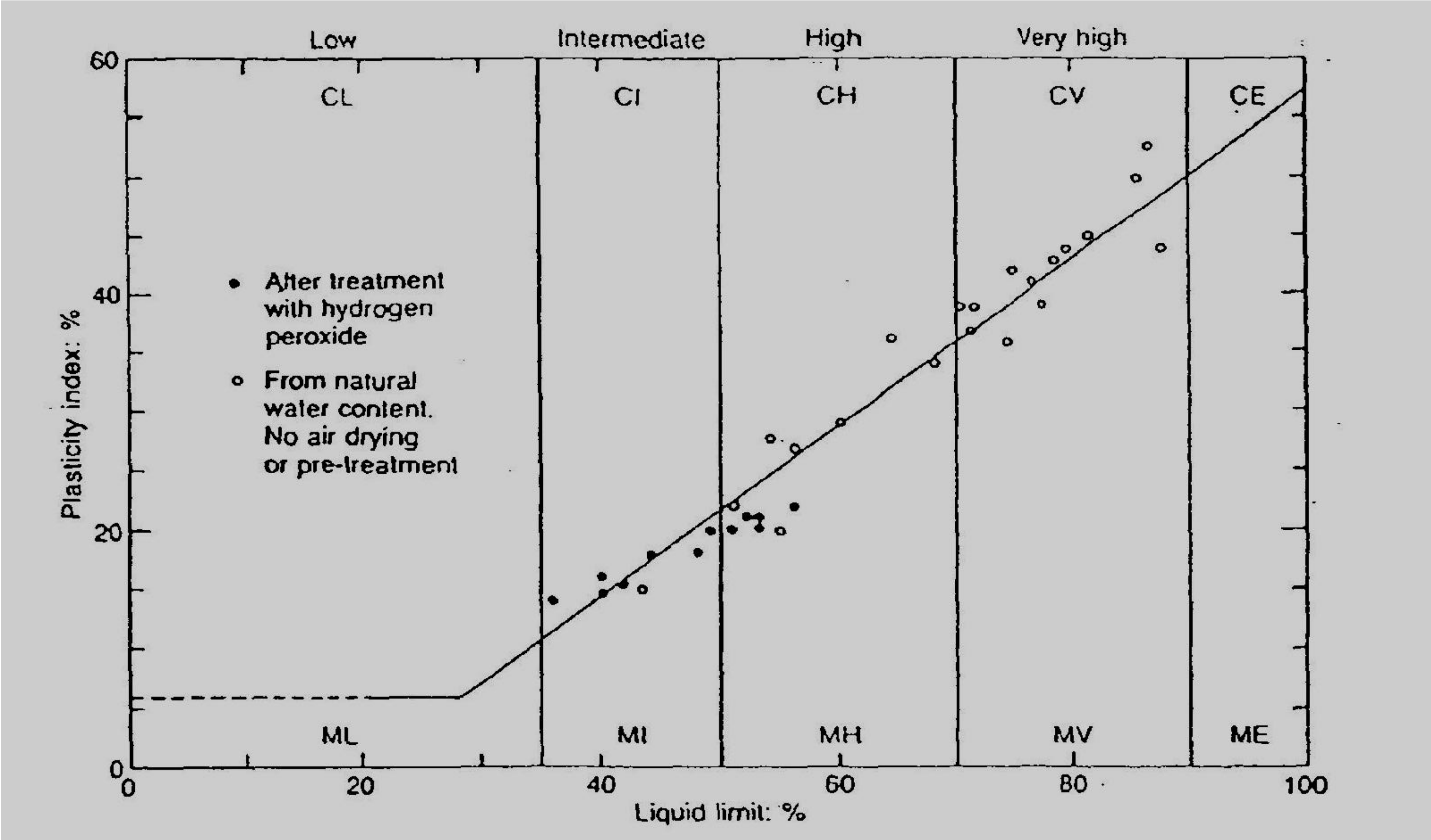


Figure 5.9: Casagrande plasticity chart before and after removal of organics from Bothkennar Clay (after Hight et al., 1992)

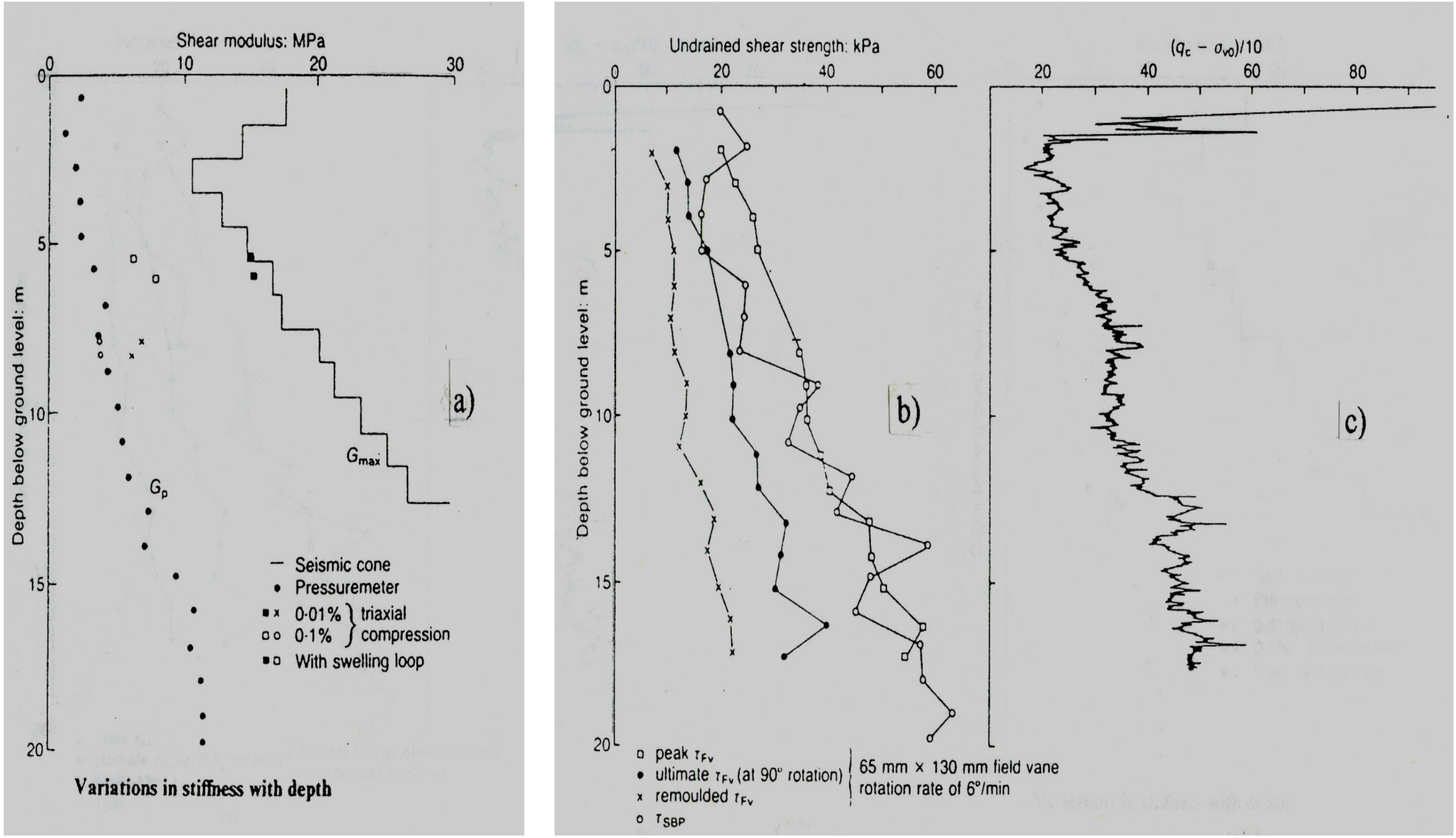


Figure 5.10: (a) Variation in stiffness with depth of Bothkennar Clay. (b) Undrained shear strength profiles for Bothkennar Clay from field vane and pressuremeter tests and (c) from cone penetration testing (after Hight et al., 1992).

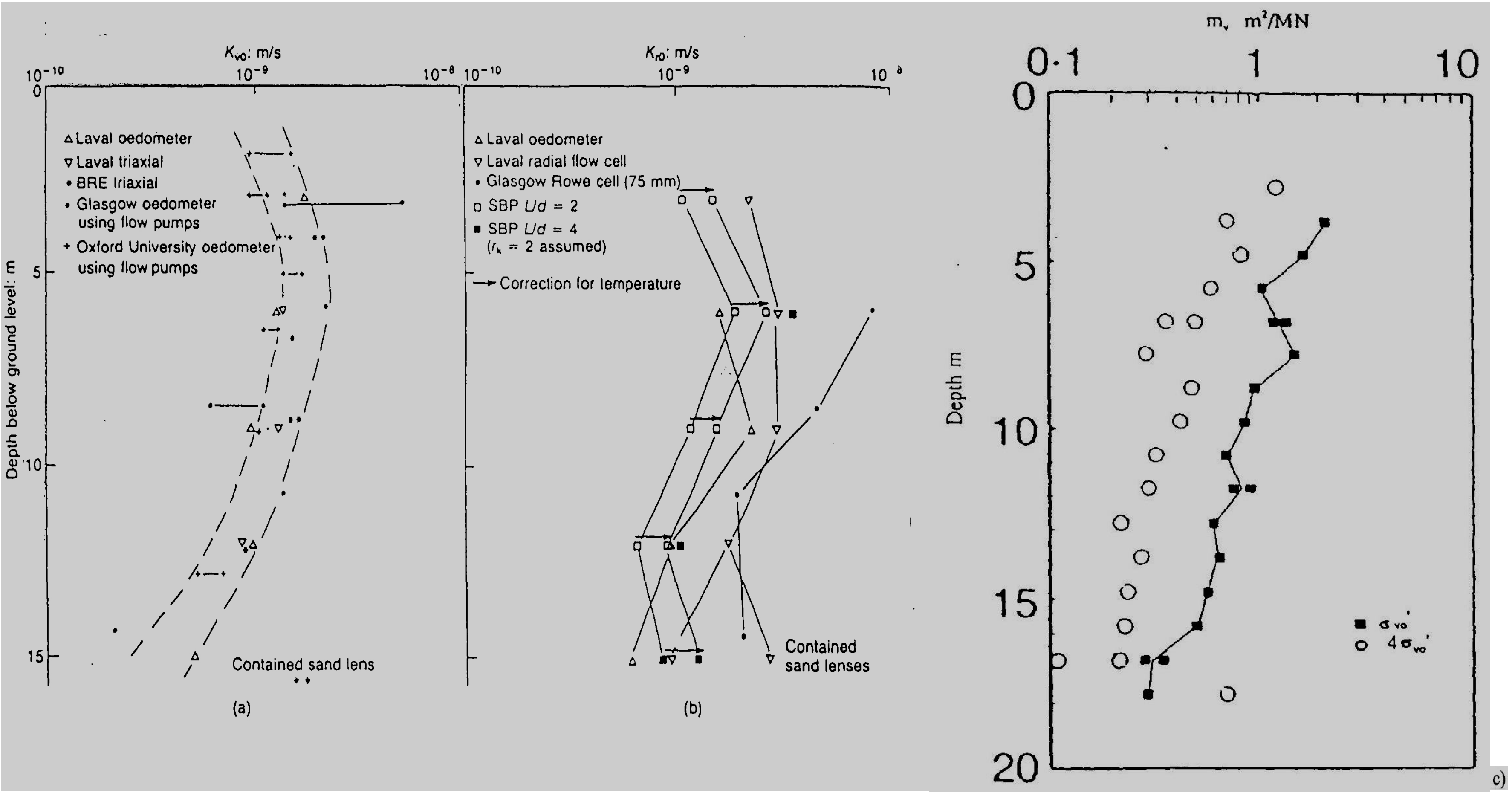


Figure 5.11: (a) and (b) Profiles of vertical (K_{V0}) and horizontal (K_{ro}) permeability at in-situ void ratio (after Hight et al., 1992). (c) Coefficient of volume change (m_v) within upper upper 18 m of Bothkennar profile (after Nash et al., 1992a).

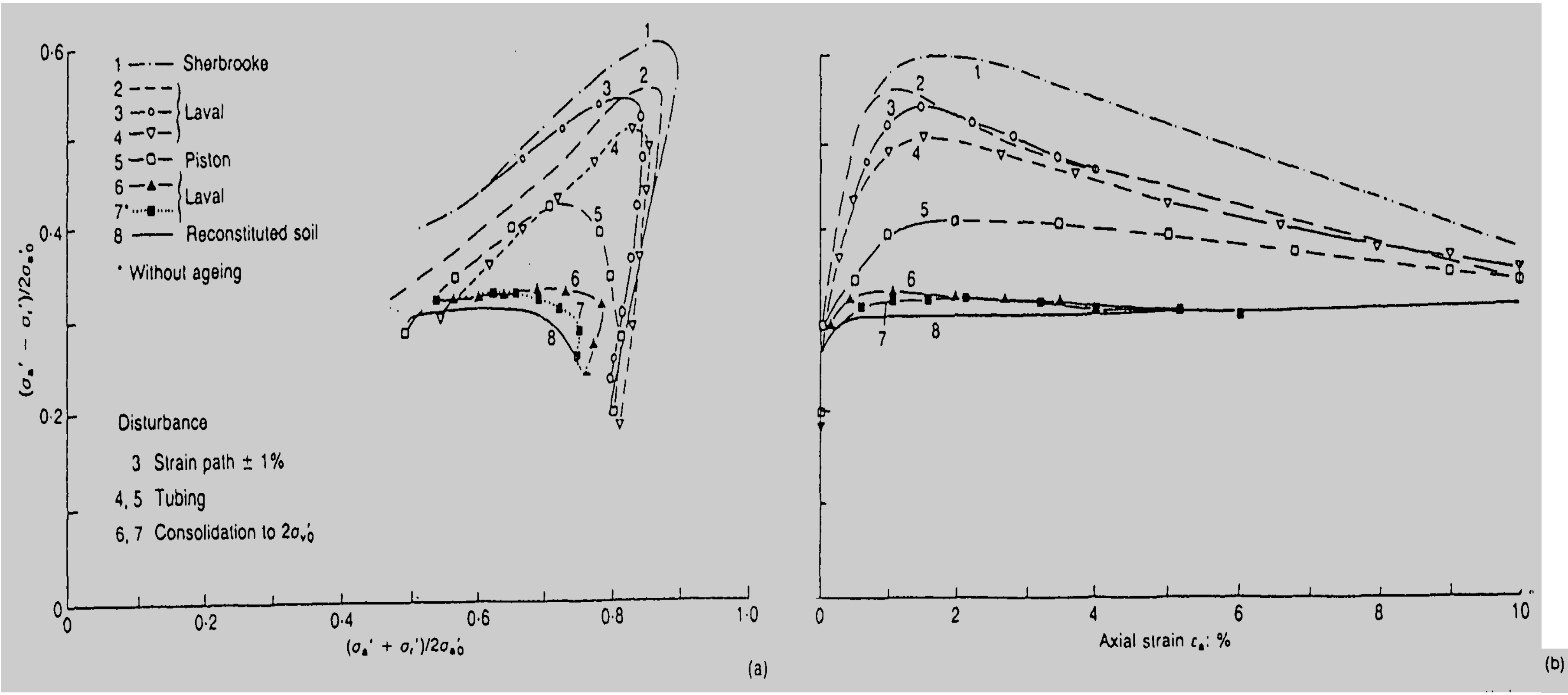


Figure 5.12: (a) and (b) Disturbance resulting from attempts at 'undisturbed' sampling of Bothkennar Clay using different sampling techniques (after Hight et al., 1992).

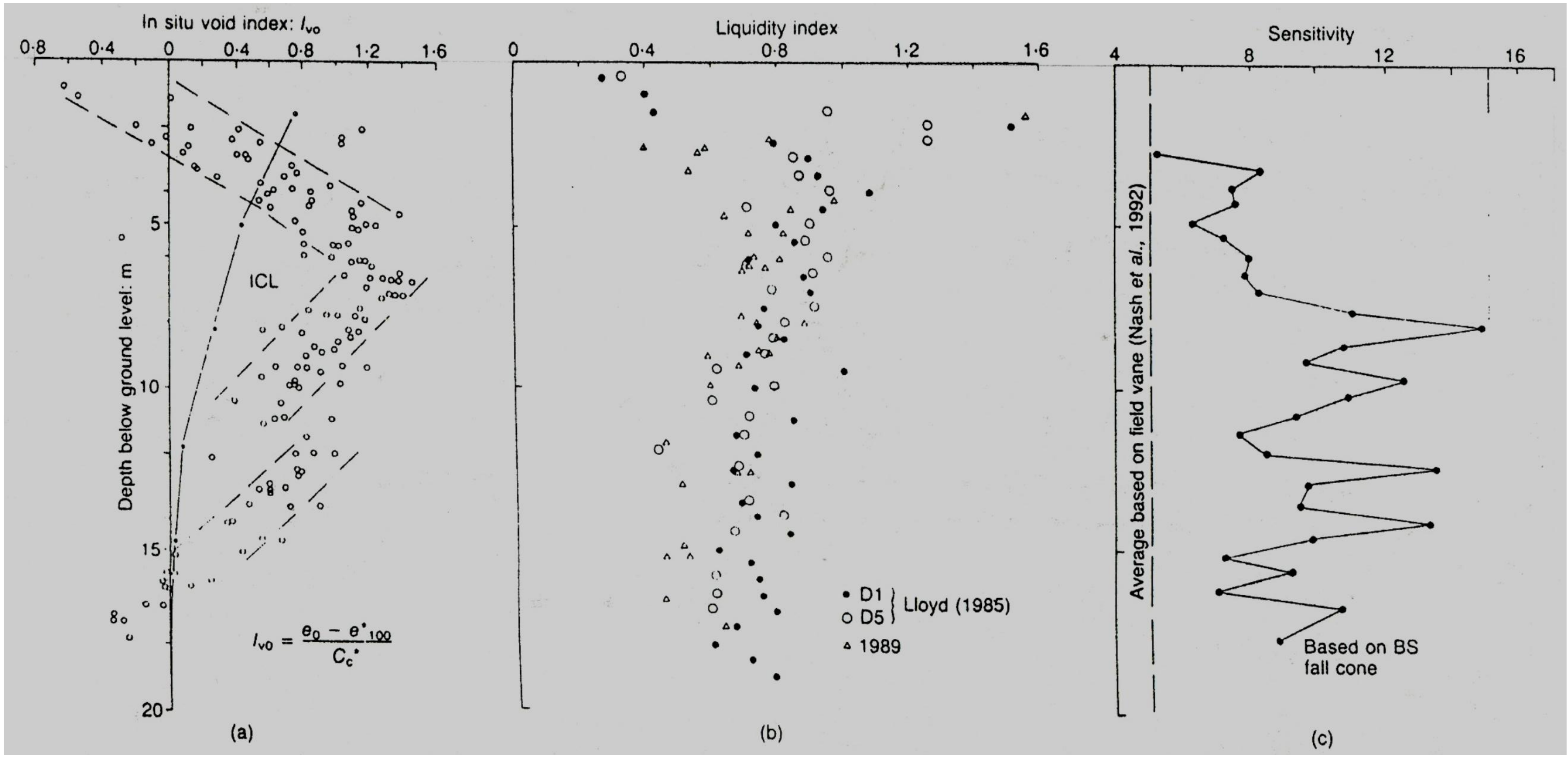


Figure 5.13: (a) In-situ void index for the Bothkennar Clay. (b) Liquidity index profile for the Bothkennar Clay. (c) Soil sensitivity based upon field vane and fall cone (after Hight et al., 1992).

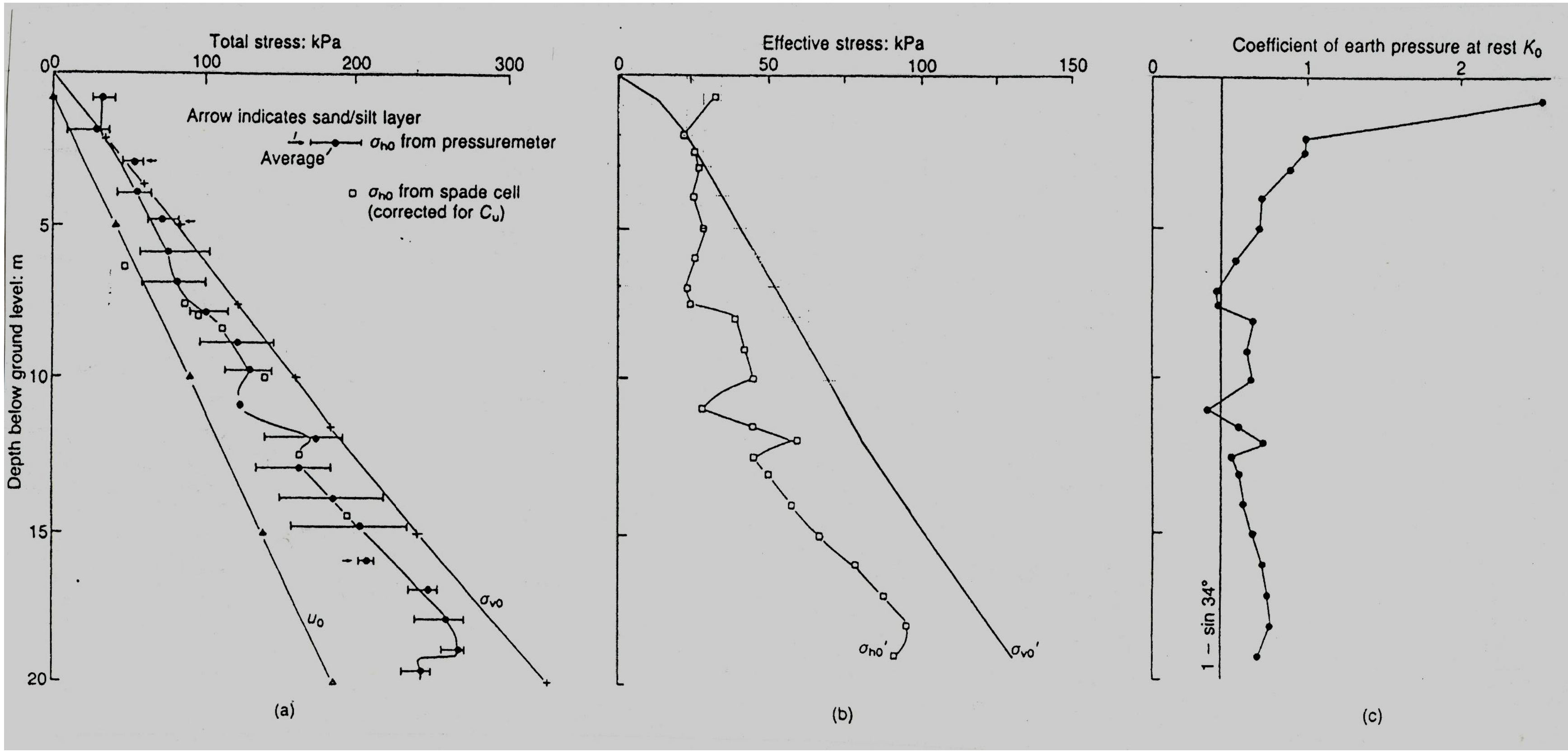


Figure 5.14: (a) Profile of total stress and (b) effective stress within the Bothkennar Clay. (c) Profile of coefficient of earth pressure at rest (K_O) within the Bothkennar Clay (after Hight et al., 1992).

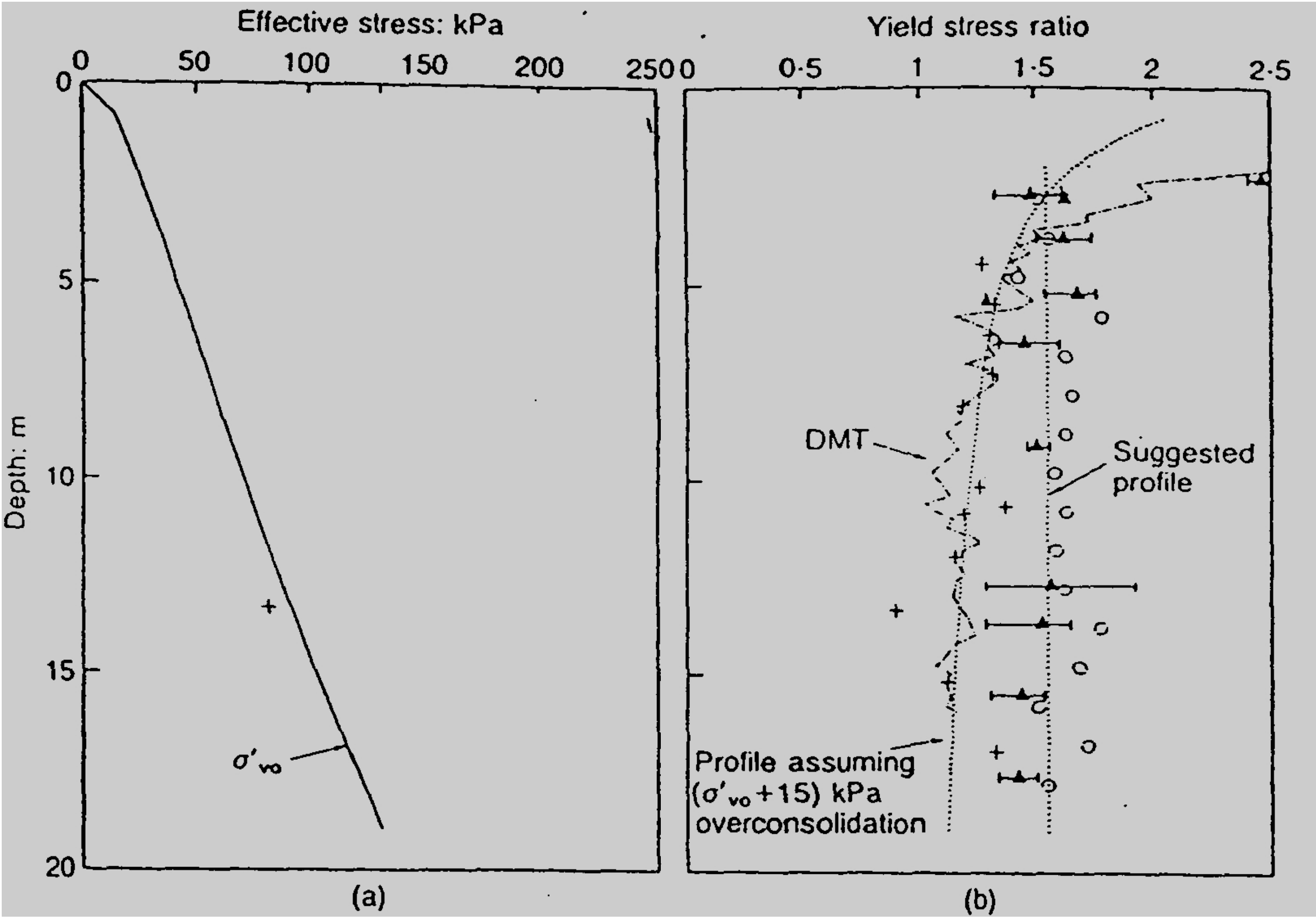


Figure 5.15 (a) and (b): Yield stress and yield stress ratio from incremental load consolidation tests on Bothkennar Clay (after Nash et al., 1992b).

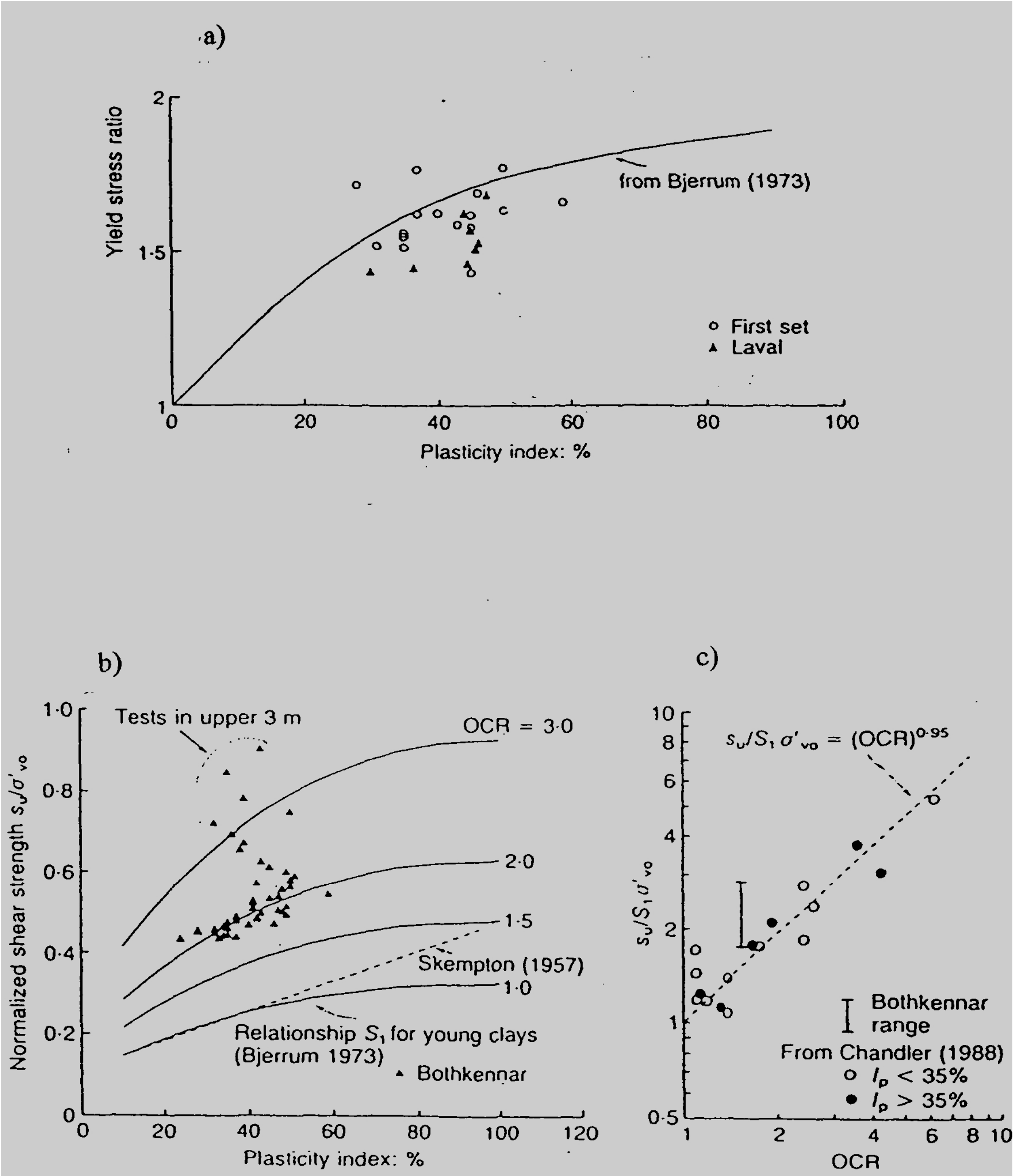


Figure 5.16: (a) Variation of yield stress ratio from incremental load consolidation tests with plasticity index (after Nash et al., 1992b). (b) Correlation of in situ vane strength with plasticity index and (c) OCR (after Chandler, 1988).

Figure 5.17: (a) and (b) Readout leads being ducted in plastic piping just below ground level.

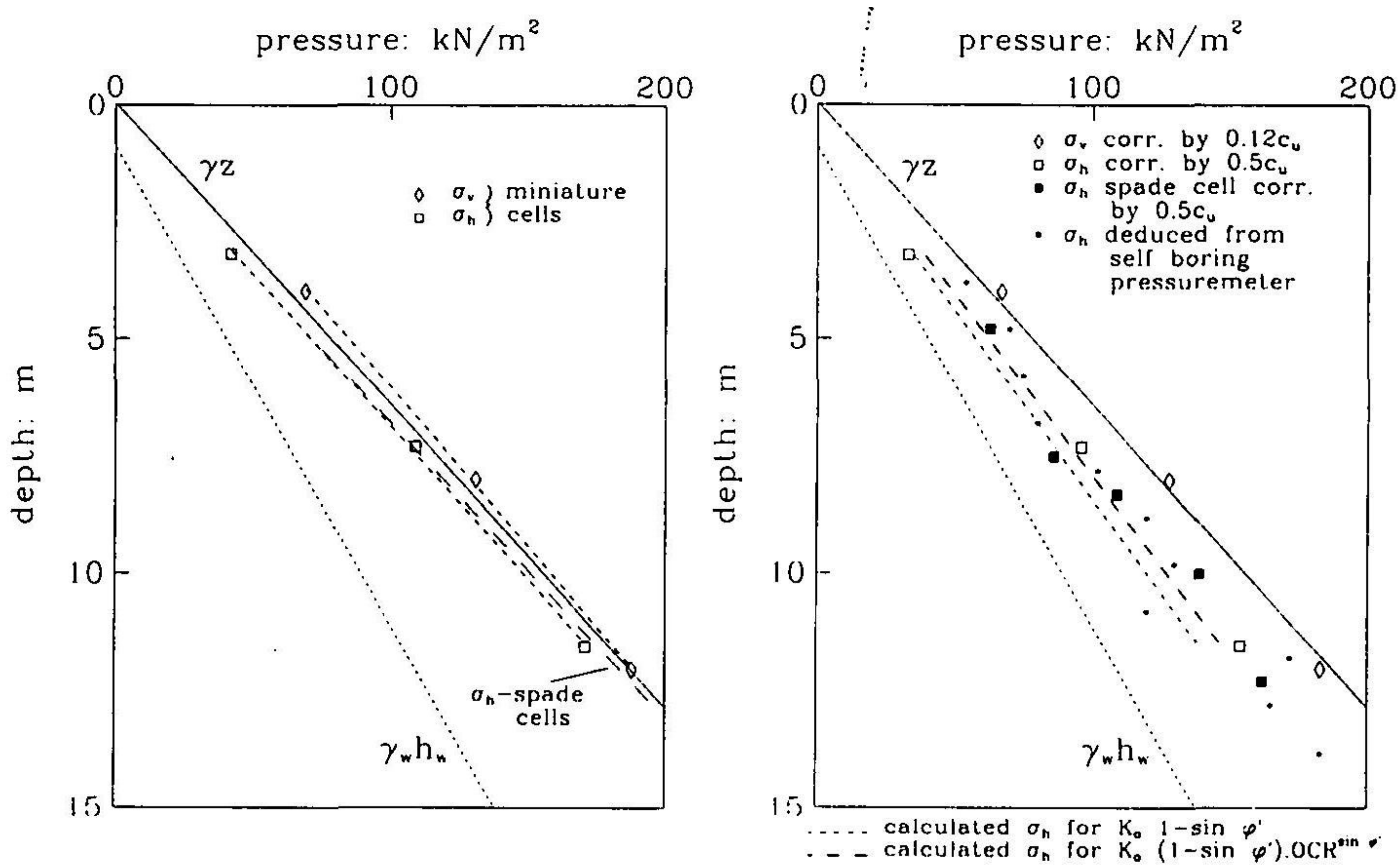


Figure 5.18: Correlation diagrams for BRE pressure cells (after Watts and Charles, 1988).

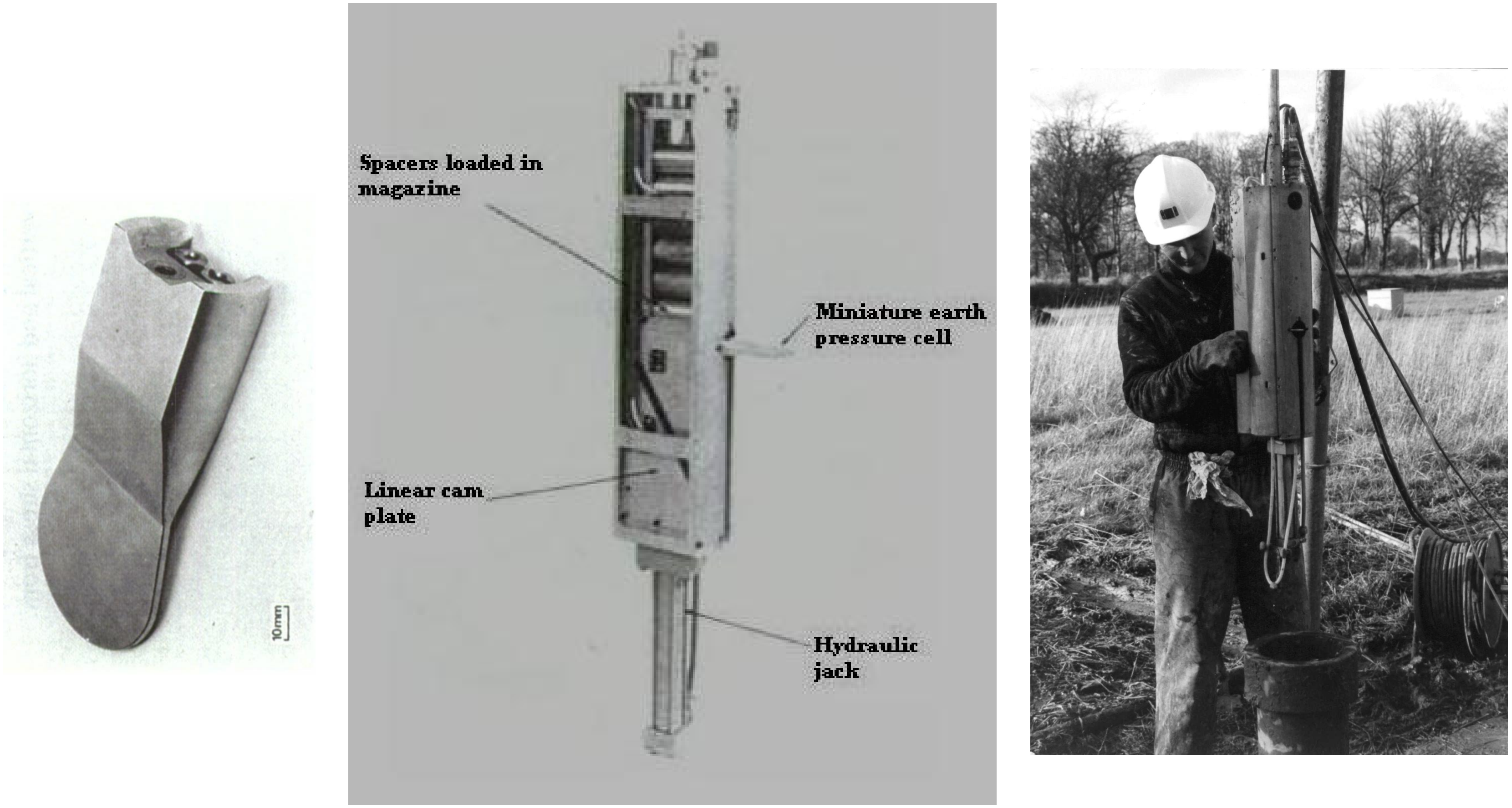


Figure 5.19: Miniature earth pressure cell and placing (installation) device.

Figure 5.20: Installation of purpose-built electro -level inclinometer.

Figure 5.21: Installation of pneumatically operated piezometers

Figure 5.22 : Installation of 300 mm diameter (flatjack) pneumatic total earth pressure cell to investigate contact pressure distribution between stone columns and intervening soil at Bothkennar.

a)

b) Trial footing levelling point

Figure 5.23: (a) Precise levelling system to measure surface settlements of trial footings at Bothkennar. (b) Close- up view of levelling stud.

Figure 5.24: Mobile crane placing kentledge on trial (concrete) footings at Bothkennar.

Figure 5.25: Completed trial strip (rectangular) and square pad footings prepared for placement of concrete kentledge blocks.

Figure 5.26: Applied first and second load increments to trial (concrete) footings at Bothkennar.

Chapter 6 Ground response to stone column installation

6.1 Visual observations

During installation of each of the trial stone columns (Figure 4.10b; Table 4.1), the construction sequence using the dry bottom-feed technique, including ground response to column installation, was carefully monitored. Spreadsheet outputs with recorded data for the trial stone column installations are given in Appendix 6.1. Parameters recorded manually by the rig operator on daily site record sheets (see Figure 6.1) were as follows:

- Stone column reference number.
- Start and finish time with regard to stone column installation.
- Depth of installation of the stone column (read from a graduated scale on the vibroflot (vibrating poker)).
- Number of bucket loads (of known volume) of stone aggregate used to construct each stone column.
- Penetrating (boring) and compacting pressures during stone column construction.
- Details of any anomalies and/or down-time during stone column installation.

Manual monitoring was also undertaken by the author, i.e. independent of the rig operator, with respect to some of the stone column installation parameters. This included recording of:

- Stone column construction time and depth.
- Proportion of aggregate introduced into different sections of the column.

Readings of the installed field instrumentation was also undertaken at different stages during the column construction, including measurements of the in situ stresses, pore pressures and lateral soil displacement. After penetrating to design depth, a procedure was adopted to construct an 'end bulb' to form a base within the soft clay, from which the rest of the column could be constructed to the ground surface (see section 5.5).

The initial insertion of the vibroflot visibly displaces the host soil laterally as it advances into the ground, resulting in the expansion of a cavity from zero to a value approximately equal to the diameter of the bottom-feed vibroflot, i.e. around 0.5 m. It was evident that the hydraulic pressure recorded in the vibroflot system represents the energy consumed during column installation, giving an indication of the radial force necessary to expand the cavity during initial vibroflot penetration. Later, the energy was consumed in the compaction of the introduced gravel and also in a slight increase of the cavity to its final diameter during column construction. As the vibroflot visually penetrated the ground, local shear failure and in some cases significant ground disturbance (heave), was observed around the top of the stone column installation point. Visually, most heave observed at the surface appeared to occur during construction of the 'end-bulb' described earlier, which was accompanied by high stone consumption and energy input. Whilst it was apparent that the air jetted through the nose-cone of the vibroflot assisted its penetration into the ground, it was clear that this was not its prime objective. The main purpose would appear to be to relieve frictional drag and also counteract the suction effects that would be generated as the vibroflot was slowly withdrawn during column construction, and confirmed by the fact that no significant resistance to vibroflot extraction from the bore was met, in turn implying that there were no significant extraction forces being generated.

Although the vibroflot in the trial(s) was fixed to a guide leader, there was a certain amount of play or give evident, particularly when some initial resistance to vibroflot penetration was exhibited by the upper part of the stiffer 'crust', sufficient in theory to permit the maximum deviation (verticality) of 1 in 20, normally specified, to be exceeded. Although there was no evidence during the trials of the vibroflot exceeding this maximum deviation from the vertical, it did nevertheless encroach close to this maximum tolerance value and it was therefore considered appropriate that some improvements could be made to the equipment, principally for quality control purposes. On this basis a guide device (with nylon rollers), was designed for fitting to the lower section of the bottom-feed installation rig and through which the vibroflot would pass and maintain a verticality of better than 1 in 20. The device produced is illustrated in Figure 6.2.

The local shear failure of the ground around the top of the stone columns had the effect of inducing an increase in stone column diameter at the ground surface, attributable in part to the lack of overburden pressure and therefore lateral restraint at this level. This was accompanied by ground heave at the surface as described earlier. The observed surface ground heave during the field trials was recorded at selected locations and the data is summarised in Table 6.1. Three stone columns installed at 2.0 m centres (trial footing 2) produced an average local increase in the ground surface of 107 mm; four stone columns at 1.5 m centres (trial footing 1) produced an average of 143 mm of heave, i.e. some 25% higher than for the wider column spacings beneath trial footing 2. Whilst heave seems to be rarely measured in the field, some published examples have been located and also included in Table 6.1, in order to permit comparison with the recorded field trial data at Bothkennar. Castro (2007) reported observation of a conical zone of surface heave with a maximum recorded value of 290 mm following the installation of a trial group of stone columns in the field, using the dry bottom-feed method. Seven columns with an average diameter of 0.8 m were installed through soft to firm alluvial clay strata to a depth of 9.0 m at 2.8 m centres. Figure 6.3 shows the column layout and the locations of piezometers installed prior to the stone column construction. It is unclear, however, if the heave was a local maximum and the overall heave would not appear to have been reported. Egan et al. (2008) describe a soft clay site in Lincolnshire, UK, where 150 mm of surface heave was measured during the installation of stone columns on a 2.0 m square grid pattern and to a depth of 5 m through soft alluvial clay (20 kN/m^2 < C_u< 50 kN/m^2, 30% < W<70%) over a relatively large area of 80 m x 70 m. The average recorded stone column diameter was 450 mm and the volume of heave (based upon levelled sections) equated to 75% of the volume of stone aggregate in the columns. For another project described by Egan et al. (2008) and referred to as 'Contract B', three separate rows of columns, each comprising five columns, approximately 5 m long and with average diameters of 550 mm were installed in soft Carse Clay near Stirling in Scotland. Contours of the average magnitude of heave estimated from a grid of levels taken around one of the column groups, are shown in Figure 6.4. The average maximum heave was around 70 mm along the centreline, diminishing with distance from the column positions. In this case, the volume of heaved ground was estimated to be approximately 35% of the volume of stone aggregate installed in the columns, determined from levelling and contouring of recorded heave around the stone column positions, i.e. levelled sections.

It is perhaps also important to note that the influence on the stress state becomes essential when this heave movement is restrained e.g. by an overlying stiff layer, i.e. surface 'crust'. Serridge (2001) describes field trials of partially penetrating vibro stone columns to support an 8.1 m square concrete raft foundation with thickened edge beam (Figure 6.5), at Bothkennar. Some 25 stone columns, up to 6 m long were installed by the dry bottom-feed method, with the installation procedure following that adopted in the current trials, namely the practice of forming an enlarged 'end-bulb' at the toe of each stone column. A borehole magnet extensometer installed prior to treatment between stone column positions 8 and 9 (Figure 6.5) which were required to support the perimeter edge beam (downstand) of the Bothkennar raft, was used to measure the magnitude and distribution of ground movements (including soil heave) along the column length. Figure 6.5 also shows the top magnet marker, located about 0.5 m below ground level between columns 8 and 9. A reported maximum of 425 mm of heave, with overall final ground heave of about 370 mm, was recorded for the (Bothkennar) raft area, which developed as the treatment progressed and stone column intensity increased. The extensometer gauge demonstrated that the largest (>50%) upward displacements (heave) occurred in the lower part (bottom 2 m) of the column(s), where a substantial proportion of the stone column aggregate was placed to form the 'end-bulb' and also to a lesser extent immediately below the crust (Figure 6.6a). It was estimated, based upon recording of stone consumption during column construction, that around 60% of the total stone aggregate volume was consumed over the bottom third of the column length, (Figure 6.6b) with the total volume of heave recorded equating to around 27% of the stone aggregate introduced into the ground.

It is recognised amongst ground improvement practitioners that some degree of surface heave is to be expected in soft clay soils, certainly where the dry bottom-feed technique (a displacement technique), is adopted. Whilst it could be argued that this demonstrates that the stress regime around the column is being altered and improvement of the ground is being achieved and by inference therefore not detrimental to the final foundation performance, this may also explain why heave is rarely measured in the field. However, this is arguably a simplistic approach as heave at depth, associated with 'end-bulb' construction for example, or heave generally, which will be influenced by construction method and site controls, could recover prior to or during application of foundation loads, resulting in additional settlement(s) over and above that predicted by conventional vibro stone column settlement calculations. Figure 6.7 for a raft on partial depth stone columns at Bothkennar (Serridge, 2001) described earlier, shows partial recovery (100 mm) of the ground heave induced by treatment, plotted for a period of one year before foundation construction and loading. This shows the recovery to be in proportion to the original displacement.

It should be recognised that very little quantitative guidance is available to estimate how much heave can be expected for any given arrangement of stone columns. However, defining the Perimeter Ratio *P* and Heave Ratio H_p according to the following equations (after Egan et al., 2008), allows an approximate relationship to be produced:

$$P = \frac{\Sigma \text{ Column perimeters in a group}}{\text{External perimeter of the group}} \quad \text{--} \quad 6.1$$

$$H_p = \frac{\text{Volume of heaved ground}}{\Sigma \text{ Volume of stone columns}} \quad \text{--} \quad 6.2$$

where: the column perimeter corresponds to the volume of the installed stone columns (i.e. surface area of column multiplied by its length) and the external perimeter of the group refers to the area or geometry of the 'foundation' envelope enclosing the stone column group.

Values of *P* and H_p have been calculated for the heave recorded during the trials undertaken at Bothkennar and are summarised in Table 6.2, together with data reported by Serridge (2001) for the raft on partial depth vibro stone columns at Bothkennar, and also Egan et al. (2008) for a soft clay site in Lincolnshire, and 'Contract B' in Carse Clay, as previously mentioned. The heave data are plotted in Figure 6.8 and indicate a trend which implies some form of relationship between stone column layout (spacing and lateral extent of stone column treatment) and amount of heave observed. The closer the column spacing for a given column arrangement, the greater is the magnitude of heave, as has been observed and demonstrated in the current field trials. It is evident that the process of stone column construction causes lateral and vertical soil displacement and that the proportion of lateral to vertical strain is a function of the lateral confinement of the host soil and adjacent previously installed stone columns. Single columns, rows and small groups of columns (e.g. under small or narrow footings), exhibit less vertical heave than large grids of columns (e.g. under a raft or slab). It is important to recognise that the dry bottom-feed technique is a displacement technique, so one would expect significantly more heave with this technique than with the wet top-feed technique, which is a replacement technique.

At several stages during the installation of the trial stone columns at Bothkennar, build-up and subsequent sudden release of air pressure from the annulus around the vibroflot in the bore was evident, accompanied by ejection of clods of very soft dark grey-black clayey silt (slime). Similar material was observed forming a smear on the external surface of the vibroflot as it was slowly withdrawn from the bore during stone column construction (Figure 6.9). This is considered to be attributed to air jetting pressures being too high, in combination with excessive disturbance and re-moulding, at least temporarily, of the soft sensitive Bothkennar Clay soil, resulting from the vibratory effects of the vibroflot. It is perhaps important to recognise that the benefit(s) of a tightly compacted (closely spaced), stone column arrangement may be offset by the extent of disturbance caused by imparting excessive energy to the ground. This was observed during the current field trials – exacerbated by forming the 'end-bulb' and leaving the vibroflot in the ground for too long, leading to the excessive soil disturbance and remoulding described above and impacting on pore pressure dissipation rates.

Similar observations were made for the Bothkennar raft on stone columns described previously, Serridge (2001). This also emphasises that the practice of constructing an 'end-bulb' for partial depth vibro stone columns should in fact be best avoided if heave and level of soil disturbance is to be controlled. For this reason it is advocated that partial depth vibro stone columns in soft cohesive deposits (clays and silts), 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.

Equipment is available where the frequency of vibration in the vibroflot can be controlled, with main applications to date having been for earthquake liquefaction mitigation in essentially sandy soils (Slocombe, 2001) outside the UK. The wet top-feed technique would potentially cause less ground disturbance, as intimated previously, but its high water demand and requirements for effluent control and disposal would most likely preclude its use as discussed in Chapter 1. Clearly the ability to modify frequency of vibration when constructing stone columns in soft clay soils, particularly sensitive clays, utilising the dry bottom-feed technique, warrants further research and investigation and in order to limit disturbance effects.

6.2 Determination of stone column diameter

Stone column diameter (which is a reflection of vibroflot equipment dimensions, installation method, soil undrained shear strength and stone consumption), in soft clay soils is one of the most important influences on load capacity and also magnitude of settlement reduction within the treated depth (Greenwood, 1991). Whilst the expected range of stone column diameter typically achieved with the dry bottom-feed method was eluded to in Chapter 1- Section 1.2.3, average values of stone column diameter have therefore been obtained from both direct and indirect sources during the field trial(s), as described below:

- *Direct sources:* Excavation of the upper 4.0 m of the two 6.2 m long trial (test) stone columns (column 45 (test column 1) and 46 (test column 2)) installed at either end of the field trial location (Figure 4.10b) showed the columns to be well formed and not contaminated with fine-grained inclusions (clay/silt) from the surrounding (host) soil. A narrow zone of discoloured dark grey clay/silt surrounding the columns was observed and is considered to represent disturbed soil from a darker soil horizon near the base of the column which had adhered to the vibroflot and smeared the sides of the bore during stone column construction, as intimated and discussed previously (Section 6.1). Figure 6.10a shows one of the two test stone columns exposed at 1.8 m depth below existing ground level. In Figure 6.10b column radius (r_o), to reflect the fact that this parameter is used in conventional vibro design calculations, is plotted against depth for the two test columns. In Figure 6.12 this data has been expanded to include installed trial stone columns exposed at founding level following their installation and prior to trial footing construction (see Figure 6.11a and b). The results (Figure 6.12) show the average measured diameter of these (test) stone columns was fairly consistent at 0.75 m (0.375 m radius) within the upper 4.0 m of the soft Bothkennar Clay (although marginally less within the overlying thin crust), with no evidence of a larger diameter 'end-bulb' visible within this depth range. This value corresponds reasonably well with that recorded for stone columns constructed by the dry bottom-feed technique in soft clays of similar undrained shear strength, for example at Guiyhurn in the Fenland of East Anglia (UK), Figure 1.3b and at Grangemouth (Egan et al., 2008). Equivalent average diameters were of the order of 700-750 mm in the Bothkennar crust.

- *Indirect sources:* In Figure 6.12 actual direct measurements of stone column diameter are compared with estimates of stone column diameter based upon stone aggregate consumption, shown as (average) % of stone aggregate introduced into different sections of the overall stone column length, derived from recording of the number of batches (of known volume) of stone aggregate introduced at each stage of column construction to the surface. The data confirm the existence of a basal 'end-bulb' extending over a length (vertical height) of between 1.5 m and 2.0 m at the toe of each stone column and having a diameter of between 1.2 m and 1.4 m corresponding to a factor of between 1.6 and 1.8 higher than the average diameter of the main 'shaft' of the column - this was slightly larger than anticipated, where the assumption was made that the average diameter of the basal 'end-bulb' was around 1.25 m. The manual method of monitoring stone consumption has led to some scatter in the calculated diameters, although there are also similar variations in the actual (direct) measured values. For comparison purposes, the stone consumption estimate of the rig operator for the installed trial stone columns was also considered. The total quantity of stone aggregate delivered to site, as determined from the stone delivery tickets, was 132.3 tonnes and with the total reported as having being used in the construction of the stone columns corresponding to around 128.2 tonnes, with the balance of 4.1 tonnes representing surplus and some loss through wastage and spillage during loading and tipping of aggregate into the supply hoppers on the bottom-feed installation rig. Based upon the foregoing values, an average overall aggregate consumption of 1.20 tonnes/linear metre of stone column was calculated. It should be recognised, however, that because of the construction of the 'end-bulb' stone consumption was inevitably higher at the toe compared to the main shaft of the column as previously intimated. Also the top diameter is smaller because of the effect of the stiff crust. Based upon a compacted bulk density of 18-20 kN/m^3 an equivalent average diameter of around 0.95-1.0 m is obtained for the entire installed column length. There is no evidence at this stage in the trials to suggest that there was any significant advance of stone column aggregate vertically into the soil below the maximum penetration depth of the vibroflot, with the majority of the introduced aggregate at the base (toe) of the column having been displaced laterally into the surrounding soft soil. The 'torpedo' shape of the vibroflot tip and the vibratory motion, principally in the horizontal plane, would tend to encourage this anyway. The 750 mm stone column diameter recorded in the field for the main 'shaft' of the columns (utilising the direct and indirect methods described above), exceed an initial design assumption of 700 mm diameter columns, which had been based upon discussions with Specialist Contractors and the authors previous experience of stone column installation at Bothkennar, Serridge (2001).

Figure 6.13 illustrates monitoring of field instrumentation during stone column installation. Figure 6.14 shows a typical spreadsheet record for the construction of column 30 (trial footing 4).

Whilst there is reasonably good agreement between the column diameter determined by both the direct and the indirect methods, this was based upon fairly strict site controls, where in addition to the rig operator recording the number of calibrated bucket loads of aggregate used in the construction of each stone column, independent manual monitoring of number of bucket loads used at each stage of column construction over its entire length was undertaken (albeit in the absence of electronic monitoring), and which may not necessarily be achieved on a standard vibro stone column project. It is evident, however, that a method of being able to monitor stone consumption electronically by, for example, batch weighing would be useful to facilitate accurate estimates of stone consumption in the column length during its incremental construction and in order to facilitate better quality control.

6.3 Lateral displacements and stresses

Measurements before and after installation of column 31 at the location of trial footing 4 showed that the top of the special (electrolevel) inclinometer gauge (described in Chapter 4 and 5) had displaced sideways by 100 mm from its orginal position of 0.5 m from the centre of the stone column installation point. Analysis of readings from the lower electro-level units revealed similar lateral displacements over the 2.5 m depth of the gauge. The data is presented in Figure 6.15. This appears to be in line with what might be expected for stone column installation in a soft clay deposit for the chosen stone column spacings. Assuming 100 mm displacement in all horizontal directions and noting a combined bottom-feed vibroflot and stone feed pipe arrangement dimension of equivalent to 450-500 mm, this corresponds reasonably well with the recorded diameter of 750 mm for the main shaft of the column within the soft Bothkennar Clay, allowing for soil displacement and some interpenetration of introduced aggregate into the (host) clay during column construction.

By way of comparison, for the trial raft on partial depth vibro stone columns at Bothkennar discussed previously (Serridge, 2001), about 105 m^3 of stone aggregate was used in the construction of the columns but the average overall surface heave was only equivalent to a volume of about 35 m^3. Only very small compression of the soil below the treatment depth took place and it is evident therefore that lateral ground displacement of about 70 m^3 took place. Assuming this occurred outside the treatment area only, an overall lateral ground displacement of about 330 mm would have taken place around the outside of the treatment area. Whilst there were no actual measurements made of lateral ground movements around the raft treatment area, measurement of around 100 mm close to a single perimeter column in the current field trials beneath narrow footings (described earlier), suggest an overall displacement of 330 mm is not unreasonable. However, measurements of earth pressures and pore water pressures during and subsequent to column installation may also be important factors in the overall effect observed.

The results of typical earth pressure measurements for stone column installation during the trials are plotted for stone columns 30 and 34 in Figures 6.16a and b, with additional raw data given in Appendix 6.1. Pressure cells alongside columns within the critical zone (zone of anticipated column bulging), measured a sharp increase in lateral (horizontal) earth pressure as the vibroflot passed the (pressure) cell positions during initial penetration, with even higher values obtained when constructing the stone columns just above or below the cell positions, and attributed to the sideways displacement of the soil. Clearly the maximum (peak) stress is reached, when the depth of the vibrating tip of the vibrator (represented by + during initial penetration phase and x during column construction phase in Figure 6.16a and b) reaches the level of the gauge (pressure cell). It was further observed that during column construction the recorded cell pressure dropped during slight withdrawal of the vibroflot from the base of the bore to permit stone aggregate to flow out and subsequently rose again on re-penetration of the vibroflot to compact the introduced charge of aggregate. This can be observed to some degree in Figures 6.16 a and b. Such changes were detectable throughout the length of stone column construction and are attributed to contact pressure fluctuations (cyclic vibrations) as the vibroflot was raised (low frequency) and lowered into the ground to compact the introduced stone aggregate (high frequency-approximately 50Hz). Similar and perhaps more refined observations and recordings were made by Castro and Sagaseta (2012), (Figure 6.17) employing electrically driven vibroflots and where it was also evident from research by these authors, that the (total) stress state rises to a value of up to around 1.5-1.6 times the initial stresses due to the ground displacement when the location of a column installation encroaches closer to the measurement location. Once a critical distance (of about four to five times the column diameter) is reached, the displacing virtue is apparently superimposed by stress reducing effects which can be addressed as remoulding (disturbance) and dynamic excitation leading to a significant (temporary) loss of soil strength.

Therefore, looking at the stress development during the installation of each single column, from the observations in this research and also research by Castro and Sagaseta (2012), one can essentially distinguish three different phases during stone column installation or construction: 1) – insertion of the vibroflot into the soil, 2) – alternating lift and sag cycles during introduction and subsequent compaction of the introduced stone aggregate, supported by air pressure assistance, 3) – pausing to recharge the stone feed hopper on the bottom-feed equipment. Therefore the lift and sag cycles invoking the lateral displacement of the stone columm material can be readily identified in the stress development.

Again, by way of comparison, for the trial raft on partial depth vibro stone columns at Bothkennar, described by Serridge, 2001, a total of eight BRE miniature push-in pressure cells had been installed beneath the raft footprint to measure total earth pressure alongside stone column positions during and also subsequent to the installation of stone columns. Four cells were installed to measure lateral earth pressure alongside the upper 3 m of selected columns. In Figure 6.18, Figure 6.19a and Figure 6.19b the location and depth of the cells in relation to columns are illustrated and the measured pressures are shown on the left hand plot during the column installation. They indicated that the pressure increases at 1.6 m and 3 m depth were greater between columns than outside. This might be anticipated in view of the 'confining' effect between closely spaced (1.5 m centre) columns. The rises were significant during the installation period (up to about 8 m head of water, or 80 kPa, excess pressure was measured). It is perhaps more surprising that up to 5m head of water, or 50kPa changes in vertical pressure were also measured during similar column installation in the same treatment area. Although the major proportion of these increases was not sustained after treatment, but returned to their equilibrium values within a short period, some residual pressures remained for several months, perhaps influenced to some degree by the confining action of the closely spaced stone columns, but probably more significantly by disturbance forces during column installation.

For the current field trials (trial footings) vertical pressure was recorded 0.3 m below the toe of stone columns 28 (footing 3); 30 (footing 4); 32 (footing 5) and 34 (footing 6) (Figure 4.11b ; Figure 4.11c ; Figure 6.20) during their installation. The toe depths of the columns corresponded to 4.2m; 6.2 m; 8.2 m and 6.9 m respectively from the installation rig working platform levels. During initial vibroflot penetration all the cells responded with an increase in pressure recorded as the vibroflot penetration depth increased. Values increased by as much as 100 kN/m^2 during initial vibroflot penetration and with a further increase of 20 kN/m^2 recorded below column 30 (see Figure 6.16a, also Figure 4.10b for location) as the 'end-bulb' of the column was being constructed between 6.2 m and 5.7m depth (instrument level = -3.75 m AOD). During the construction of the remainder of the stone column the pressure reduced to approximately 80 kN/m^2, but which remained fairly constant during construction of the upper section of the stone column to the surface. The cells below the toe responded during compaction of the uppermost section of the instrumented columns, irrespective of length, i.e. short 4.2 m or long 8.2 m columns (Figure 6.16a and b; 6.20; Appendix 6.1), indicating that there was stress transfer from the impact forces imparted by the vibroflot down the entire installed stone column length (perhaps also reflecting the temporary remoulded condition of the soil if one relates this to the Hughes and Withers (1974) minimum column length predictions described in Chapter 4 and 5 for peak and remoulded soil strengths). It is important to reiterate that during column construction the recorded pressure dropped during slight withdrawal of the vibroflot from the base of the bore to permit stone column aggregate to flow out and subsequently rose again on re-penetration of the vibroflot to compact the plug of stone aggregate. Furthermore, such changes were detectable throughout the length of stone column construction. The preceding displacements could cause shear failure so that p_{max} could be equal to $\sigma_v k_p + 2C_u / k_p$. It should be noted that some of the pressure cells e.g. beneath the toe of column No's 28 (footing 3) and 32 (footing 5), were still registering pressures in the range 5 to 15 kN/m^2 some 6 days after stone column installation. This is considered to be attributed to the fact that the cells will also effectively be registering pore pressures. Air flush escape into the ground and general construction vibration may have contributed to (pore) pressures remaining elevated. Another consideration is that the soil is unlikely to be behaving as an elastic material. However, one cannot rule out some damage to the pressure cell, particularly if some stone aggregate was forced downwards into the (pressure) cell during construction of the column toe.

In addition to observations during the current field trials, and observations by Serridge (2001), Kirsch (2008) have also reported increases in total stress after column installation, but as noted by Mc Cabe et al. (2009), it is the equalised effective stress around columns (once pore pressures have dissipated), which influences column performance under load. It is also evident that the positive effects of column installation in soft soils are due to the increase of effective horizontal stresses after the consolidation process that follows the expansion of the cavity. For example, Priebe (1995) already assumed in his analysis a value of the soil lateral earth pressure coefficient of 1, which is higher than the initial value at rest for most soils. The only published field measurements of the post-installation lateral earth pressure coefficient found by the author were from Kirsch (2006) at two different field trial sites. The soil at the first field trial site was a silty clay with a relatively high initial lateral earth pressure coefficient at rest, K_o = 0.91, while the second trial was undertaken in a silty sand with K_o = 0.57. The columns were constructed using the dry bottom-feed method and their diameter was 0.8 m. Despite the differences between the two field trial sites, the same range of values and pattern of variation with distance from the column axis of the normalised lateral earth pressure coefficient were found. The lateral earth pressures (K_o) clearly influence the improvement factor achieved with stone column treatment as it gives the amount of lateral support for the column and influences its yielding. The K_o value is therefore an important state parameter in stone column design and should be recognised as such.

6.4 Pore pressure changes

All piezometers (P7 to P10) adjacent to column 30 (trial footing 4) and column 34 (trial footing 6) at 0.5 m and 1.1 m depth below proposed field trial founding levels (-0.5 m for footing 4 and -1.20 m for footing 6) responded to the stone column installation. Highest recorded pore pressures by the field instrumentation at the trial footing locations, during initial penetration of the vibroflot, occurred as it passed the instrumentation (piezometer) locations (Figure 6.20, see also Figure 4.11b and 4.11c) and most notably at the same instrumentation locations (and depths) during subsequent stone column construction – once stone column placement and compaction started, pore pressures rose sharply (Figure 6.16a and 6.16b), with a maximum 68 kN/m^2 excess pore pressure recorded (which subsequently dissipated) in response to the total stress increase in the ground due to dynamic excitation from the vibroflot and compaction of the introduced stone column aggregate. It was evident from the field trial observations of the column installation that the penetration of the vibroflot was not monotonic, but was withdrawn up and down several times, as indentified previously. The lifting cycle causes slight contraction of the cavity (bore) and sudden decrease of pore pressure that is recovered once the vibroflot re-penetrates, compacting stone aggregate introduced into the bore and pushing or forcing the stone backfill radially outwards into the adjacent soil. The radial expansion is accompanied by vertical displacement of the soil, which manifests itself as ground heave at the surface, and which has been discussed previously. The increase and then decrease of pore pressures arising from column installation indicate that the ground is squeezed laterally as the columns are installed, causing an immediate undrained increase in pore water pressure.

Field measurements relating to more widely spaced stone columns covering a wider foundation area (Serridge, 2001; Kirsch and Sondermann, 2003; Gäb et al., 2007; Castro, 2007) also clearly show that pore pressures rose extremely rapidly during initial vibroflot penetration. The pore pressures reach a peak during subsequent column construction (with highest levels achieved when the depth of the vibroflot is coincident with the depth of the piezometer instrumentation) and later dissipated. Figure 6.16a and b reproduces some of the field data for the Bothkennar trials (see also Appendix 6.1) and Figure 6.17 provides some data from Castro and Sagesta (2012) supporting the above trends. Gäb et al. (2007) reported, during field trials to investigate the performance of a floating stone column foundation under a widespread load, that pore pressure rapidly increases when a stone column is being built. The closer the vibroflot encroaches on installed piezometer instrumentation the higher and more pronounced are the amplitudes of the excess pore water pressures, with the maximum recorded value occuring at the location of the piezometer, during column construction in soft clay. The combined earth pressure cells in the Bothkennar field trials confirm these readings. The values of the peaks of excess pore pressure during the construction of trial columns 30 (footing 4) and 34 (footing 6) are shown respectively in Figure 6.16a and b. The observation of recorded peak pore pressures coinciding with the vibroflot passing the piezometer instrumentation level appears to be generally in line with equivalent data for displacement piles (Gavin and Lehane, 2003; Mc Cabe et al., 2008). It is important to recognize that unlike a driven pile, a vibroflot (vibrating poker) will pass any given horizon more than once as the stone is compacted in lifts.

Dissipation of excess pore pressures during the current Bothkennar trials commence immediately after column construction beneath the trial footings (Figure 6.16a and b; Appendix 6.1). Pore pressures generally returned to pre-treatment levels within 6 days of column construction for columns that took between 14 and 22 minutes to complete. While the response of piezometers will be sensitive to soil permeability at the particular location being monitored, the above observation indicates relatively rapid dissipation of pore pressures and re-consolidation of the soil between columns and implying that any soil strength reduction due to soil disturbance during installation was potentially only temporary. It is important to highlight, however, that where column installation took 28 minutes, for practical reasons associated with stone aggregate supply and delays due to monitoring of the field instrumentation, pore pressures remained elevated for up to 48 days (Appendix 6.1). The above observations emphasise the importance of minimising column construction time in soft sensitive cohesive soils. This would suggest that in order to prevent excessive soil disturbance of the soil fabric, the vibroflot should only be permitted to penetrate the introduced charge of stone aggregate between one and two times and not be held unnecessarily in contact with soft sensitive clay soils during column construction.

By way of comparison, it is again useful to introduce some pore pressure observations for field trials described by Serridge (2001) for the Bothkennar raft on partial depth vibro stone columns. A total of six pneumatic piezometers (P1 to P6) had been installed prior to column installation at different locations and depths within or close to the area where stone columns were to be installed beneath the proposed raft footprint. In Figure 6.21 the location and depths of piezometers between column positions is illustrated. During the construction of adjacent stone columns (beneath edge beams) high pore pressures were generated within the silty clay deposits, in particular at piezometer installation levels 5 m below ground level, where the columns' enlarged bases were formed, but also to a lesser extent at a depth of 3.0 m. A maximum 100 kN/m^2 rise in pore water pressure (maximum 10 m head of water - excess pore water pressure) was observed between stone columns during column construction at the deep (5m) piezometer (P6) location for columns spaced at 1.5m apart (and again corresponding to the location where the base 'bulbs' were being formed (Figure 6.20)). The elevated pore pressure reduced to around 30 kN/m^2 within 24 hours and had returned to pre-treatment levels after about 2 months. The piezometers continued to measure pressure in excess of pre-treatment values (i.e. elevated values) for at least two months after treatment, with pressure at 5 m depth between column ('end-bulbs') remaining elevated for up to four months. This was attributed to the vibroflot having remained in the ground, i.e. in direct contact with the soft clay, for too long during column construction (particularly during the construction of the 'end-bulb') resulting in excessive soil disturbance. Figure 6.21b illustrate the pore pressures measured at the different piezometer positions during and after stone column installation. Following pore pressure dissipation small observed variations were attributed to seasonal fluctuations in natural ground water levels, confirmed by standpipe readings from a piezometer installed about 5 m outside the treated area. In contrast, it is important to note that a piezometer at 1.6 m depth (Figure 6.5; Figure 6.21a) registered little sustained excess pore pressure during column installation. This was attributed to the presence of a more permeable shelly horizon. Piezometer P2 at 3 m depth between columns indicated lower excess pore pressure than P3 located at the same depth 0.75 m outside of the treatment area. This was considered to reflect the reduced drainage path lengths available between columns. P4 outside the treatment area also registered several metres excess head of pressure and P5 indicated the lowest rise overall. The response of the piezometers and the magnitude of increase in pressure suggest that most of the apparent increases in total stress were, in fact, changes in pore pressure. Whilst an increase of excess pore pressures with depth has been measured in field trials by Castro (2007) no detailed explanation is given, but which may stem from the increase of undrained shear strength with depth, which can be theoretically proven for an elastic-perfectly plastic material in plane strain (Randolph and Wroth, 1979). Furthermore, the observations compare with general trends observed for displacement piles.

The foregoing observations based on the current research and also research by Serridge (2001), carry implications for the timing of load application onto stone column reinforced clay soils via narrow (or widespread) concrete foundations, the construction of which should generally not take place until pore pressures have dissipated to an acceptable level. It is suggested that foundation construction should generally not commence sooner than one week following column construction in soft sensitive clay soils (to allow for satisfactory pore water pressure dissipation). The primary consolidation process appears to take place within a period of a few weeks following column installation, during which time the ground between the columns increases in strength. This process will usually have concluded during the early stages of construction - the evidence being successful completion over many years of a large number of vibro projects in soft clay-albeit historically this would have meant employment of the wet top-feed technique and with the stone column(s) acting as a very efficient drainage pathways for pore water pressures.

It is considered useful to include maximum excess pore pressure ratios $\Delta_{Umax}/\sigma'_{vo}$ (where $\Delta_{U\ max}$ is the free-field vertical effective stress) data for some stone column case histories based upon some recent work by Mc Cabe et al. (2009), in Figure 6.22 with additional supporting information, including approximate YSR values in Table 6.3 where *r* is the distance (or average distance) of the measuring device from the column centre, and *R* is the column radius. The significant sand content of the soil from the Castro (2007) data is likely to be responsible for the lower $\Delta_{Umax}/\sigma'_{vo}$ values. Moreover, the higher $\Delta_{Umax}/\sigma'_{vo}$ values from Gäb et al. (2007) reflect the greater size of the stone column group. In general, the data suggest that the pore pressures are cumulative as each successive column is installed, but the cumulate values will of course be a function of, among other things, the average duration between successive column installations. The value from the Bothkennar trials (which has been taken by Mc Cabe et al. (2009) from Serridge and Sarsby, (2008), based on the current research), plots quite high compared with the rest of the data, indicative of over-working of the ground during the installation of the columns (particularly the 'end-bulb').

6.5 Post installation undrained shear strength

Average pre-treatment peak field vane strengths for the field trial location at Bothkennar varied from about 20 kN/m^2 at 2.0 m below ground level, rising in direct proportion to the effective over-burden pressure, to 60 kN/m^2 at around 20 m below ground level (see Figure 5.10b and 6.23). During excavation of two additional test stone columns (38 and 39 – see Figure 4.10b for location) five days after their installation, measurements of undrained shear strength were undertaken using a hand shear vane tester within about 0.1 m of the edge of the installed stone columns. These values suggested little or no loss of strength when compared to previously measured (pre-column installation) peak values (Figure 6.23) within the zone of anticipated stone column bulging. This would suggest (together with the change in the stress regime resulting from the displacement process previously described), that any disturbance or re-moulding of the soft sensitive clay due to a combination of air jetting and the vibratory action of the poker, was only temporary and that improvement of the ground was being achieved and therefore not detrimental to the final foundation performance. By way of comparison Aboshi et al. (1979) presented field data for a clay soil after installation of compozer columns (Sand compaction piles (SCP) - see Chapter 1) indicating initial reduction of strength by 10% to 40%, which recovered after 1 month (30 days) to reach undrained shear strengths up to one and a half times the undisturbed value. Ogawa and Ichimoto (1963) showed pre-consolidation loads improved by 50% to 100% for no loss of strength from tests on normally consolidated natural clays after remoulding by composer treatment (SCP installation). Whilst these comments are supported by Egan et al. (2008), the researcher is of the opinion, however, that if construction control is inadequate and overworking of the ground takes place, this is likely to result in excessive remoulding of the soil, combined with excessive heave which will have implications for the magnitude of any long term settlements.

Location	Pre installation level, (m AOD)	Post installation level, (m AOD)	Heave (m)
Trial footing 1, (cols 21-24)	2.777	2.92	0.143
Trial footing 2, (cols 25-27)	2.691	2.798	0.107
Serridge (2001)Bothkennar Raft, (cols 1-20);	2.754	3.18	0.435
Castro (2007)	-	-	0.290
Egan et al. (2008)	-	-	0.150

Table 6.1: Recorded heave for stone columns installed in soft clay.

Reference	No. of columns in group	Column length (m)	Average column diameter (mm)	Arrangement	Perimeter ratio, *P*	Heave percentage, Hp %
Castro (2007)	7	9	800	2.8 m spacing	1.05	12
Serridge (2001) 8.1 m raft	25	6	940	1.5 m centres around edge of raft 5 in centre	2.28	27
Bothkennar trials 2 Columns	2	6	940	-	0.99	0
Bothkennar trials 3 Columns	3	6	940	-	0.99	3
Bothkennar trials 4 Columns	4	6	940	-	1.21	5
Mc Cabe et al. (2009) Contract B	5	5.5	550	Single line 1.5 m centres	0.63	35
Mc Cabe et al. (2009) Contract A	Infinite'	5	450	Infinite 2 m square grid	6.60	75

Table 6.2: Case histories of recorded heave, incorporating data from current Bothkennar trials.

Reference	Site location	Approx YSR	Column length (m)	Average column diameter (m)	No. of columns	Piezometer depth (m)
Venmans (1998)	Holendrecht-Abcoude, Netherlands	~ 1.0	5.5	0.65	Large grid	_
Egan et al. (2008)	Scotland, UK	1.2	5.5	0.55	5	2,4
Serridge and Sarsby (2008)	Bothkennar, UK	1.5	5.7	~ 0.75	4	1.6
Castro (2007)	Valencia, Spain	~ 1.0	9.0	0.80	7	4,7
Gab et al. (2007)	Klagenfurt, Austria	~ 1.0	14.5	0.70	36	12
Mc Cabe et al. (2009) Contract B (unpublished)	Scotland, UK	1.2	5.5	0.55	>10	3

Table 6.3 : Case histories with stone column installation pore pressure measurements, (after Mc Cabe et al., 2009)

Bauer Foundations | **Daily Report**

Site		Loc. No.	Sheet No.
Date	Day of Week	Starting Time	Finishing Time

No.	Compaction Point		Time (Hrs.)		Depth (m)	B/fill Bucket Loads	Pressures min/max (p.s.i.)	
	Consec.No.	Ref.No.	Start	Finish			Boring	Compact
1								
2								
3								
4								
5								
6								
7								
8								
9								
10								
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
Totals	Total this report							
	Previous total							
	Total to date							

HOURS WORKED

Job	Name	Hours
F/M		
C.D.		
F.M.		
D.D.		
JCB		
Total Man Hours		

PLANT HOURS

Type	Work-ing	Idle	Break down	Total
Vibroflot				
Powerpack				
Crane				
JCB/Drott				
Dumper				

MOB & DELAY TIMES

Operation	Hours
Setting-up site	
Clearing site	
Setting out	
Plant repairs	
Delay time	

Remarks (Progress Notes, Site Instructions, variations, obstructions, breakdown):

For Client agreed by:

....

Signature

....

Foremans' Signature

Figure 6.1: Sample daily record sheet for reporting stone column installation at Bothkennar

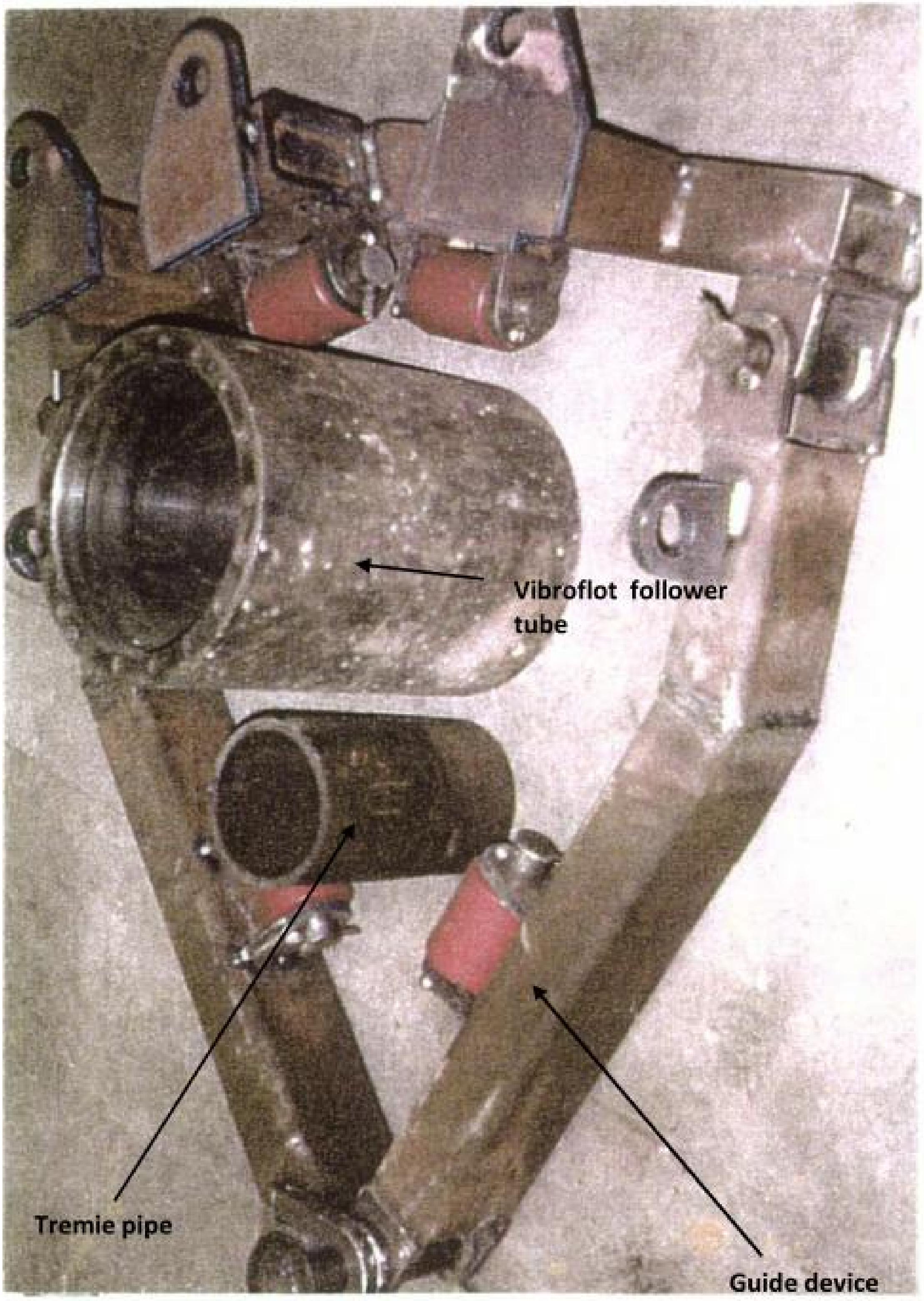


Figure 6.2: Vibroflot ‘guide’ device developed to address verticality issues.

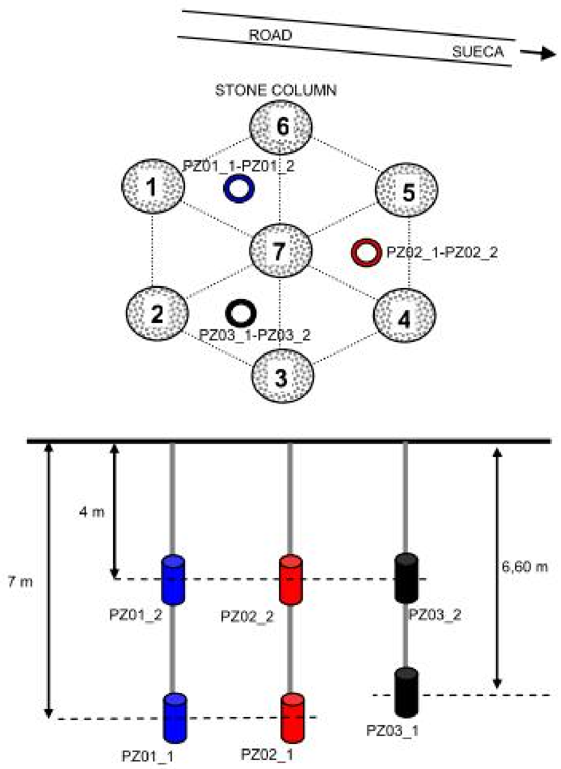


Figure 6.3: Stone column and instrumentation layout for reported heave by Castro (2007)

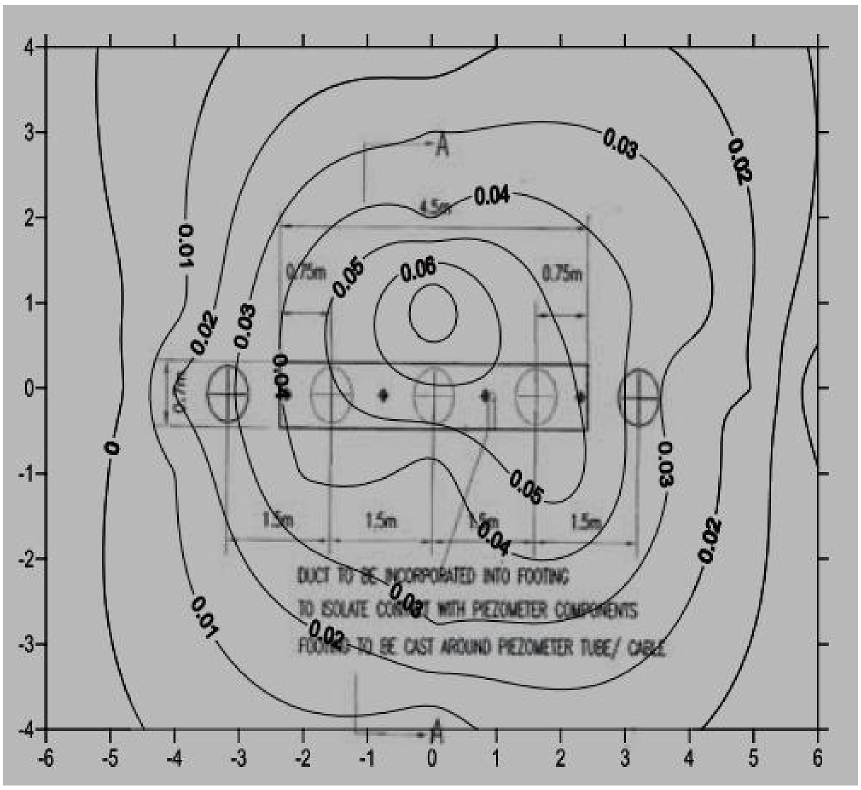


Figure 6.4: Contours of heave around a line of stone columns (after Egan, 2008)

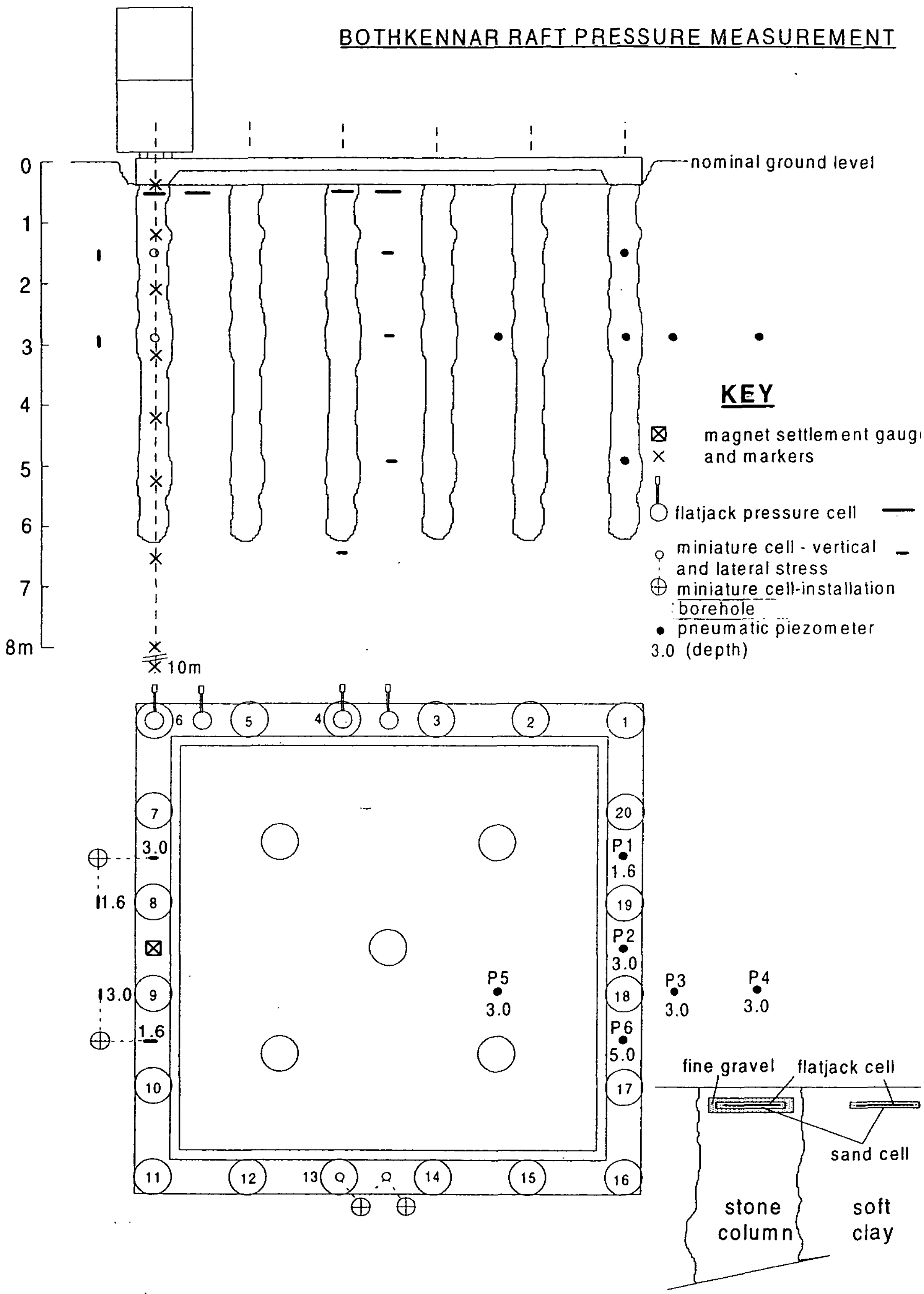


Figure 6.5: Instrumentation installed to monitor ground improvement and foundation performance beneath the Bothkennar raft (after Serridge, 2001).

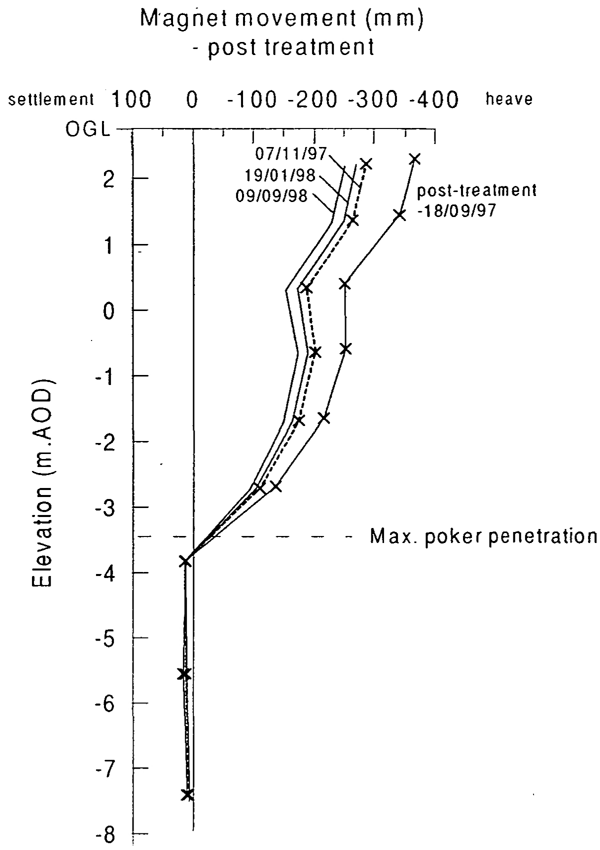


Figure 6.6: (a) Vertical ground movements between Bothkennar raft stone column No's 8 and 9 both during and subsequent to stone column installation (after Serridge, 2001).

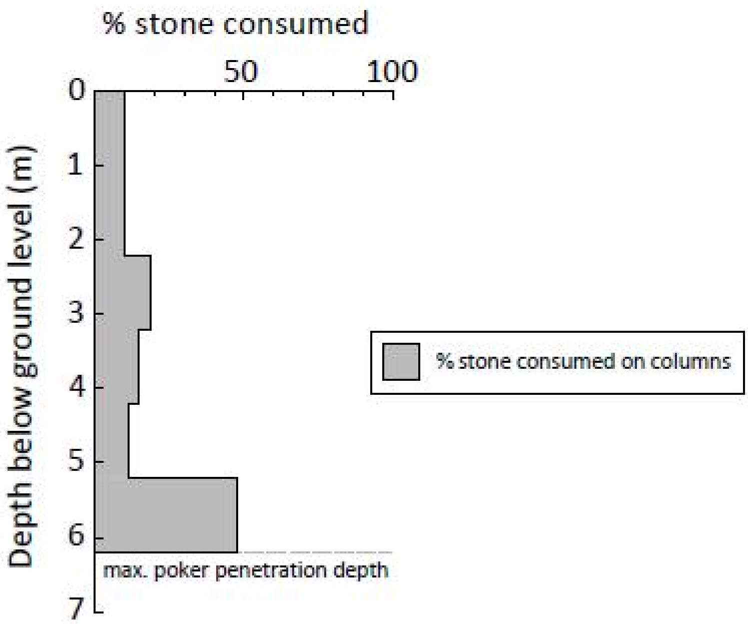


Figure 6.6: (b) Percentage of stone aggregate consumed at different depths during the column construction (incorporating some field data from Serridge, 2001).

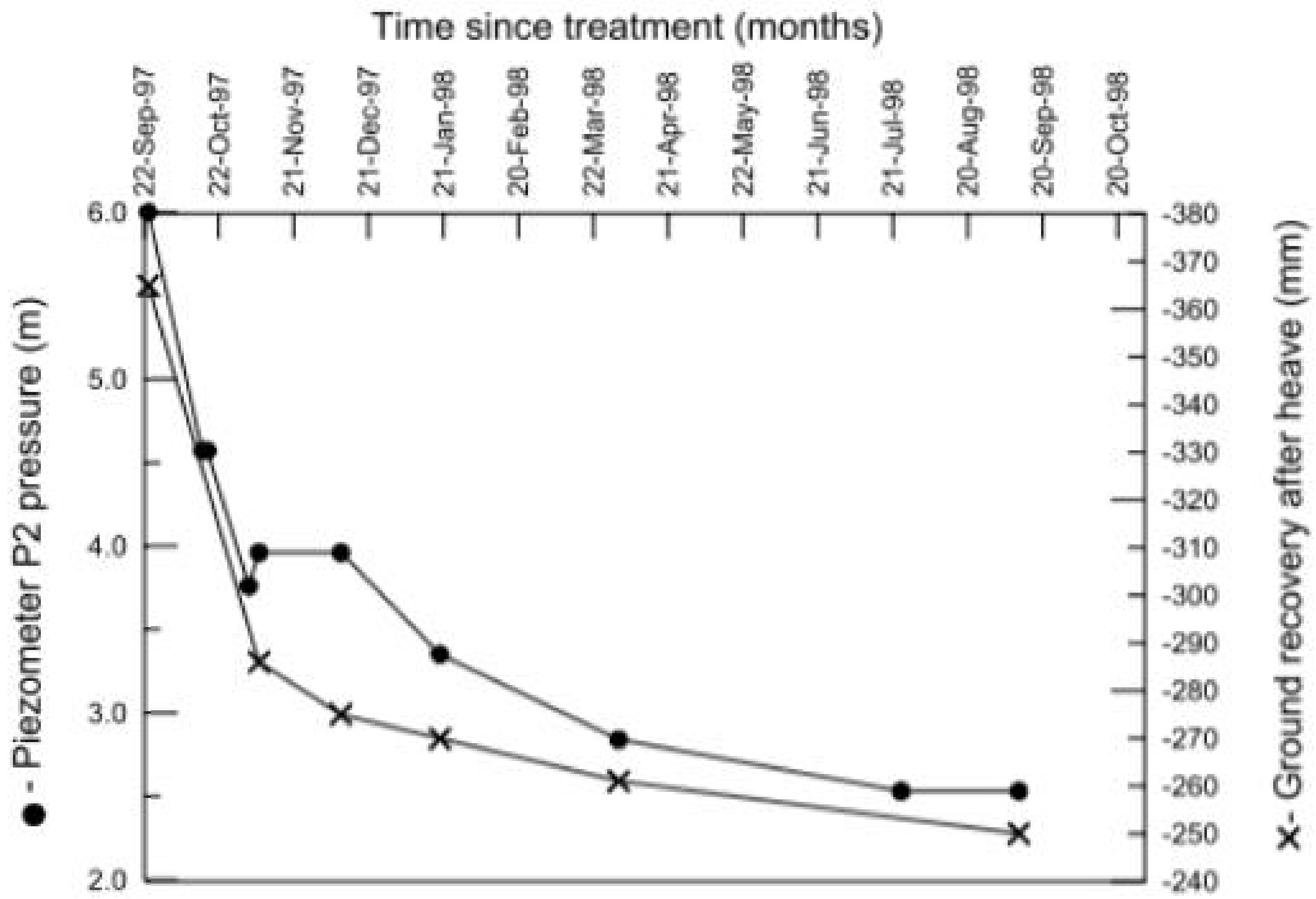


Figure 6.7: Pore pressure dissipation and heave recovery during 12 months following installation of raft columns (after Serridge, 2001).

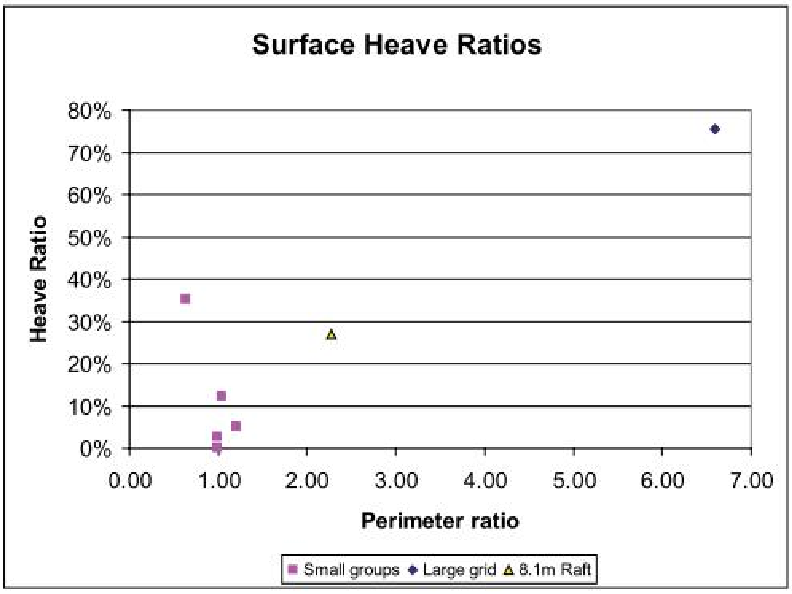
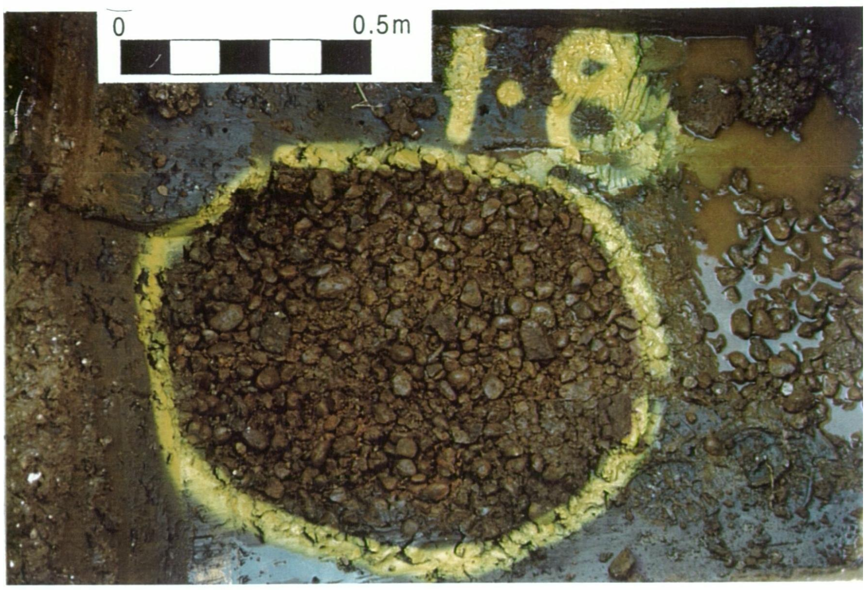
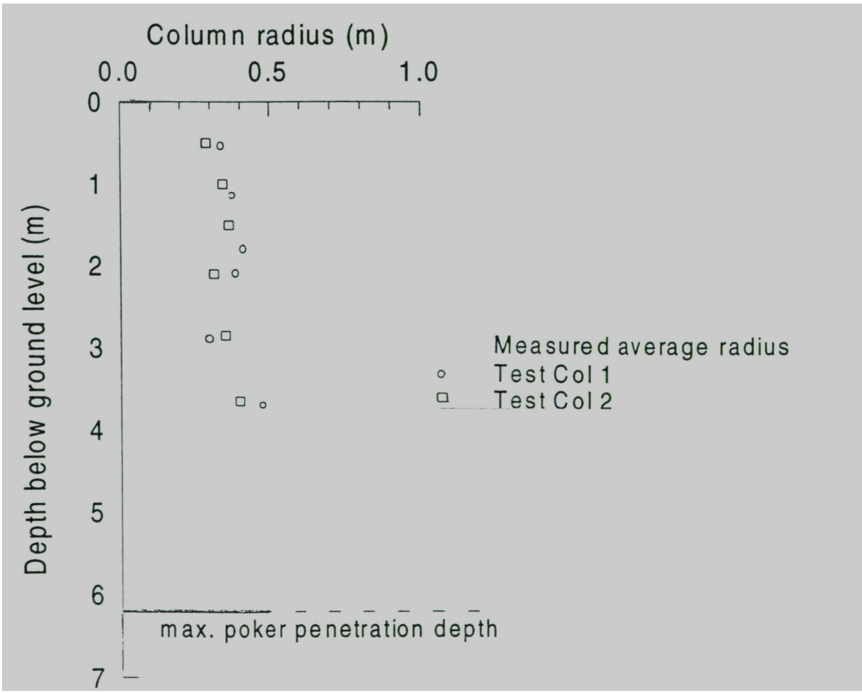


Figure 6.8: Ground surface heave ratios for stone column installation (after Egan et al., 2008).

Figure 6.9: Soil disturbance attributed to the vibratory action of the vibroflot for the current Bothkennar trials.



a)



b)

Figure 6.10: (a) Exhumed stone column exposed at 1.8 m depth (b) Plot of recorded stone column radius from exhumation of test columns.

a)

b)

Figure 6.11: Exposed stone columns at 0.5 m depth within Bothkennar crust (a) Footing 1 (1.5 m column spacings). (b) Footing 2 (2.0 m column spacings).

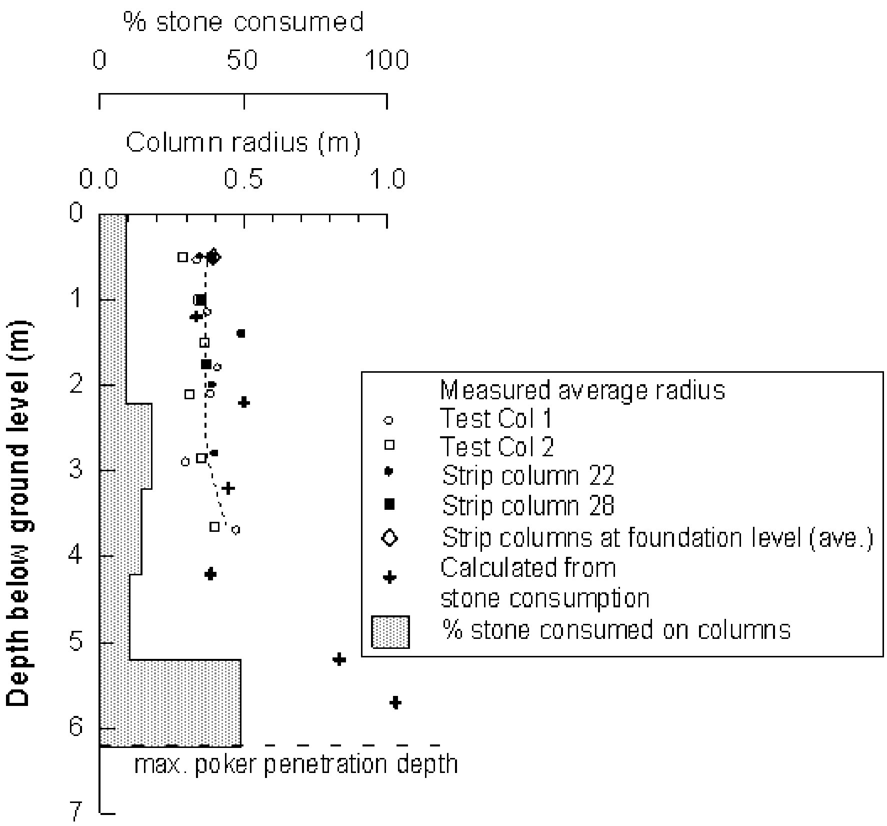


Figure 6.12: Comparison of direct and indirect (+) methods for determination of stone column diameter. Stippled zone = percentage of overall stone consumption at different depths within installed stone columns (including data from Serridge, 2001).

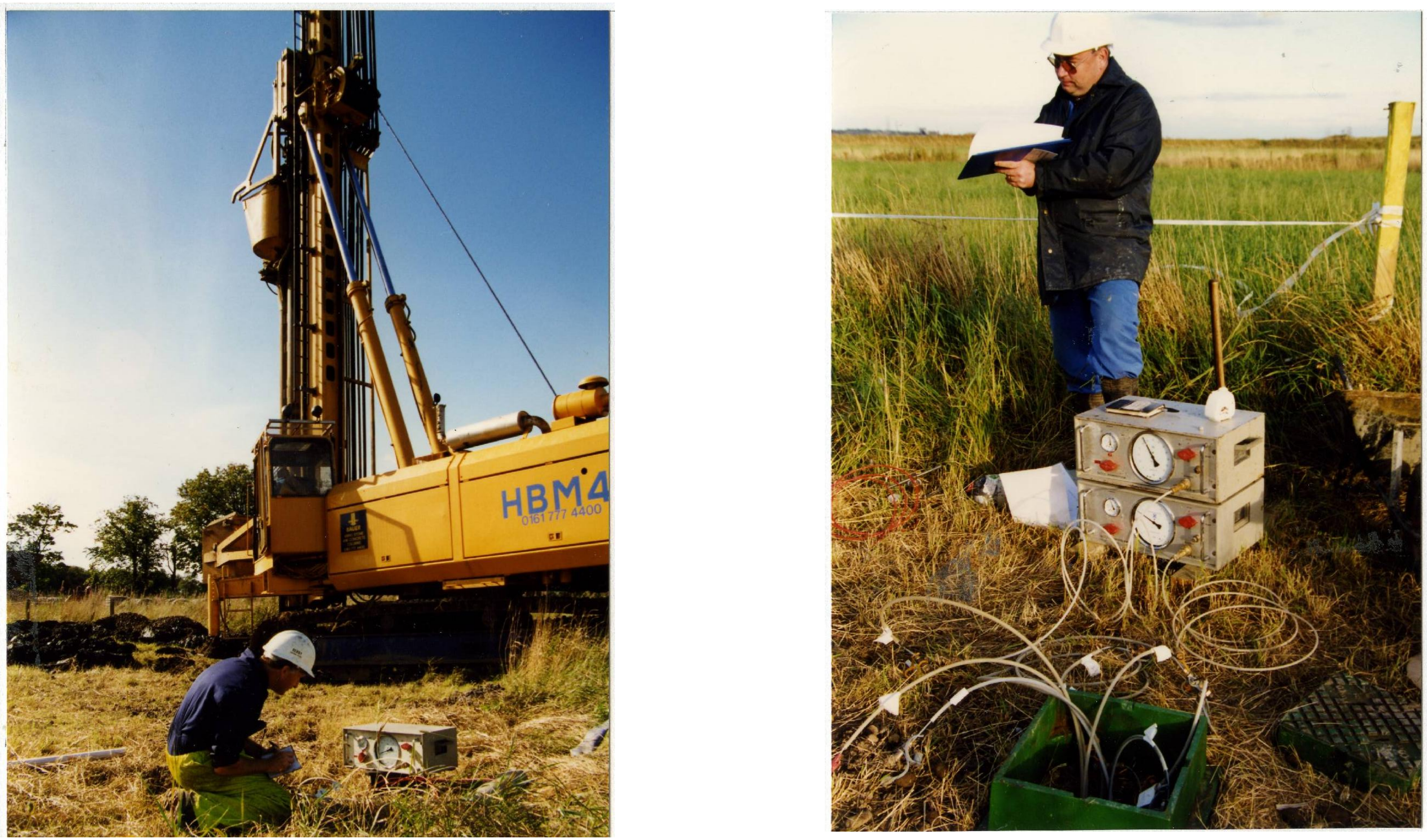


Figure 6.13 : Monitoring of installed instrumentation during stone column installation for Bothkennar field trials.

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	30	
Ground level	2.744	m. AOD
Fill level	2.744	m. AOD (2)
Required column base depth	6.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	5.7	m. (1) = (o)-(m)
Poker penetration level	-2.956	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.88	cub.m
No of buckets:	6	

Associated instrumentation

Mini cell No. 28 - Vert. stress @ 6.5m beneath col 30

Cell zero	1.05	Bar
Box zero	0.11	Bar
Cell elevation	-3.756	m. AOD

Mini cell No. 21 - Horiz. stress @ 1.5m alongside col 30

Cell zero	0.93	Bar
Box zero	0.11	Bar
Cell elevation	1.144	m. AOD

Mini cell No. 22 - Horiz. stress @ 1.0m alongside col 30

Cell zero	1.1	Bar
Box zero	0.11	Bar
Cell elevation	1.744	m. AOD

Piezometer No. 7 - Pore water pressure @ 1.6m alongside col 30

Cell zero	0.15	Bar
Box zero	0.11	Bar
Cell elevation	1.144	m. AOD

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Cell 28 Glotzl box reading (Bar)	-116 Cell 28 Stress elev. m AOD	Cell 21 Glotzl box reading (Bar)	-104 Cell 21 Stress elev. m AOD	Cell 22 Glotzl box reading (Bar)	-121 Cell 22 Stress elev. m AOD	Piezo 7 Glotzl box reading (Bar)	-26 Piezo 7 Stress elev. m AOD	Comments
16/09/97	15:31	0	0	2.744	2.36	9.60	1.13	3.18	1.34	4.19	0.28	2.47	
16/09/97	15:35	0					1.6	7.97	1.65	7.35			Start and Bucket
16/09/97	15:38	3	5.7	-2.956	3.05	16.63			1.32	3.99	0.49	4.61	
16/09/97	15:38	3.5			2.7	13.06							
16/09/97	15:38	3	6.2	-3.456	3.3	19.18							on push to depth
16/09/97	15:39	4			3.5	21.22							Bucket
16/09/97	15:41	6	5.5	-2.756									
16/09/97	15:42	7	5.5	-2.756									Bucket
16/09/97	15:43	8	5	-2.256									
16/09/97	15:44	9	4	-1.256	3	16.12	1.4	5.94	1.55	6.33	0.6	5.73	Bucket
16/09/97	15:46	11	3	-0.256									Bucket
16/09/97	15:47	12	2	0.744			1.47	6.65					Bucket
16/09/97	15:49	14	0	2.744			1.47	6.65	1.75	8.37	0.72	6.95	Finished
16/09/97	18:15	160		installed	2.75	13.57	1.25	4.41	1.45	5.31	0.47	4.41	installed
22/09/97	10:30	8335			2.44	10.41	1.1	2.88	1.38	4.60	0.24	2.06	pre excavation
22/09/97	16:00	8665			2.42	10.21	1.1	2.88	1.36	4.39	0.21	1.76	post excavation
23/09/97	16:00	10105			2.42	10.21	1.09	2.77	1.38	4.60	0.21	1.76	wet blinding
24/09/97	15:10	11495			2.42	10.21	1.1	2.88	1.39	4.70	0.2	1.65	wet concrete
17/10/97	15:10	44615			2.43	10.31	1.1	2.88	1.42	5.01	0.22	1.86	
04/11/97	10:50	70275			2.43	10.31	1.08	2.67	1.4	4.80	0.22	1.86	pre-loading

Figure 6.14: Typical spreadsheet record for trial stone column (column 30) installation.

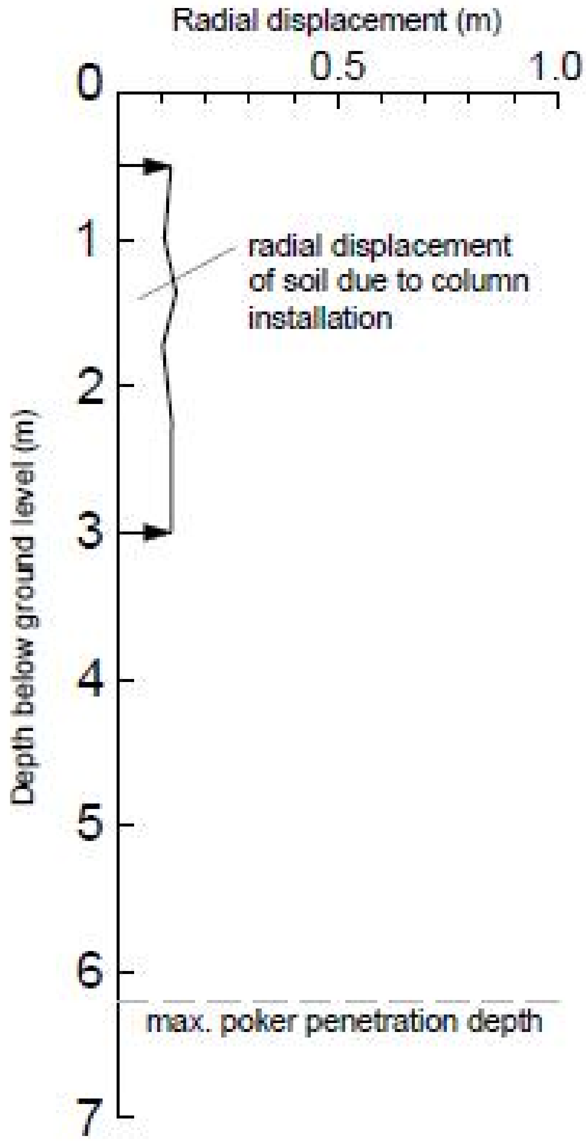


Figure 6.15: 100 mm radial (lateral) displacement (in direction of arrows) recorded during installation of stone column 31 (trial footing 2).

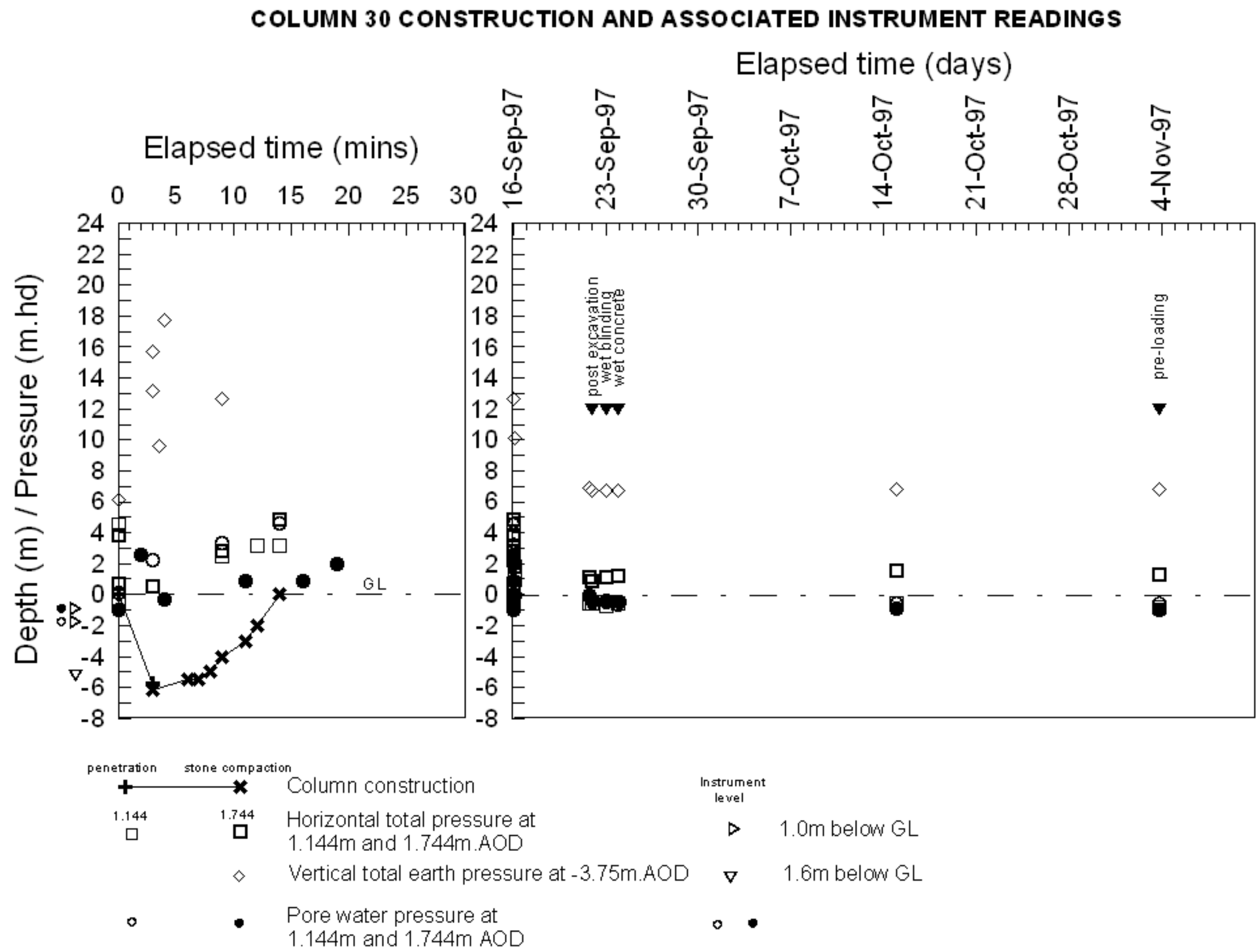


Figure 6.16: (a) Earth pressure and pore (water) pressure measurements associated with Installation of stone column 30.

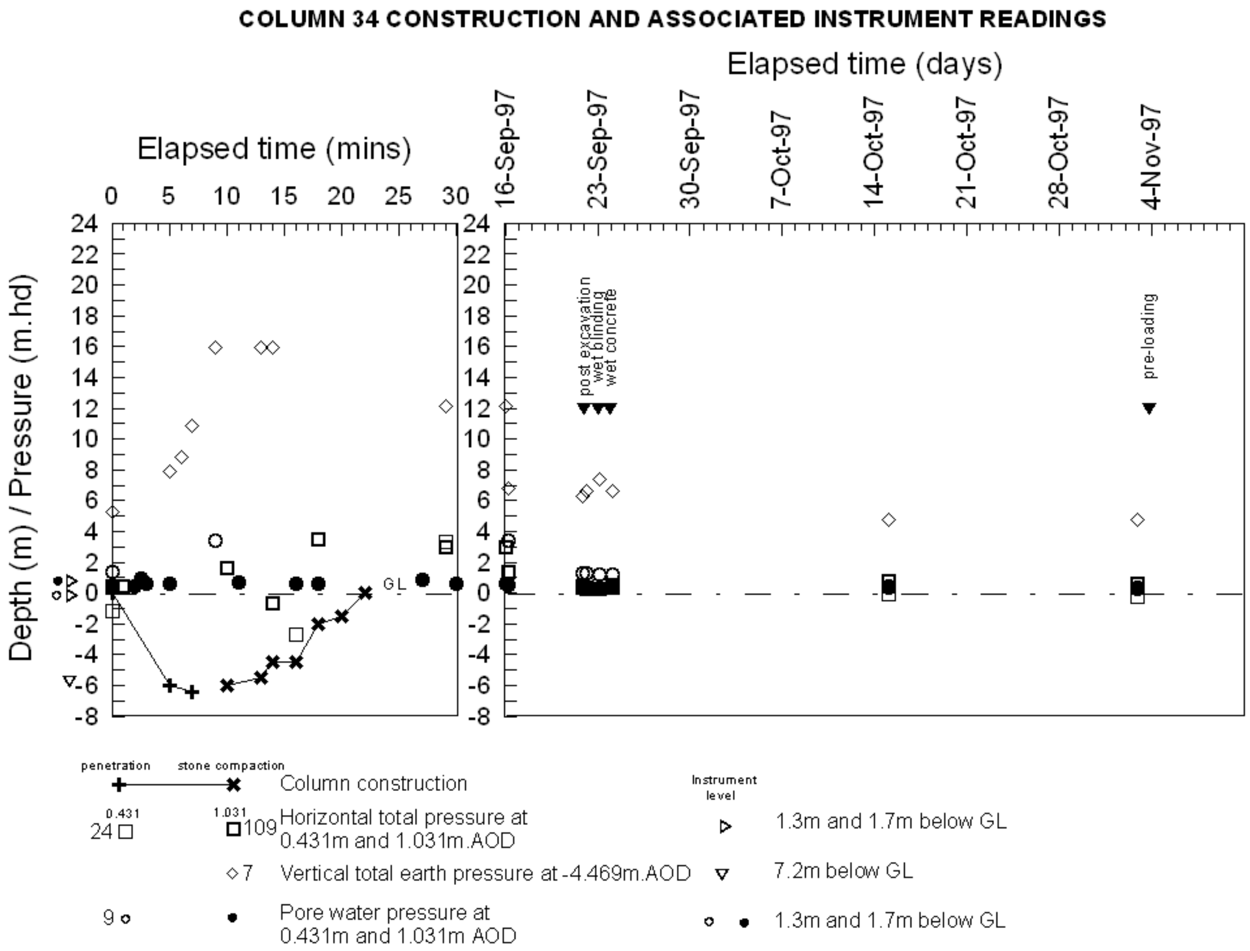


Figure 6.16: (b) Earth pressure and pore (water) pressure measurements associated with Installation of stone column 34.

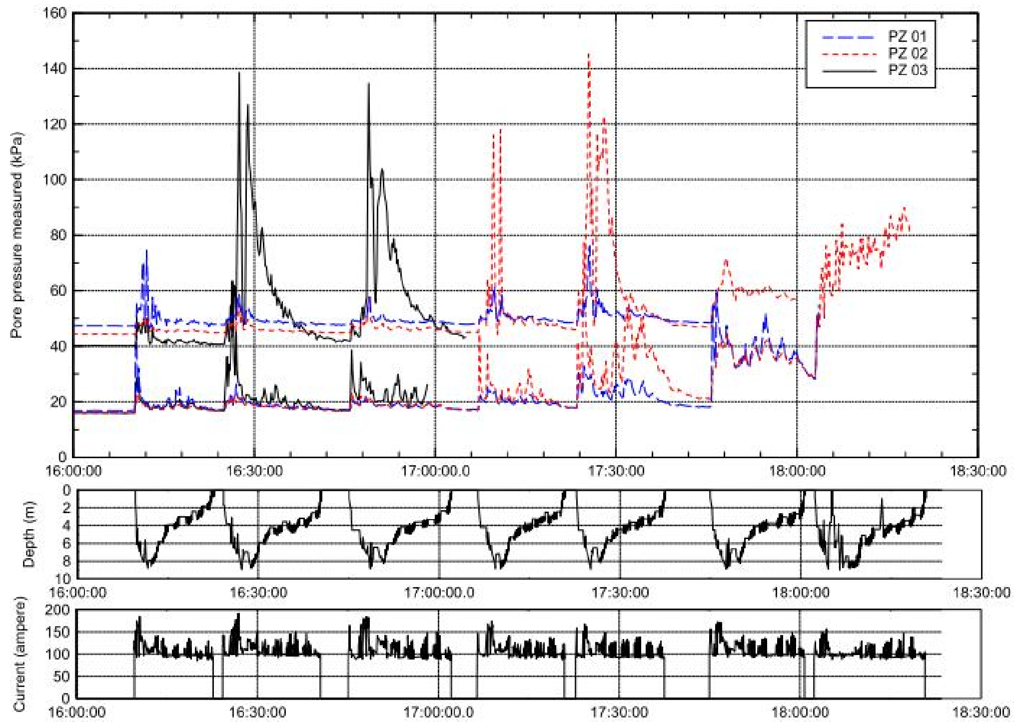


Figure 6.17: Stress development during stone column installation (after Castro and Sagesta, 2012)

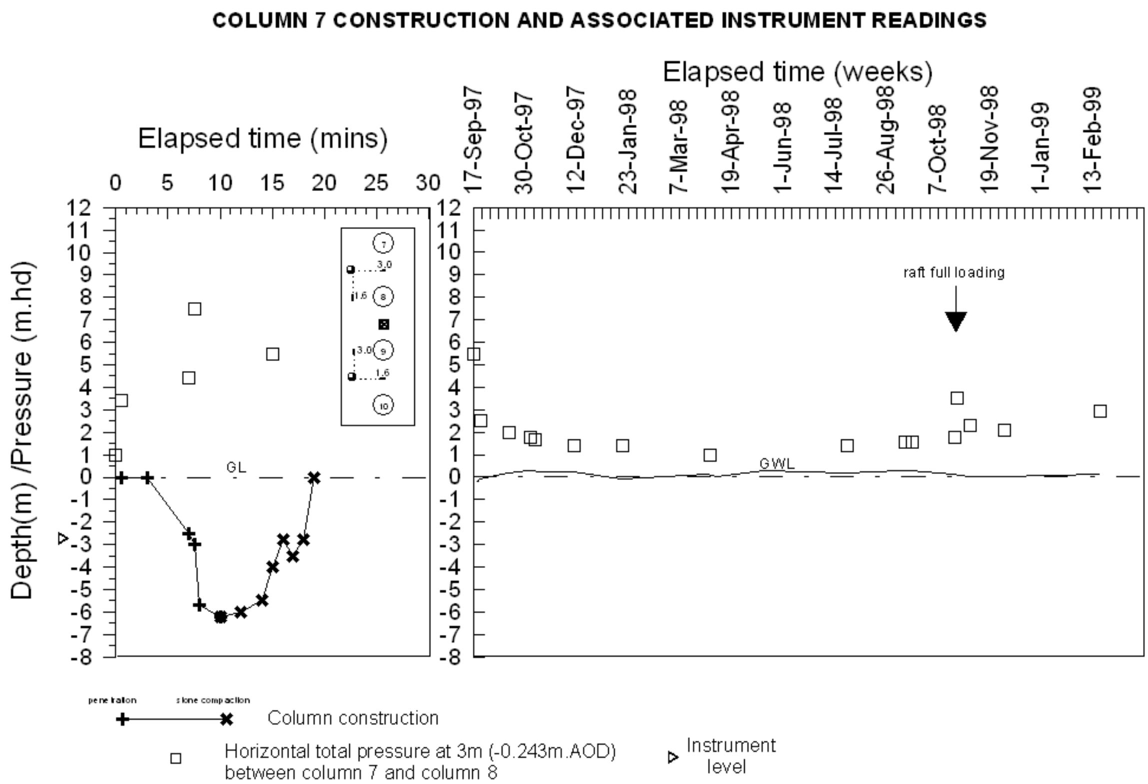
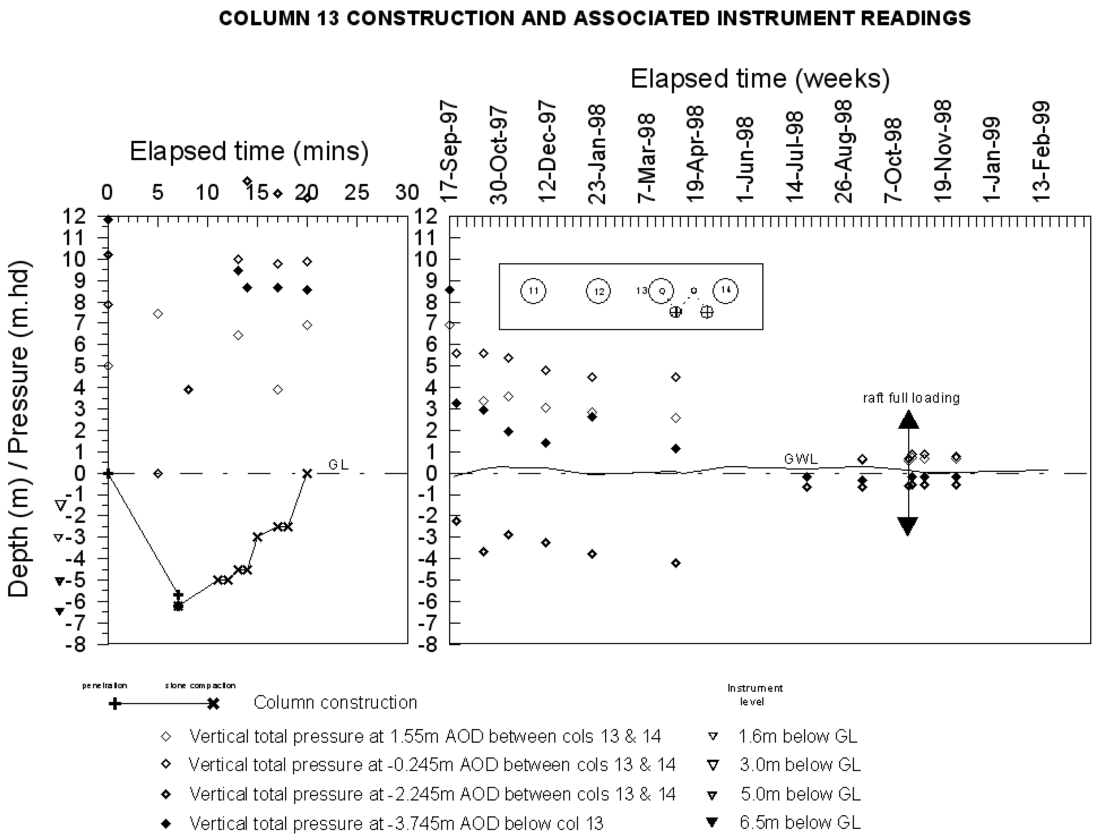
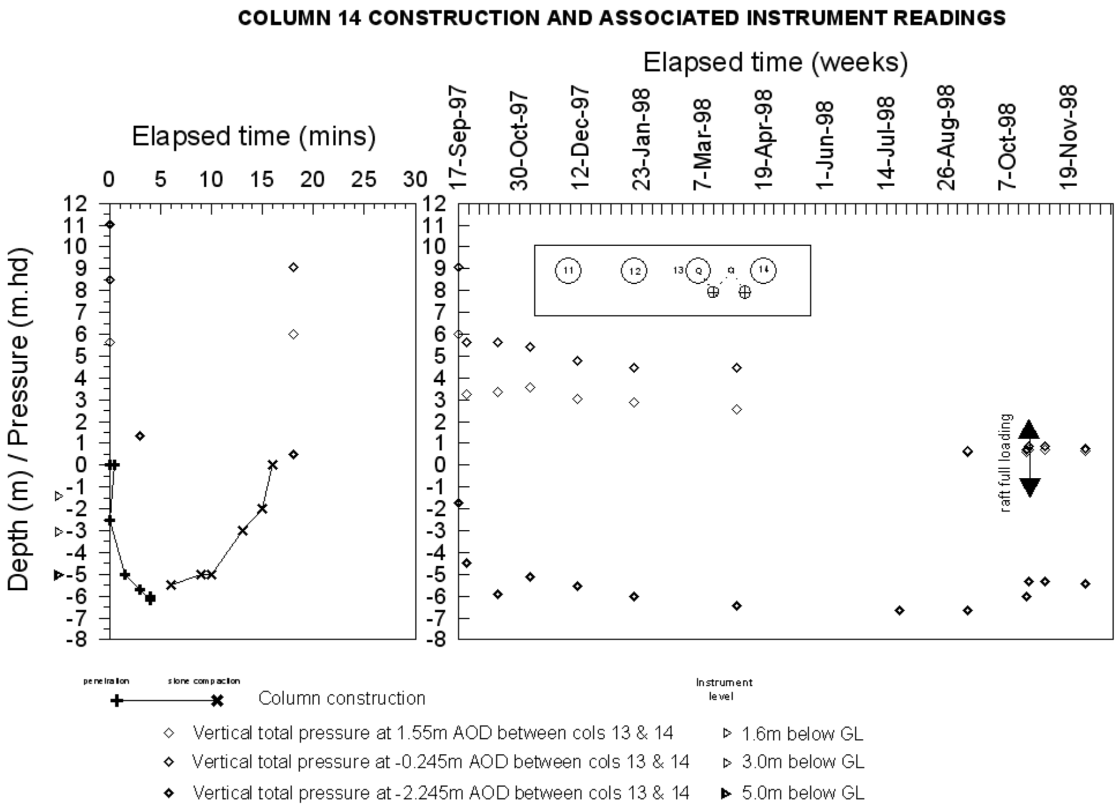


Figure 6.18: Earth pressure measurements associated with column installation (column No.7 for Bothkennar raft), after Serridge, 2001. (Note: + = penetration; x = stone compaction)



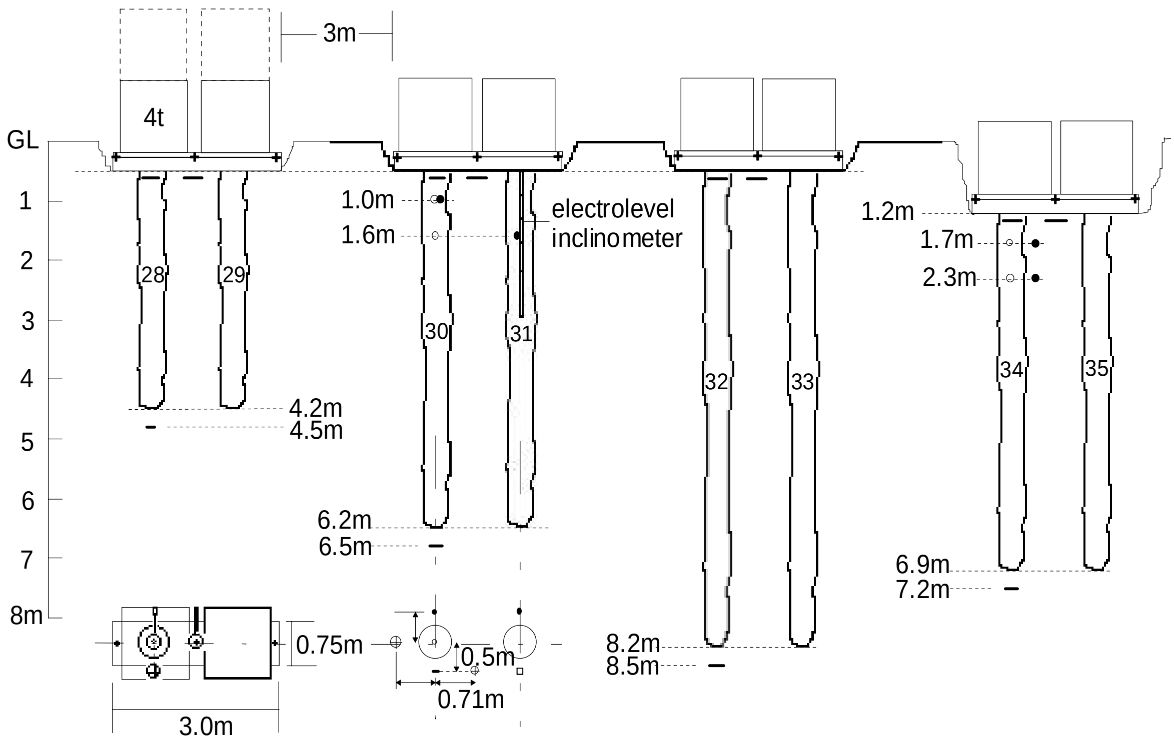
a)



b)

Figure 6.19: Vertical total pressure measurements during installation of (a) Stone column 13 and (b) Stone column 14 for Bothkennar raft (after Serridge, 2001)

Foundations to study depth of treatment and crust effect



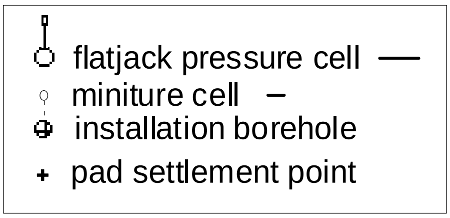


Figure 6.20: Typical Instrumentation locations associated with trial stone column installation (Bothkennar).

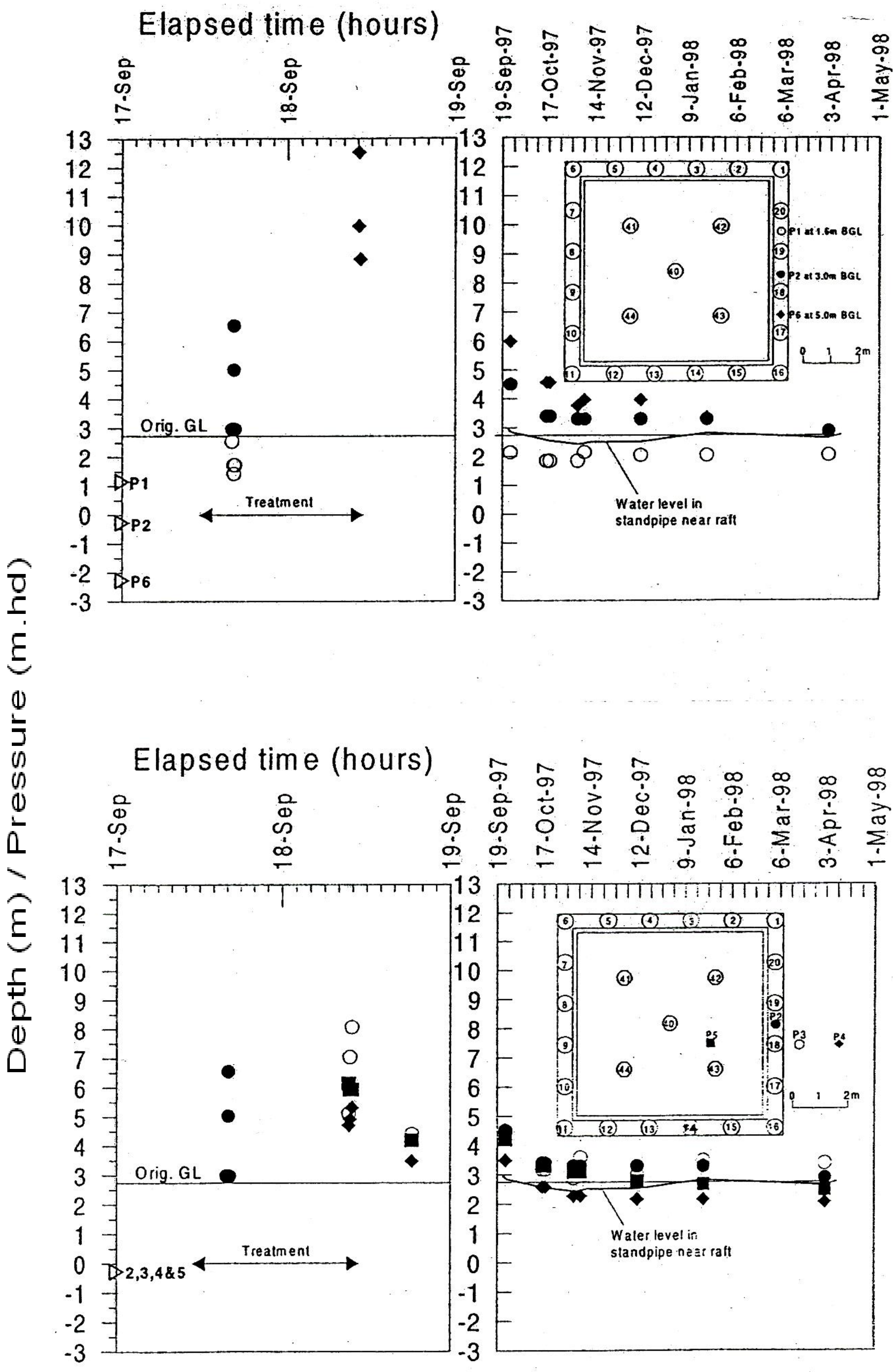
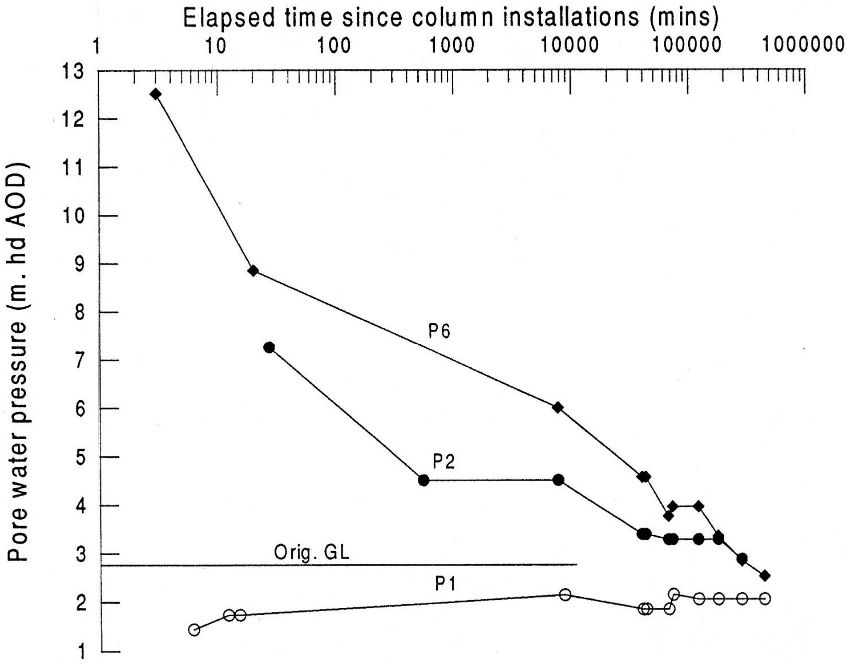


Figure 6.21: (a) Changes in pore water pressure during and after Bothkennar raft stone column installation (after Serridge, 2001).



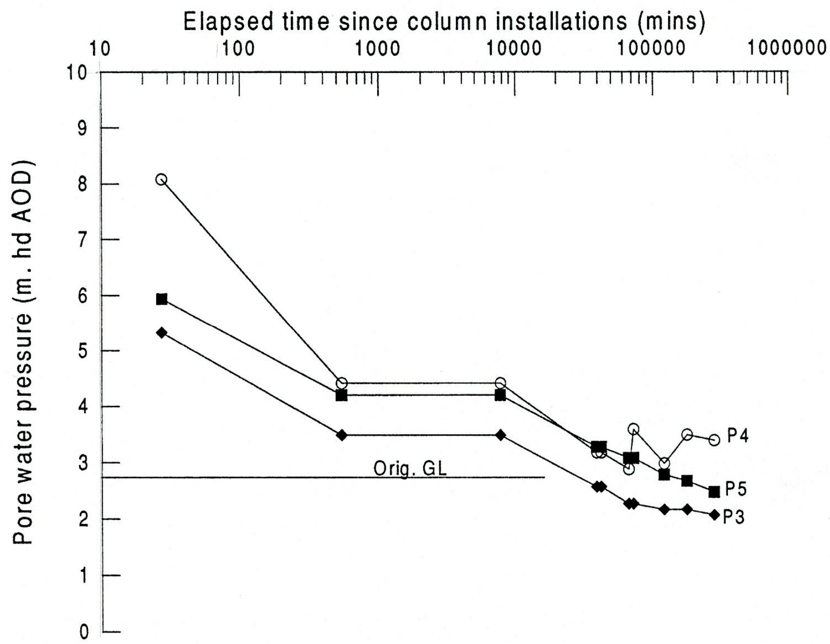


Figure 6.21: (b) Changes in pore water pressure during and after column installation (log plot) for Bothkennar raft (after Serridge, 2001).

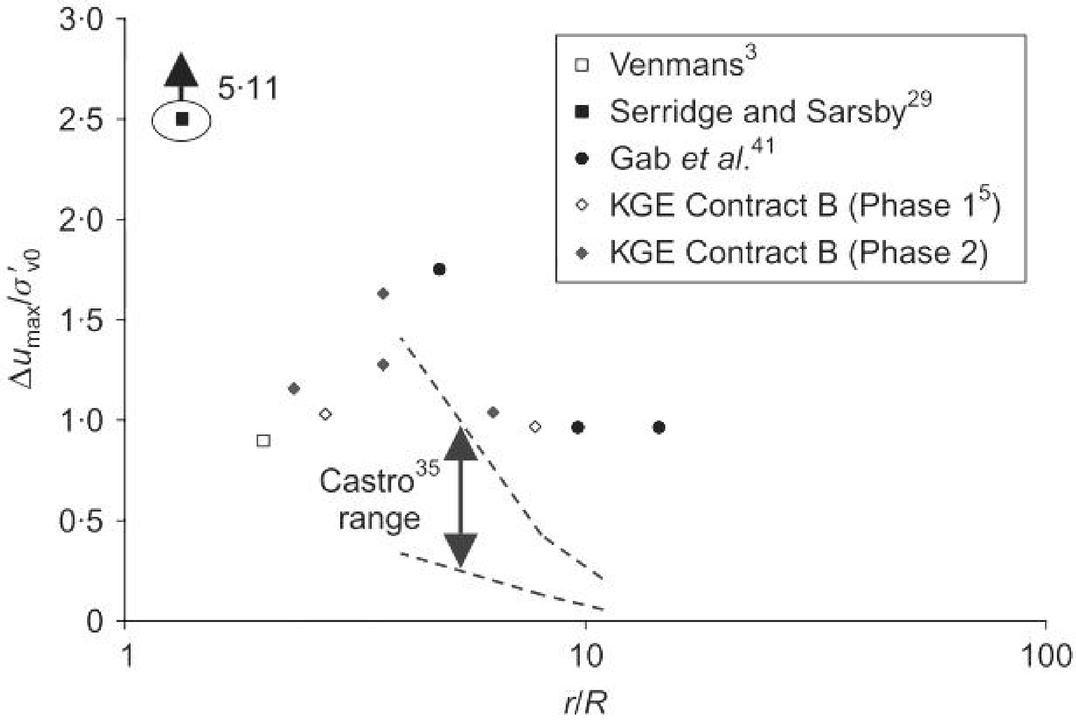


Figure 6.22: Maximum excess pore pressures around stone columns (after Mc Cabe et al., 2009).

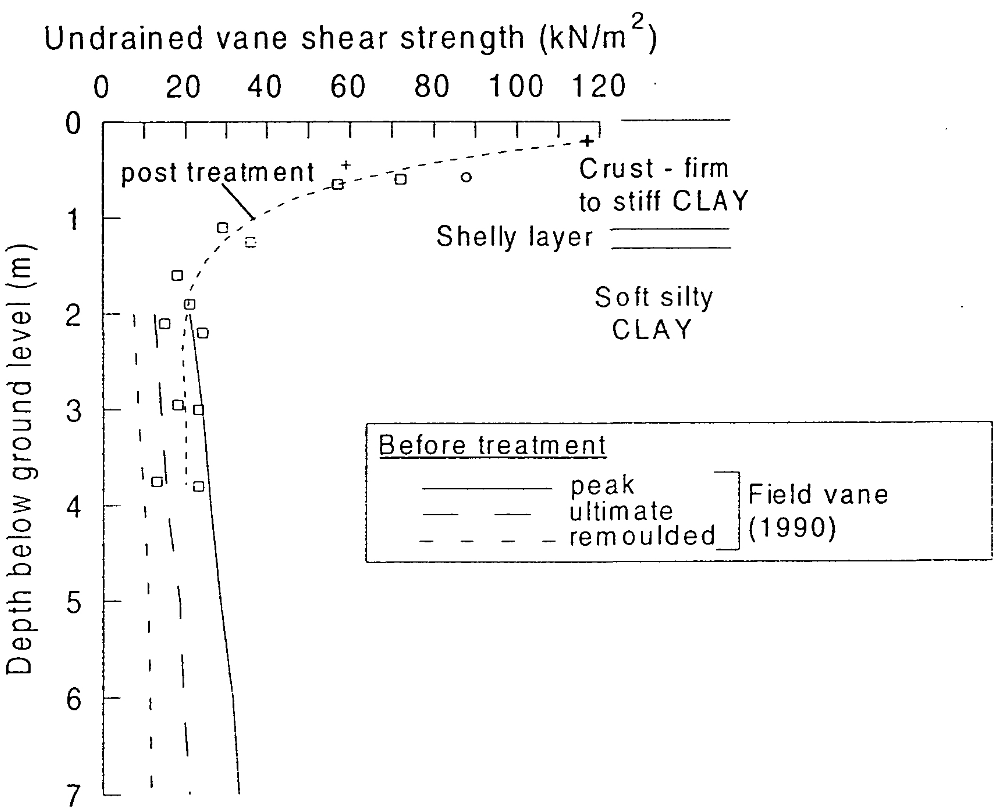


Figure 6.23: Summary of undrained shear strength measured about 5 days after installation of (test) columns.

Chapter 7 Response of stone column reinforced soft clay to load

7.1 Introduction

Details of the footing arrangements and the suite of instrumentation used in the field trials, together with the loading increments applied to the trial footings, are given in Tables 4.1; 4.3 and 4.4; Figures 4.10a-b and Figures 4.11a-e. Where vertical stress changes were measured at the top of stone columns and at the same level in the soil between columns, it has also been possible to calculate the proportion of vertical stress carried by both and in turn the stress (concentration) ratio (S_r). These are summarised in Table 7.1. Recorded settlements under the two main load increments for all trial footings are detailed in Figures 7.1a and b. The settlement data relating to both the two main load increments and the third and fourth load increments applied to footings 4 and 6 (treated) and 8 (untreated) are summarised in Figure 7.2a. Figure 7.2b and c shows the four load increments in place on these footings. The ground response to application of load to the trial footings constructed over the differing arrangements of partial depth vibro stone columns (footings 1-7) and also the footing constructed over untreated ground (footing 8) are discussed below for each of the individual trial footings:

7.2 Effect of stone column spacing (footing 1 and 2) L/d = 7.5

The effect of stone column spacing on the performance of the partial depth vibro - stone columns was investigated at the locations of trial footing 1 (1.5 m spacings) and 2 (2.0 m spacings), see Table 4.1 and 4.3. The field instrumentation responses to application of the two main load increments (see Table 7.2; Figure 7.3 and Figure 7.4) are discussed below:

7.2.1 Vertical stresses

During application of the first load increment: 35.5 kN/m^2 (footing 1) and 32.9 kN/m^2 (footing 2), (see Table 7.2; Figure 7.3 and Figure 7.4), the more widely spaced (instrumented) stone column 26 (footing 2) attracted the majority of the applied vertical stress (100% + on the basis of the instrumentation readings), and corresponding to almost 2.5 times more vertical stress than for the more closely spaced column 22 (footing 1), which attracted around 50-55 % of the applied vertical stress. In addition, whilst column 22 shows a slight overall reduction (around 14%) in attracted vertical stress during this first load increment cycle, the recorded vertical stress on stone column 26 reduced by around 50% at the mid-point with subsequent recovery to initial values by the end of the load increment cycle. Whilst there is likely to be some influence from foundation 'bedding-in' effects, this would suggest that even under the relatively low applied stresses of the initial loading increment, because the more widely spaced stone columns are attracting a significantly higher proportion of the applied vertical stress, compared to the more closely spaced stone columns, there will be greater load (stress) transfer down the stone columns into the underlying soft clays until an equilibrium condition is met. In the case of the closer column spacings, one could be witnessing accelerated soil consolidation due to the significantly shorter drainage path lengths provided by the stone columns, in conjunction with the presence of the permeable shelly horizon immediately below the crust, or response to a higher composite stiffness per unit area of foundation attributed to the 'reinforcing' effect of the higher frequency of stone columns. As the intervening soil drains and consolidates one would expect some load shedding from the stone column onto the intervening soil (as its stiffness improves). Monitoring of vertical stress in the clay soil between stone columns demonstrates that around 70% more vertical stress is attracted by the soil between stone columns spaced at 2.0 m centres compared to those at 1.5 m centres. This is considered to be attributed to the greater contrast in stiffness per unit area of foundation for the different column spacings as implied above. It may also be attributed to the fact that with the wider spacings the clay is undrained (at least initially) and behaving in an incompressible manner, therefore attracting more load. It was also observed that the vertical stress recorded in the soil between stone columns at both spacings was around 50% lower than that recorded at the instrumented stone column positions under this first load increment.

It is clear from the above observations that the more widely spaced stone columns are registering higher stress levels than those actually applied to the trial footing. Whilst such a situation would seem implausible, Greenwood (1991) made similar observations during field trial investigations of a single instrumented stone column at Uskmouth, UK (see Chapter 2 Section 2.3), where it was observed that at higher loadings during a plate load test on the stone column, installed through a normally consolidated soft clay profile, the upper of two pressure cells (installed in the column during its construction), registered higher stress levels than those applied at the surface. A possible explanation put forward by Greenwood (1991) was that stress re-distribution due to deformation of the soft clay below the surface 'crust' caused the crust to transfer its weight to the stone column by skin friction.

During application of the second load increment (see Table 7.2; Figure 7.3 and Figure 7.4), the attracted vertical stress recorded by stone columns 22 and 26 (beneath footing 1 and 2 respectively), increased throughout the duration of this load increment cycle, but with the wider spaced column 26 (Table 7.2 and Figure 7.4) attracting twice as much vertical stress as that attracted by the closer spaced column 22 (Table 7.2 and Figure 7.3). The difference in vertical stress recorded in the soil between the two column spacings varied by around 30% during the first half of the load increment cycle. However, by the end of the load increment cycle this increases to around 75% for footing 1 with wider column spacings, i.e. with around 3 times more vertical stress recorded in the soil between the wider spaced stone columns compared to the closer spaced columns, (see Table 7.2; Figure 7.3 and Figure 7.4), suggesting that at higher stresses and with wider column spacings, the stone columns settle transferring load back to the soil and the crust gradually becomes less effective in its ability to support and distribute the applied vertical (foundation) stresses. Similar observations were made by Greenwood (1991). A further significant observation was that for the more widely spaced columns (footing 2), the combined average vertical stress recorded for the stone column and intervening soil, expressed as a percentage of the total applied vertical (foundation) stress, for the first and second load increment cycles, is around 80% and 100% respectively. This compares with values of around 40% and 50% respectively, i.e. around half, for the two load increments applied to footing 1 with the closer spaced columns. This would suggest that the closer stone column spacing, in combination with the crust properties, provides a stiffer (composite) soil, which in turn has a significant influence on soil-structure interaction, with in the case of footing 1 about 50% of the applied vertical stresses being transferred elsewhere, most probably towards the edges of the foundation (where unfortunately there was no instrumentation), by some form of 'arching' or 'rafting' mechanism. Whilst other contributory factors relating to these load discrepancies could be the reliability of the field instrumentation, the researcher (author) considers the former explanation to be more plausible, particularly as similar observations were made by Greenwood (1991) for an embankment trial on stone columns at Humberside, UK (see Chapter 2 Section 2.3), where heavy compaction of chalk fill, placed above a normally consolidated clay profile with a lightly over-consolidated partly saturated clay crust, had left a residual stress similar to that in an over-consolidated crust, corresponding to about twice the overburden weight above pressure cells which had been installed to monitor stress ratio between columns and intervening soil (see Figure 2.13a-d). Based upon observations by Greenwood (1991) at Humberside, if both the soil and columns had been yielding uniformly to load, pore pressure dissipation should have increased resistance to bulging allowing the column stress to increase or at least be maintained; and corresponding settlement would continue instead of levelling off (Figure 2.13b). Some levelling off of settlement was observed in the field trials at Bothkennar. However, as Greenwood (1991) indicated, if the chalk fill at Humberside was rafting to some extent over the soil, the effect of pore pressure dissipation would be to allow radial consolidation as bulging occurred with consequent reduction of stress on the column as observed and reflecting similar observations at Bothkennar.

7.2.2 Stress ratio (S_r)

Recorded stress ratios under the first and second load increment cycles are given in Table 7.1 (see also Table 7.2). Under the first load increment the stress ratio is around 2.0 for the wider column spacings (footing 2), with a very slight decrease at the mid-point, before subsequent recovery to a value of around 2.0 towards the end of the load increment cycle. For the closer column spacings (footing 1) the stress ratio (S_r) increases to around 2.5 following initial load application, increasing to a maximum of 3.4 at the mid-point, before reducing to around 1.5 by the end of the load increment cycle (indicative of consolidation of the intervening soil and therefore loading shedding onto the increasingly stiffer soil). Under the second load increment cycle the recorded stress ratio (S_r) following load application is around 1.6 for footing 2, increasing to around 3.0 at the mid-point and then reducing to around 1.4 by the end of the load increment cycle. For footing 1 an initial S_r of around 1.2 is recorded, increasing to around 2.0 by the end of the load increment cycle, suggesting drainage-consolidation of the intervening soil, and therefore increasing the lateral confining pressure on the stone column, allowing it to behave as a stiffer element and attract more load. This is supported by the settlement data for this footing, which is discussed below.

7.2.3 Settlement

Recorded settlements are detailed in Figure 7.1a and b. For the first and second load increments total settlements respectively of 27.5 mm and 50 mm (footing 1) and 25 mm and 42.5 mm (footing 2) were recorded over a combined period of around 10 months. It is therefore evident that footing 1 supported on four stone columns at 1.5 m centres settled marginally more (10%) than footing 2 supported by three stone columns at 2.0 centres under the first load increment, with the difference increasing to 15% under the second load increment. Although one would expect the closer column spacings to provide a stiffer, composite, less compressible soil within the treated depth, the magnitude and rate of settlement is higher for footing 1 implying that the shorter drainage paths for pore water pressure dissipation associated with the closer column spacings, combined with the permeable shelly layer (horizon), is a significant factor in the context of the recorded accelerated primary consolidation settlements. This is also reflected in the fact that about half the recorded settlement took place within the first few days of (each) load application.

7.3 Effect of stone column length (trial footing 3: L/d = 5),(trial footing 4: L/d = 7.5),(trial footing 5: L/d= 10)

During the application of the two main load increments to trial footings 3; 4 and 5 with the shortest, intermediate and longest column lengths respectively (and the third and fourth load increments (Table 4.4) to footing 4), the vertical stress changes in the instrumented stone columns and intervening soil were monitored and recorded as for earlier stone column arrangements. In addition, lateral stress changes and pore pressures within the anticipated (zone) of stone column bulging, together with vertical stresses beneath the toe of selected stone columns, were also monitored and recorded (Table 7.3

and Figures 7.5; 7.6a-c; Figure 7.7). From review of the data the following trends can be identified:

7.3.1 Vertical stresses

During the application of the first load increment the instrumented columns with the shortest (footing 3), intermediate (footing 4) and greatest column length (footing 5) attracted around 40%, 60% and 80% respectively of the applied vertical stress(es), with the highest values recorded at the commencement of load application (Table 7.3 and Figure 7.5; Figure 7.6a; Figure 7.7). This is also supported by observations by Greenwood 1975; 1991. It was also observed that these columns all recorded minimum values of vertical stress at the mid-point of the first load increment cycle, with significantly higher values recorded at the commencement and end of the load increment cycle. This seems to be unique to the first load increment application and may be attributed to 'bedding-in' effects as mentioned previously, until an equilibrium condition is met with regard to effective stresses. For the instrumented soil (horizon) between stone columns the (average) proportion of applied vertical stress recorded during the first load increment cycle for footings 3; 4 and 5 corresponded to 17%; 20% and 32% respectively, showing a similar trend of increase (with adjacent increasing column length), to that for the instrumented columns (Table 7.3; Figure 7.5; Figure 7.6a and Figure 7.7). It should be noted, however, that whilst these values remained relatively constant beneath footings 3 and 4 for the duration of the first load increment cycle, the recorded value beneath footing 5 rose further to a maximum of around 45% of the applied vertical stress at the mid-point of the cycle. This would suggest that the greater the stone column length in the soft Bothkennar Clay below the crust, the greater the load transfer onto the intervening soil, which the author considers is attributed to settlement of the (adjacent) stone column(s), but noting that since the vertical settlement of the stone column and surrounding soil is approximately the same, stress concentration occurs in the stone column since it is stiffer than a cohesive soil. The stone column-soil composite in this situation is more responsive to load application until an equilibrium condition is met, with what appears to be complex (inter-dependent) stress changes between column and soil.

During application of the second load increment (see Table 7.3 and Figures 7.5; Figure 7.6a; Figure 7.7) the (average) proportion of applied vertical stress recorded for the (instrumented) columns beneath footings 3; 4 and 5 was around 60%; 75% and 100% respectively. Lowest values were recorded at the commencement of the load increment cycle, gradually increasing to maximum values of around 70% (footing 3); 90% (footing 4) and 100% (footing 5), by the end of the load increment cycle, which is clearly a much slower response than during the first load increment. These values are 20% higher than values recorded during the first load increment cycle, however, clearly implying that with increasing vertical stress the columns are required to work harder and support a greater proportion of the applied vertical stress, in turn suggesting that under these higher applied vertical stresses the crust is progressively less effective in supporting and distributing these stresses. The (average) proportion of applied vertical stress recorded by the soil instrumentation between stone columns for footings 3; 4 and 5 under the second load increment is very consistent at around 25%, which is marginally higher than values recorded under the first load increment for footings 3 and 4, but significantly lower than values recorded at the location of footing 5, confirming significantly greater stress concentration onto the longer stone column(s). It is notable that highest (vertical stress) values for the soil in between stone columns are recorded at the commencement of the second load increment cycle with some slight overall decrease in recorded values by the end of the load increment cycle, probably in response to re-equilibration of applied stresses, again suggesting complex load transfer mechanisms and consolidation and stiffening of the intervening soil (the latter to be investigated by post trial in-situ hand shear vane testing).

During application of the third and fourth load increment(s) to footing 4, the percentage of the theoretical applied vertical stress (Table 7.3) recorded by the instrumented column 30 increases to around 100%, indicating significant stress transfer on to the stone column(s). The corresponding measured vertical stress in the soil between stone columns remained relatively constant at around 25%.

7.3.2 Lateral stresses

Lateral (horizontal) stresses were monitored by pressure cells installed adjacent to the instrumented column 30 (footing 4) at two levels, corresponding to depths of 0.5 m and 1.1 m beneath the -0.5 m founding level (Figure 4.11b) as previously indicated. Reference to Figures 7.6 b-c demonstrate that during the first load increment very nominal stress changes of between 1 and 2 kN/m^2 were observed and recorded at both cell locations, corresponding to around 4 to 5% of the applied vertical stress and probably attributed to foundation 'bedding-in' effects. During the second load increment cycle, no significant change in recorded lateral stress values were observed. However, during the relatively short period (maximum 2 months) between application of the third and fourth load increment cycles (in relatively quick succession) and completion of the field trials, both pressure cells responded immediately to the higher applied loads with recorded total lateral pressures corresponding to around 10% of the applied vertical stress, which although relatively small (particularly when compared to the lateral stresses recorded during stone column installation – see Chapter 6, Section 6.3), is considered to be potentially indicative of a small amount of column dilation (bulging) at the location of the pressure cells.

7.3.3 Pore pressures

Pore pressures were monitored adjacent to the instrumented column 30 (footing 4) at two depths, again corresponding to 0.5 m and 1.1 m below the - 0.5 m founding level, (see Figure 4.11b). Reference to Figure 7.6b demonstrates that throughout the two main load increment cycles a consistent pattern emerges, namely that higher pore pressures are registered in response to load application by the shallower cell at 0.5 m depth beneath the foundation compared to the deeper cell at 1.1 m depth (with the difference generally being around 5%). Whilst this could be attributed to the stress distribution beneath the foundation : If one takes the trial footing width of 0.75 m and a length of 3.0 m, for z (shallower instrumentation depth) = 0.5 m, using Fadum analysis, L/2Z = 3, B/2Z = 0.75. For Z = 1.1 m (deeper instrumentation depth) L/Z = 1.5, B/Z = 0.34, clearly demonstrating different stress influence values, the differences could also be potentially explained by the fact that the deeper cell is closer to the permeable shelly layer present in the upper Bothkennar Clay profile, (see Figure 4.3) providing, in conjunction with the stone columns, significantly shorter drainage paths for the dissipation of excess pore water pressures, as intimated previously.

7.3.4 Toe pressures

Whilst no positive toe pressures were recorded beneath the toe (base) of the instrumented stone columns installed beneath footings 4 and 5 (See Figures 4.11b; Figure 7.5; Figure 7.6a and 7.7) under the two main load increments applied, significant positive values (stress transfer) were recorded beneath the toe of the instrumented shorter column 28 beneath footing 3, at all applied load increments (see Figure 4.11b and Figure 7.5), equating to between 10 and 15% of the applied vertical stress, i.e. 5-6 kN/m^2, demonstrating the field validity of the Hughes and Withers (1974) model for behaviour of stone columns shorter than calculated minimum stone column length in soft cohesive soil, i.e. significant stress transfer to the toe of the column (and also validity of fieldwork by Hughes et al. (1976) among others).

7.3.5 Stress ratio (S_r)

Under the first load increment the average recorded stress ratio (S_r) (Table 7.1) was 2.3; 2.8 and 2.4 respectively for the short (L/d = 5.0 : footing 3) intermediate (L/d = 7.5: footing 4) and long (L/d = 10 : footing 5) stone column lengths, increasing to around 2.7, 3.3 and 4.0 respectively under the second load increment cycle. It is clear therefore that the S_r increased significantly with both length of stone column and applied load. Similar observations have been made by Hu (1995); Mc Kelvey et al. (2004) and Black et al. (2010) among others.

7.3.6 Settlement

Settlement data for trial footings 3; 4 and 5 are detailed in Figure 7.1a; 7.1b and 7.2a. Footing 3 over the shorter stone columns settled the least (around 17.5mm under the first load increment and 37.5 mm under the second load increment), with footing 5 settling marginally less than footing 4 (22 mm under the first load increment and 40 mm under the second load increment, compared with 20 mm under the first load increment and 42.5 mm for the second load increment for footing 4). With increasing stone column length, the crust also clearly has less impact on the control of settlements. Maximum cumulative total settlements of the order of 75 mm (Figure 7.2a) were recorded following application of the third and fourth load increments to footing 4, but which did not result in bearing capacity failure.

7.4 Effect of crust

This aspect of the field trials was intended to assess the impact of placing the trial footing (footing 6) on stone column reinforced soft clay at the base of the crust, i.e. 1.2 m depth below ground level. Details of recorded data are given in Table 7.4 and Figures 7.8a-c. Vertical stresses were measured on top of stone column 34, during the loading increments. Unfortunately a pressure cell could not be monitored below trial footing 6 to measure vertical stress in the soil between stone columns, due to damage. However, lateral stress changes and pore pressure changes within the anticipated range of stone column bulging, together with vertical stresses beneath the toe of selected stone columns were monitored and recorded (see Table 7.4) as for previous footings (footings 3;4 and 5). From review of the data the following trends can be identified:

7.4.1 Vertical stresses

The vertical stresses attracted by column 34 during the two main load increment cycles (and the subsequent third and fourth load increments, applied in quick succession) are presented in Table 7.4; Figure 7.8a. Under application of the first load increment column 34 immediately attracted in excess of 100% (recorded as 146%) of the applied vertical stress, (34.2 kN/m^2), suggesting significant stress concentration onto the stone column compared to the weaker intervening soil. During the remainder of the load increment cycle the value reduces marginally to around 90% by the mid-point, with subsequent recovery to around 100% by the end of the load increment cycle. This suggests, as previously (intimated) some 'bedding-in' effects or load re-distribution, until an equilibrium level is reached. Under the second load increment column 34 recorded a vertical stress at commencement of around 100% of the applied vertical stress (69.6 kN/m^2) which increased marginally to 111% by the mid-point and subsequently remaining fairly consistent at this level for the remainder of the load cycle. Between application of the third load increment and the end of the fourth load increment cycle, vertical stresses ranged from around 106% of the applied vertical stress at commencement to 98% at the finish. This suggests that the column will not support any additional load and is potentially approaching a failure condition (with potential significant load transfer to the intervening soil, although this could not be substantiated in the absence of soil instrumentation in the intervening soil (and could be in response to the consolidation process)).

It is important to recognise that whilst a pressure cell could not be installed in the intervening soil between columns 34 and 35 at founding level beneath trial footing 6, to measure vertical stress in the soil between stone columns, the recorded vertical stresses at the top of column 34 were higher than the values recorded under all other trial footings, implying that a much greater proportion of the applied load (stress) was carried by the columns, where founding depth for the footing was at the base of the crust. In the absence of instrumentation described above between stone columns, the explanation could be described as tenuous, although the author (researcher) would describe it as plausible. Instrumentation error or drift cannot be totally ruled out however.

7.4.2 Lateral stresses

Lateral (horizontal) stresses were measured as under previous trial footings at two depths (corresponding to 0.5 m and 1.1 m respectively below founding level), within the zone of anticipated bulging and with results summarised in Table 7.4; Figure 7.8b-c.

During application of the first load increment, the shallower cell recorded a lateral stress increase of up to 5 kN/m^2 (representing around 15% of the theoretical applied vertical stress). This compares with the deeper pressure cell where nominal lateral stress changes of only between 1 and 2 kN/m^2 were recorded (representing around 6% of the (theoretical) applied vertical stress). During application of the second load increment the shallower cell initially recorded a lateral stress increase of around + 2 kN/m^2 to a total value of 7 kN/m^2 (10% of the applied vertical stress). However the lateral (horizontal) stress increased significantly during the second half of the load increment, rising to a maximum of 40 kN/m^2 (representing around 57% of the applied vertical stress). Whilst this may represent exceedance of pre-consolidation pressure, the stresses measured above are significantly in excess of those calculated from normal stress distribution in an elastic medium and it is considered more likely to be indicative of significant column dilation (bulging) at this level (approximating to 1.5-2.0 column diameters below founding depth), within the top of the soft Bothkennar Clay below founding level, particularly as the increase in lateral pressure was progressive. With application of the third and fourth load increments the shallower cell recorded an immediate very nominal lateral stress change of 1 kN/m^2 to a total of 41 kN/m^2 (representing around 30-35% of the applied vertical stress). This compares with the deeper cell where the recorded lateral stresses had increased by around 8 kN/m^2 to 15 kN/m^2 (corresponding to around 10-15% of the applied vertical stress and half about that recorded by the shallower cell). The above observation suggests that column bulging (dilation) was more pronounced at 0.5 m below founding level. However, the observation of lateral stresses falling during application of the third and fourth load increments suggests that 'punching shear' at the toe of the columns was perhaps taking over from bulging near the top of the column. This is borne out by the pore pressure measurements discussed below. However, one could simply be looking at soils having gained strength through consolidation, therefore reducing tendency to bulge and therefore reducing lateral pressures. It is clear, however, that more response and higher stress increases were observed at the shallower cell location compared to the deeper cell location. Whilst there may be some influence for the stress bulb beneath the foundation, it is considered that this does not completely explain the problem. It is important to recognise that the lateral stresses measured at the cell locations were in excess of those calculated from normal stress distribution in an elastic medium. This indicates that the column may be bulging at a depth equivalent to two times its diameter and confirms that there was significant stress transfer down the stone column.

7.4.3 Pore pressures

Pore pressure measurements are plotted in Figure 7.8b. Review of this demonstrates that during the first load increment cycle no significant pore pressures were measured at the commencement of monitoring. By around mid-way through the load increment cycle, the shallower cell was registering around 5 kN/m^2, with the deeper cell registering around half this value. By the end of the load increment cycle, the respective values had reduced by 50%. This would appear to be again indicative of some 'bedding-in' effects as described previously. Under the second load increment (application) an immediate initial increase in pore pressure was evident at both cell locations, but with the shallower cell registering a total of around 7.5 kN/m^2 and for the deeper cell again around half this value. Upon application of the third and fourth load increments the shallower cell recorded a total value of around 10 kN/m^2 at commencement of the third load increment, which had reduced to around 4 kN/m^2 at the end of the fourth load increment, i.e. after a period of around 2 months. This compares to values of 1.5 kN/m^2 and 2.0 kN/m^2 respectively for the deeper cell. The pore pressures clearly respond to load, but recorded values are not significant, suggesting rapid dissipation of pore pressure attributed to the presence of the (free-draining) stone columns, in conjunction with the shelly horizon previously described, coinciding with the approximate level of the deeper cell. It is evident that there is a clear relationship between the lateral stresses described above and pore water pressures.

7.4.4 Toe pressures

Upon application of the first load increment to footing 6 the toe pressures recorded beneath column 34 (Figure 7.8a) ranged from +12% to -12% of applied vertical stress, with similar observations noted during the application of the third and fourth load increments, with values ranging from +15% to -15% of the applied vertical stress. This suggests a nominal increase in toe pressure with increasing load, but reliability of the instrumentation or potential for displacement or damage of the pressure cell during initial stone column installation seems likely, particularly when negative values are being recorded. This potential displacement or damage to the instrumentation during stone column installation unfortunately does not allow one to confirm conclusively whether 'punching' is occurring at the toe. Whilst the instrumentation proved generally reliable the earth pressure cell beneath the toe may have been a casualty of the significant soil displacement observed during the construction of the 'end-bulb'.

7.4.5 Stress ratio (S_r)

For reasons described earlier, it was not possible to measure stress ratio beneath trial footing 6. However, based upon the proportion of the applied stress attracted by the instrumented stone column 34, it was evident that significant transfer onto the stone column was taking place, potentially higher than for all other stone column arrangements.

7.4.6 Settlement

The settlement recorded for the two main load increments applied to trial footing 6 (see Figures 7.1a and b) were higher than for all other trial footings, corresponding to 24 mm under the first load increment and 45 mm under the second load increment. This increased further to a cumulative total settlement of around 80 mm following application of the third and fourth load increments to the trial footing (Figure 7.2a), but did not experience failure.

7.5 Investigation of footing shape

7.5.1 Settlement

Trial footing 7 was used to assess the impact of footing shape on settlement performance based upon a 0.5 m founding depth within the crust. The footing was constructed as a 1.5 m x 1.5 m pad supported by two stone columns (36 and 37) and corresponding to the same area replacement ratio (A_r) as that beneath trial strip footings containing columns at 1.5 m centres.

The settlement (Figure 7.1a and b) was comparable to trial footing 4 with identical column lengths and replacement ratio (A_r). However, the stress depth influence below founding depth for a 2.5 m square pad would be expected to be around 6.0 m, whereas that of a 0.75 m wide footing around 2.7 m. This would suggest that the crust is having a greater impact on soil-structure interaction in the context of a pad footing. Column length would appear to have minimal impact on settlement in this context.

7.6 Summary of stress ratios and foundation settlements (for all trial footings)

Whilst it is evident from the field trials that stress ratio is sensitive to small variations in stress and can sometimes be difficult to measure with certainty, average values in the range 1.9 to 3.3 and 1.9 to 3.8 were recorded during the first and second load increments respectively, under trial footings 1 to 5. For footings 3 to 5, founded at 0.5 m depth in the crust and supported on two stone columns, the ratio increased significantly with length of column and applied load. A cell could not be installed below footing 6, founded at the base of the crust to measure stress in the soil between stone columns. However, vertical stresses recorded at the top of column 34 under trial footing 6 were higher than under all other trial footings on stone columns, implying a much greater proportion of the applied load was carried by the columns under this footing. Horizontal stress in the soil close to the top of column 34 increased significantly during the second load increment. The application of further loads resulted in a rise in vertical stress beneath the column toe, suggesting stress transfer and penetration at the toe was taking over from bulging at shallow depth, closely following the Hughes and Withers (1974) model. A significant stress transfer was also measured beneath the toe of short column 28 (trial footing 3) at all applied loads, but not columns equal to, or longer than design length according to Hughes and Withers (1974) analysis, also verifying their analysis for column behaviour in cohesive (fine-grained) soil. The stress measurements demonstrate that the behaviour of the composite stone column-soil-foundation system is complex, with simultaneous and inter-dependent changes in pore pressures, soil stress ratios and resulting stiffness of both soil and columns. The stress ratios from the Bothkennar trials have been superimposed on field data reported by Greenwood (1991) for the Humber Bridge approach (see Chapter 2, Section 2.3) in Figure 7.9. There are generally good comparisons at low applied stresses but a greater scatter is evident at higher applied stresses. Nevertheless it is evident that there is a trend over a significant stress range. The differences may be attributed to widespread loads associated with embankment structures having a greater confining action on stone columns compared to individual stone columns under isolated footings, where there is no confinement beyond the edge of the footings (see Serridge and Synac, 2007).

Settlement of the trial footings due to the first two load increments, corresponding to average bearing pressures of 33 kN/m^2 and 70 kN/m^2 respectively, is summarised in Figure 7.1a and 7.1b. Bearing pressures of 33 kN/m^2 resulted in settlements of between 18 mm and 28 mm settlement for the treated trial footings over the first five months. Applying the second load increment to give a total applied bearing pressure of 70 kN/m^2 resulted in total (cumulative) settlements of between 39 mm and 51 mm after a further five months. Interestingly, about half the settlement recorded during each increment occurred within the first few days of load application. This is similar to observations made by Egan et al. (2008) for footings in the Carse Clay (see Chapter 3 Section 3.2). Trial footing 1 supported by four stone columns at 1.5 m centres settled slightly more than footing 2 supported by three stone columns at 2.0 m centres. Here it is the shorter drainage paths provided by the closer stone column spacings which is influencing recorded settlement(s). For trial footing 3 (shortest) to 5 (longest) supported by increasing stone column lengths, contrary to what one might expect settlements increased with increasing column length (see Section 7.3.6). This is considered to be attributed to a combination of significantly reduced drainage path lengths with increasing column lengths (assisted by the permeable shelly horizon), accelerating primary consolidation settlements under the applied load increments (and also load transfer to the toe of the deeper column, particularly if remoulding has taken place within the sensitive Bothkennar Clay during column installation, which would influence the depth of load transfer down the column, based upon Hughes and Withers (1974)). With increasing stone column length, the crust also clearly has less impact on the control of settlements. Footing 6 founded at the base of the crust on 5.7 m long columns settled the most. Predictions of settlement for trials footings 1-7 inclusive, without stone columns, under the first (Table 4.5a) and second (Table 4.5b) load increments were between 21 mm and 23 mm under the first load increment, similar to the average recorded settlements for treated trial footings under the first loading increment and between 43.5 mm and 46.5 mm under the second load increment, slightly higher than the average observed values with stone columns, although settlement of the treated trial footings was still ongoing at the end of each load increment. Originally estimated settlements for the treated situation, based on design charts proposed by Priebe (1995), were around 50% of the untreated situation. It is clear therefore that the large energy used to install the stone columns, particularly the 'end-bulb' has resulted in significant soil disturbance and heave which has contributed to poorer settlement performance than anticipated. However, given that around half of the recorded settlement occurred within the first few days of load application it could be argued that a significant component of the settlement would be built out during construction. It is clear however, that the stone columns significantly enhanced the bearing capacity characteristics of the soil.

7.7 Absence of treatment - Performance of the untreated trial footing 8

7.7.1 Settlement

Trial footing 8 investigated the performance of an untreated footing (i.e. without stone column reinforced soil), founded at 0.5 m within the crust, to facilitate comparison of settlement (and bearing capacity) performance with the treated trial footings 1-7 (and also an untreated footing described by Jardine et al. (1995), founded at 0.75 m within the crust at Bothkennar). In this regard only settlement was therefore monitored for trial footing 8. From review of the data it is evident that footing 8 on untreated ground (dotted line in Figure 7.1a-b;7.2a) settled about half the average of trial footings on treated ground and only about 40% of that predicted for untreated ground at low applied bearing pressures. Also the rate of settlement was much less than the treated footings at the end of each load increment (this is attributed to the significantly shorter drainage paths provided by the stone columns, in conjunction with the shelly layer, as described previously). Figure 7.1b shows log time plots for settlement of both the treated footings and untreated footing (represented by the dotted line), under the first and second load increments. Figure 7.2a shows the resultant load-settlement curves for (treated) trial footings 4 and 6 and the untreated footing 8 (dotted-line) for the two main load increments and subsequent third and fourth load increments (applying a maximum bearing pressure of 125 kN/m^2).

Load-settlements plots for trial footing 4 founded at 0.5 m depth in the crust and trial footing 6 founded at the base of the crust (1.2 m) are presented in Figure 7.10. Also included in Figure 7.10 is the load-settlement plot for trial footing 8 founded at 0.5 m depth within the crust without stone column support. The results from earlier field trials at Bothkennar performed by Jardine et al. (1995) at slightly greater founding depth in the crust i.e. 0.75 m founding depth, on untreated ground are also annotated. The relationship between settlement and load for both treated trial footings (4 and 6) was fairly linear within the wide load range shown, although the settlements might be considered excessive for certain applications. This should be seen in the context of the curve for the untreated footing 8, however, which shows large settlements as the load was increased and indicates the onset of bearing failure at around 125 kN/m^2. Actual field trial results from the current research and the projected trend for settlement over a similar time period for the Jardine et al. (1995) data in Figure 7.10 shows very large settlements, even at low applied loads, where the crust does not contribute significantly to overall bearing capacity and also demonstrates the sensitivity of founding depth within the crust (when compared to trial footing 8) to both achievable bearing capacity and settlement. It is therefore clear that the stone columns reinforced the weak soil below the crust, reducing settlement and providing an appropriate factor of safety against bulging failure and bearing capacity failure of the composite stone column-soil system, over a stress range normally associated with foundations for low-rise buildings. It is important to note that whilst the magnitude of settlement beneath footing 8 was significantly less than for the stone column reinforced footings under comparable loading conditions, the magnitude of increase in settlement between the end of the first load increment and end of the second load increment was around 20% higher for the untreated footing 8.

7.7.2 Bearing capacity

The conditions under which trial footing 8 was loaded in the field was interpreted as essentially axisymmetric, similar to those encountered in the undrained triaxial test. The movement (displacement) of footing 8 in response to the applied loading was essentially non-linear, inelastic, hysteretic and time dependent. This was apparent even under relatively light loads and with settlement relatively even under the footing until the load factor (ratio of applied load to failure load) exceeded a value of between about 0.7 and 0.75. As the footing approached 'failure' there was evidence of some very slight tilting of the foundation preferentially to one side, although in general the settlement profile across the foundation was relatively even, which is indicative of a 'punching type' failure mechanism, Figure 7.11a rather than the classical Prandtl wedge failure mechanism, Figure 7.11b. On unloading of the trial footing (Figure 7.1a ; Figure 7.2a) some 14-15% 'elastic' rebound was recorded, indicative of a high degree of plastic deformation at failure. The 'failure' load of the untreated footing 8 has been compared with available case history information on other sites containing a normally consolidated to lightly over-consolidated clay profile. A selection of these sites are summarised in Table 7.5 and include data reported by Skempton (1942) for Kippen (located some 20 km west of Bothkennar); Thorburn (1976), for Grangemouth (to the east of Bothkennar); Schnaid et al. (1993) for Shell Haven in the Thames Estuary (UK) and Jardine et al. (1995) for Bothkennar. Although the soft clay profile at Shell Haven in the Thames Estuary is known to have a similar stress history to the Bothkennar Clay, Schnaid et al. (1993) found that the soft clays from the Thames Estuary showed a tendency to be less sensitive than the Bothkennar Clay. On this basis one would expect recorded ultimate bearing capacity to be at least equal, if not greater than the Bothkennar Clay. However, if allowance is made for footing shape, the bearing capacity failure load at Shell Haven is about 50% lower than the average of the two failure loads reported for Bothkennar in Table 7.5. It is known that the Bothkennar Clay has a higher friction angle (φ'), attributed to a higher silt fraction (see Chapter 5 Section 5.3.3), than the Shell Haven Clay and that the Bothkennar Clay exhibits evidence of some 'cementing' and is structured. The Shell Haven Clay on the other hand does not have any 'cementing' characteristics. These differences could well explain the differences in reported failure loads. Shields and Bauer (1975) recorded only moderate settlements (typically less than 200 mm), at bearing pressures of up to 300 kN/m^2 in the Leda Clay deposit in Ottawa, Canada. Although the Leda Clay has high sensitivity, it is notable for having considerable bond strength i.e. is structured, which enhances very significantly its bearing capacity. This provides some support to the explanation for the differences in performance of the Shell Haven and Bothkennar Clay described above. However, it should be recognised that in soil mechanics terms the observed bearing capacities in the Bothkennar Clay were relatively short term and it is evident from the research trial(s) that with sensitive soils one tends to witness progressive creep settlement.

Bearing capacity prediction for footing 8 using the shear strength profile in Figure 4.3 and the theory of Davis and Booker (1973) yields an ultimate bearing capacity about 20% higher than the actual value recorded in the field. Whilst this provides additional support for the Bothkennar Clay being structured, it is important to recognise that it is not easy to define exact shear failure for loading on a sensitive clay soil. Some back analysis has been undertaken using conventional bearing capacity theory for undrained failure of a clay soil:

$$Q_{ult} = N_c.C_u + P_o \qquad \text{---} \quad 7.1$$

where N_c is the undrained bearing capacity factor, which depends on shape and other factors and P_o is the overburden pressure at the base of the trial footing. Although analytical solutions have been published for cases where undrained shear strength (C_u) varies with depth, the classical constant strength case approach models the parabolic shallow Bothkennar profile very effectively. Based upon data published by Eason and Shield (1960), N_c may be taken as 6.1 for a square or circular pad bearing on the surface of an undrained clay. Applying Brinch Hansen's (1970) depth correction multiplication factor of (1+ 0.4 z/D) gives an overall N_c of 7.0 for trial footing 8. Substituting this into the equation and accounting for the dead weight of the concrete footing results in a back calculated average (operational) undrained shear strength of 19 kN/m^2. One explanation is that some form of progressive failure has occurred which may be a phenomenon in certain soft sensitive clay soils as intimated previously. This is perhaps not surprising however, if one takes account of the fact that standard bearing capacity theory relies on the assumptions that the soil behaves as an isotropic rigid elastic (almost perfectly elastic) material. It is known, however, and clear from the trials, that this is not the case for soft sensitive clay soils, in particular those which behave in a highly non-linear manner, with a significant amount of what is effectively unrecoverable plastic strain developing at even relatively modest applied loads.

7.8 Comparison of observed settlements with other data

By way of comparison it is useful to refer to the work by Jarrett et al. (1974) who undertook a review of available settlement monitoring data (covering a period of 20 years), for low-rise structures at the ICI works located on the southern boundary of Grangemouth in the Forth Valley, to the east of Bothkennar. Such data are often rare or absent, so it was considered appropriate to briefly review this. It is understood that no piling or ground improvement had been adopted beneath the structures which were monitored. The soil profile is very similar to that at Bothkennar with the exception of a 1.5 m thick shelly sand and gravel layer encountered at a depth of 5.5 m (soil layer 3 in Figures 7.12a and 7.12b). Buildings monitored were light one to one and half storey brick infilled frame structures with pitched roofs of light construction. Results from the settlement monitoring are presented in Table 7.6 and it was reported that differential settlement had not caused structural distress in these buildings. In order to obtain an indication of the soil compressibility under the varying loading conditions imposed by these buildings, the average settlements presented in Table 7.6 were plotted by Jarrett et al. (1974) against the logarithms of the corresponding stress increases (Figure 7.13). As might be expected there is quite a considerable scatter of the results, but a reasonable and useful curve was fitted to the points. It is clear that the use of a rigid hollow raft to ameliorate the settlements and structural damage due to flexural differential settlements appear(s) to have been successful at this site underlain by a deep soft soil profile. The differences between the above recorded settlement data at Grangemouth and those for the untreated trial footing 8 at Bothkennar supports the theory that one is dealing with progressive creep settlement over a relatively long period. Even under the small loadings reported settlements over a 20-25 period beneath raft foundations at Grangemouth ranged from around 50 mm to 240 mm. In terms of implications for modern low-rise structures over weak compressible soils, although these structures are generally not monitored for settlement, settlements in the range 50 to 240 mm (as recorded at Grangemouth) are unlikely to be acceptable.

Based upon the results of the Bothkennar field trials, which have formed the basis of this thesis, and comments above, shallow, narrow footings are perhaps best supported on partially penetrating vibro stone columns, particularly where the crust is relatively thin (<1.5 m), as at Bothkennar. However, for more widespread loads, these are likely to perform better with an appropriately detailed raft foundation founded in the crust, rather than a raft supported by partially penetrating vibro stone columns, (particularly if there is not scope to place a temporary surcharge on the stone column reinforced ground prior to raft construction to address any ground heave (or accelerated consolidation issues). This is supported by evidence from Bothkennar, where the performance of a raft foundation located in the crust without vibro stone column ground treatment was compared with the performance of a raft foundation on partial depth vibro stone columns, Figure 7.14 and 7.15 (after Serridge, 2001; Chown and Crilly, 2000). It is perhaps important to recognise that if shallow foundations are chosen as the preferred foundation solution (without ground improvement), on normally consolidated clay profiles, they should be constructed where possible at a depth which utilises the strength of the crust. This will create a more slender, economic and potentially better performing shallow foundation than foundations placed below the crust. Whilst shallow foundations in soft clay soil profiles are susceptible to tilt and rotational movements, as described earlier, providing there is careful detailing given to the design and that over dig (excavation) in the crust is avoided (since a shallow foundation will be relying heavily on the intact strength of the crust), a typical application could be the use of a raft foundation or in some instances a shallower, narrow footing. There are essentially two types of raft foundation which could be supported on soft ground. Firstly the edge-beam raft which is regarded as being 'semi-rigid' and secondly the 'plane slab' raft, regarded as flexible. Both types of raft have been shown to perform adequately in given situations (Chown and Crilly, 2000), however, in general the use of stiffer rafts allows for greater ground movement whilst retaining a high level of performance. A raft on partial depth vibro stone columns at Bothkennar (Serridge, 2001) has been shown not to be as effective as a raft founded within the crust (without ground improvement - Chown and Crilly, 2000) at least in the short term (see Figure 7.14 and Figure 7.15) although effective performance of the treated raft may have been masked by soil disturbance and heave associated with forming the 'end bulbs' during stone column construction. Ongoing research by the researcher is suggesting that constructing partial depth vibro stone columns in soft clay soils without end bulbs is providing superior performance.

7.9 Post (stone column) trial observations

Following completion of the field trials, footings 1 and 5 were carefully lifted and turned on their side to expose the sub-grade formation, to facilitate examination of any deformation effects within the upper sections of the stone columns below founding level. This incorporated the staged excavation (Figure 7.16a) and measurement of stone column diameter (and therefore any deformation characteristics), to a maximum of 3.0 m below founding level. A deformed shape was observed during this exercise and is recorded in Figure 7.16b. The bulged column is analogous to a failed triaxial sample with a height to width ratio of around 2. The bulging profile is also geometrically similar to that observed by Hughes and Withers (1974) laboratory based studies, Hughes et al. (1976) field based trials, and laboratory based studies by Brauns (1978), Hu (1995) and Mc Kelvey (2002). In particular, the bulging is confined to the upper zone of the column (in line with what would be anticipated, given the practical influence of the 'crust' over the soft clay). For the post trial exhumed stone column at Bothkennar the maximum diameter of 1.0 m was recorded at a depth of around 0.4-1.5 m beneath the underside of the - 0.5 m founding depth. This compares with a pre-loading stone column diameter of 0.75 m, determined prior to commencement of the trials (see Chapter 6 Section 6.2). Bulging clearly took place within the soft clay immediately below the crust (and is supported by the lateral stress changes described previously during load application to the footings), with the crust having been deformed at constant volume under load. This confirms the Hughes and Withers (1974) hypothesis, namely that with stress transfer through skin friction to the soil the direct vertical stress in a column would rapidly diminish so that a single column would be unlikely to bulge except near the top. This is considered to be attributed to the fact that the direct stresses are highest at this level and the containing radial stress is likely to be a minimum because there is little overburden weight (constraint), and also the strength of the normally consolidated clay just below the crust is low. Thus one observes bulging just below the crust with little stress transferred downwards beyond about five column diameters as evidenced in the investigation of varying stone column length earlier in the chapter (Section 7.3). By way of comparison, field trials described by Greenwood (1991) at Uskmouth, UK (see Chapter 2 – Section 2.3) demonstrated (following column exhumation), an average stone column diameter of 850 mm (range 810-890 mm) for a stone column installed by the wet top feed technique through a normally consolidated soft clay and subsequently subjected to plate load testing. The maximum diameter of 890 mm was recorded at 2 m (i.e. below a thin surface crust), again demonstrating the practical influence of a stiff crust over soft material as at Bothkennar.

Following completion of the Bothkennar trials and removal of trial footings 1-5, opportunity was also taken to undertake some hand shear vane testing within the Bothkennar Clay profile, in the clay soil surrounding the stone columns (Figure 7.17a). The results (Figure 7.17b) demonstrate enhanced values of undrained shear strength (cohesion), compared to pre-treatment values, by a factor of up to 1.5 and which was more pronounced in the soft Bothkennar Clay immediately beneath the crust. This represents a significant improvement from the recorded post stone column installation 'remoulded' values (Figure 7.17b). The observed improvement in undrained shear strength is almost identical to that recorded by Greenwood (1991) following field plate load testing of a stone column extending through a soft clay profile with a surface crust at Uskmouth, UK, previously discussed (see Chapter 2 - section 2.3). As intimated previously it is postulated that the shelly layer (see Figure 4.3), combined with the presence of stone columns, provided very efficient drainage path(s) for (excess) pore water dissipation via the stone columns during the loading increments applied to the trial footings at Bothkennar, leading to an improvement in soil stiffness under the applied loads. Indeed, if one undertakes a consolidation analysis, allowing for the permeable shelly horizon and the shorter drainage paths provided by the stone columns, the reduced settlement value at completion of primary consolidation can be related back to the improved undrained shear strength and therefore stiffness of the soft clay soil between stone columns within the treated depth. This observation also accords well with Aboshi et al. (1979) in their K_o laboratory studies (mentioned in Chapter 2). It was also found by Aboshi et al. (1979) that the stress ratio increases as primary consolidation proceeds. Similar observations were made in the Bothkennar trials as described previously.

Parameter investigated	Trial Footing no.	Stone Column Spacing (m)	Stone Column Length Beneath Underside of Foundation (m)	Stress Ratio	
				1st Load Increment	2nd Load Increment
Stone column spacing	1	1.5	5.7	2.09	1.67
	2	2	5.7	2.00	1.75
Stone column length	3	1.5	3.7	2.29	2.63
	4	1.5	5.7	2.8	3.26
	5	1.5	7.7	2.44	4.00
Absence of crust	6	1.5	5.7	----	----

Table 7. 1: Recorded stress ratios for trial footings over stone column reinforced soft clay at Bothkennar.

Loading stage for trial footing 1 (1.5 m column spacing)	% of applied load attracted by stone column	% of applied load attracted by Intervening soil
First applied load increment ($35.5\ kN/m^2$) .	Start : 28 Mid-point : 22 End : 20	Start : 11 Mid-point : 6-7 End : 14
Second applied load increment ($72.0\ kN/m^2$).	Start : 24.5 Mid-point: 30. End : 35	Start : 20 Mid-point : 15-18 End : 18

Loading stage for trial footing 2 (2.0 m column spacing)	% of applied load attracted by stone column	% of applied load attracted by Intervening soil
First applied load increment ($32.9\ kN/m^2$) .	Start : 60 Mid-point : 38 End : 58	Start : 30 Mid-point : 21 End : 27
Second applied load increment ($67.1\ kN/m^2$).	Start : 45 Mid-point: 65 End : 74.5	Start : 29 Mid-point : 22 End : 53

Stress ratio: trial footing 1	Stress ratio: trial footing 2
First applied load increment: Start : 2.54 Mid-point : 3.38 End : 1.43	**First applied load increment:** Start : 2.00 Mid-point : 1.81 End : 2.14
Second applied load increment : Start : 1.23 Mid-point : 1.81 End : 1.94	**Second applied load increment :** Start : 1.55 Mid-point : 2.95 End : 1.41

Table 7.2: Percentage of applied vertical stress attracted by stone column and intervening soil beneath trial footings 1 and 2, and stress ratios.

Loading stage for trial footing 3 (1.5 m column spacing)	% of applied load attracted by stone column	% of applied load attracted by Intervening soil
First applied load increment (33.1 kN/m^2) .	Start : 42 Mid-point : 30 End : 45	Start : 17-18 Mid-point : 17 End : 17
Second applied load increment (67.8 kN/m^2).	Start : 39 Mid-point: 64 End : 72	Start : 25 Mid-point : 18-19 End : 22

Loading stage for trial footing 4 (1.5 m column spacing)	% of applied load attracted by stone column	% of applied load attracted by Intervening soil
First applied load increment (34.9 kN/m^2) .	Start : 71 Mid-point : 50 End : 57	Start : 20 Mid-point : 20 End : 22
Second applied load increment (71.1 kN/m^2).	Start : 57 Mid-point: 79-80 End : 88	Start : 25 Mid-point : 21 End : 22.5
Third applied load increment (108.1 kN/m^2).	Start : > 100 Mid-point : -- End : >100	Start : 28 Mid-point : -- End : 25
Fourth applied load increment (125.1 kN/m^2).	Start : - Mid-point : -- End : > 100	Start : -- Mid-point -- End : 24 :

Loading stage for trial footing 5 (1.5 m column spacing)	% of applied load attracted by stone column	% of applied load attracted by Intervening soil
First applied load increment (32.1 kN/m^2) .	Start : 93.5 Mid-point 62: End : 78	Start : 23 Mid-point : 44 End : 28
Second applied load increment (67.0 kN/m^2).	Start : 71 Mid-point: 100 End : >100	Start : 26.5 Mid-point : 22 End : 22

Table 7.3 : Percentage of applied vertical stress attracted by stone column and intervening soil beneath trial footings 3, 4 and 5, and stress ratios.

Loading stage for trial footing 6	% of applied load attracted by stone column
First applied load increment (34.2 kN/m^2)	Start: > 100% Mid-point: 92% End : 100 %
Second applied load increment (69.6 kN/m^2).	Start : 100% Mid-point: > 100% End : > 100%
Third applied load increment (105.5 kN/m^2).	Start : 100% Mid-point -- End --
Fourth applied load increment (122.4 kN/m^2).	Start -- Mid-point -- End : 98%

Loading stage for trial footing 6	Recorded lateral stress (kN/m^2) at 0.5 m below founding depth	Recorded lateral stress (kN/m^2) at 1.1 m below founding depth
First applied load increment (34.2 kN/m^2) .	Start : 0.5-1.0 Mid-point : 5.0 End : 4.0	Start : 3.0 Mid-point : 0.0 End : -2.5
Second applied load increment (69.6 kN/m^2).	Start : 7.5 Mid-point : 6.0 End : 4.0	Start : -2.5 Mid-point : 0.0-1.0 End : 5.0
Third applied load increment (105.5 kN/m^2).	Start : 41 Mid-point -- End --	Start : 15 Mid-point -- End --
Fourth applied load increment (122.4 kN/m^2) .	Start -- Mid-point -- End: 34	Start -- Mid-point -- End: 15

Table 7.4: Percentage of applied vertical stress attracted by stone column 34 (footing 6) and recorded lateral stresses.

Case and Reference	Footing dimensions and founding depth	Soil	Reported failure load (kPa)
Kippen, Forth Valley, Scotland (Skempton, 1942)	2.4 square; 0.7 m deep.	Carse Clay	120
Ottawa, Canada (Shields and Bauer, 1975)	3.1 m square; 0.7 m deep.	Leda Clay	>300
Grangemouth, Forth Valley, Scotland (Thorburn, 1976)	2.4 m square; 0.7 m deep.	Carse Clay	148
Shell Haven, Thames Estuary (Schnaid et al, 1993)	14 m x 5 m; 0.4 m deep.	Thames alluvial Clay	84
Bothkennar, Forth Valley, Scotland (Jardine et al, 1995)	2.2 m square; 0.75 m deep.	Carse Clay	138
Bothkennar, Forth Valley, Scotland (Serridge, 2013)	3.1 m x 0.75 m; 0.50 m deep.	Carse Clay	125

Table 7.5: Case histories of bearing capacity failure loads for foundations in soft clays.

Identification	*Buildings*	*Plan dimensions* (m)	*Foundation type*	*Nett loadings* Δq kN/m^2	*Average settlement* δ_{ave} mm
a	Analytical Lab. East	12.8 × 33.0	Strip footings	21.9	107
b	East Area Lab.	11.6 × 18.2	Hollow raft	20.2	128
c	West Area Lab.	11.6 × 18.2	Hollow raft	20.2	49
d	G2 Shed	24.3 × 54.8	Hollow raft	31.0	237
e	Q2 Shed	22.8 × 63.3	Hollow raft	24.5	171
f	H2 Shed	24.3 × 45.6	Hollow raft	26.0	152
g	L3 Shed	22.8 × 42.6	Hollow raft	25.0	70
h	West Area Workshop	6.1 × 16.7	Strip footings	13.0	11
j	Welding Shop	12.2 × 18.2	Strip footings	14.7	11
k	Instrument Workshop	11.2 × 21.2	Strip footings	15.6	12
l	New Laundry Block	12.2 × 20.0	Strip footings	13.6	15
m	East Area Workshop	6.1 × 19.8	Strip footings	12.6	8
n	Engineering Lab.	8.2 × 18.2	Strip footings	12.8	12
p	Amenities Block West	24.3 × 56.2	Hollow raft	12.4	18
q	Main Office Extension	12.8 × 17.3	Hollow raft	15.1	40

Table 7.6: ICI Works, Grangemouth. Building details and settlements (after Jarrett et al., 1974).

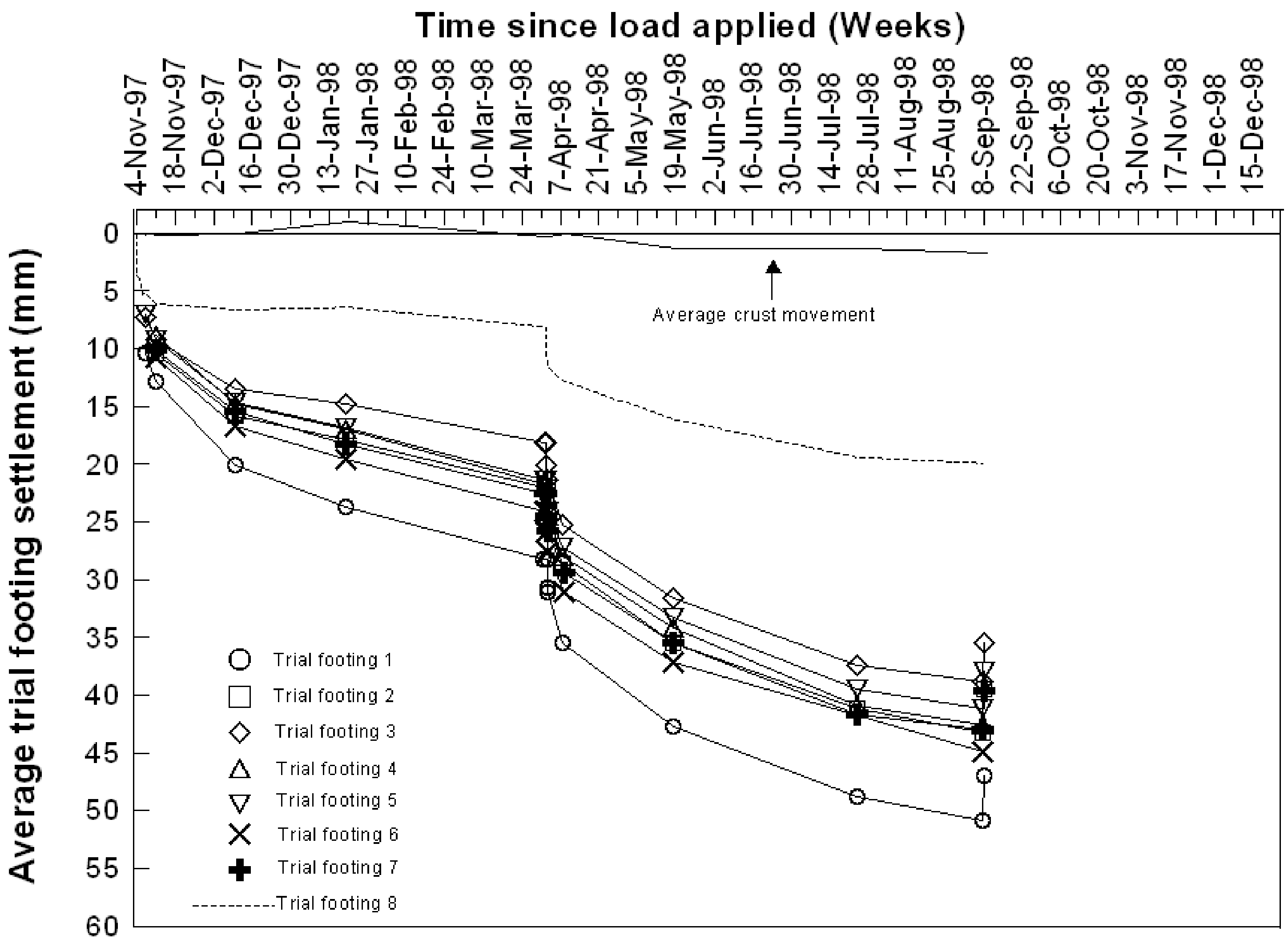


Figure 7.1: (a) Time - settlement plot for the treated and untreated trial footings under the first and second load increments.

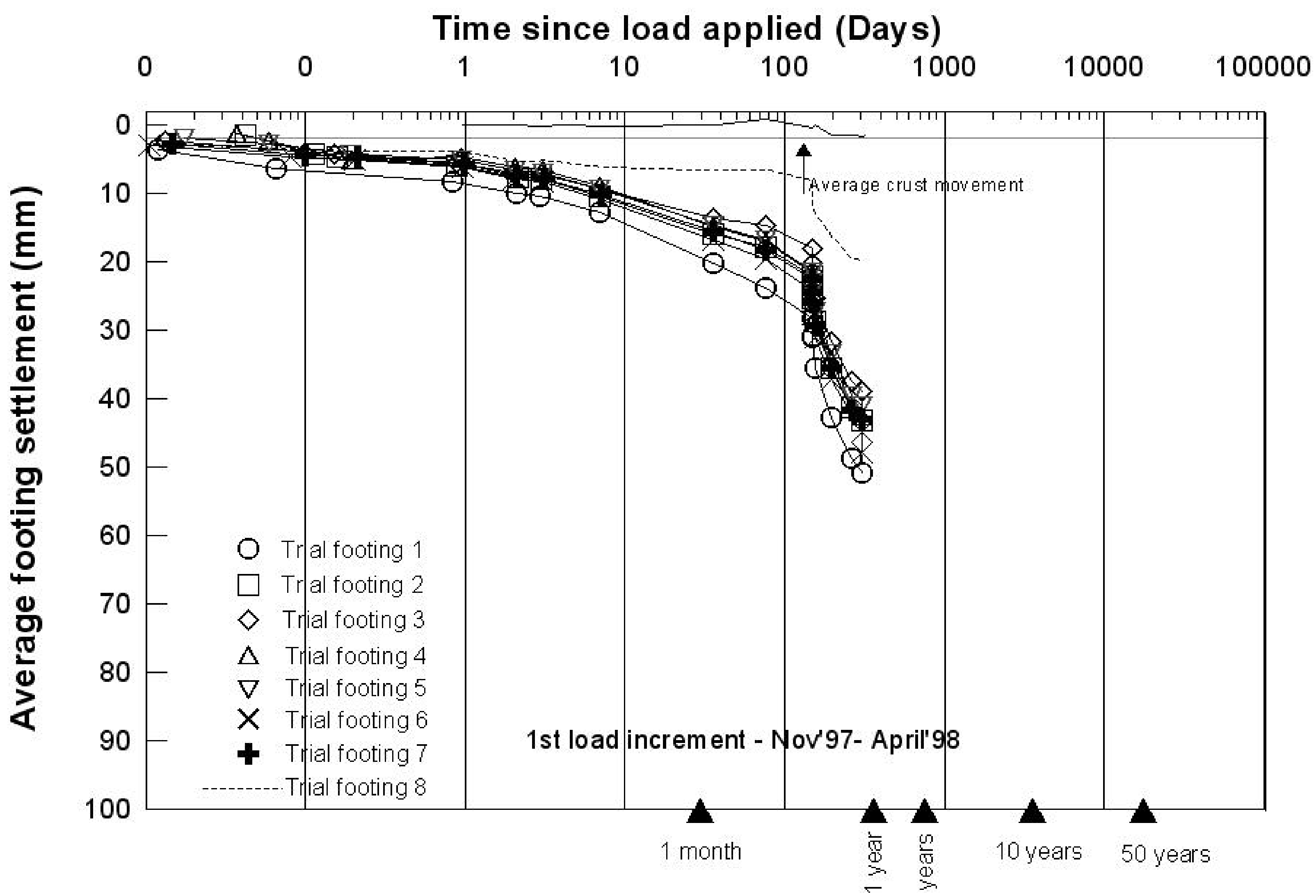


Figure 7.1: (b) Log time plot for settlement of the treated and untreated trial footings under the first and second load increments.

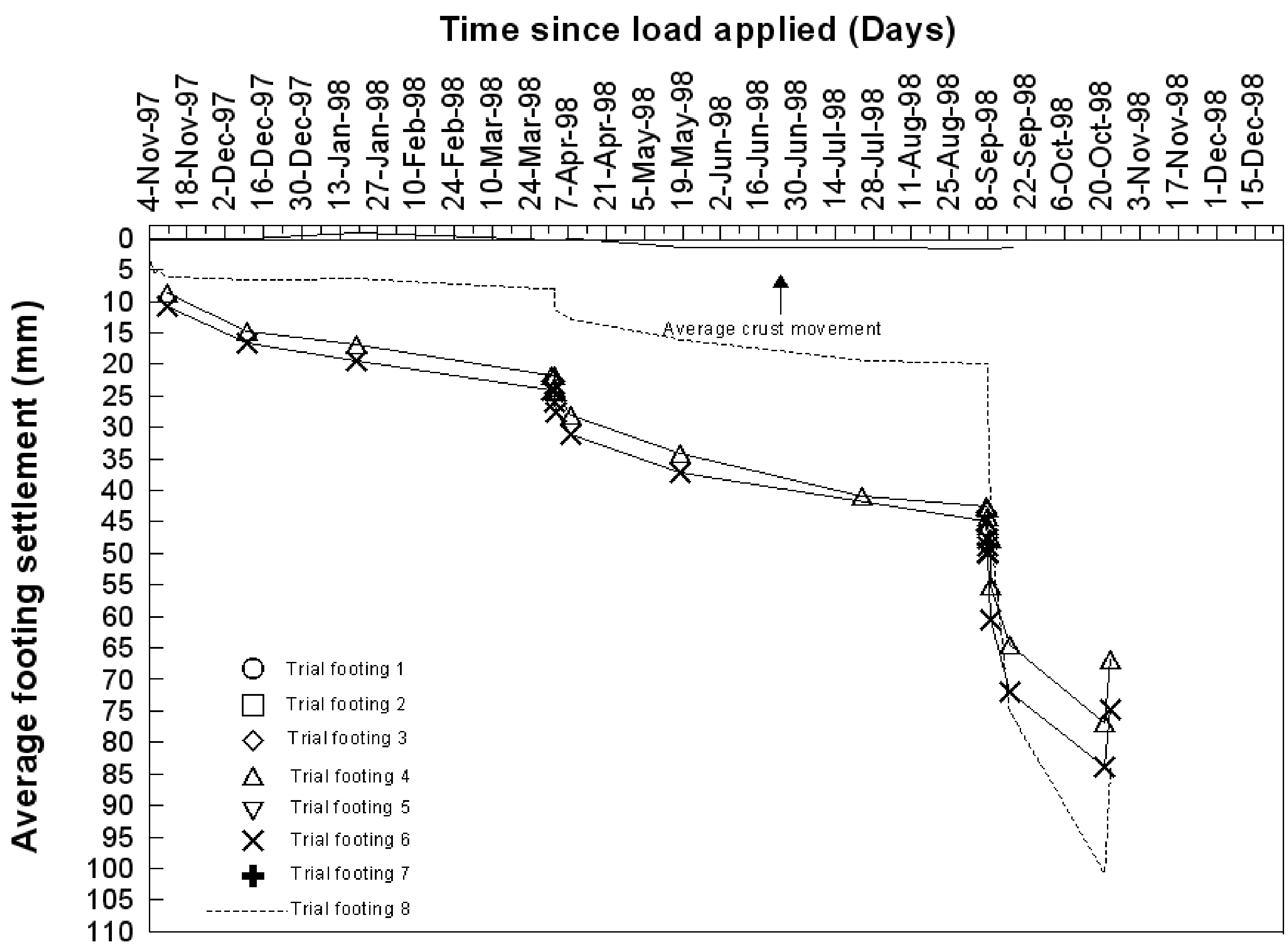


Figure 7. 2: (a) Time- settlement plot for the treated (trial footing 4 and 6) and untreated (trial footing 8) footings under the first and second load increment, together with third and fourth load increments.

b)

c)

Figure 7.2: (b) Four load increments in place on treated trial footings 4 and 6 and untreated trial footing 8. (c) Precise levelling being carried out on trial footing 4 following fourth load increment application.

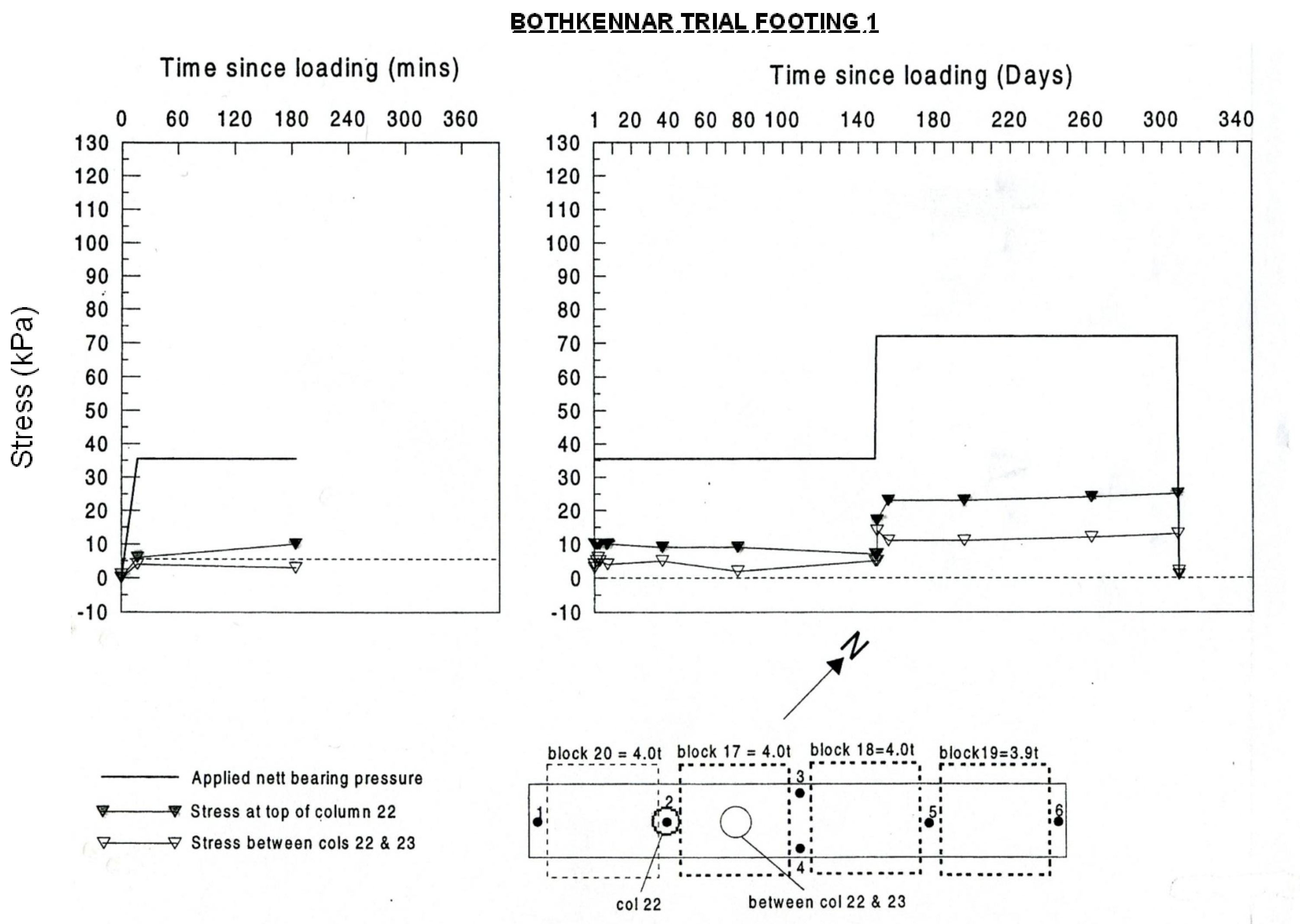


Figure 7.3: Response of instrumentation to loading of trial footing 1 (two load increments).

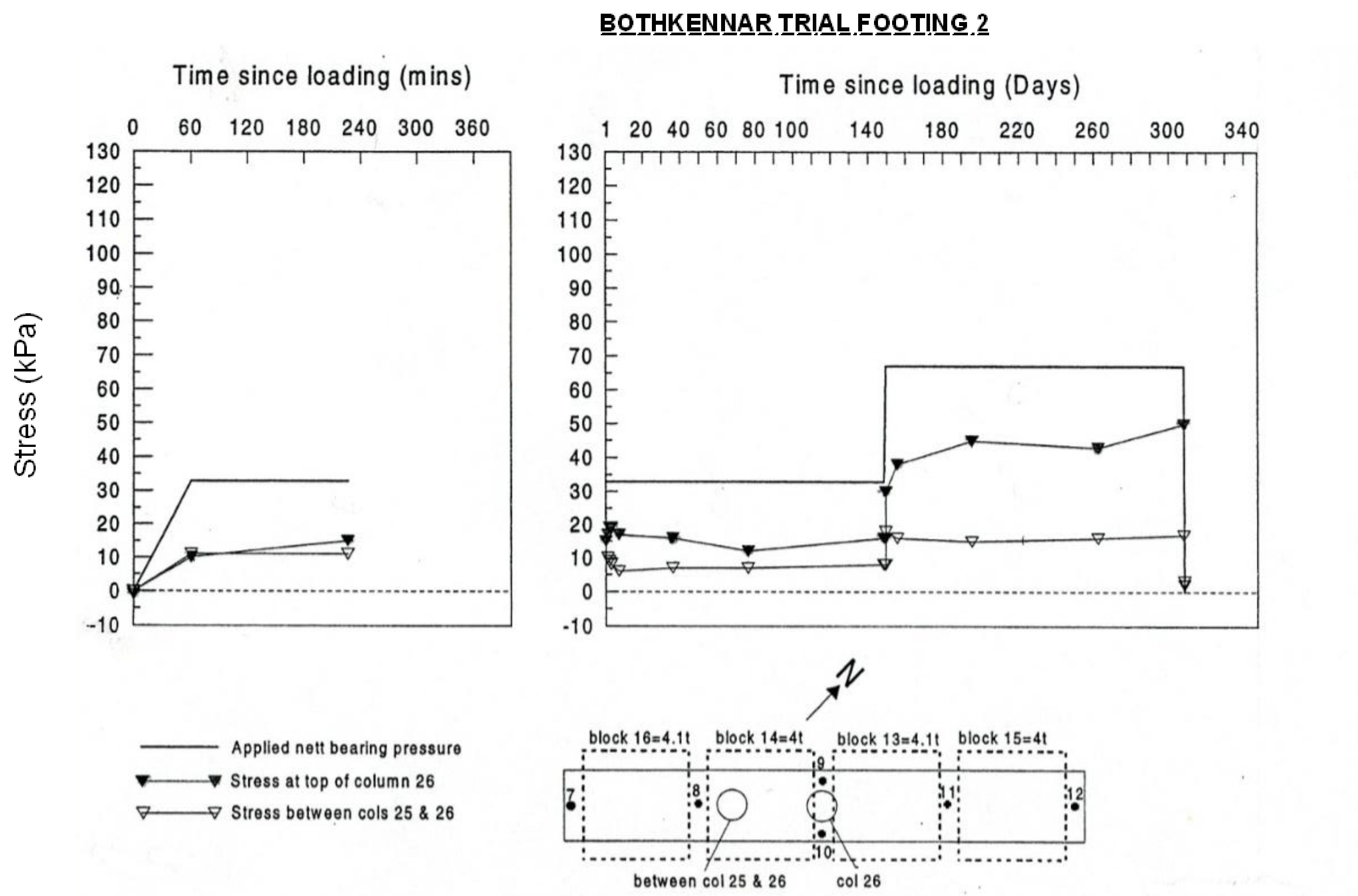


Figure 7.4: Response of instrumentation to loading of trial footing 2 (two load increments)

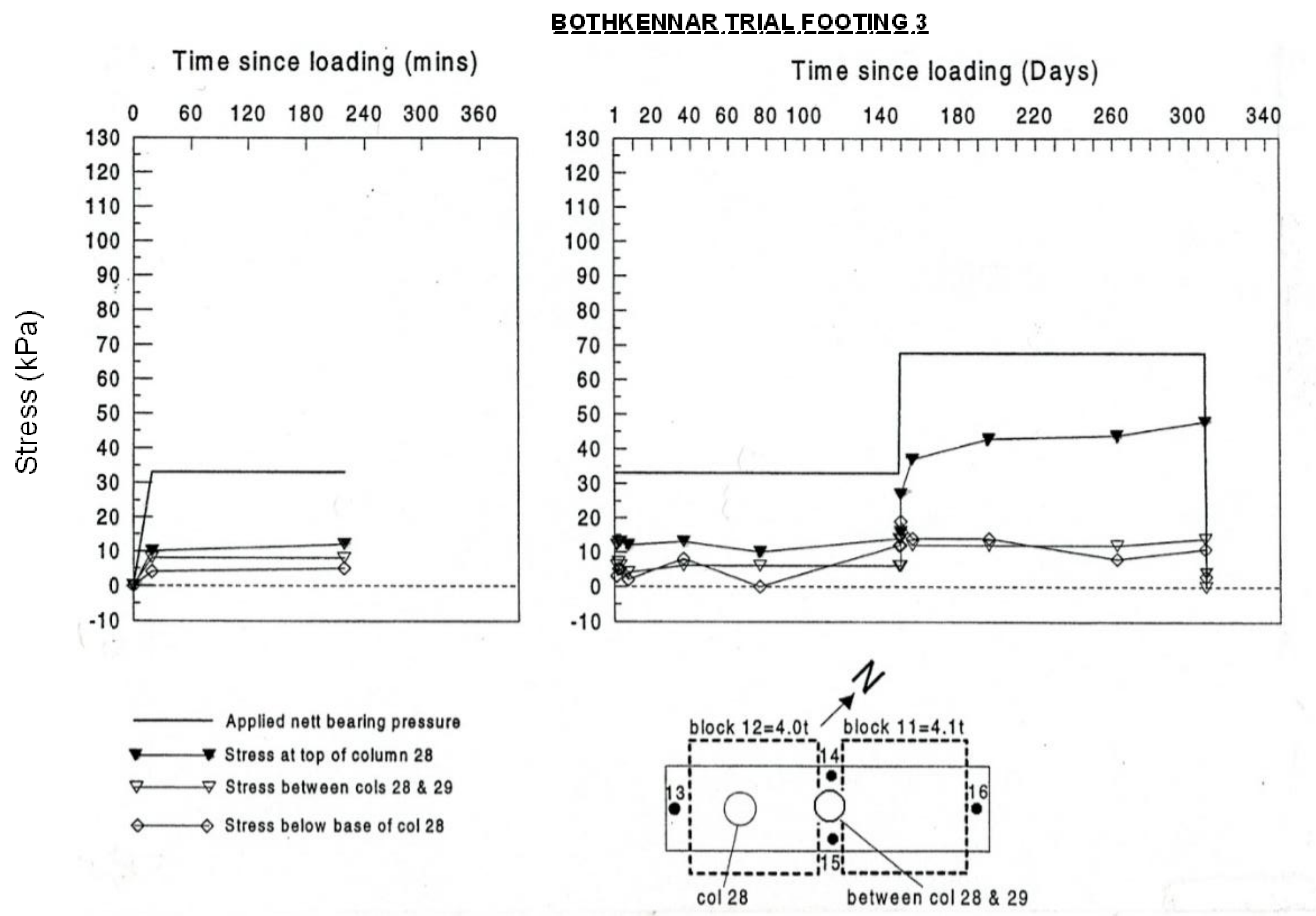


Figure 7.5: Response of instrumentation to loading of trial footing 3 (two load increments)

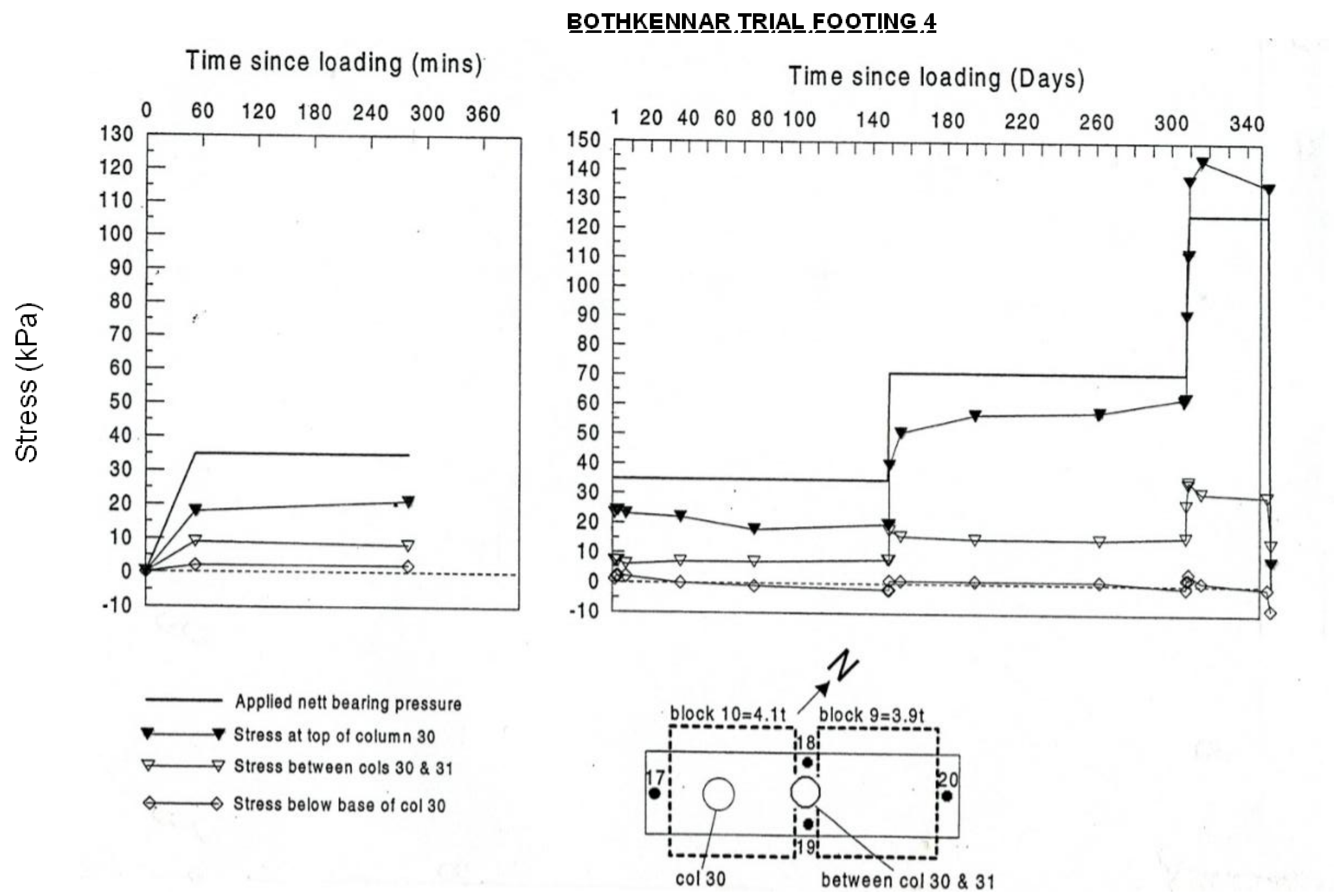


Figure 7.6: (a) Response of instrumentation to loading of trial footing 4 (four load increments)

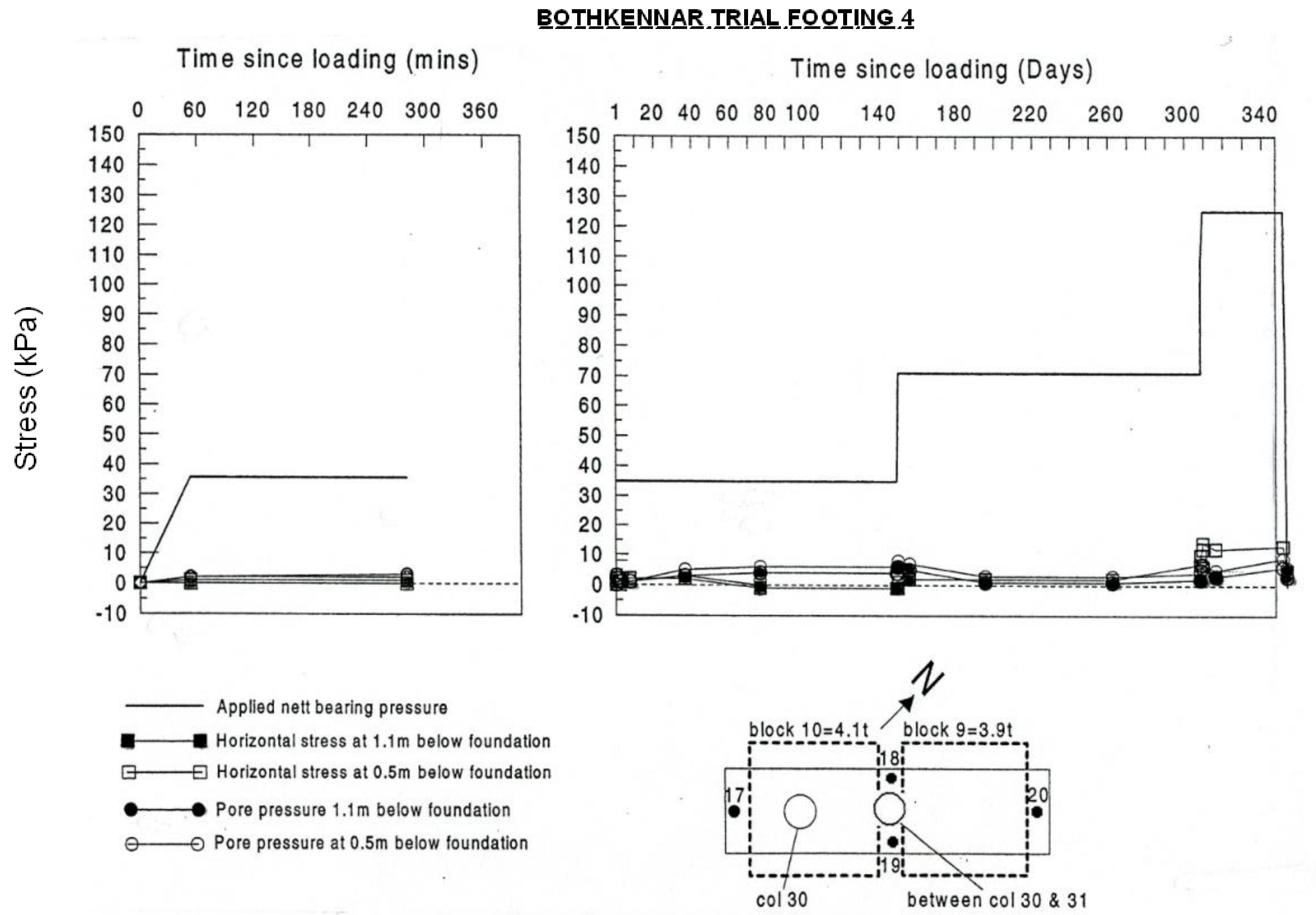


Figure 7.6: (b) Response of instrumentation to loading of trial footing 4 (four load increments)

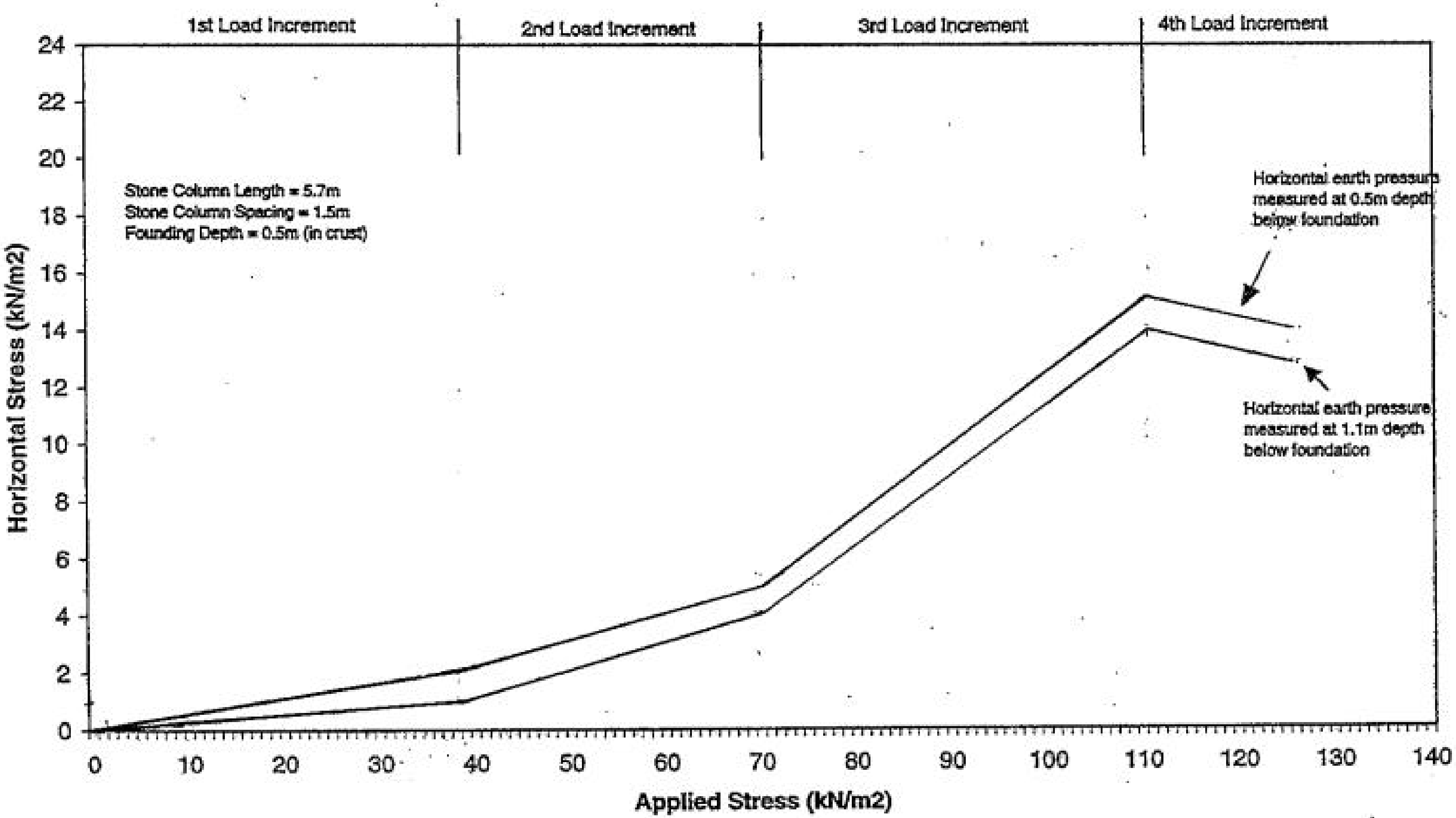


Figure 7.6: (c) Peak horizontal (lateral) earth pressure(s) recorded at each of the four load increments applied to trial footing 4.

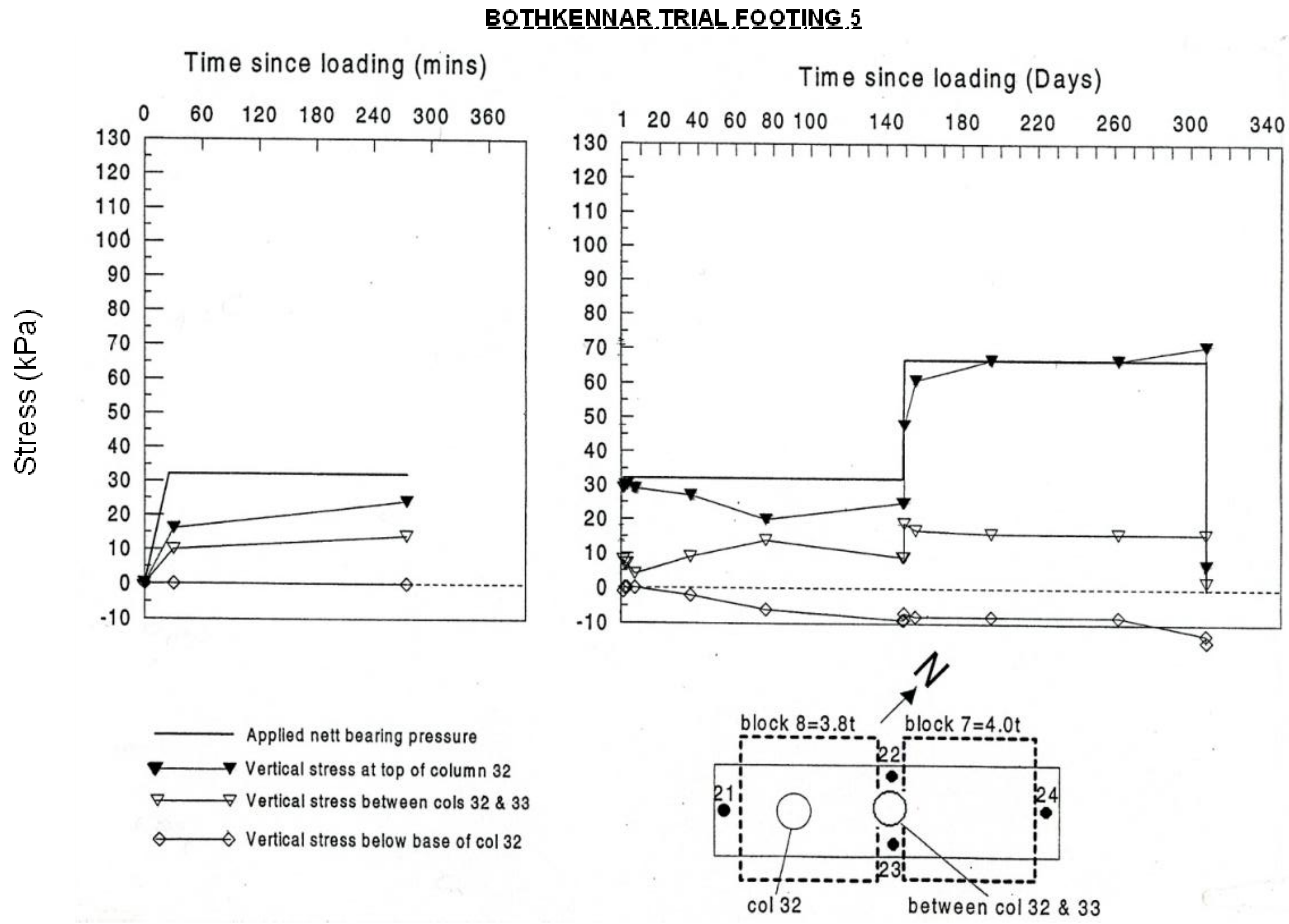


Figure 7.7: Response of instrumentation to loading of trial footing 5 (two load increments).

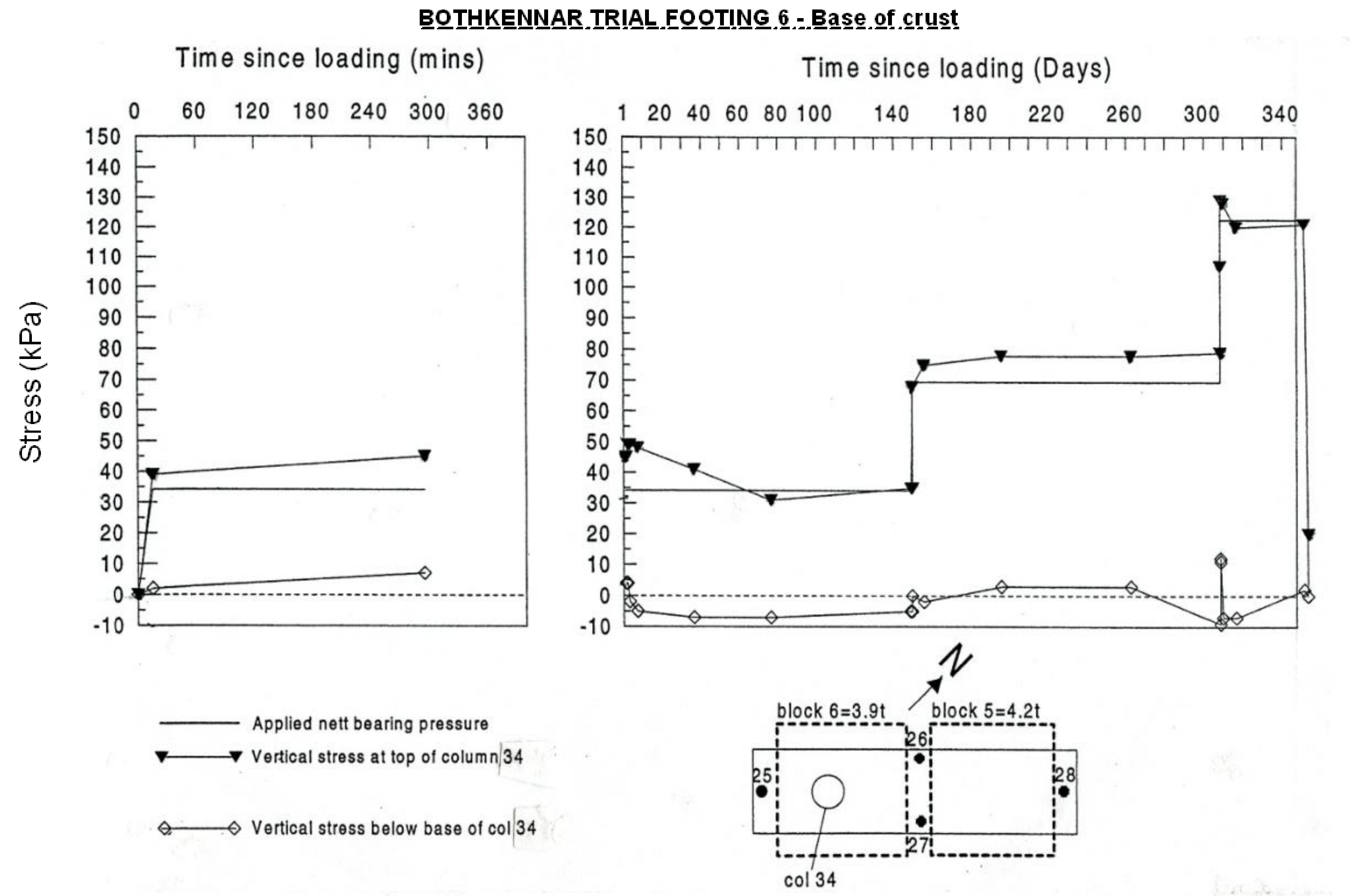


Figure 7.8: (a) Response of instrumentation to loading of trial footing 6 (four load increments).

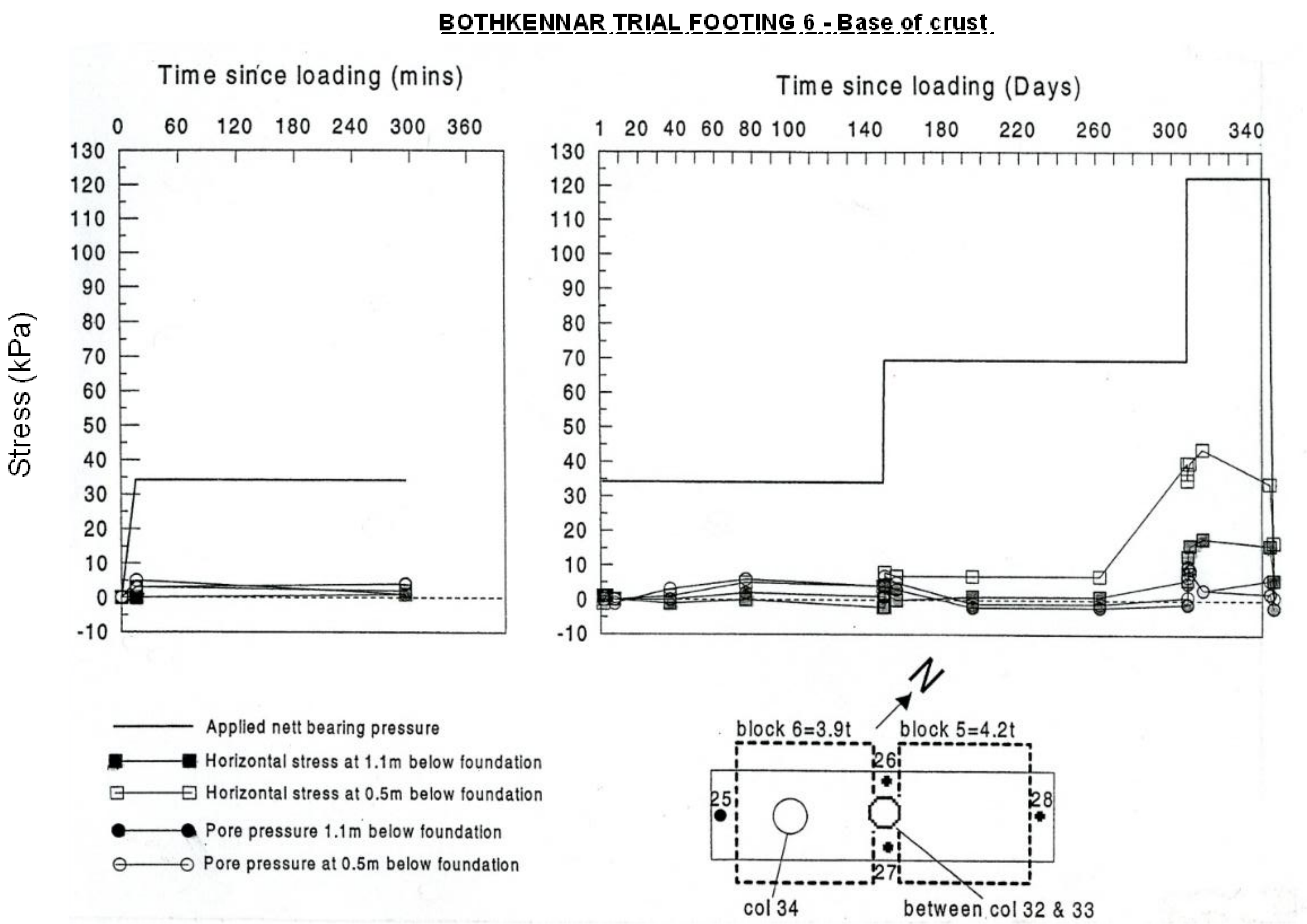


Figure 7.8: (b) Response of instrumentation to loading of trial footing 6 (four load increments).

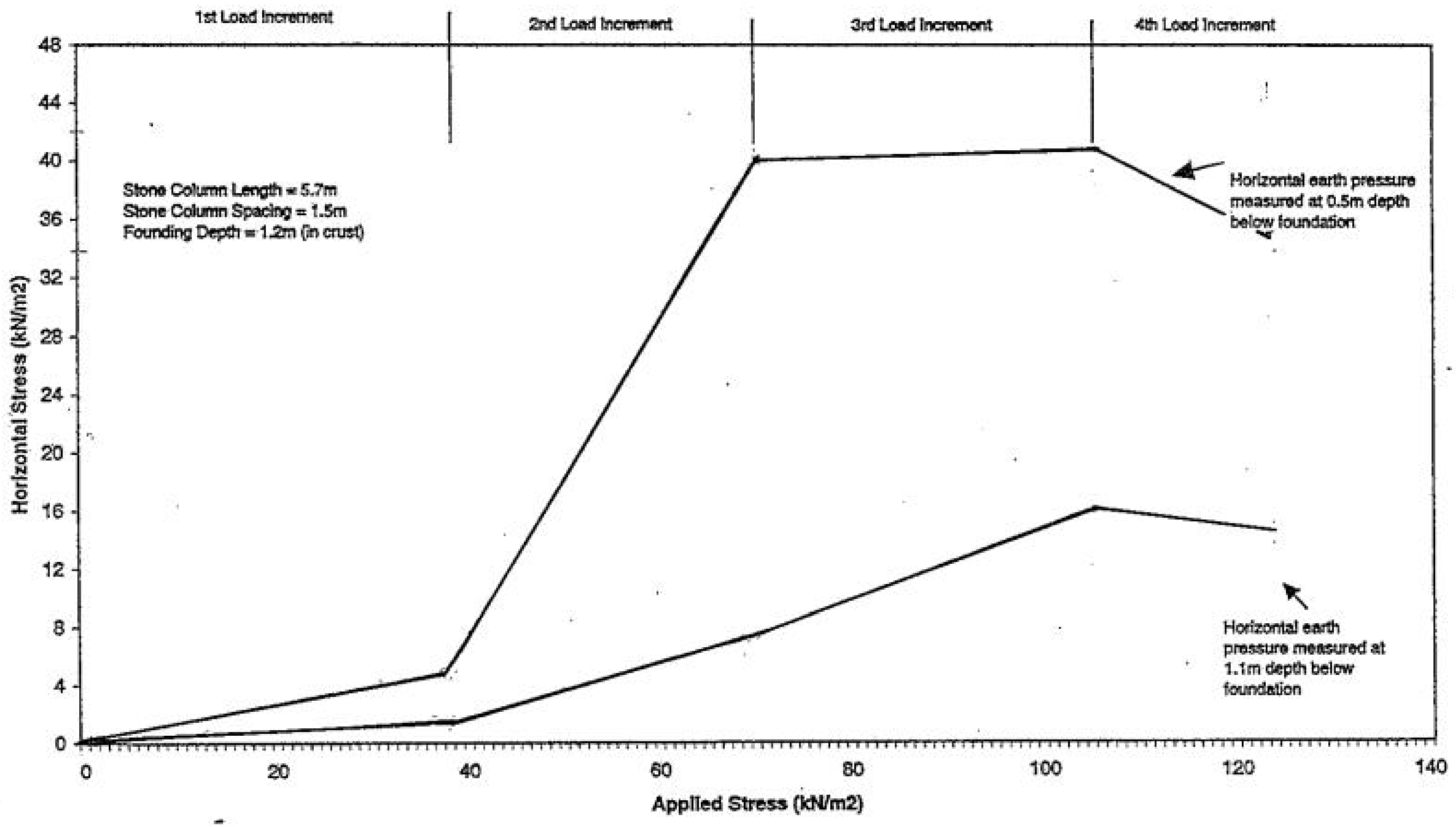


Figure 7.8: (c) Peak horizontal (lateral) earth pressure(s) recorded at each of four load increments applied to trial footing 6.

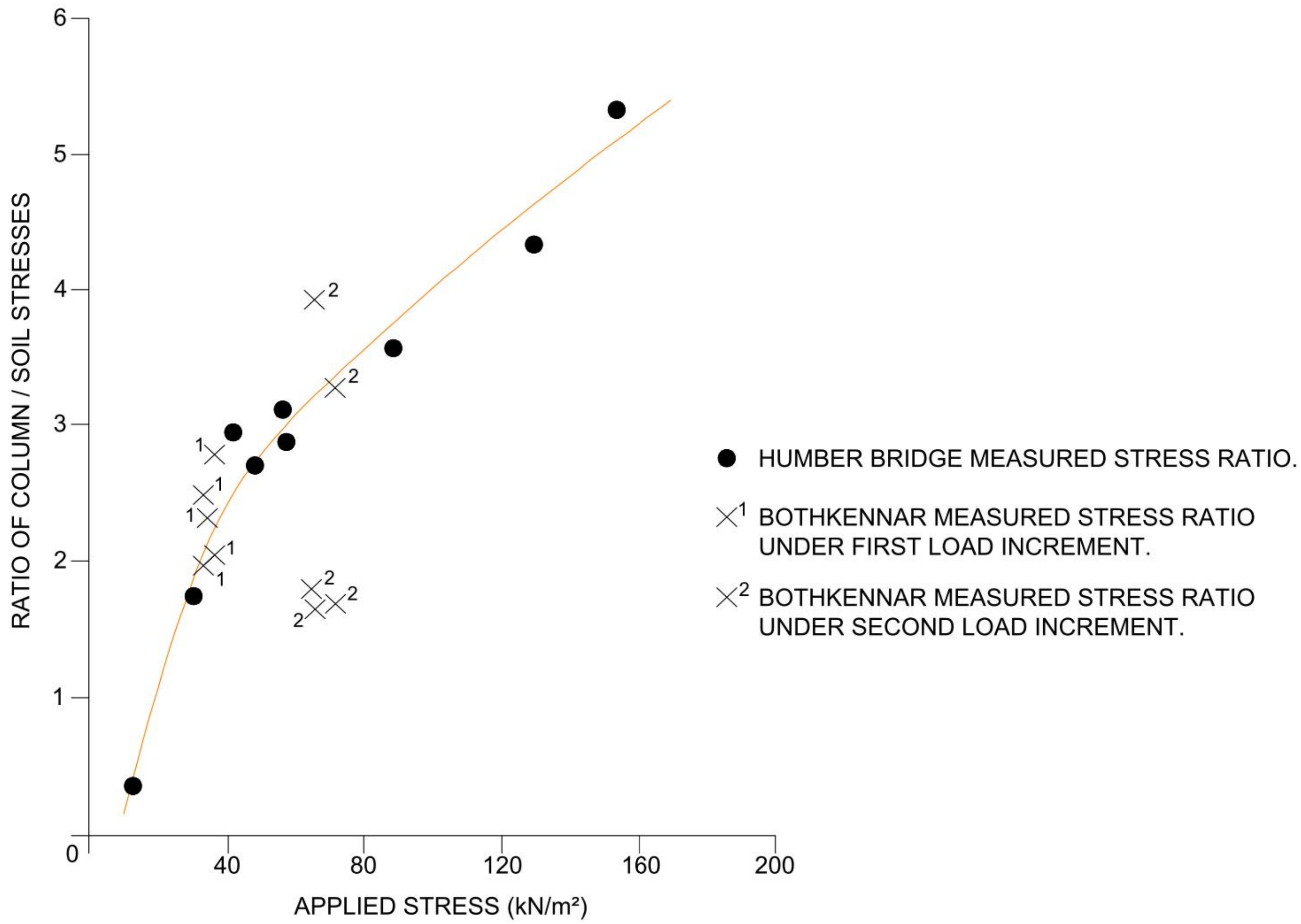


Figure 7.9: Stress ratio values from the Bothkennar field trials superimposed on data from Humber Bridge approach. Modified from Greenwood, 1991.

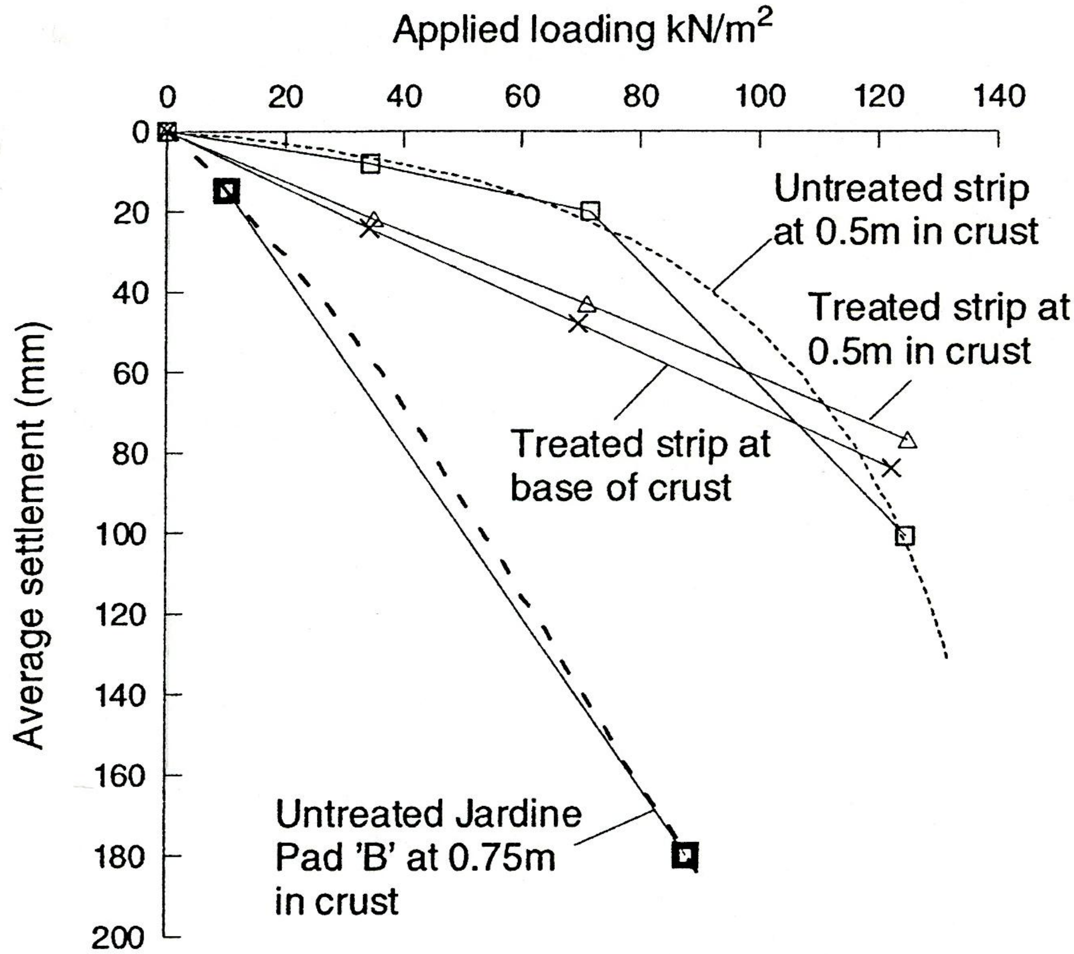
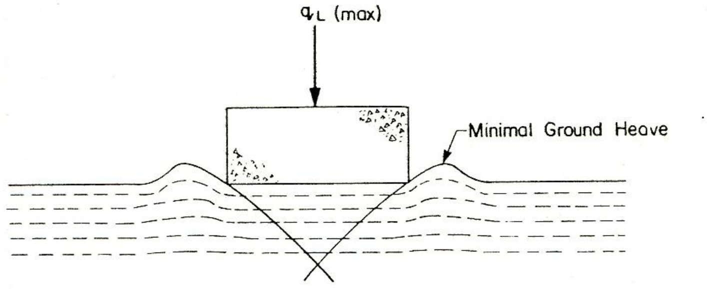
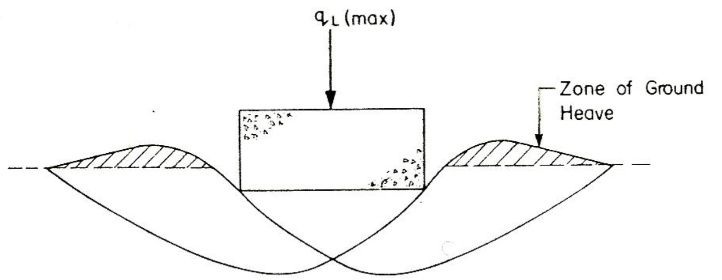


Figure 7.10: Load-settlement curves for some of the treated trial footings (footing 4 founded at 0.5 m depth; footing 6 founded at 1.2 m depth (base of crust) and the untreated trial footing (footing 8)), compared with earlier trials performed by Jardine et al. (1995) on untreated ground at Bothkennar.



Punching failure mechanism.

a)

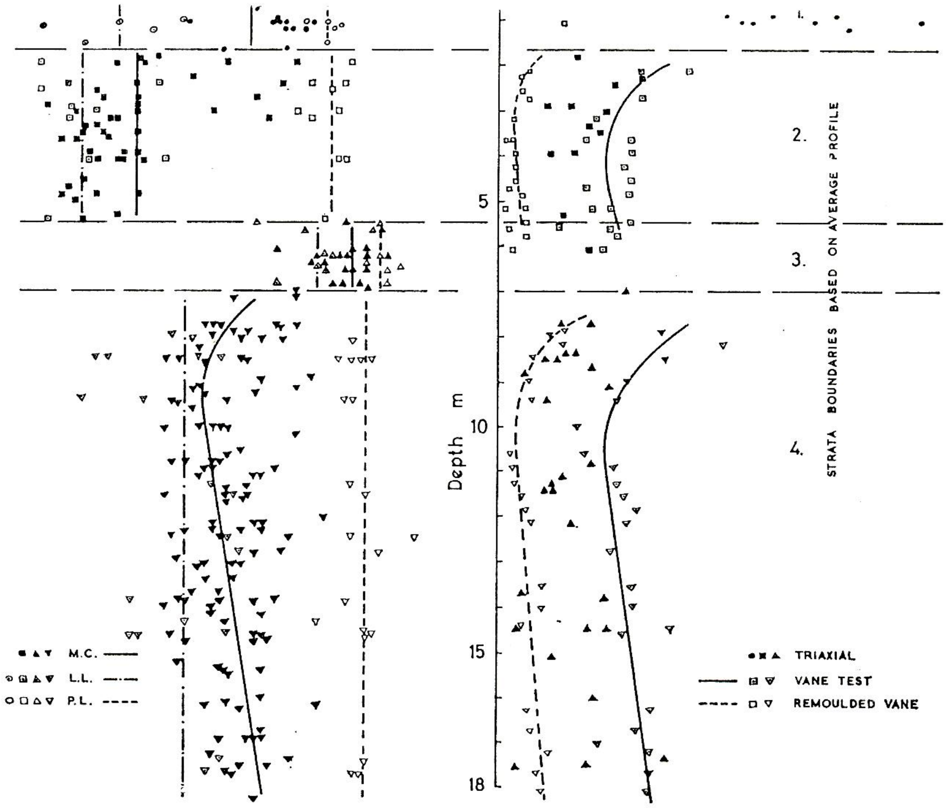


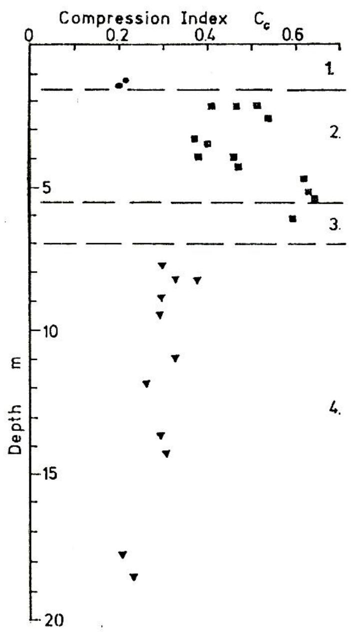
Classical Prandtl wedge failure mechanism

b)

Figure 7.11: (a) Punching failure mechanism and (b) Classical Prandtl wedge failure mechanism.

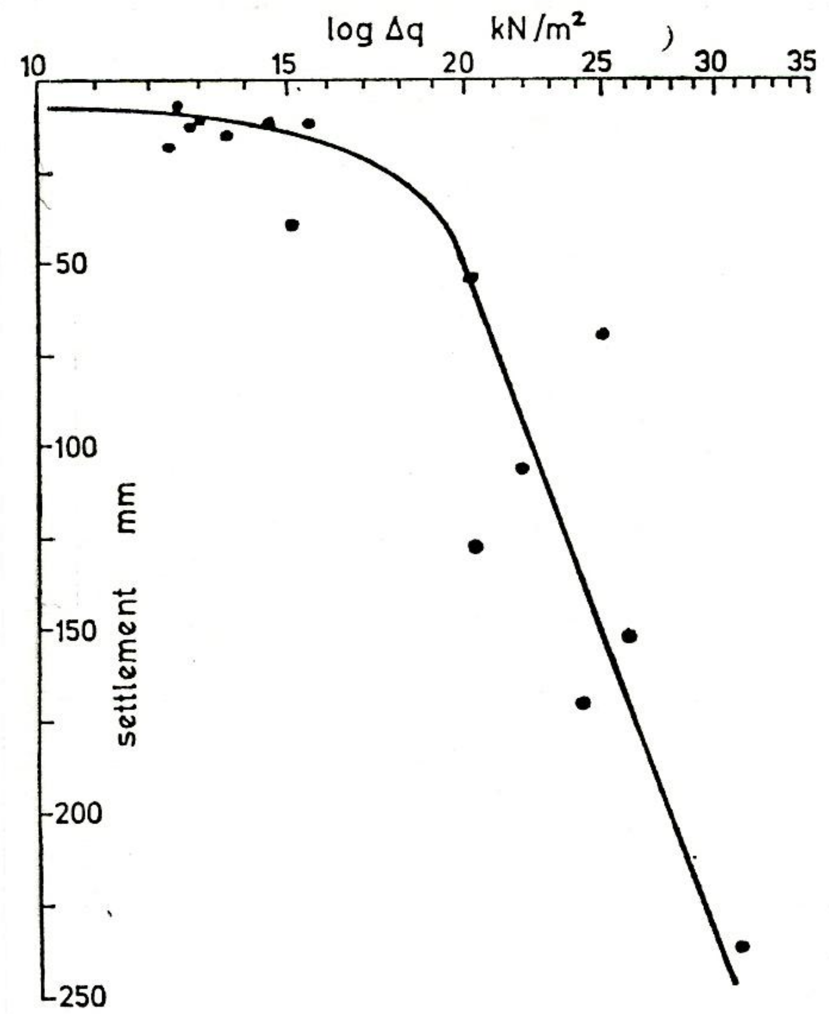
a)





b)

Figure 7.12: (a) Index properties and shear strength results. (b) Compression index values – ICI Works, Grangemouth (after Jarrett et al., 1974).



Nett loadings versus settlement

Figure 7.13: Average settlements recorded for monitored structures plotted against logarithms of the corresponding stress increases – ICI Works, Grangemouth (after Jarrett et al., 1974).

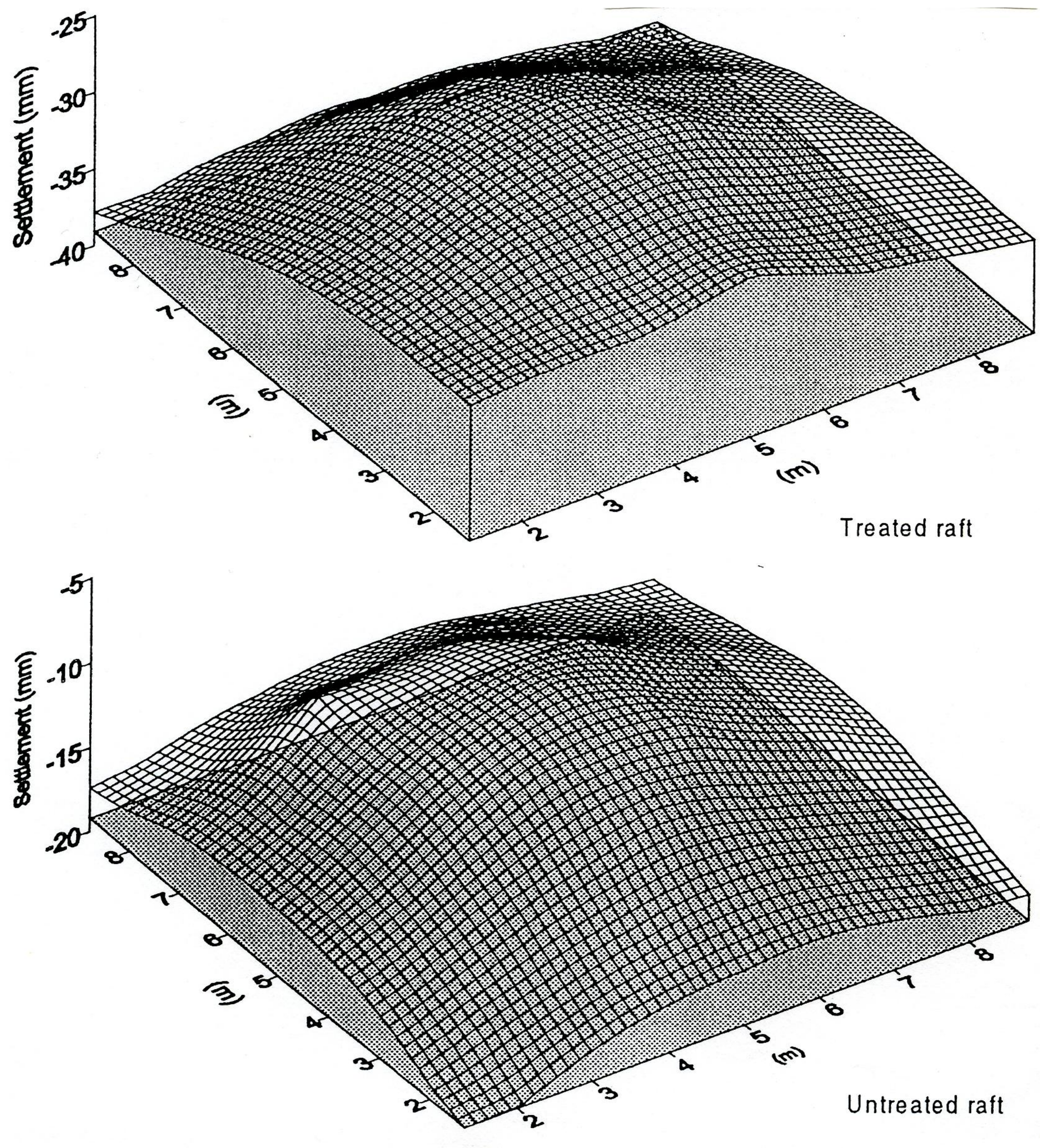


Figure 7.14: Comparison of deformation of the treated raft (Serridge, 2001) with the untreated raft (Chown and Crilly, 2000), 35 days after application of full outside line load of 50 kN/m run.

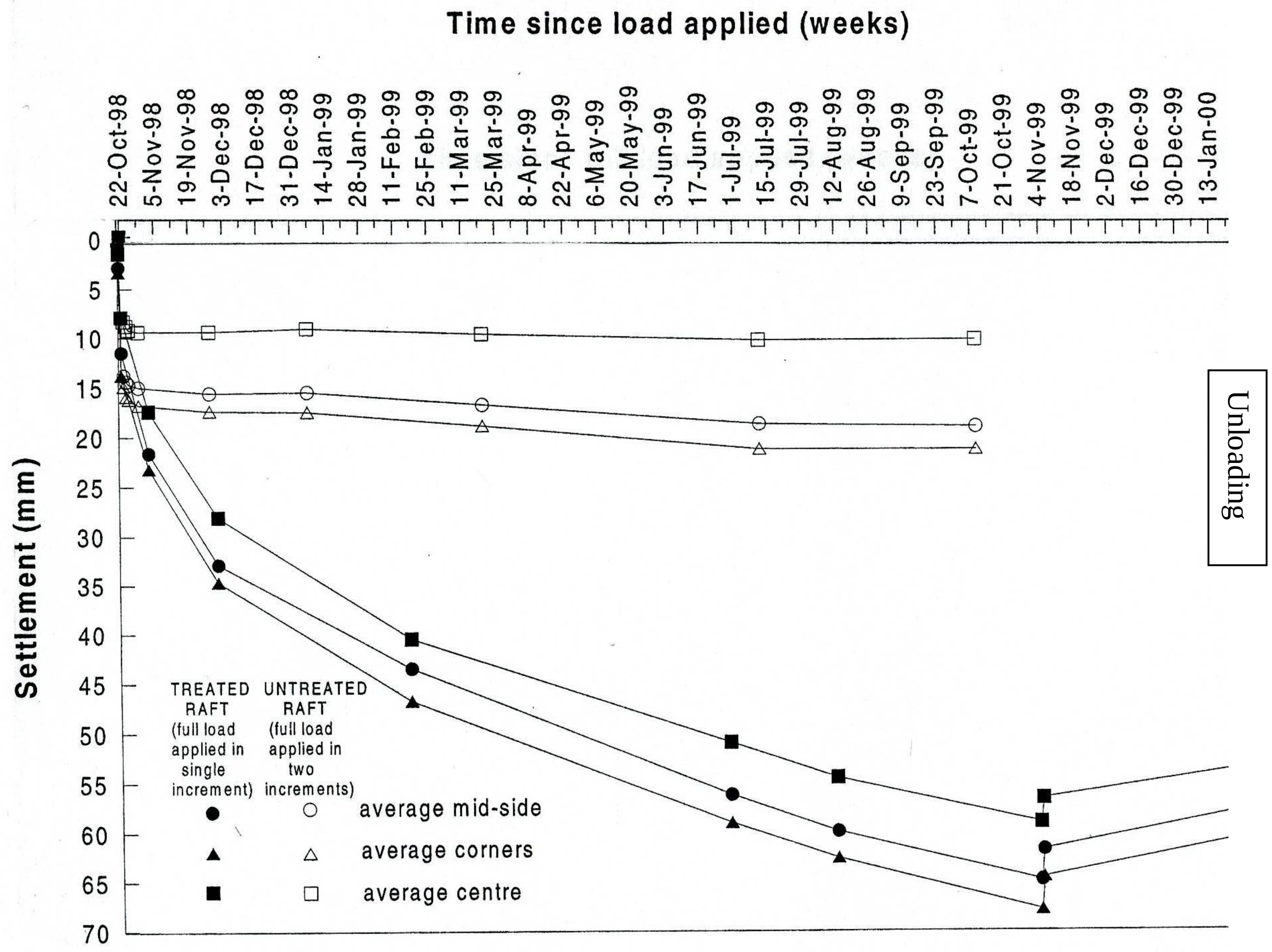
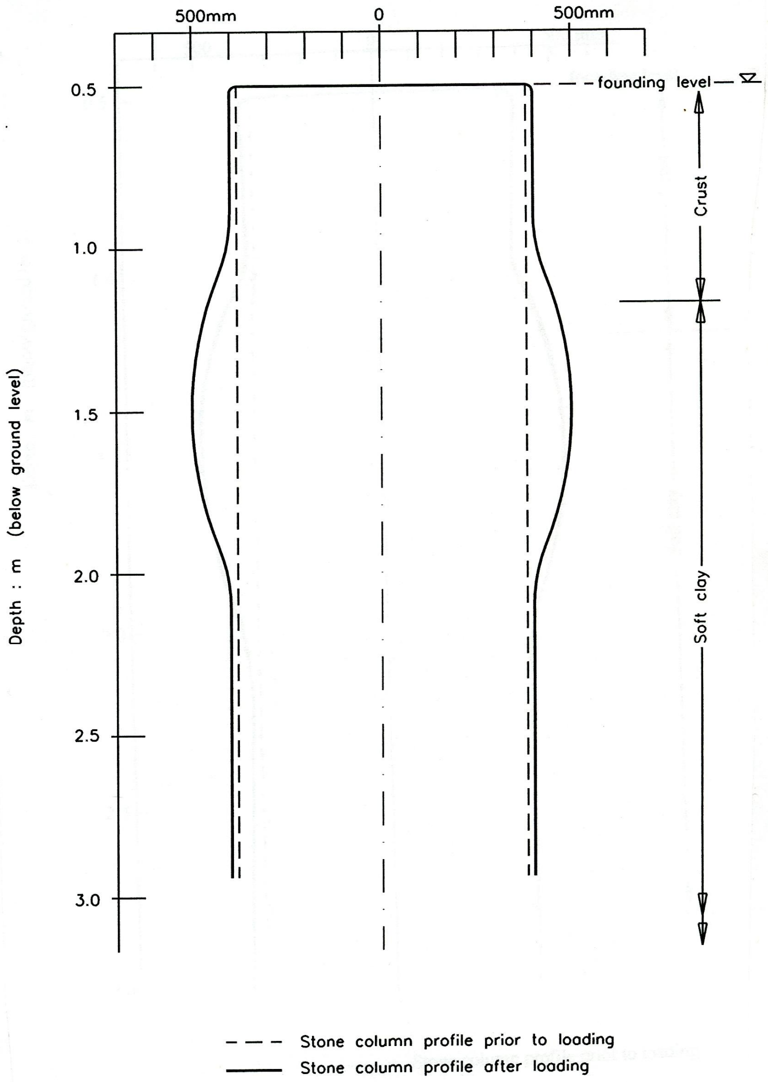


Figure 7.15: Settlement of the raft over stone column reinforced Bothkennar Clay (partial depth treatment) for the full loading duration, with settlement for the untreated raft (reported by Chown and Crilly, 2000; Serridge, 2001) superimposed.

Figure 7.16: (a) Investigation of deformation characteristics of stone column following completion of the Bothkennar field trials. The colour differentiation between the surface crust (brown) and underly ing soft Bothkennar Clay (dark grey-black) is clearly visible.



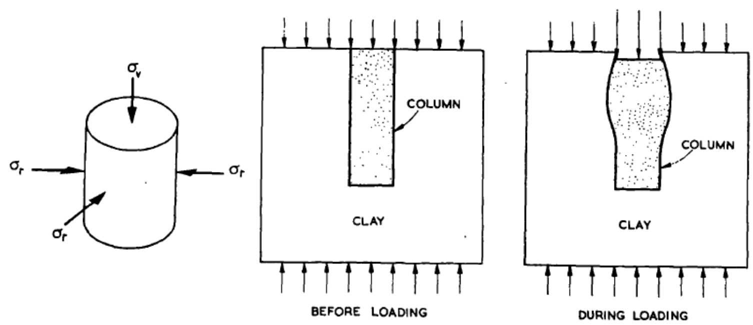


Figure 7.16: (b) Investigation of deformation characteristics of stone column following completion of the Bothkennar field trials.

Figure 7.17: (a) Investigation of undrained shear strength characteristics of the clay soil surrounding installed stone columns (using hand shear vane tester), upon completion of Bothkennar field trials and removal of footings.

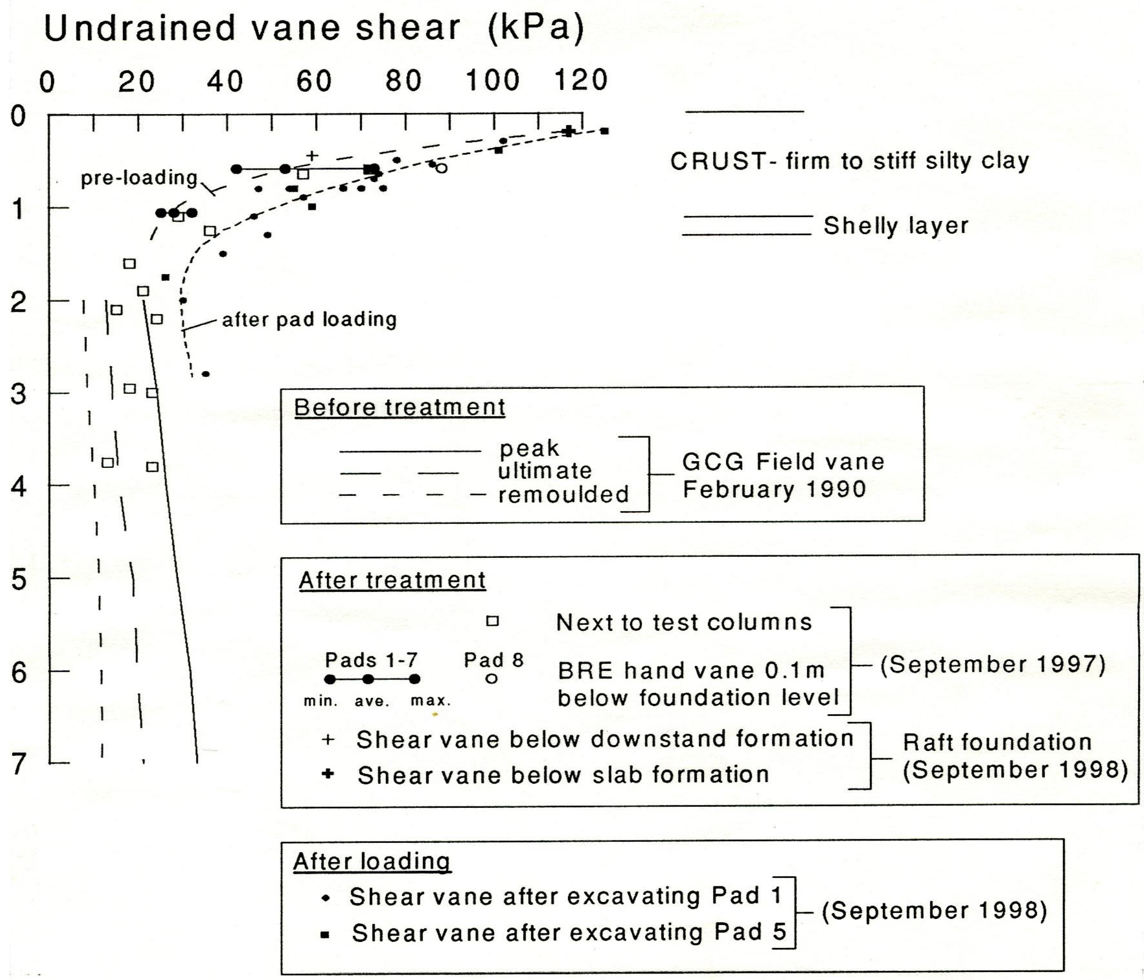


Figure 7.17: (b) Plot of undrained shear strength data for upper clay soil following completion of the Bothkennar field trials. Note: Pad = trial footing.

Chapter 8 Numerical modelling

8.1 Introduction

Optimisation of stone column design beneath narrow footings represents one of the more difficult aspects of stone column design in soft clay soils, with current design practice relying heavily on empirical methods for partial-depth treatment as analytical theory is less well developed in this area. There is significant potential for use of the finite element method (FEM) in an applied sense, as analytical approaches have many shortcomings and high quality field data (as exists for the Bothkennar soft clay research site), is scarce. As intimated in Chapter 2, publications relating to the application of the finite element method to model ground improved with stone columns mostly relate to wide area loading, using either a unit cell (i.e. Domingues et al., 2007) or 2-D axisymmetric, i.e. Elshazly et al., 2008, approximation. Some 3-D modelling of wide-area loading has also been carried out, i.e. Gäb et al., 2007, however, hardly any 3-D modelling of footings has been published, apart from some preliminary work on rigid square pad footings by Killeen and Mc Cabe (2010). Little if any numerical 3-D modelling of footings, notably narrow footings, has been published. In this chapter the Plaxis 3-D Foundation (Version 2.2) geotechnical software package has been used to attempt to model the key design variables investigated during the Bothkennar field trials, namely column spacing, column length and founding depth within a thin surface 'crust'.

The Hardening Soil (HS) model, which is an extension of the hyperbolic stress-strain model developed by Duncan and Chang (1970), and where there is a distinction between primary loading and unloading, was selected to simulate the behaviour of the weathered crust, Carse Clay and stone column aggregate in the Bothkennar field trials. The HS model can be used to account for anisotropy or small strain stiffness. This second order model can also be used to simulate the behaviour of sand and gravel as well as softer types of soil such as clays and silts.

8.2 Bothkennar soil model parameters

The clay and stone column material properties adopted for the numerical modelling draw on the suite of geotechnical data available for the Bothkennar site, as previously discussed (Chapter 4 and 5). For the purposes of the soil model the soil profile has been stratified as follows: Crust; Upper Carse Clay and Lower Carse Clay. A high critical state friction angle ($\varphi' = 34^\circ$), attributable to a high proportion of angular silt particles, discussed previously in Chapter 5, Allman & Atkinson (1992), is used for the Carse Clay, and a nominal cohesion value of 1 kPa is used for numerical stability. A slightly higher cohesion value of 3 kPa was used for the weathered crust layers. Nash et al. (1992a) report the variation of yield stress ratio which is equivalent to the overconsolidation ratio (OCR) measured in an oedometer, and *in situ* lateral earth pressure coefficient (K_o) with depth, suggesting that the stress state of the Carse Clay may have been influenced by erosion of material, a relative drop in sea level and fluctuating groundwater levels (equating to a 15 kPa drop in vertical effective stress). In choosing the friction angle of the stone backfill, reference was made to Mc Cabe et al. (2009) who measured settlement improvement data from the field to suggest that the conventionally used value of $\varphi' = 40^\circ$ may be conservative for columns in soft cohesive soils constructed using the dry bottom-feed system. Subject to adequate workmanship, the value of $\varphi' = 45^\circ$ shown in Table 8.1 should be readily achievable. Whilst 42.5˚ had been used for preliminary predictions of settlement for the field trials, using empirical methods, the above comments, combined with the fact that a sample of the stone column aggregate tested in large triaxial cell apparatus (Chapters 4 and 5) had yielded a friction angle in excess of 45˚, substantiated the use of a 45˚ friction angle for the aggregate in the Plaxis analysis. The angle of dilatancy (ψ) was calculated based on the relationship $\psi = \varphi' - 30^\circ$.

8.3 Field calibration of PLAXIS program

The well documented field trial described by Jardine et al. (1995) at Bothkennar for a pad footing founded at 0.75 m depth within the 1.5 m thick surface 'crust' and without the support of stone columns, and discussed in Chapter 7, was simulated using the Plaxis 3-D Foundation software program in order to substantiate the adoption of the geotechnical parameters detailed in Table 8.1. For the field trials described by Jardine et al. (1995) a 2.2 m square (0.8 m thick) pad footing had been loaded incrementally to a failure condition over a period of 3 days using kentledge blocks, with loading pauses overnight and whenever settlement rates exceeded 8 mm.hr. The Carse Clay was modelled as effectively undrained attributed to the short duration of the load test; concrete was modelled as a linear elastic material (Young's modulus E_{conc} = 30GPa; Poission's ratio υ_{conc} = 0.15). The load-settlement response of the footing recorded by Jardine et al (1995) was closely aligned with Plaxis 3-D prediction by Killeen and Mc Cabe (2010) (Figure 8.1). The fact that both curves are in relatively good agreement, both affirmed and gave confidence in the selection of the adopted soil profile and properties for both soil and stone column material, as did an independent verification of the curves by the author.

8.4 Finite element analyses (and modelling issues)

Within the constraints of the current research the stone columns have been 'wished-in-place', i.e. ground properties have not been modified to reflect changes induced by the installation of the columns as observed in Chapter 6. This will form the topic of future research. The interaction between the stone columns and the surrounding soil is simulated using elasto-plastic interface elements. Owing to the process of stone column construction, as described in Chapter 1, the stone column aggregate is tightly interlocked with the surrounding soil and it was assumed for the purposes of this research that a perfect bond (total adhesion) occurs along the interface between the soil and stone column elements. This approach follows that adopted by others, e.g. Guetif et al. (2007).

In generating the model for the field trial data, two issues had to be addressed:

1. Generation of an appropriate mesh to facilitate accurate capture of the stresses in the stone column reinforced soil and resultant settlements.

2. Establishing appropriate boundary conditions. The Plaxis 3-D software package has a convenient default setting to generate standard boundary conditions. A set

of general boundary conditions are imposed on the model geometry. In this research the displacements are assigned to zero in both the x and y orientation at the bottom of the model and only in the x orientation at the sides. Changing of the size of the model was undertaken to determine the influence of the boundary conditions on displacement magnitude and distribution. The width of the model was chosen to ensure the boundary conditions did not introduce any constraint to the model.

Details of the above refinements are given in Figures 8.2, 8.3 and 8.4.

8.5 Parametric study and discussion

Whilst the importance of bearing capacity has been demonstrated in the field trials at Bothkennar, settlement rather than bearing capacity tends to be more significant. Key variables influencing settlement, and addressed in the field trials, include column length column spacing, and founding depth within a thin surface crust. A preliminary parametric finite element study (investigation) with an advanced soil model (Plaxis 3-D) was therefore carried out to simulate the trial footings supported by stone columns in the field trials at Bothkennar. The loading was applied as a uniformly distributed load in two increments and according to the (average) loading scheme adopted in the actual field trials. The boundaries of the 3-D finite element mesh had to be refined as mentioned above (see Figure 8.2 and 8.3), in order to minimise the effects of model boundaries on the analysis. The height of the finite element model was selected as a maximum of 12 metres. The first 1.2 m corresponded to the 'crust' with the underlying remaining 10.8 metres corresponding to the soft normally consolidated Bothkennar Clay.

The various parametric combinations considered in this component of the research are labelled LC2-1 to LC6-1 inclusive in Table 8.2. The footing is founded 500 mm below ground level (with the exception of footing 6 (LC6-1) which is founded at 1.2 m, i.e. at the base of the 'crust'). A stone column diameter of 750 mm is adopted based upon direct measurement of stone column diameter in the field trials.

8.6 Discussion of results and limitations

The analysis (see outputs in Figures 8.2-8.14 inclusive) has shown the importance of considering boundary effects when setting upon the model and refining the mesh. The settlement outputs associated with the parametric investigation are summarised in Table 8.3. Whilst the Plaxis analysis has yielded comparable settlement to predictions using empirical methods (Priebe, 1995) for the field trials for shorter column lengths and wider column spacings, it is evident that the Plaxis analysis predicts significantly less post treatment settlement where columns are longer and more closely spaced. It is also interesting to note that LC6-1 (trial footing 6) associated with the greater founding depth yielded the least settlement in the Plaxis analysis output, which is likely to be a reflection on the increasing over-burden constraint with depth on the column, which will cause it to act as a stiffer element. The empirical analysis may have been less sensitive to this. There has not been opportunity to investigate parameters such as: (a) stone column stiffness (the value of E = 70 MPa suggested by Killeen and McCabe (2010) in Table 8.1 is higher than the value typically used in stone column design for soft clay soils which is closer to E = 50 MPa), (b) installation effects of vibro stone columns (and also column bulging effects under applied load), in the numerical modelling within the scope of the current research, which clearly was of significance in the field trials - in terms of performance, due in part to time constraints and in part to limitations of the current software version. Possible methods of building in an installation effect prior to the loading phases of the numerical analyses need to be developed. A large cavity expansion approach perhaps needs to be included in a numerical analyses if the process of stone column installation is to be captured at a fundamental level. It is apparent that the small strain or other limitations inherent in Plaxis 3-D FE analysis, restricts its use for modelling the installation process. Further development work needs to be carried out to provide Plaxis 3-D with the capability of modelling larger strains associated with the column installation, as observed in the field trials. Whilst attempts have been made in some quarters to model installation effects using temperature gradients to mimic the cavity expansion, these would appear to have been of limited value to date.

8.7 Conclusions

The following conclusions may be drawn, which are specific to a type (i) drained analysis for the ground profile modelled:

- Refinement of the mesh and boundary conditions are important when setting up the model to limit influence of boundary effects.

- The Plaxis analysis suggests significant load concentration onto the stiffer stone column elements and is more pronounced for shorter stone columns and for columns supporting a footing founded at the base of the surface 'crust'.

- The settlements obtained are comparable to the predictions made using empirical predictions in advance of the trials. The analyses were unable to directly model the field response to stone column installation, which has clearly had an impact on field performance.

- The Plaxis 3-D output suggest that settlement performance continues to improve to at least a column length to diameter ratio (L/d) = 10. Settlement performance also improves with a reduction in stone column spacing and increase in founding depth.

- The settlement reduction (n) values converge with depth and long stone columns are relatively insensitive to column spacing.

		Crust	Upper Carse clay	Lower Carse clay	Stone backfill
Depth	(m)	0.0–1.5	1.5–2.5	2.5–14.0	–
γ	(kN/m^3)	18.0	16.5	16.5	19.0
ϕ'	(°)	34	34	34	45
ψ	(°)	0	0	0	15
c′	(kPa)	3	1	1	1
OCR	(−)	1.0	1.0	1.5	–
POP	(kPa)	15	15	0	–
K_0	(−)	1.5	1.0	0.75	0.3
[1] E_{50}^{ref}	(kPa)	1068	506	231	70000
E_{ur}^{ref}	(kPa)	5382	3036	1164	210000
p_{ref}	(kPa)	13	20	30	100
m	(−)	1.0	1.0	1.0	0.3

[1] E_{50}^{ref} assumed equal to E_{oed}^{ref} based upon Elshazly et al., (2008)

Table 8.1: Parameters used to model Bothkennar field trials in Plaxis 3-D (modified from Killeen and Mc Cabe, 2010)

Load Case	**Parameter investigated**
LC2-1	2.0 m spacing and 5.7 m column length below footing.
LC3-1	1.5 m spacing and 3.7 m column length below footing.
LC4-1	1.5 m spacing and 5.7 m column length below footing.
LC5-1	1.5 m spacing and 7.7 m column length below footing.
LC6-1	Founding depth at base of the crust (1.2 m) and 5.7 m column length below footing.

Note: LC2-1 and LC4-1 investigate column spacing and LC3-1; LC4-1 and LC5-1 investigate variation in column length.

Table 8.2: Parametric study details –Plaxis 3-D Analysis of Bothkennar field trials.

Load Case	**Stone column spacing and length below founding depth**	**1st load increment**	**Plaxis 3-D settlement ouptut**	**2nd load increment**	**Plaxis 3-D settlement ouptut**
LC2-1	2.0 m spacing and 5.7 m column length below footing (trial footing 2)	33 kPa	12.51 mm (10.55mm)	70 kPa	24.57 mm (21.78 mm)
LC3-1	1.5 m spacing and 3.7 m column length below footing (trial footing 3).	33 kPa	12.33 mm (10.92 mm)	70 kPa	21.33 mm (22.51 mm)
LC4-1	1.5 m spacing and 5.7 m column length below footing (trial footing 1and 4).	33 kPa	11.29 mm (11.00 mm)	70 kPa	18.71 mm (23.44 mm)
LC5-1	1.5 m spacing and 7.7 m column length (trial footing 5).	33 kPa	9.76 mm (10.01 mm)	70 kPa	16.48 mm (21.45 mm)
LC6-1	Founding depth at base of the crust and 5.7 m column length below footing (trial footing 6).	33 kPa	6.50 mm (11.42 mm)	70 kPa	13.14 mm (22.81 mm)

Note: Founding depth = 0.5 m for LC2-1; LC3-1; LC4-1; LC5-1 and 1.2 m for LC6-1.
The settlement figures in brackets represent the average values predicted using the Priebe (1995) empirical approach, undertaken prior to commencement of field trials (see Chapter 4)

Table 8.3: Settlement output from Plaxis 3D for the two main load increments (average) applied during the Bothkennar field trials.

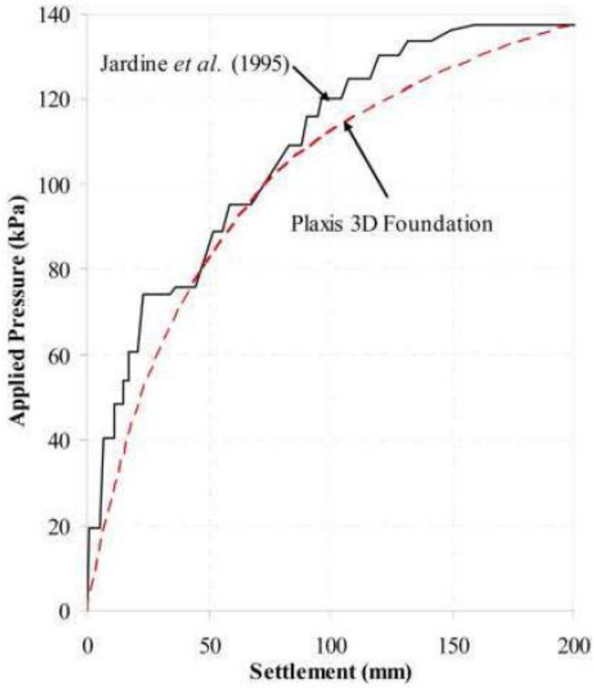


Figure 8.1: Calibration of Plaxis 3-D model against published Bothkennar field data (Jardine et al., 1995), after Killeen and Mc Cabe, 2010.

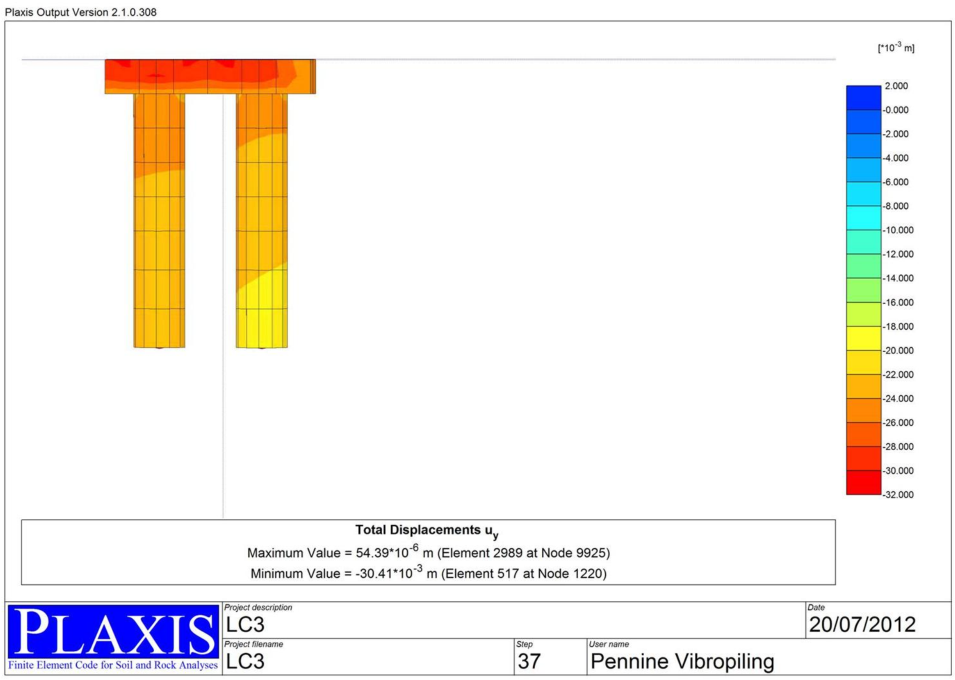


Figure 8.2: Influence of boundary effects on settlement output.

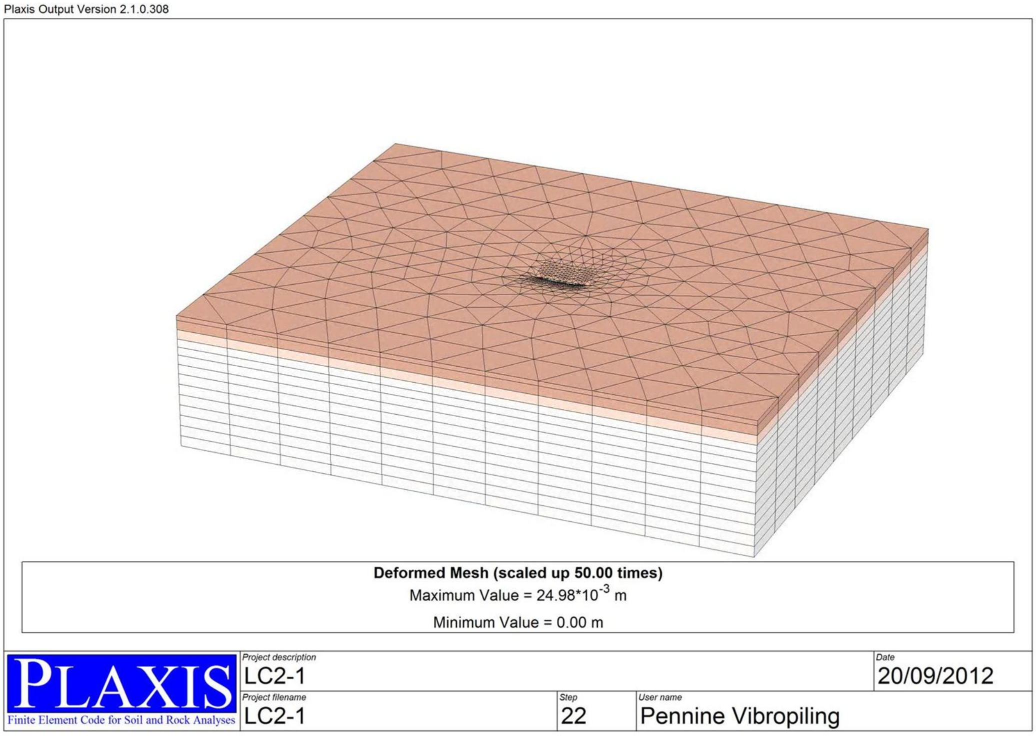


Figure 8.3: Load Case LC2-1: Refinement of mesh for boundary effects.

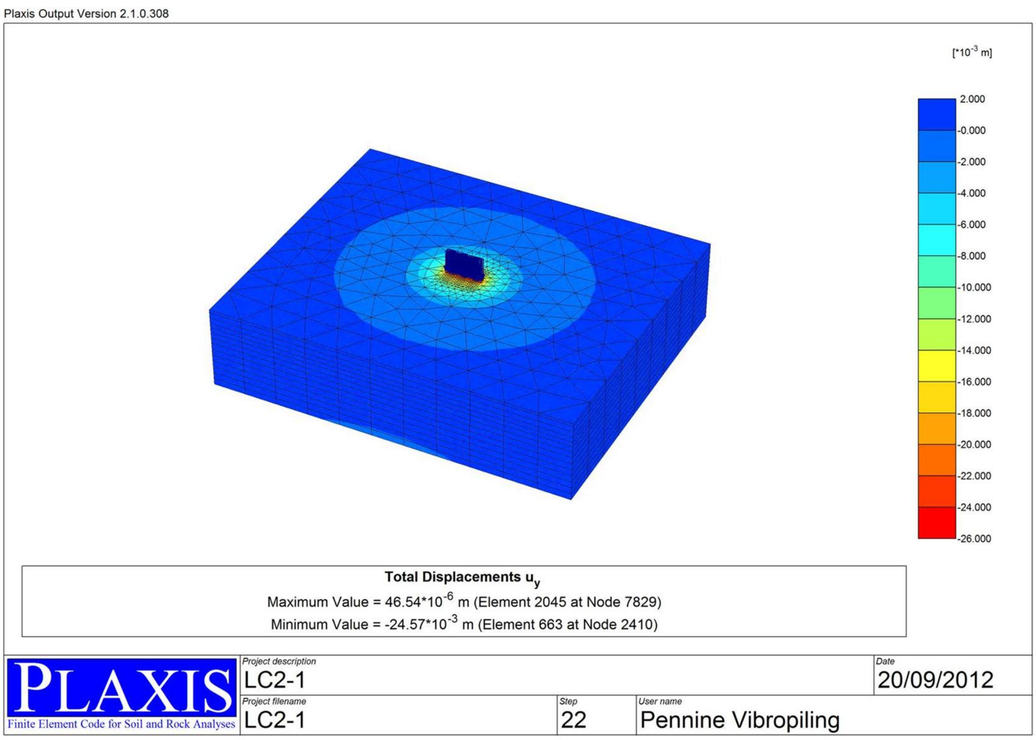


Figure 8.4: Load Case LC2-1: Total displacement distribution with refined boundary.

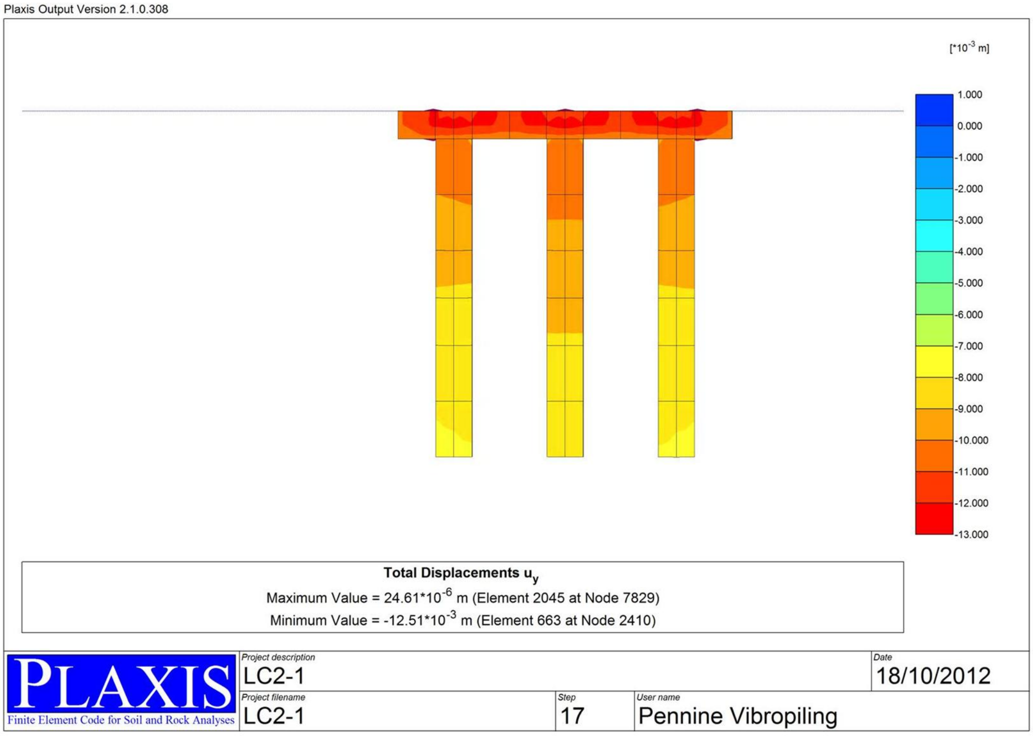


Figure 8.5: Load Case LC2-1: Total settlement under the 1st load increment (33 kPa)

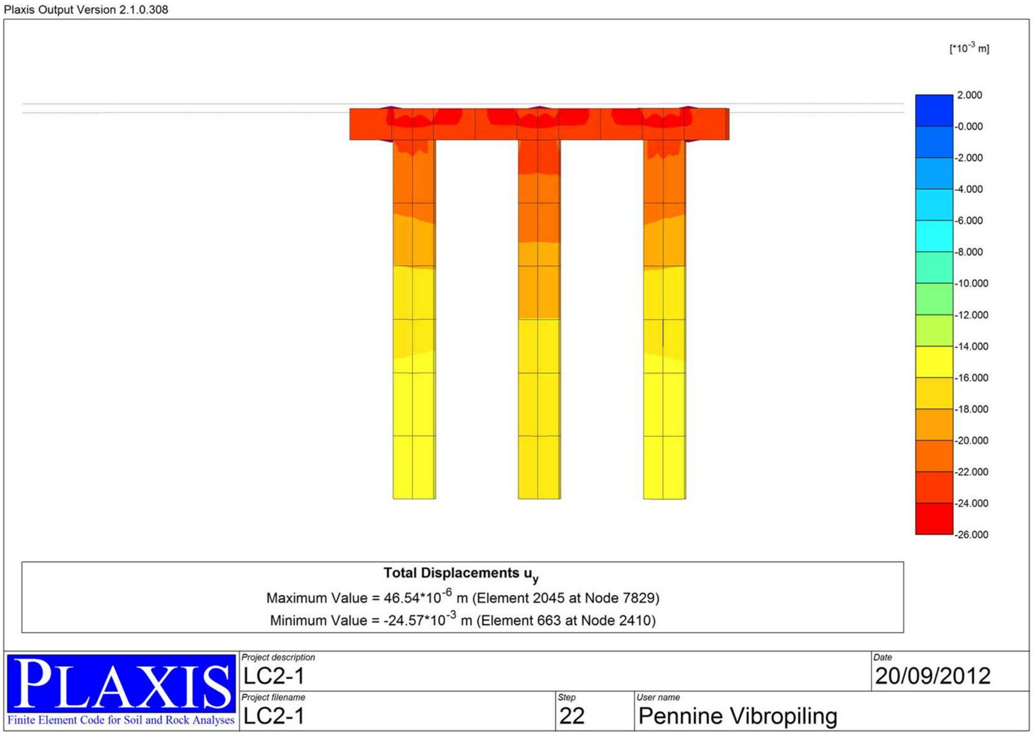


Figure 8.6: Load Case LC2-1: Total settlement under the 2nd load increment (70 kPa)

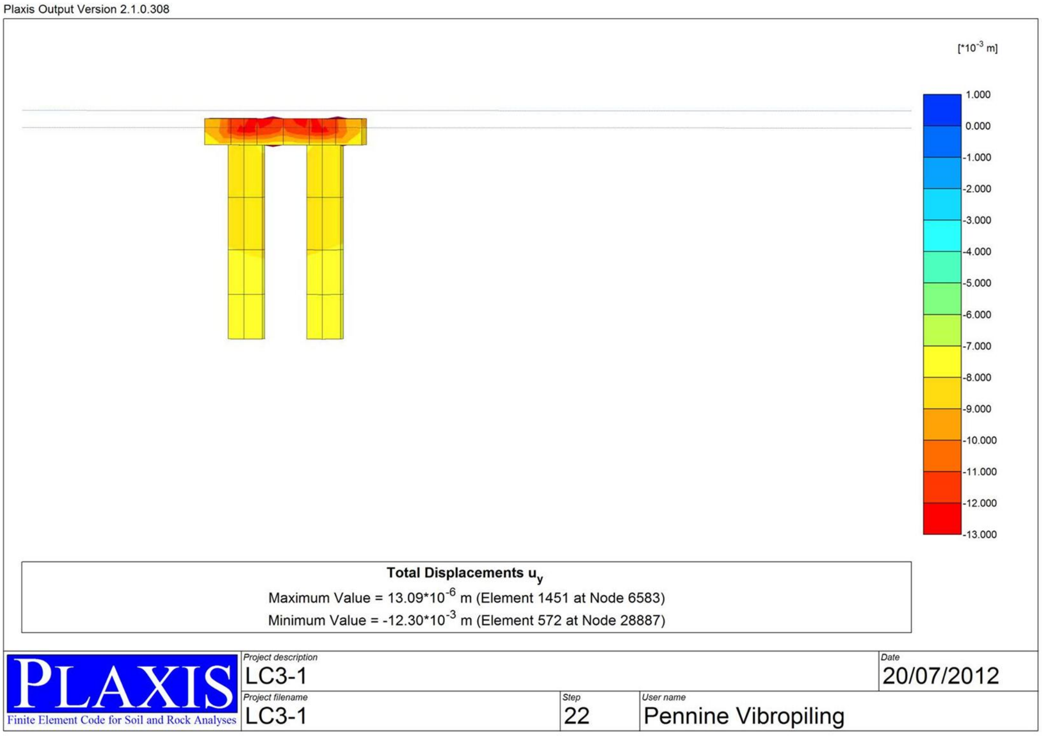


Figure 8.7: Load Case LC3-1: Total settlement under the 1st load increment (33 kPa)

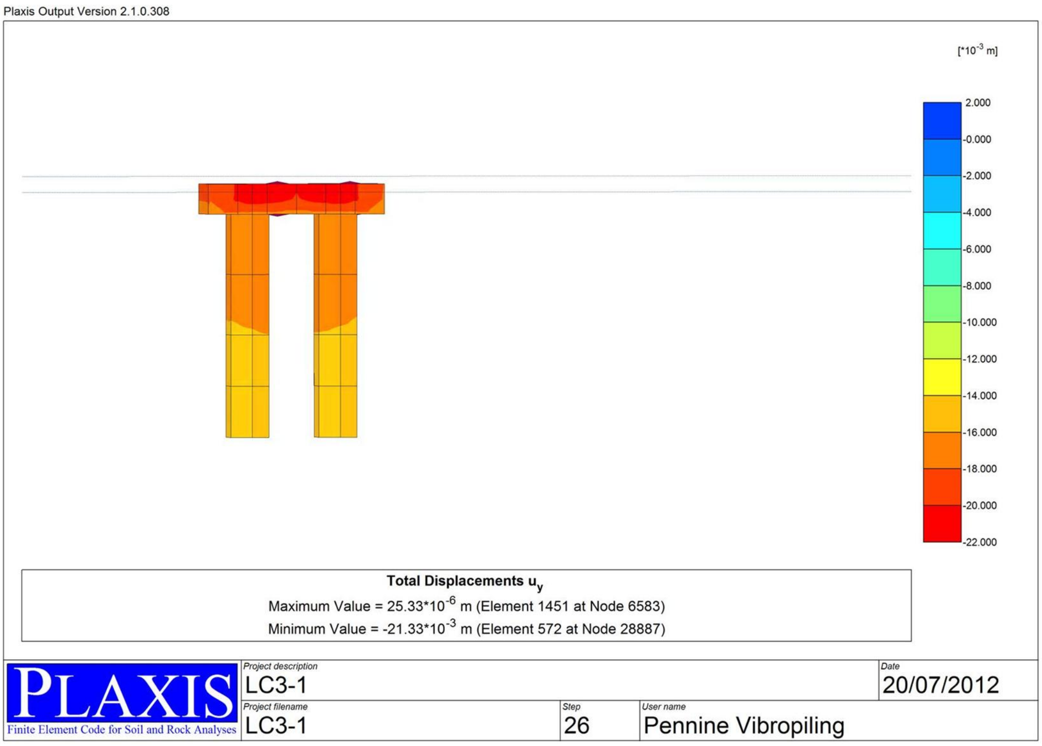


Figure 8.8: Load case LC3-1: Total settlement under the 2nd load increment (70 KPa)

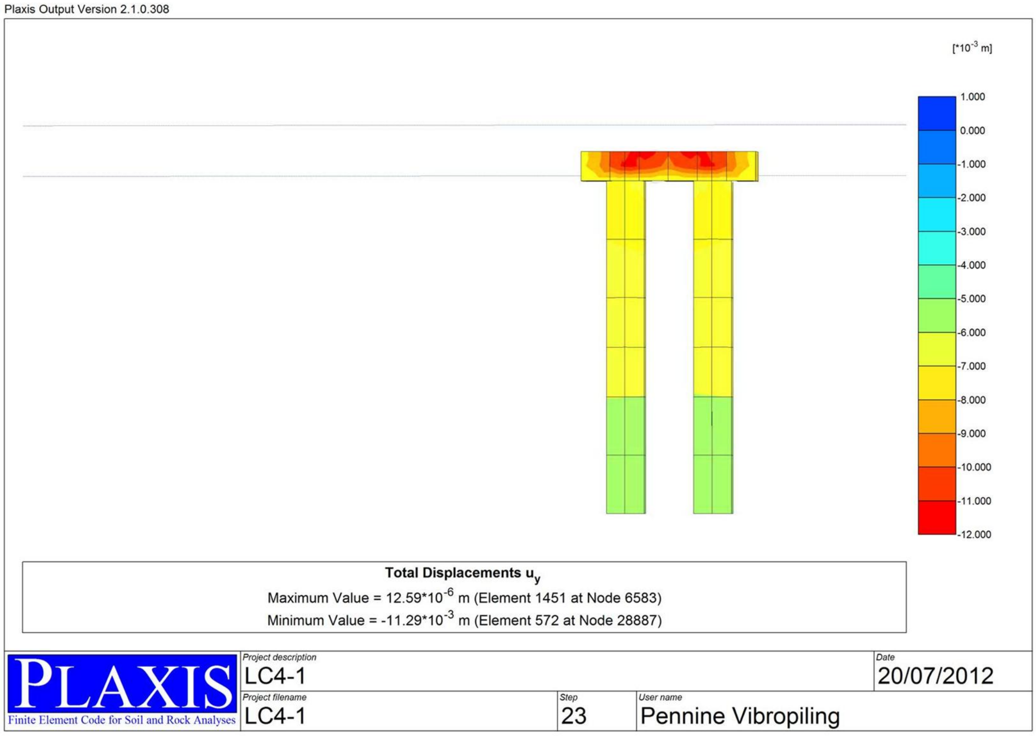


Figure 8.9: Load Case LC4-1 : Total settlement under the 1st load increment (33 kPa)

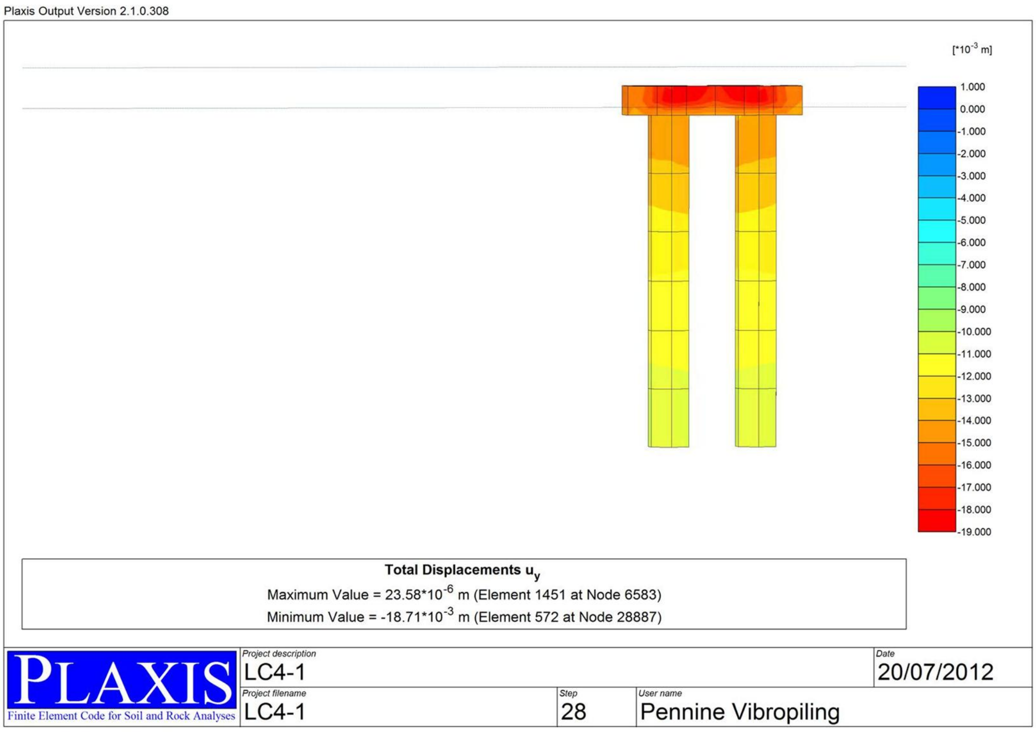


Figure 8.10: Load Case LC4-1 : Total settlement under the 2nd load increment (70 kPa)

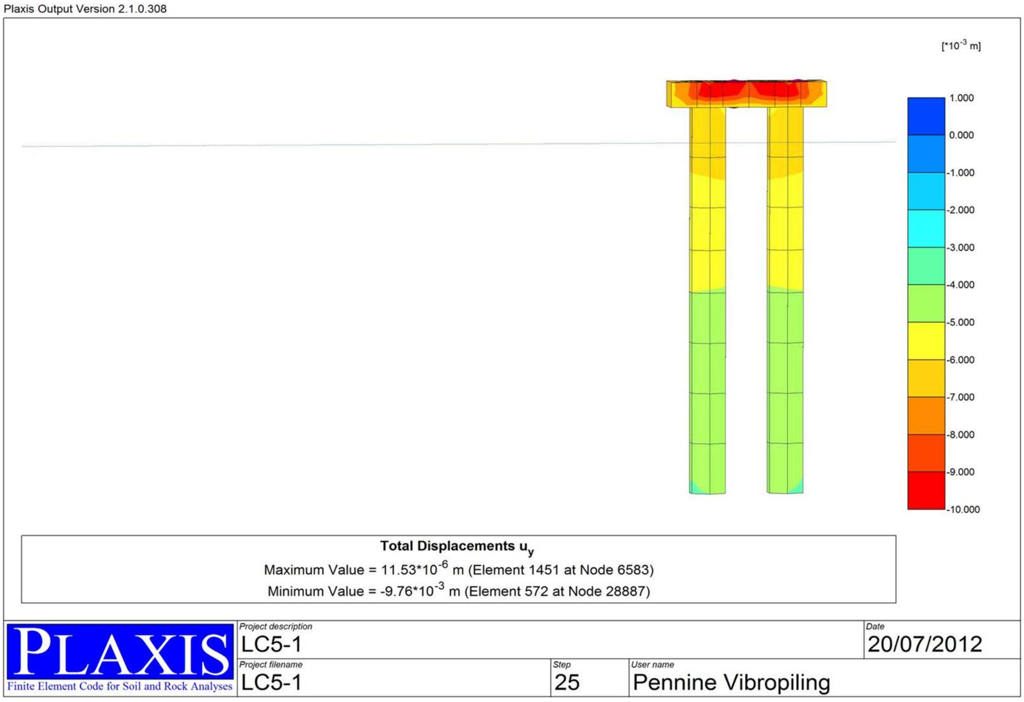


Figure 8.11: Load Case LC5-1 : Total settlement under the 1st load increment (33 kPa)

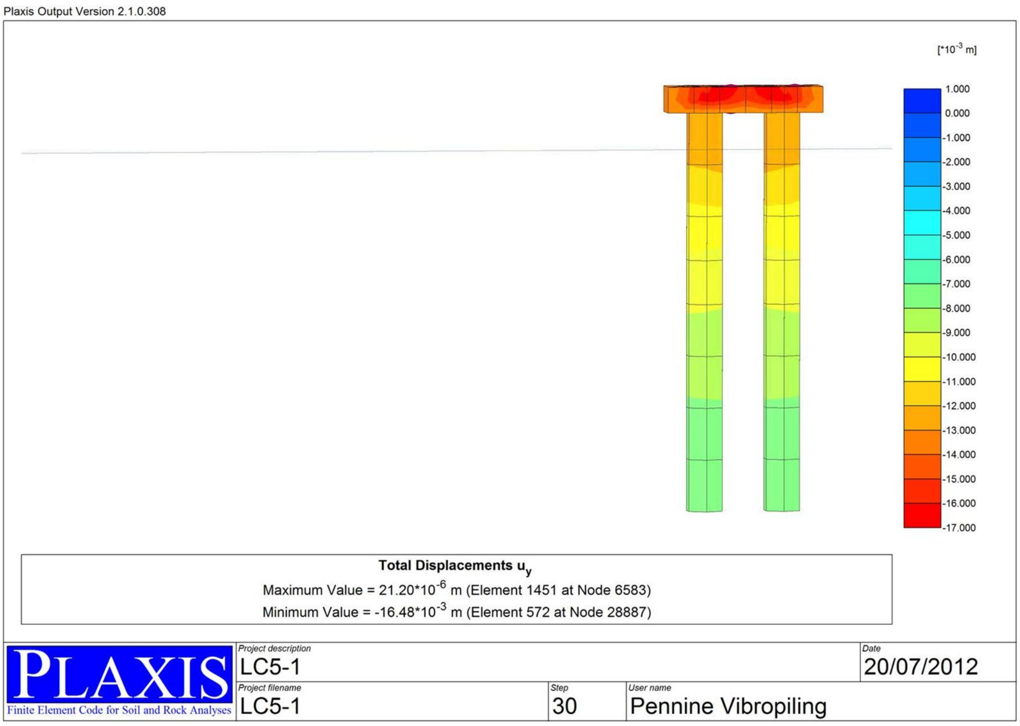


Figure 8.12: Load Case LC5-1: Total settlement under the 2nd load increment (70 kPa)

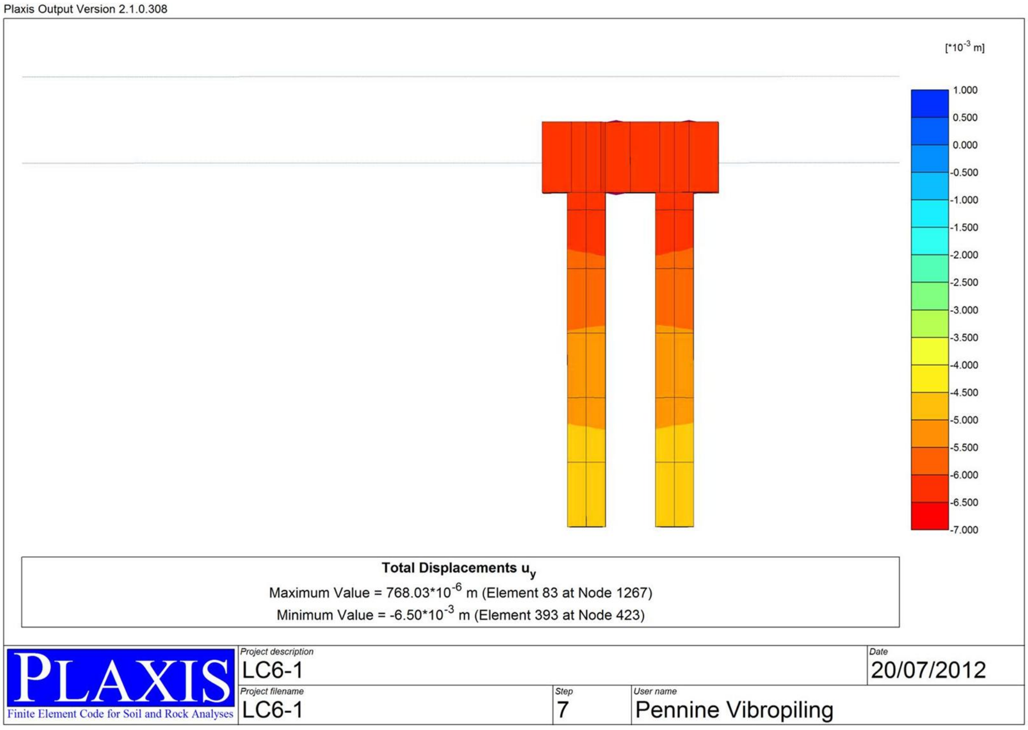


Figure 8.13: Load Case LC6-1 : Total settlement under the 1st load increment (33 kPa)

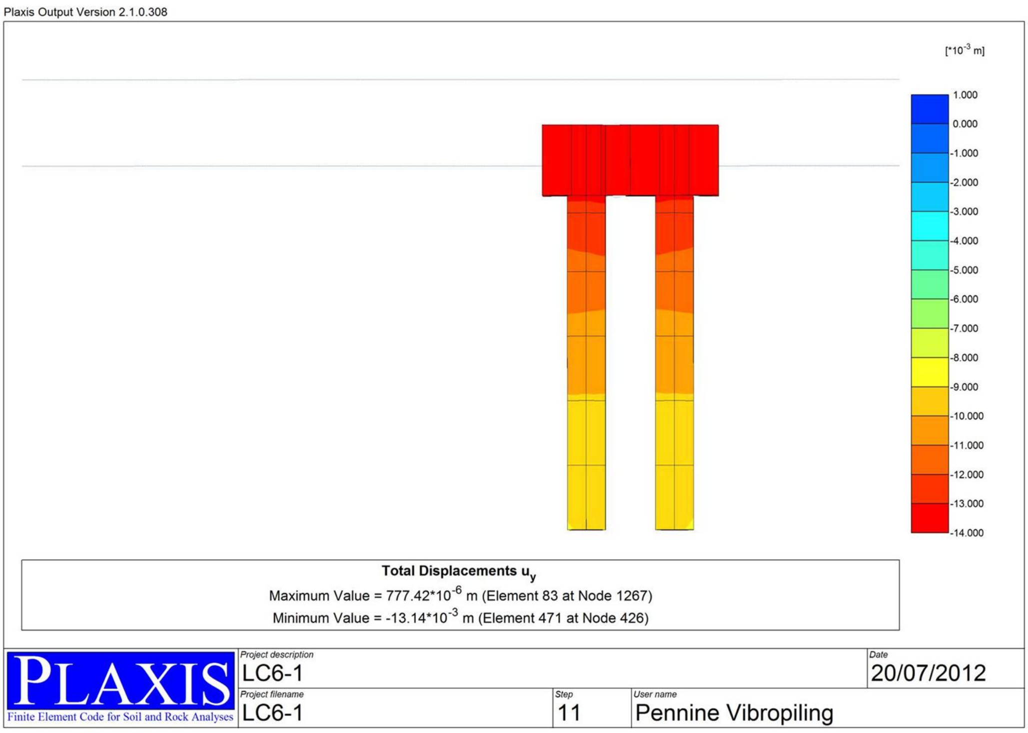


Figure 8.14: Load Case LC6-1: Total settlement under the 2nd load increment (70 kPa)

Chapter 9 Conclusions and recommendations for further research

9.1 Conclusions

It is evident from a review of previous investigation of stone column behaviour (see Chapter 2) that vibro stone column design in soft clay soils is largely empirical. Chapter 2 demonstrates that there are several methods for the determination of the bearing capacity and load-settlement behaviour of stone column reinforced foundations, ranging from experience-based empirical estimates to sophisticated finite element analyses, which have been proposed. Whilst elastic methods of settlement analysis have their shortcomings the completely elasto-plastic methods of analysis are not sufficiently advanced to date to be of practical use, particularly within the Ground Improvement industry, although this is being addressed to some degree by some of the modern finite element software packages. The method of analysis which is most commonly used in design is the Priebe (1995) approach. This approach is empirical and a compromise between elasticity and elasto plasticity; the column is considered to be in a state of failure whilst the surrounding soil behaves elastically. The method employs a settlement reduction factor which means that the accuracy of the settlement prediction for the treated ground will be reliant upon the accuracy of the predicted settlement for the untreated ground, which in turn is dependent upon satisfactory site geotechnical characterisation.

Review of some of the limited number of case histories relating to poor vibro stone column performance (Chapter 3) suggests that there are four principal factors associated with unsatisfactory performance:

- Those relating to ground risk, including inadequate site and ground investigation information and failure to produce a (geotechnical) ground model.

- Those relating to a lack of understanding of vibro stone column techniques and their application, together with selection of appropriate validation testing and interpretation of results.

- Failure to understand the (settlement) implications of re-grading of site levels, notably placement of upfill on deep soft soil deposits, particularly where only partial depth vibro stone column treatment is proposed.

- Poor quality control and operator experience, together with a lack of monitoring of design parameters in the field.

This research, as defined in the aims and objectives in Chapter 1, has primarily investigated both ground response to installation of partial depth dry bottom-feed vibro stone columns in a deep soft sensitive clay profile, together with investigation of the effect (impact) of stone column spacing and length, foundation shape and also founding depth within a thin surface crust on the performance of shallow, narrow footings subsequently constructed over the installed stone column reinforced clay soil, and subject to incremental loading to replicate loads similar to those applied by low-rise, lightweight structures. This has been compared with the performance of similar foundations on untreated ground.

The instrumentation installed prior to commencement of the vibro stone column installation for the (Bothkennar) field trials unusually generally survived the column installation process and the general arduous conditions of the trials, with the exception of earth pressure cells located below the toe of some of the trial stone columns. In addition to giving reasonably consistent results, providing confidence in their reliability, the suite of installed instrumentation has provided valuable (and in some cases unique) insight and data in relation to vibro stone column installation in soft clay soil, including ground responses. Field instrumentation measurements during the field trials have shown the effects of column installation, such as increase in pore pressures and horizontal stresses, and temporary remoulding of the surrounding soil caused by vibroflot insertion and soil-structure (foundation) interaction when trial footings were subsequently constructed and incrementally loaded over the stone column reinforced soft Bothkennar Clay soil.

The stress measurements recorded by the installed field instrumentation during foundation loading demonstrate the complex nature of the stone column reinforced soil-foundation system, with simultaneous and interdependent changes in pore pressures, soil stress ratios and resulting stiffness of both soil and columns. The soil-structure interaction is highly dependent on design criteria such as column spacing and diameter (and therefore area ratio (A_r)), together with column length. Other influencing factors are stress concentration ratio (S_r) between column and soil, the shear strength of the soil and the presence of a thin surface 'crust', all of which have been shown to have a significant impact on trial footing performance at Bothkennar. Recorded stress ratios during the field trials range from 1.6 to 4.0 which accords well with published values for vibro stone columns in soft clay over a similar stress range.

Despite the complexity of the load transfer mechanism and the need for simplified assumptions in any analysis, the results obtained by the more common design approaches for stone columns in (soft) clays are often in reasonable agreement with each other and with field measurements. The application of realistic soil parameters, especially in soft clay soils, is a matter of judgement and experience, particularly where site and ground investigation information is limited. The piezocone (CPTU) is gaining increasing application for geotechnical parameter acquistion. A significant factor, as implied above, and probably the most imponderable is the installed stone column diameter in a particular soil: again experience is required to estimate this. It has been demonstrated during the field trials that indirect methods of assessing stone column diameter can give comparable results to direct measurements of column diameter, providing the indirect methods are suitably robust, requiring appropriate field calibration and drawing on experience from other sites with similar ground conditions where diameters have been confirmed by direct methods.

It is evident from the field trials at Bothkennar that a high level of quality control is required with regard to stone column installation (construction) in soft clay soils to avoid potential problems. Supervision of the stone column installation, particularly in soft sensitive clay soils, is very important as the quality of column construction relies heavily on the installation rig operator and their experience. It is also clear from the field trials that whilst stone columns can be satisfactorily constructed using the dry bottom-feed method within the soft sensitive Bothkennar Clay, the vibroflot should not remain in the ground for longer than is necessary to achieve an acceptable level of workmanship with regard to stone column construction. Air jetting should be kept to the minimum necessary to mitigate soil suction effects on the vibroflot whilst also limiting soil disturbance, particularly in sensitive clay soils.

It was also evident that some of the benefits of the stone columns may be offset by the extent of disturbance caused by imparting excessive energy to the soft sensitive Bothkennar Clay, particularly in the context of the practice of ramming stone aggregate at the toe of the partial depth column(s) to form an enlarged end-bulb or base from which to construct the remainder of the stone column. For this reason it is concluded and recommended that partial depth stone columns in soft (sensitive) cohesive (clay/silt) deposits should ideally be approximately uniform in diameter over their length. Achievement of this is best facilitated by constructing to a stone aggregate consumption rather than a rig compaction energy criterion.

Vibro stone columns reduce ground movement but do not eliminate it. Whilst the vibro stone columns did not perform as well as predicted in the context of recorded foundation settlement, it is clear that the stone columns reinforced the weak soil, providing a significantly increased factor of safety against bearing failure. Moreover, a significant part of the recorded settlement during each load increment was observed within the first few days of load application, so arguably would be built out during construction. However, where structures will be sensitive to very small ground movements, it is clear that the suitability of vibro stone columns must be carefully appraised.

The field trial results confirm some of the Hughes and Withers (1974) hypotheses, notably with regard to the critical depth (depth at which bulging occurs). Field instrumentation also demonstrated that significant stress was transferred below the base (toe) of a stone column which was installed shorter than the minimum design length proposed by Hughes and Withers (1974), but was not transferred below the base of stone columns with length exceeding the minimum design length predicted by Hughes and Withers (1974), based upon adoption of pre-stone column installation peak undrained shear strength parameters. The recorded stresses in the field trials imply that the dominant failure mechanism would be 'punching' for the short columns. In contrast, bulging took place at shallow depth in longer columns and only at high footing loads did transfer of load to the base of the columns take place (partly aided by the presence of the crust). It has been shown that shorter columns, i.e. in excess of 3.7 m, give adequate capacity, but longer columns may be needed to control settlements. Nevertheless, the observations made permit optimisation of stone column design and performance and in turn costs. However, in soft sensitive clays any remoulding of the soil will potentially increase the minimum column length according to Hughes and Withers (1974), if remoulded shear strengths have to be used in design.

When undisturbed, i.e. in the absence of stone columns, the relatively thin surface crust (<1.5 m) at Bothkennar made a significant contribution to reducing settlement at low applied bearing pressures beneath footings (< 70-75 kN/m^2). However, the founding depth is critical and a small increase in founding depth (in the absence of stone columns), from 0.5 m (untreated trial footing 8 for current Bothkennar field trials) to 0.75 m (in an earlier trial at Bothkennar on untreated ground, described by Jardine et al., 1995), resulted in unacceptably large settlements and the onset of bearing failure. The practical influence of even a relatively thin surface crust over a deep layer of soft sensitive clay in the presence of partial depth vibro stone columns has been demonstrated.

Preliminary attempts at numerical modelling of the field trial data using the Plaxis 3-D finite element software package (with the stone column elements 'wished in-place' and adoption of the Hardening Soil (HS) model approach), has largely replicated the pre-trial predictions using the Priebe (1995) empirical approach for the differing trial column arrangements, but requires further development and research to model installation effects and potential impacts on settlement performance, in respect of more accurately modelling the field data. During the analysis refinement of the mesh was deemed necessary to mitigate the impacts of boundary conditions or effects.

9.2 Further research

Whilst the current research is considered to have made an important contribution to the greater understanding of the installation and subsequent performance of partial depth dry bottom-feed stone columns beneath narrow, shallow footings in a deep soft sensitive clay profile, it would be beneficial to further develop the field trials carried out at

Bothkennar as part of the current research, by assessing stone column installation and performance without construction of the 'end bulb' and also (whilst recognising that environmental constraints are limiting its use on some sites), consider column installation by the (vibro-replacement) wet top-feed technique, the latter of which researchers e.g. Greenwood (1991), have argued is technically more appropriate for sensitive soft clay soils, such as at Bothkennar and which may still be a relevant technique to achieve best results on some soft clay sites, dependent upon site specific circumstances. The use of geotextile encased granular columns appears to have received limited if any attention in the UK. It is considered that the system warrants further research and investigation through appropriate instrumented field trials in ultra soft and organic soils, where historically there have been concerns regarding inadequate lateral support for granular (stone) columns and potential for excessive bulging under applied foundation loads and in turn excessive settlement.

The beneficial effects of the surface crust at Bothkennar has been highlighted from the field trial data. It is considered that these effects could be further investigated in the context of exploitation of surface crusts, associated with normally consolidated clay profiles, as a 'natural' load transfer platform (LTP).

The extent to which stone columns can reduce secondary consolidation settlement (creep), seems to be relatively poorly understood and also warrants further research and development.

Further research and development is required with regard to automated monitoring systems, to monitor stone column installation parameters in real time, particularly stone aggregate consumption, together with variable frequency vibroflots which could be 'tuned' to individual soil types, notably for application in soft (sensitive) clay soils, where use of the dry bottom-feed technique may only be permitted, in order to mitigate excessive soil disturbance and remoulding and optimise soil response and performance.

Research and development of a quick, simple, cheap and reliable means of measuring the beneficial effect (at an early stage), in terms of enhancement of relative stiffness of clay in stone column reinforced soft clay soils, employing techniques such as continuous surface wave (CSW) testing for measurement of improvement in soil 'stiffness' is required. The current practice of plate load testing of vibro stone columns in soft clay soils is of limited use, generally only confirming level of workmanship with regard to stone column construction. Larger scale zone load tests used to assess the composite effect of the stone column-soil system, take time to construct and monitor and can be cost prohibitive. Time constraints may also dictate that such tests are of short duration, so that creep effects may not be fully realised. However, the importance of stone column diameter in the field cannot be underestimated and its close monitoring can be more significant than any load testing.

Whilst some preliminary work has been carried out in this research trial it is considered that the data base of information generated during the field trials at Bothkennar, which have formed a significant part of this research, could be used to verify more advanced constitutive modelling. In particular, detailed numerical understanding can only come from fully three-dimensional modelling with properly developed constitutive models for the stone column-soil composite. Column installation effects need to be more accurately modelled.

Development of a 'carbon-calculator' tool is required for the vibro stone column ground improvement technique in order to understand more fully its green credentials and sustainability advantages where compared with other ground improvement and deep foundation techniques. The author is currently involved with the European Federation of Foundation Contractors (EFFC) and Deep Foundations Institute (DFI) Europe in the development of such a tool which can be applied to vibro ground improvement techniques throughout Europe, to address this.

References

Aboshi, H., Ichimoto, E., Enoki, M., Harada, K. (1979). The Compozer – a method to improve the characteristics of soft clays by inclusion of large diameter sand columns. In *Proceedings of International Conference on Soil Reinforcement: Reinforced Earth and other Techniques,* Paris Vol. 1, pp. 211-216.

Aboshi, H., and Suematsu, N. (1985). The state-of-the-art on sand compaction pile method. In *Proceedings of 3rd NTI International Geotechnical Seminar,* Singapore, pp. 1-12.

Alexiew, D., Horgan, G.J, and Brokemper, D. (2008). Load capacity and geotextile selection. In *Proceedings of the BGA International Conference on Foundations,* Dundee, Scotland, 24-27 June 2008, IHS BRE Press, pp. 81-90.

Allman, M.A. and Atkinson, J.H. (1992). Mechanical properties of reconstituted Bothkennar soil. *Géotechnique* 42(2), pp. 289-301.

Almeida, M.S.S. (1984). Stage construction embankments on soft clays. *PhD thesis, Cambridge University.*

Ambily, A.P. and Gandhi, S.R. (2007). Behaviour of stone columns based on experimental and FEM analysis. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE* 133(4) : pp. 405-415.

Bachus, R.C. and Barksdale, R.D. (1984). Vertical and lateral behaviour of model stone columns. *Proceedings of the International Conference on In Situ Soil and Rock Reinforcement,* Paris, France, pp. 99-104.

Bachus, R.C. and Barksdale, R.D. (1989). Design methodology for foundations on stone columns. Vertical and horizontal deformations of foundations and embankments, *In Proceedings of Settlement' 1994, Texas, ASCE Geotechnical Special Publication No. 40,* pp. 244-257.

Balaam, N.P., Poulos, H.G. and Brown, P.T. (1977). Settlement analysis of soft clays reinforced with granular piles. *Proceedings of the fifth South East Asian Conference on Soil Engineering,* Bangkok, Thailand, pp. 81-92.

Balaam,N.P.(1978). Load settlement behaviour of granular piles. *PhD Thesis, University of Sydney.*

Balaam, N.P. and Booker, J.R. (1981). Analysis of rigid rafts supported by granular piles. *International Journal of Numerical and Analytical Methods in Geomechanics,* Vol.5, No. 4, pp. 379-403.

Balaam, N.P. and Poulos, H.G. (1983). The behaviour of foundations supported by clay stabilised by stone columns. *Proceedings of the Eighth European Conference on Soil Mechanics and Foundation Engineering,* Helsinki, Vol. 1, pp. 199-204. Balkema, Rotterdam, Netherlands, Balaam, N.P. and Booker, J.R. (1985). Effect of stone column yield on settlement of rigid foundations in stabilized clay. *Int. J. for Numerical and Analytical Methods in Geomechanics,* Vol. 9, No. 4, pp. 331-351.

Barksdale, R.D. and Bachus, R.C. (1983). Design and construction of stone columns, *Volume 1. Report No. FHWA/RD-83/026, Federal Highway Administration*, National Technical Informatiom Service, Springfield, Virginia, USA.

Barksdale, R.D. and Goughnour, R.R. (1984). Settlement performance of stone columns in the U.S. *Proceedings of the International Conference on In Situ Soil and Rock Reinforcement,* Paris, France, pp, 105-110.

Barras, B.F. and Paul, M.A. (2000). Post-reclamation changes in estuarine mudflat sediments at Bothkennar, Grangemouth, Scotland. In *Pye, K. and Allen, J.R.C. (eds) Coastal and Estuarine environments: sedimentology, geomorphology and geoarchaeology.* Geological Society of London, Special Publication:175, pp. 187-199.

Bauer Technical Brochure (2001).

Baumann V. and Bauer G.E.A. (1974). The performance of foundations on various soils stabilized by the vibro-compaction method. *Canadian Geotechnical Journal,* 11. No.4, pp. 509-530.

Bell, A.L. (2004). The development and importance of construction technique in deep vibratory ground improvement. In *Ground and Soil Improvement* (Raison, C.A. (ed)). Thomas Telford, London, pp. 103-111.

Bergado, D.T. and Lam, F.L. (1987). Full scale load test of granular piles with different densities and different proportions of gravel and sand on soft Bangkok Clay, *Soils and Foundations Journal,* Vol. 27, No. 1 pp. 86-93.

Bevan, D. and Johnson, D. (1989). Use of geogrid to enable treatment of peaty soils by vibrodisplacement.*Ground Engineering* 22 (3) March 1989, pp. 21-24.

Biot, M.A. (1941). General theory of three-dimensional consolidation. *Journal of Applied Physics,* Vol.12, No.2, pp. 155-164.

Bishop. A.W. (1976). Discussion: Ground treatment by Deep Compaction, Institution of Civil Engineers (ICE), London. pp. 138-140.

Bjerrum, L. (1967). Engineering geology of Norwegian consolidated marine clays as related to the settlement of buildings. Géotechnique, Vol. No. 2, pp. 83-118.

Bjerrum, L. (1973). Problems of soil mechanics and construction on soft clays and structurally unstable soils (collapsible, exposure and others). In *Proceedings of. 8th International. Conference on Soil Mechanics and Foundation Engineering,* Moscow, Vol 3, pp. 111-159.

Black, J.A., Sivakumar, V., Madhav, M.R. and Mc Cabe, B.A. (2006). An improved experimental test set-up to study the performance of granular columns. *ASTM Geotechnical Testing Journal*, 29 (3), pp. 193-199.

Black, J.A., Sivakumar, V. and McKinley, J.D. (2007a). Performance of clay samples reinforced with vertical granular columns. *Canadian Geotechnical Journal*, 44 (1), January, pp. 89-95.

Black, J.A., Sivakumar, V., Madhav, M.R. and Hamill, G.A. (2007b). Reinforced stone columns in weak deposits: Laboratory model study. *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 133 (9), September, pp.1154-1161.

Black, J.A., Sivakumar, V. and Bell. A. (2010). Performance and observations of model stone column foundations. In *7th International conference on physical modelling in Geotechnics,* Zurich, pp. 1315-1320.

Bouassida M., Ellouze, S. and Hazzar, L. (2008). Investigating Priebe's method for settlement investigation of foundation resting on soil reinforced with stone columns. In *Proceedings of the 2nd International workshop on the geotechnics of soft soils*, Glasgow, 2008, pp. 321-326.

Boussinesq, J. (1885) .*Application des potentials a l´e´tude de l´e´equilibre et de movement des solids elastiques.* Gauthier-Villars. Paris.

Brauns, J. (1978). Initial bearing capacity of stone columns and sand piles. In *Proceedings of the symposium on soil reinforcing and stabilising techniques,* Sydney, pp. 477-496.

Brinch Hansen, J. (1970). *A revised and extended formula for bearing capacity*. Bulletin 28, pp. 5-11, Copenhagen: Danish Geotechnical Institute.

British Standards Institution BS 812 (1990). *Methods for Sampling and Testing of Mineral Aggregates and Fillers- Part 110: Method for Determination of Aggregate Crushing Value (ACV); Part 111: Method for Determination of Ten Per Cent Fines Value (TFV).*London: BSI, BS812: 100 Series.

British Standards Institution BS 8002 (1994). Code of Practice for Earth Retaining Structures (Incorporating Amendments No's 1 and 2 and Corrigendum No. 1) September 2001 BSI, London.

British Standards Institution BS EN 1097-2 (1998). *Test Methods-Physical and Mechanical Part 2- Methods for the Determination of Resistance to Fragmentation, Los Angeles Abrasion (LAA) test.* BSI, London. BS EN 1097.

British Standards Institution (2004). *Eurocode 7: Geotechnical Design.* BSI, Milton Keynes, 2004, BS EN 1997: Part 1. General rules.

British Standards Institution BS EN 14731 (2005). *Execution of Special Geotechnical Works-Ground treatment by deep vibration,* BSI, 24 November 2005, London.

Browne, M.A.E., Graham, D.K. and Gregory, D.M. (1984). Quaternary Deposits in the Grangemouth area, Scotland. *Report of the British Geological Survey,* 16, No.3. British Geological Survey. HMSO, Edinburgh.

Building Research Establishment-BRE (1989). Use of 'vibro' ground improvement techniques in the United Kingdom Information Paper IP 5/89.

Building Research Establisment-BRE Bothkennar soils database (1988-1998).

Building Research Establishment-BRE (2000). *Specifying Vibro Stone Columns.* BRE Report BR 391. Garston, UK: CRC.

Burland, J.B. (1969). Contribution to discussion, *Proceedings of Conference on In-situ Investigation on Soils and Rocks.* British Geotechnical Society, London, pp.61-62.

Burland, J.B. (1976). Discussion,- In Ground treatment by deep compaction. *Institution of Civil. Engineers.,*(I.C.E)., London, pp. 142.

Burland, J.B. (1990). The compressibility and shear strength of natural clays. Thirtieth Rankine Lecture *Géotechnique* 40, No. 3, pp. 329-378.

Butterfield, R. and Banerjee, P.K. (1971). A rigid disc embedded in elastic half space. *Journal S.E. Asia National Society of Soil Engineering.* 2, No.1, pp.35-52.

Castro, J. (2007). Pore pressures during stone column installation. *Proceedings of the 17th European Young Geotechnical Engineers Conference,* Ancona, Power-point presentation 2007.

Castro, J. and Sagaseta, C. (2012). Pore pressure during stone column installation. *Proceedings of the Institution of Civil Engineers. Ground Improvement.* Vol 65, No. 2 May 2012, Issue G12, pp. 97-109, Thomas Telford, London.

Chandler, R.J. (1988). The in-situ measurement of the undrained shear strength of clays using the field vane. *Vane shear strength testing in soil: field and laboratory studies.* American Society for Testing and Materials (ASTM),STP 1014, pp. 13-44.

Charles, J.A. and Watts, K.S. (1983). Compressibility of soft clay reinforced with granular columns. *Proc. 8th European Conf. on Soil Mech and Found.Engng,* Helsinki, pp.347-352.

Charles, J.A. and Watts, K.S. (2002). *Treated Ground-Engineering Properties and Performance.* CIRIA Report C572. London: Construction Industry Research and Information Association.

Cheney, J.E. (1989). Long term heave of a building founded on a clay soil after tree removal. *Instrumentation in Geotechnical Engineering.*

Chown, R.C. and Crilly, M.S. (2000). Performance of a semi-rigid raft foundation on soft ground under vertical load. *Ground Engineering,* 33 (3) March, pp. 36-40.

Clayton, C.R.I., Hight, D.W. and Hopper, R.J. (1992). Progressive destructuring of Bothkennar Clay: Implications for sampling and reconsolidation procedures. *Geotechnique* 42, No.2., pp. 219-239.

Cooper, M.R. and Rose, A.N. (1999). Stone column support for an embankment on deep alluvial deposits. *Proceedings of the Institution of Civil Engineers, Geotechnical Engineering,* 137, pp.15-25.

Das, B.M. (1990). Principles of Foundation Engineering, 2nd ed., PWS-KENT, Boston.

Davis, E.H. and Booker, J.R. (1973). The effect of increasing strength with depth on the bearing capacity of clays. *Géotechnique,* 23, No.4, pp. 551-563.

DIN Standards 4017 (2006). Soil-calculation of design bearing capacity of soil beneath shallow foundations. Standard Published 03/01/2006. Deutches Institut Fur Normung E.U. (German National Standard).

Domingues, T.S., Borges, J.L. and Cardoso, A.S. (2007). Stone columns in embankments on soft soils. Analysis of the effect of the ground deformability. In *Proceedings of the 14th European Conference on Soil Mechanics and Geotechnical Engineering,* Madrid, Spain, 24-27 September 2007. Millpress, Amsterdam, Netherlands, Vol 3, pp. 1446-1449.

Duncan, J.M., and Chang, C-Y. (1970). Non-linear analysis of stress and strain in soil. *ASCE J. of the Soil Mech and Found. Div.* Vol. 96, pp. 1629-1653.

Eason, G. and Shield, R.T. (1960). The plastic deformation of a semi-infinite soil by a perfectly rough circular punch. Z. Angew. *Math, Phys,* 11, No. 1, pp. 33-38.

Egan, D., Scott W. and Mc Cabe, B.A. (2008). Installation effects of vibro replacement stone columns in soft clay. In *Proceedings of the 2nd International Workshop on the Geotechnics of Soft Soils,* Glasgow, 2008 pp. 23-30, Taylor and Francis, New York, USA.

Egan,D.(2008). The ground: Clients remain exposed to unnecessary risk. In *Proceedings of the Institution of Civil Engineers. Geotechnical Engineering.* 161, August 2008 Issue GE4, pp. 189-195.

Elshazly, H., Hafez, D. and Mossaad, M. (2008). Back calculating vibro installation stresses in stone column reinforced soils. *Proceedings of the Institution of Civil Engineers, Ground Improvement.* Vol 10. No.2, pp. 47-53.

Engelhardt, K. and Kirsch, K. (1977). Soil Improvement by deep vibratory techniques. *Proc. 5th South East Asian Conference. on Soil Mechanics and. Foundation Engineering.* Bangkok, Thailand, pp. 377-388.

Environment Agency (2001). *Piling and Penetrative Ground Improvement Methods on Land Affected by Contamination: Guidance on Pollution Prevention.* Environment Agency 2001, NC/99/73.

Gäb, M., Schweiger, H.F., Thurner, R., and Adam, D. (2007). Field trial to investigate the performance of a floating stone column foundation. *In Proceedings of the 14th European Conference on Soil Mechanics and Geotechnical Engineering,* Madrid, Spain, 24-27 September 2007. Millpress, Amsterdam, Netherlands, Vol 3 pp. 1311-1316.

Gäb, M., Schweiger, H.F., Kamrat-Pietraszewska, D. and Karstunen, M. (2008). Numerical analysis of a floating stone column foundation using different constitutive models. In *Proceedings of the 2nd International Workshop on the Geotechnnics of Soft Soils,* Glasgow 2008, pp. 137-142.

Gavin, K.G. and Lehane, B.M. (2003). The shaft capacity of pipe piles in sand. *Canadian Geotechnical Journal,* 40, No.1, pp. 36-45.

Gibson, R.E. and Andersen, W.F. (1961). In-situ measurement of soil properties with the pressuremeter. *Civ. Engng. and Public Works Review,* 56, (658), pp. 615.

Giroud, J.P. (1971). Stresses under linearly loaded rectangular area. *Journal. Soil Mech., Found Div., ASCE,* 96, No. SMI, pp. 263-268.

Gostelow, T.P. and Lambert, J.T. (1979). Evaluation of Dutch cone soundings as an Engineering Geological mapping tool in the Quaternary sediments of the Firth of Forth. *Bulletin of the International Association of Engineering Geology,* 19, pp. 216-226.

Gostelow, T.P. and Browne, M.A.E. (1986). Engineering geology of the upper Forth Estuary, *1981 Open File Report on Contract DG 482/87 to the D.O.E., published in 1986 as British Geological Survey Report 16,* No. 8.

Goughnour, R.R. and Bayuk, A.A (1979a). Analysis of stone column-soil matrix interaction under vertical load. In *Proceedings of the International Conference on Soil Reinforcement: Reinforced Earth and Other Technologies,* Paris, Vol.1 pp. 271-277.

Goughnour, R.R. and Bayuk, A.A. (1979b). A field study of long term settlements of loads supported by stone columns in soft ground. In *Proceedings of the International Conference on Soil Reinforcement: Reinforced Earth and Other Technologies,* Paris, pp. 279-285.

Goughnour, R.R. (1983). Settlement of vertically loaded stone columns in soft ground. In *Proceedings 8th European Conference on Soil Mechanics and Foundation Engineering,* Helsinki, Vol 3, pp. 235-240.

Goughnour, R.R., Sung, J.T. and Ramsey, J.S. (1990). Slide correction by stone columns. In *Deep Foundations and Improvements: Design, Construction and Testing* (Esrig, M.I. and Bachus R.C. (eds)), American Society for Testing and Materials. Philadelphia, P.A. ASTM STP 1089, pp. 131-147.

Greenwood. D.A. (1970). Mechanical Improvements of soil below ground surface. *Proceedings of the Conference on Ground Engineering*. Institution of Civil Engineers, London, Vol. 2, pp. 11-22.

Greenwood, D.A. (1975). Vibroflotation: Rationale for design and practice. *In Methods of treatment of unstable ground, ed. F.G. Bell, Newness.* Butterworth, pp. 189-209.

Greenwood, D.A. (1976a). Discussion. - In *Ground Treatment by Deep Compaction.* Institution of Civil Engineers, London, pp. 107-109.

Greenwood, D.A. (1976b). Discussion. - In Ground treatment by deep compaction. *Institution of Civil Engineering,* London, pp. 123-125

Greenwood, D.A. and Kirsch, K. (1984). Specialist ground treatment by vibratory and dynamic methods. *In Piling and Ground treatment,* London: Thomas Telford, 1984. pp. 17-45.

Greenwood, D.A. (1991). Load tests on stone columns. In *Deep Foundations and Improvements: Design, Construction and Testing* (Esrig, M.I. and Bachus R.C. (eds)), American Society for Testing and Materials. Philadelphia, P.A. ASTM STP 1089, pp. 148-171.

Greenwood, D.A. (2004). Discussion.-In *Ground and Soil Improvement* (ed. C.A. Raison). Institution of Civil Engineers, Thomas Telford, London, p. 144.

Guetif, Z, Bouassida, M. and Debats, J.M. (2007). Improved soft clay characteristics due to stone column installation. *Computers and Geotechnics* 34, March, Issue 2 (2007); pp. 104-111.

Hawkins, A.B., Larnach, W.J., Lloyd, I.M. and Nash, D.F.T. (1989). Selecting the location and the initial investigation of the SERC Soft Clay test bed site. *Quarterly Journal of Engineering Geology,* London. Vol 22, pp. 281-316.

Head, K.H. (1994). *Manual of soil laboratory testing.* Volume 2 (Second Edition) John Wiley and Sons Inc.

Herle I., Wehr J. and Arnold, M. (2008). Soil improvement with vibrated stone columns: influence of pressure level and relative density on friction angle. *Proceedings of the 2nd International Conference on the Geotechnics of Soft Soils,-Focus on Ground Improvement.* Glasgow, pp, 235-240.

Hight, D.W, Bond, A.J. and Legge, J.D. (1992). Characterisation of Bothkennar clay – an overview. *Géotechnique,* 42, No. 2, pp. 303-348.

Hope,V.S.(1988). Behaviour of small groups of granular columns in clay. *PhD Thesis.* Birmingham University, UK.

http://www.plaxis.nl

Hu, W. (1995). Physical modelling of group behaviour of stone column foundations. *PhD Thesis.* University of Glasgow, UK.

Hughes, J.M.O., Withers, N.J. (1974). Reinforcing of soft cohesive soils with stone columns. *Ground Engineering. May, pp. 42-49.*

Hughes, J.M.O., Withers, N.J. and Greenwood, D.A (1976). A field trial of the reinforcing effect of a stone column in soil. *Ground treatment by Deep Compaction,* Inst. Civ. Engnrs., London, pp. 31-44.

Institution of Civil Engineers (ICE) (1987). *Specification for Ground Treatment,* London: Thomas Telford.

Institution of Civil Engineers (1992). Eighth *Géotechnique,* Symposium in Print. Bothkennar soft clay test site: characterisation and lessons learned. *Geotechnique,* 42. pp 159-378.

Janbu,N.,Bjerrum,L.and Kjaernsli,B.(1956).Veiledning ved Ifsning av Fundamentering-Soppgaver. *Norwegian Geotechnical Institute,* Oslo, Publication 16, pp. 30-32.

Jardine, R.J., Lehane, B.M., Smith, P,R, and Gildea, P.A. (1995). Vertical loading experiments on rigid pad foundations at Bothkennar. *Géotechnique,* 45, No. 4, pp. 573-597.

Jarrett, R., Stark, R. and Green, C. (1974). A settlement study within a geotechnical investigation of the Grangemouth area. In *settlement of structures, Pentech Press, London, pp. 99-105.*

Jefferson, I., Gaterell, M., Thomas, A.M. and Serridge, C.J. (2010). Emissions assessment related to vibro stone columns. *Proceedings of the Institution of Civil Engineers. Ground Improvement.* 163, February 2010 Issue G11, pp 71-77.

Johnson, D. (1994). Stone columns to prevent slip failure at a coal terminal. *Proc. 13th Int. Conf. on Soil Mechanics and Foundation Engineering, New Delhi,* Jan., Vol. 3, pp. 1203-1206.

Keller technical brochure (2008).

Kempfert, H.G. (2003). Ground Improvement methods with special emphasis on column type techniques. *In Proceedings of International Workshop on Geotechnics of Soft Soils – Theory and Practice, 17-19 September, 2003.* Vermeer, P.A, Schweiger, H.F, Karstunen, M. and Cudny, M. (eds). pp 101-112, Verlag Gluckauf Gmbh, Germany.

Killeen, M.M. and McCabe, B.A. (2010). A numerical study of factors governing the performance of stone columns supporting rigid footings on soft clay. In *Numerical methods in Geotechnical Engineering (Benz and Nordal (eds)).* Taylor and Francis Group, London.

Kirsch, F. and Sondermann, W. (2003). Field measurements and numerical analysis of the stress distribution below stone column supported embankments and their stability. *Proceedings of the International Workshop on Geotechnics of Soft Soils – Theory and Practice.* VGE, Essen, Germany pp. 110-115.

Kirsch, F. (2006). Vibro stone column installation and its effect on ground improvement. In Triantafyllidis, Th (ed.), *Numerical Modelling of Construction Processes inGeotechnical Engineering for Urban Environment.* pp 115-124. Taylor and Francis, New York, USA,

Kirsch, F. (2008). Evaluation of ground improvement by groups of vibro stone columns using field measurements and numerical analysis. In *Karstunen et al* (Eds), *Geotechnics of Soft Soils – Focus on Ground Improvement*, Proceedings of the Second International Workshop on Geotechnics of Soft Soils, Glasgow, CRC Press/Balkema.

Kitazume, M. (2005). The sand compaction pile method. 232 Pages. Taylor and Francis Group Plc, London, UK.

Kjellman, W. (1948). Accelerating consolidation of fine-grained soils by means of cardboard wicks. *Proceedings of 2nd International Conference on Soil Mechanics*, Rotterdam, 1, 302.

Lee, J.S. and Pande, G.N. (1994). Analysis of stone column reinforced foundations. *Departmental Research Report CR/835/94,* University College of Swansea.

Leroueil, S., Lerat, P., Hight, D.W. and Powell, J.J.M. (1992). Hydraulic conductivity of a Recent estuarine silty clay at Bothkennar, Scotland, *Géotechnique,* 42. No. 2, pp. 275-288.

Leslie, D.D. (1963). Large scale triaxial tests on gravelly soils. 2nd Pan American Conference on Soil Mechanics and Foundation Engineering, Sao Paulo, 1963, pp. 181-202.

Lloyd, I.M. (1989). The location and investigation of a test-bed site for research on soft clay. *PhD. Thesis,* University of Bristol.

Marchetti, S. (1975). A new in-situ test for the measurement of horizontal soil deformability. *Proc. Am. Soc. Civ. Engs Speciality Conf. In-situ measurements of soil properties.* Raleigh 2, pp. 255-269.

Massarch, K.R. (1975). New method of measurement of lateral earth pressure in cohesive soils. *Canadian Geotechnical Journal.* 12. No. 1: pp. 142-146.

Mattes, N.S. and Poulos, H.G. (1969). Settlement of single compressible pile. *Journal of Soil Mechanics and Foundation Division.* American Society of Civil Engineers (ASCE) Vol. 95, SMI, Jan., pp. 189-207.

Matthews, M.C., Hope., V.S. and Clayton, C.R.I. (1996). The use of surface waves in the determination of ground stiffness profiles. *Proc. Instn. Civ. Engrs Geotech. Engng,* 119, April, pp. 84-95.

Mc Cabe, B.A., Gavin, K.G. and Kennelly, M. (2008). Installation of a reduced-scale pile group in silt. *Proceedings of the BGA Conference on Foundations,* Dundee, 2008. Vol.1, pp. 607-616.

Mc Cabe, B.A., Nimmons, G.J. and Egan, D. (2009). A review of field performance of stone columns in soft soils. *Proceedings of the Institution of Civil Engineers: Geotechnical Engineering.* 162(6), pp. 323-334.

Mc Kelvey, D. and Sivakumar, V. (2000). A review of the performance of vibro stone column foundations. *Proceedings of 3rd International Conference on Ground Improvement techniques, Singapore* (Pinto MI (ed.)). C.I. Premier Ltd, Singapore, pp. 245-254.

Mc Kelvey, D. (2002). The performance of vibro stone column reinforced foundations in deep soft ground. *PhD Thesis.* Queens University, Belfast.

Mc Kelvey, D., Sivakumar, V., Bell, A.L. & Graham, J. (2004). Modelling vibrated stone columns in soft clay. *Proceedings of the Institution of Civil Engineers: Geotechnical Engineering*. 157(3), pp. 137-149.

Mc Kenna, J.M. (1968). *Site Investigation of a Slip in the East Brent (Somerset) trial embankment.* Soil Mechanics report No. 5101/1. Unpublished.

Mc Kenna, J.M., Eyre, W.A. and Wolstenholme, D.R. (1976), Performance of an embankment supported by stone columns in soft ground. *Géotechnique,* 25, 1, pp. 51-60.

Menzies, B.K. and Matthews, M.C. (1996). The continuous surface wave system: A modern technique for site investigation. *Special Lecture: Indian Geotech. Conf.,* Madras.

Mitchell, J.K. (1981). 'Soil Improvement' State-of-the-Art report. *Proc. 10th Int. Conf. on Soil Mech. Found. Engng.,* Vol. 4, Stockholm, Sweden, pp. 509-565.

Mitchell, J.K. and Huber, T.R. (1985). Performance of a stone column foundation. *J. Geotech Engng., Am. Soc. Civ. Engng.,* Vol. 111, No 2. February 1985. pp. 205-223.

Moreau, N. and Moreau, M. (1835). Foundations – Emploi due sable. *Annales des Points et Chaussees. Memoires.* No. 224, pp. 171-214.

Moseley, M.P. and Priebe, H.J. (1993). Vibro techniques (Chapter 1). In *Ground Improvement.* (Ed. Moseley, M.P.), pp. 1-19. Blackie Academic and Professional, Glasgow.

Moxhay, A.L., Tinsley, R.D. and Sutton, J.A. (2001). Monitoring of soil stiffness during ground improvement using seismic surface waves. *Ground Engineering (1) Jan.*

Munfakh, G.A. (1984). Soil reinforcement by stone columns-varied case applications, In *Proceedings of the International Conference on Insitu Soil and Rock Reinforcement,* Paris, pp 157-162.

Munfakh, G.A., Sarker, S.K. and Castellic, R.J. (1984). Performance of a test embankment founded on stone columns. *In Piling and Ground treatment.* Thomas Telford Ltd., London, pp. 259-265.

Murayama. S. (1962). Considerations on the composer methods as applied to cohesive soils. *Mechanization Association.* No. 150. pp. 10-15.

Nash, D.F.T., Powell, J.J.M. and Lloyd, I.M. (1992a). Initial investigations of the soft clay site at Bothkennar. *Géotechnique,* 1992, 42(2), pp.163-181.

Nash, D.F.T., Sills, G.C. and Davidson, L.R.(1992b). One-dimensional consolidation testing of soft clay from Bothkennar. *Géotechnique,* 42(2), pp. 241-256.

National House Building Council (NHBC), (1989). *Chapter 4.6 Vibratory Ground Improvement techniques.* NHBC Standards, Amersham.

National House Building Council (NHBC), (2011). *Chapter 4.6 Vibratory Ground Improvement techniques.* NHBC Standards, Milton Keynes.

Nayuk, N.V. (1982). Recent innovations in ground improvement by stone columns. *Proc. Symp. Recent Advances in Ground Improvement Techniques.* Bangkok, pp. 17-29.

Ogawa, M and Ichimoto, T. (1963). Application of vibro-compozer methods on sandy and clayey ground. Tuchi-to-Kiso, Vol 13, No.2 pp. 77-82 (In Japanese).

Pande, G.N. (1994). Analysis of irregularly inhomogeneous soils: Application to stone column reinforced foundations. Final Report to EPSRC Grant No.: GR/H/17299, Dept. of Civil Engineering, CR/835/94, University College Swansea.

Paul, M.A., Peacock, J.D. and Wood, B.F. (1992). The Engineering Geology of the Carse Clay of the National Soft Clay Research Site, Bothkennar. *Géotechnique,* 42, No.2, pp. 183-198

Petroski, H. (2006). Success through failure: The paradox of design. Princetown University Press, U.S., 256p.

Phear, A.G. and Harris, S.J. (2008). Contributions to Géotechnique 1948-2008: Ground Improvement. *Géotechnique*, Volume 58, Issue 5 1 June 2008, pp. 399-404

Plaxis (2008). Plaxis 3D Foundation Version 2 Manual (Brinkgreve, R.B.J. and Swolfs, W.M. (eds.)), Plaxis BV, Delft, Netherlands.

Priebe,H.J.(1976). An evaluation of settlement reduction in a soil improved by cram consolidation. *Die Bautechnik,* 53. Vol. 5, pp. 160-162.

Priebe,H.J.(1988).Zur Abschatzung des Setzungsverhalten eines durch Stopfverdichtung verbesserten Baugrundes. (Assessing the settling behaviour of a soil improved by vibro-replacement). *Die Bautechnik*, Vol. 65, No. 1. pp. 23-26.

Priebe,H.J.(1991).Vibro-replacement: Design criteria and quality control. Deep-foundation improvements: Design, construction and testing. *ASTM STP 1089,* Edited by Bachus, R.C., ASTM, Philadelphia.

Priebe, H.J. (1993). Design criteria for ground improvement by stone columns.In *Proceedings of Fourth National Conference on Ground Improvement.* January 18-19, 1993, Lahore, Pakistan.

Priebe, H.J. (1995). The design of vibro-replacement. *Ground Engineering.,* Vol. 28, No. 10, December, pp. 31-37.

Raffle, J.F. and Greenwood, D.A. (1961). The relation between the rheological characteristics of grouts and their capacity to permeate soil. In *5th International Conference on Soil Mechanics and Foundation Engineering.* Paris,Vol II, 1961, pp. 789-793.

Raithel, M., Kempfert, H.G., Kirchner, A. (2002). Geotextile-encased columns (GEC) for foundation of a dike on very soft soils. Proceedings 7th ICG International Conference on Geosynthetics, Nice, France, September, pp. 1025-1028.

Raju, V.R. (1997). The behaviour of very soft soils improved by vibro-replacement. In *Ground Improvement Geosytems: Densification and Reinforcement.*(Davis, M.C.R. and Schlosser F. (eds)). Thomas Telford, London, pp. 253-259.

Randolph, M.F., Carter, J.P. and Wroth, C.P. (1979). Driven piles in clay – the effects of installation and subsequent consolidation *Géotechnique* 29 (4) : pp. 361-393.

Rogers (2012). Chapter 25: The role of ground improvement. In Burland J, Chapman T, Skinner H, Brown M (Eds), *ICE Manual of Geotechnical Engineering.* ICE Publishing, London, Vol. 1, pp, 271-280.

Saadi, A. (1995). The behaviour of strip footings on stone columns. *PhD Thesis, South Bank University,* London, UK.

Schnaid, F., Wood, W.R., Smith, A.K.C. and Jubb, P. (1993). An investigation of bearing capacity and settlements of soft clay deposits at Shellhaven. *Predictive Soil Mechanics, pp. 609-627.* Thomas Telford, London

Schofield, A.N. (1980). 'Cambridge geotechnical centrifuge operations'. *Géotechnique* 30 (3): pp. 227-268.

Schweiger H.F. and Pande, G.N. (1986). Numerical analysis of stone column supported foundations. *Journal of Computers and Geotechnics* Vol 2: 347-372.

Serridge, C.J. (2000). Bauer Internal document.

Serridge, C.J. (2001). A review of vibro stone columns in soft clay (Incorporating field trials of partial depth stone columns at the Bothkennar soft clay research site). *MSc thesis,* University of Bolton (formerly Bolton Institute of Higher Education).

Serridge, C.J. (2005). Achieving sustainability in vibro stone column techniques. *Proceedings of the Institution of Civil Engineers, Journal of Engineering Sustainability.* 158 (ES4), London: Thomas Telford, pp. 211-222.

Serridge, C.J. (2006a). Sustainability in the context of vibro stone column ground improvement techniques. In *Proceedings of the 5th ICEG. Environmental Geotechnics.* Thomas Telford, London, 2006, pp. 1052-1059.

Serridge, C.J. (2006b). Some applications of ground improvement techniques in the urban environment. In *Engineering Geology for Tomorrow's Cities* (Culshaw, M.G., Reeves, H.J., Jefferson, I. and Spink, T.W.(eds.)). Engineering Geology Special Publication 22, Paper 296 (CD-Rom), The Geological Society of London.

Serridge, C.J. and Synac, O. (2007). Ground improvement solutions for motorway widening schemes and new highway embankment construction over soft ground. *Proceedings of the Institution of Civil Engineers. Ground Improvement,* 11(4), pp. 219-228.

Serridge, C.J. and Sarsby, R.W. (2008). A review of field trials investigating the performance of partial depth vibro stone columns in a deep soft clay deposit. In *Proceedings of the 2nd International Workshop on the Geotechnics of soft soils – Focus on Ground improvement.*(Karstunen, M. and Leoni, M. (eds)). Taylor and Francis, London, pp. 293-298.

Serridge, C.J. (2008). Site characterization and ground improvement applications for embankment construction over soft ground. In *Proceedings of the BGA International Conference on Foundations,* Dundee, Scotland, 24-27 June 2008, IHS BRE Press, pp. 1403-1414.

Serridge, C.J. (2010). Development and applications of marine vibro ground improvement techniques. In *Proceedings of Twin International Conference on Geotechnical and Geo-environmental Engineering and (7th) Ground improvement techniques.* 23-25 June 2010, Seoul, South Korea., pp. 105-112.

Serridge, C.J. and Sarsby, R.W. (2010). Assessment of the use of recycled aggregates in vibro-stone column ground improvement techniques. In *Construction for a sustainable environment (Sarsby and Meggyes (eds))* pp. 75-89, Taylor and Francis Group, London.

Serridge, C.J. and Slocombe, B.C (2012). *Ground Improvement, Chapter 84* In Institution of Civil Engineers (ICE) Manual of Geotechnical Engineering,Vol. 2 pp. 1217-1238. Thomas Telford, London.

Sherwood, P.T. (1995). *Alternative Materials in Road Construction: A Guide to the Use of Waste, Recycled Materials and By-products.* Thomas Telford Publications, London.

Shields, D.H. and Bauer, G.E. (1975). Determination of the modulus of deformation of sensitive clay using laboratory and in-situ tests. *In In-situ measurement of soil properties,* Vol 1, pp. 395-421, New York, American Society of Civil Engineers.

Sissons, J.B. (1966). Relative sea level changes between 10,300 and 8,300 BP in part of the Carse of Stirling. *Transactions of the Institute of British Geographers,* 39, pp. 19-29.

Sissons, J.B. (1969). Drift stratigraphy and buried morphological features in the Grangemouth-Falkirk-Airth area, Central Scotland. *Transactions of the Institute of British Geographers*,48, pp. 19-50.

Skempton, A.W. (1942). An investigation of the bearing capacity of a soft alluvial clay soil. *J. Instn Civ. Engrs 18, pp. 307-321.*

Slocombe, B.C., Bell, A.L. and Baez, J.I. (2000). The densification of granular soils using vibro methods. *Géotechnique* 50, No.6, pp. 699-708.

Slocombe, B.C. (2001). Deep compaction of problematic soils. In *Problematic Soils* (eds Jefferson, I., Murray, E.J., Faragher, E. and Fleming, P.R.). London: Thomas Telford, pp. 163-181.

Slocombe, B.C. (2003). Ground Improvement (Nature versus nurture). *Ground Engineering,* 36 (5), 20-23.

Smallridge, M.R. and Johnson, D. (1990). Case study of ground treatment by vibro displacement at a paper store in Immingham. *Proc. Int. Conf. Construction on Polluted and Marginal Land,* pp. 99-102, London.

Smith, P.R., Jardine, R.J. and Hight, D.W. (1992). Yielding of the Bothkennar Clay. *Géotechnique,* 42, No. 2, pp. 257-274.

Suematsu, N., Isoda, T. and Kanda, Y. (1984). Construction of highway on soft ground. *Proc. of Seminar on Soil Improvement and Construction Techniques in soft ground,* Singapore, pp. 144-158.

Tavenas, F.A., Mieussens, C., and Bourges, F. (1979). Lateral displacements in clay foundations under embankments. *Canadian Geotechnical Journal,* 1979, Vol. 16., pp 532-550.

Tedd, P., Powell, J.J.M., Charles, J.A. and Uglow, J.M. (1989). In-situ measurement of earth pressures using push-in spade shaped cells-Ten years experience. *Geotechnical Instrumentation in Practice: purpose, performance and interpretation, pp 701-715. London: Thomas Telford.*

Thomson, G.H. (1987). Personnel communication.

Thorburn, S. and MacVicar, R.S.L. (1968). Soil stabilisation employing surface and depth vibrations. *The Structural Engineer, 46, No. 10, pp. 309-316.*

Thorburn, S. (1975). Building structures supported by stabilised ground. *Géotechnique*, 25, No. 1, pp. 83-94.

Thorburn (1976). 'Discussion, ground treatment by deep compaction' Institution of Civil Engineers, London. Page 130-132.

Tomlinson, M.J. (1995). *Foundation Design and Construction.* 5th Edition, Addison Wesley Longman Ltd.

Van Impe, W.F. and De Beer, E, (1983). Improvement of settlement behaviour of soft layers by means of stone columns. *Proceedings of 8th European Conference on Soil Mechanics and Foundations Engineering.*Balkema, Rotterdam, Netherlands, 1, pp. 309-312.

Van Impe, W.F. (1989). Soil Improvement techniques and their evaluation. *A.A. Balkema, Rotterdam, Netherlands, 125 pages.*

Vesic, A.S. (1972). Expansion of cavities in an infinite soil mass. *Proceedings of the American Society of Civil Engineers (ASCE),* SM3, pp 265.

Watts, K.S. and Charles, J.A. (1988). In-situ measurement of vertical and horizontal stress from a vertical borehole. *Géotechnique, 38, No.4 , pp. 619-626.*

Watts, K.S. and Charles (1991). The use, testing and performance of vibrated stone columns in the United Kingdom. *Deep foundation improvement: design, constructon and testing, pp 212-223,* STP 1089, ASTM. Philadelphia.

Watts, K.S., Johnson, D., Wood, L.A. and Saadi, A. (2000). An instrumented trial of vibro ground treatment supporting strip foundations in variable fill. *Géotechnique,*Vol. 50, No. 6, pp. 699-708.

Watts, K.S., Chown, R.C., Serridge, C.J. and Crilly, M.S. (2001). Vibro stone columns in soft clay: a trial to study the influence of column installation on (raft) foundation performance. *Proceedings of the 15th International Conference on Soil Mechanics and Foundation Engineering,* Istanbul, 3, pp. 1867-1870.

Watts, K.S. (2011). – Personnel communication.

Wehr, J. (2006). The undrained cohesion of the soil as a criterion for the column installation with a depth vibrator. *Proceedings of TransVib Conference,* Paris, pp. 157-162.

Wehr, J. (2007). TC 17 Ground Improvement workshop, Madrid.Power-point presentation on vibro-compaction of reclaimed land (Keller GmBH).

Wilde, P.M. and Crook, J.M. (1992). The monitoring of ground movements and their effects on surface structures – a series of case histories. In *Ground Movements and Structures* (ed Geddes, J.D.). Proceedings of 4th International Conference, Cardiff, 8-11 July, 1991,London, Pentech Press, pp. 182-189.

Wood, L.A. and Perrin, A,J. (1984). Observation of a structural diaphragm wall in London Clay. A preliminary assessment, *Géotechnique, 34, No. 4, pp. 563-579.*

Wood, L.A. (1989). The performance of a raft foundation in London Clay. *Instrumentation, pp. 323-340, in Geotechnical Engineering, Thomas Telford, London.*

Wood, L.A., Johnson, D., Watts, K.S. and Saadi, A. (1996). Performance of strip footings on fill materials reinforced by stone colunns. *Structural Engineer. 74. No. 16, pp.265-271.*

Appendix 1.1

Serridge, C.J. (2006). Some applications of ground improvement techniques in the urban environment. In *Engineering Geology for Tomorrow's Cities* (Culshaw, M.G., Reeves, H.J., Jefferson, I. and Spink, T.W.(eds.)). Engineering Geology Special Publication 22, Paper 296 (CD-rom), The Geological Society of London (cover-sheet).

Serridge, C.J. and Slocombe, B.C. (2012) Ground Improvement (Chapter 84) In *Institution of Civil Engineers (ICE) Manual of Geotechnical Engineering*. Vol 2 of 2, pp. 1217-1238, Thomas Telford, London (cover-sheet).

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Some applications of ground improvement techniques in the urban environment

COLIN J. SERRIDGE[1]

[1] *Pennine Vibropiling (UK) Limited. (e-mail: colin.serridge@pennine-group.co.uk)*

Abstract: Within the UK, high population density and increased traffic volumes has led to the expansion of urban areas incorporating new building construction and associated infrastructure. Such developments have encroached on "brownfield" sites with a legacy of former industrial activity and "marginal sites" where weak alluvial and/or glacigenic deposits occur. Dependent upon site geological and geotechnical characterisation, this has necessitated the use of ground improvement techniques prior to construction. A brief overview is provided of one of the most widely used ground improvement techniques within the UK urban environment, namely that of (vibro) stone columns, in particular how the technique has developed in response to environmental legislation/pressure and requirements for improved performance and sustainability. The importance of detailed site investigation for stone column design in the urban environment is emphasised.

The main part of this paper then focuses on the novel application of vibro stone column (vsc) and vibro concrete column (vcc) techniques beneath new highway embankment construction over soft ground and the steps required to permit successful ground improvement implementation. This draws upon recent experiences on the M60 orbital motorway widening around south Manchester and a new relief link road construction in Kings Lynn, Norfolk (UK).

The most common types of ground related problems encountered during ground improvement relate to soil strata interfaces and boundaries (i.e. geometry not as anticipated) and the geotechnical properties of the soil profile. The usefulness of Cone Penetration Testing (CPT/CPTU) as a site investigation tool, the importance of preliminary trials (particularly for larger projects), quality control, monitoring and testing, to ensure successful implementation and performance of the treated ground is emphasised.

Résumé: Au Royaume Uni, la densité élevée de population et une augmentation du trafic a favorisé l'expansion des zones urbaines et la construction de nouveaux bâtiments et infrastructures. Ces développements ont empiété sur les sites 'brownfield' (anciennes zones d'activité industrielles) et sur les 'marginal sites' caractérisés par de faibles dépôts alluviaux et /ou glaciaux. Les caractéristiques géotechniques du sol peuvent nécessiter l'utilisation de techniques d'amélioration des sols avant de commencer toute construction. C'est pourquoi cet article présentera succinctement la technique la plus utilisée colonnes ballastées d'amélioration des sols dans les zones urbaines du Royaume Uni. La technique colonnes ballastées a été développée pour faire face á une législation environnementale de plus en plus pressante et répondre également aux exigences d'amélioration de performance et de viabilité. Sont aussi mis en évidence l'importance d'une recherche détaillée lors de l'utilisation du colonnes ballastées modèle et de l'application de ce modèle dans un environnement urbain.

Une grande partie de cet article met en exergue les nouvelles applications des techniques < vibro stone column (vsc) > et < vibro concrete column (vcc) > dans la construction des nouvelles chaussées d'autoroutes sur sols mous. On y décrira également toutes les étapes nécessaires á la réussite de l'implantation de cette technique. Toutes ces observations ont été faites récemment lors de l'élargissement du périphérique de Manchester et de la construction d'une voie de secours a Kings Lynn dans le Norfolk (RU).

Les interfaces et extrémités des strates du sol ainsi que ces propriétés géotechniques sont les deux types de problèmes fréquemment rencontrés lors de l'amélioration des sols. Cet article souligne l'utilité des tests CPT (CPTU) pour les recherches dans les sites, l'importance des tests préliminaires (particulièrement pour les grands projets), les contrôles qualité et encore de la surveillance pour s'assurer d'une implantation réussie et des performances du sol traité / amélioré.

Keywords: Environmental urban geotechnics; site investigation; cone penetration testing; geotechnical engineering; design; risk assessment.

INTRODUCTION

Within the UK, high population density and increased traffic volumes has led to the expansion of urban areas. A legacy of industrial activity has given rise to economic and environmental pressure to redevelop land with past industrial usage and other anthropogenic processes, the so called "brownfield sites" (frequently containing deep heterogeneous/miscellaneous fills (made ground deposits) and contaminated soils and groundwater), and to build on land hitherto considered marginal for development, where soft clay soil profiles prevail. This has presented both the engineering geologist and the geotechnical engineer with the challenge of improving methods of geotechnical site characterisation (including site investigation methods), and providing satisfactory foundation solutions and performance at low cost and with minimum environmental impact. For many types of low-rise development, piling through deep fill or natural soft soil deposits may be either uneconomical or impractical and some form of ground improvement/treatment is frequently necessary.

Chapter 84
Ground improvement

Colin J. Serridge Balfour Beatty Ground Engineering Ltd, Manchester, UK
Barry Slocombe Keller Limited, Coventry, UK

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CONTENTS

Ground improvement techniques can often be used as an economical alternative to piled and deep foundation solutions for a wide range of made ground, fill materials and natural soils to support houses, offices, industrial units, tanks, road embankments and similar low-rise developments. Techniques used include *in situ* compaction of clean sands using depth vibrators, adding stone or concrete during compaction to form vibro stone, or vibro concrete columns and dynamic compaction.

As the soil conditions have a large influence on the result, ground improvement techniques require an appropriate level of site and ground investigation to permit satisfactory geotechnical characterisation of the soil profile. Ground improvement also requires an appropriate level of understanding of where the differing techniques work and how to ensure correct and appropriate application. Quality control and monitoring procedures during execution of ground improvement techniques are essential to ensure successful implementation and performance.

Ground improvement techniques permit the adoption of relatively simple shallow foundations and groundbearing warehouse floor slabs. They can also provide significant sustainability advantages in comparison to more traditional deep foundation methods.

84.1 Introduction

Ground improvement involves the enhancement of ground properties, principally by strengthening or stiffening processes and compaction or densification mechanisms, to achieve a specific geotechnical performance.

Of the various ground improvement methods available, vibro techniques employing vibroflots (also referred to as vibrating pokers or depth vibrators) and dynamic methods incorporating the dropping of heavy steel tampers or weights on predetermined grid patterns, are the most commonly used. These form the main focus of discussion in this chapter.

It is important that ground engineering practitioners involved with the design, procurement and supervision of ground improvement techniques have an appropriate level of understanding of the various treatment methods being considered, the likely benefits, including selection of the most appropriate technique and any drawbacks or limitations of the technique selected for the prevailing combination of the ground conditions and applications being considered.

Choice of method will be influenced by the geotechnical requirements and engineering performance objectives of the ground improvement. Appropriate site supervision, quality control and monitoring procedures during execution of ground improvement techniques are essential to ensure successful implementation, as is an appropriate level of post-treatment testing to ensure satisfactory performance will be achieved.

Design principles for these techniques are covered elsewhere (Chapter 59 *Design principles for ground improvement*). Further guidance on these and a wider range of ground improvement techniques can be found in the referenced CIRIA Guides and Building Research Establishment (BRE) Specifications in this chapter. The ground improvement techniques of grouting and soil mixing are covered in Chapter 90 *Geotechnical grouting and soil mixing*.

Where any doubts or uncertainties exist specialist ground improvement contractors should always be consulted. Such advice is normally free of charge, although advance trials may sometimes be recommended.

84.2 Vibro techniques (vibrocompaction and vibro stone columns)

84.2.1 Introduction and history

The modern origins of vibro techniques were conceived in Germany in the mid-1930s. Vibrocompaction was the first technique to be developed, with its first application being for the *in situ* densification of loose sands for a government building in Berlin in 1937. Further development of the technique proceeded in parallel in Germany and the United States in the 1940s. The development of the vibro stone column method in the 1950s permitted the application of vibro techniques to a much wider range of soil types, most notably finer-grained soils (cohesive and mixed cohesive and granular soils), thus increasing the range of soils which could be treated by vibro techniques. Vibro stone column techniques were introduced into Great Britain and France in the late 1950s and have been used extensively worldwide.

84.2.2 Essential features of the vibroflot/vibrator equipment

The principal piece of equipment used to carry out vibro ground improvement techniques (whether for *in situ* densification or

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Ground improvement solutions for motorway widening schemes and new highway embankment construction over soft ground

C. J. Serridge* and O. Synac†
*Pennine Vibropiling Limited, Bacup, Lancashire, UK
†Formerly Pennine Vibropiling limited, Bacup, Lancashire, UK

Over recent years both motorway/highway widening and new highway construction schemes have become increasingly fast track and complex. As such it is not always possible to permit sufficient time within the construction programme to allow settlement to occur for new embankments, particularly over soft ground, without some form of ground improvement. Some important steps in achieving successful ground improvement implementation are: adequate geotechnical site investigation; an appropriate design and review process; consideration of the interfacing of different ground improvement techniques; preliminary trials; monitoring, testing and quality control. Dependent upon the soil profile and geometry, this can be achieved in critical zones using a combination of ground improvement techniques such as vibro concrete columns (VCCs), vibro stone columns (VSCs) and vertical band drains (VBDs), to control settlements and also smooth out the settlement profile on the approach to rigid (piled) bridge abutments to ensure acceptable vehicle ride quality and reduce maintenance. These aspects are discussed, based upon ground improvement experiences on recent motorway widening and new highway construction schemes over soft ground in the UK.

Keywords: embankments; ground improvement; site investigation; soft ground

Au cours des dernières années, les projets d'élargissement des grands axes routiers et de construction de nouveaux grands axes sont devenus de plus en plus «urgents» et toujours plus complexes. En conséquence, il n'est pas toujours possible de disposer de délais suffisants, dans le programme de construction, pour permettre le tassement de nouveaux talus, en particulier sur des sols boulants, sans une certaine bonification du sol. Parmi les mesures importantes permettant de réaliser un renforcement réussi du sol, on indiquera les suivantes: recherches géotechniques adéquates sur le site; méthode appropriée d'étude et de révision; examen de l'interface entre différentes techniques de bonification du sol; essais préliminaires; contrôles, essais et contrôle de la qualité. En fonction du profil et de la géométrie du sol, ceci peut être réalisé, dans des zones critiques, avec une combinaison de techniques de bonification du sol, comme les colonnes vibrobéton (VCC), les colonne de vibro-pierre (VSC) et les drains à bande verticale (VBD) pour suivre l'affaissement et réduire les profils d'affaissement à proximité de la culée de ponts rigides (sur piles) afin d'assurer la qualité acceptable de la circulation automobile et la réduction de l'entretien. Ces aspects sont discutés sur la base des expériences de bonification des sols réalisées récemment dans le cadre de l'élargissement d'autoroutes et de projets de construction de nouveaux grands axes routiers sur des terrains boulants, au Royaume-Uni.

Introduction

Over recent years, both motorway/highway widening (Fig. 1) and new highway embankment construction schemes have become increasingly fast track and complex within the UK, with many phases of traffic management. Consequently, it is not always possible to permit sufficient time within the construction programme to allow settlement to occur for new embankments, particularly over soft/weak ground, without some form of ground improvement. Fast-track construction programmes hence require an innovative and flexible (design) approach. For highway schemes, the approach to bridges (or the transition between rigid bridge abutments and the consolidating soil behind the abutment(s)), is a challenge for the geotechnical engineer who is exploring cost-effective ground improvement solutions, and whose tasks are to ensure a smooth settlement profile and vehicle ride quality and to reduce maintenance. Similarly, the engineering challenge also exists at the approach to drainage culverts crossing a highway.

Drawing upon recent experiences on two highway embankment projects in the UK—the M60 orbital motorway widening scheme (junctions 5–8), around south Manchester and a new relief road construction in Kings Lynn, Norfolk, where such an approach has been required—the following key aspects, all of which form important steps in achieving successful ground improvement implementation (Fig. 2) are discussed:

(*a*) geotechnical site investigation
(*b*) the design process and review

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Appendix 2.2

Journal Proceedings Paper:

Serridge, C.J. (2005) Achieving sustainability in vibro stone column techniques. In *Proceedings of the Institution of Civil Engineers Journal – Engineering Sustainability* 158, Dec 2005 Issue ES4, pp. 311-322, Thomas Telford, London.

Note: For copyright reasons only the front page of the paper is attached. The Publisher should be approached for a full copy of the paper.

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recycling of materials

Colin J. Serridge
Principal Geotechnical Engineer,
Pennine Vibropiling Limited,
Bacup, UK

Achieving sustainability in vibro stone column techniques

C. J. Serridge MSc, MIQ, MCS, MIHT, FGS

As part of achieving environmental sustainability in ground treatment, there is an increasing desire to use recycled materials for vibro stone column techniques. Materials used in stone column construction are required to be free-draining, hard, inert and comply with acceptable criteria in terms of material type, grading, hardness and chemical stability. Spent railway track ballast and crushed concrete probably have the greatest potential for this application in the UK. New European standards for aggregates allow the use of environmentally sustainable materials in a wider range of applications than under the British Standards they replace. However, where such materials are considered for use, as an alternative to natural primary aggregates, it is important that these materials are fit for purpose. Technical requirements for vibro stone column aggregate are discussed. Of particular importance is the fines (clay/silt) content of the aggregate, as this can have a significant effect on the angle of internal friction of the stone column material (and drainage characteristics). This in turn influences both the carrying capacity of the stone column and the settlement characteristics of the stone column–soil composite. Some applications of recycled aggregate in the context of vibro stone columns are presented, including a case history for spent railway ballast and crushed concrete. Also presented is an innovative technique for avoiding pollutant linkages via stone columns using 'vibro concrete plug' technology.

1. INTRODUCTION

Vibro stone columns have been used as a ground treatment technique in the UK since the 1960s and are currently the most common form of ground treatment employed. Useful commentary and guidance on the vibro stone column technique and its applications are given in Moseley and Priebe[1] and Building Research Establishment (BRE) Report BR 391,[2] among others. While the use of recycled aggregates in stone columns is not an entirely new concept, historically the technique has consumed predominantly freshly quarried primary aggregates. However, increasing awareness for sustainable development and construction is leading to a greater desire for use of recycled materials in vibro stone column ground treatment (with utilisation in this sector currently estimated at around 25–30% and increasing). Recycling is of significant benefit through

(*a*) reducing demand on natural (primary) aggregate resources and associated environmental impact (disturbance and transport)
(*b*) reducing disposal of material to landfill.

This contributes to the sustainability objectives of protection of the environment and prudent use of natural resources.

While a considerable amount of recycling is taking place, it is recognised that there is further scope for increasing both the amount of recycling and the level of utility at which recycled materials are used, including ground treatment applications such as vibro stone columns. It is important to recognise, however, that the best practical environmental option (BPEO) remains a key consideration in aggregate use. As far as practical, locally available materials should be used, as it is not sustainable to transport recycled or secondary materials long distances if primary aggregates are available much closer to the construction site. Similarly, natural aggregates should not be transported long distances if suitable recycled and secondary aggregates are available locally. In this regard recycling opportunities may therefore need to be assessed on a site-by-site basis.

2. THE VIBRO STONE COLUMN TECHNIQUE

Within the UK, stone column installation is generally carried out using the dry technique. The specific circumstances of bore stability (particularly in respect of cohesive soils) and groundwater regime, determine whether a top feed (Fig. 1) or bottom feed (Fig. 2) technique is selected. Charges of stone are introduced (from the surface in the case of the top feed technique and by way of a tremie pipe in the case of the bottom feed technique) and compacted in stages by the vibroflot until a dense stone column is constructed to the surface. Within the UK the dry bottom feed technique (see Figs 2 and 3) has largely superseded the wet top feed technique on environmental grounds.

3. DESIGN PHILOSOPHY AND INFLUENCE OF STONE COLUMN MATERIAL ON DESIGN

The fact that a compacted stone column is cohesionless implies that the load-carrying capacity of the column is a function of the angle of internal friction of the column material. It should be recognised, however, that the angle of internal friction of the

Appendix 2.3

Conference Keynote paper:

Serridge, C.J. and Sarsby, R.W. (2010) Assessment of the use of recycled aggregates in vibro-stone column ground improvement techniques. In *Proceedings of Green 5 Conference – Construction for a Sustainable Environment (Sarsby and Meggyes (eds)), Vilnius, Lithuania.* pp.75-89, Taylor and Francis Group, London.

Note: For copyright reasons only the front page of the paper is attached. The Publisher should be approached for a full copy of the paper.

Construction for a Sustainable Environment – Sarsby & Meggyes (eds)
© 2010 Taylor & Francis Group, London, ISBN 978-0-415-56617-9

Assessment of the use of recycled aggregates in vibro-stone column ground improvement techniques

C.J. Serridge
Pennine Vibropiling Ltd., Bacup, Lancashire, UK

R.W. Sarsby
University of Wolverhampton, Wolverhampton, UK

ABSTRACT: Natural (primary) aggregate resources are not unlimited within the UK and their extraction causes increasingly unacceptable geo-environmental impacts. As a part of achieving environmentally sustainable development within the ground engineering sector, there is an increasing desire to utilise recycled (and secondary) aggregates in vibro-stone column ground improvement techniques. However, where such materials are considered for use it is important that there are appropriate specifications and quality control/assurance procedures in place to ensure "fitness-for-purpose". Spent railway track ballast and crushed concrete currently have the greatest potential for this application in the UK and other materials are being investigated. These aspects are discussed and the results of 600 mm diameter plate load tests undertaken on vibro-stone columns constructed using recycled spent rail ballast and crushed concrete (to determine deformation modulus) are presented and these demonstrate comparable performance with natural primary aggregates.

1 INTRODUCTION

It would seem that everyone has something to say about sustainability and one of the biggest challenges for the UK building industry is sustainable construction and development. At the heart of sustainable development is the simple idea of ensuring a better quality of life for everyone, now and for future generations to come. The "sun diagram" in Figure 1 (Leiper, 2003),

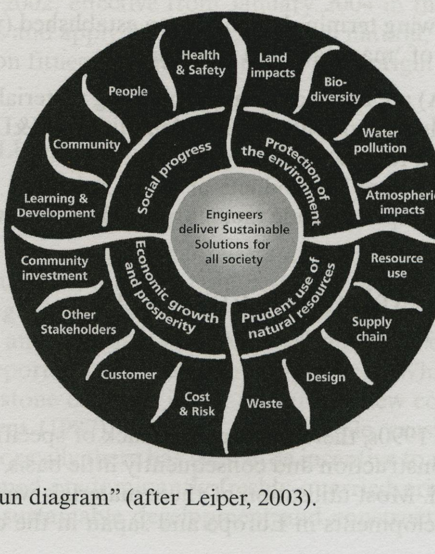

Figure 1. Sustainability "sun diagram" (after Leiper, 2003).

Appendix 2.4

Journal Proceedings Paper:

Jeffersen, I., Gaterell, M., Thomas, A.M. and Serridge, C.J. (2010) Emissions assessment related to vibro stone columns. *Proceedings of the Institution of Civil Engineers. Ground Improvement.* 163, February 2010 Issue G11, pp 71-77.

Note: For copyright reasons only the front page of the paper is attached. The Publisher should be approached for a full copy of the paper.

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Ian Jefferson
Senior Lecturer, School of Civil Engineering, University of Birmingham, UK

Mark Gaterell
Senior Lecturer, School of Civil Engineering, University of Birmingham, UK

Andrew M. Thomas
Research student, School of Civil Engineering, University of Birmingham, UK

Colin J. Serridge
Chief Engineer, Pennine Vibropiling Ltd, Bacup, UK

ice
Institution of Civil Engineers

Emissions assessment related to vibro stone columns

I. Jefferson DIS, PhD, FGS, M. Gaterell MPhil, PhD, DIC, CEnv, MCIWEM, A. M. Thomas MSc, MICE and C. J. Serridge MSc, EurGeol, C.Geol, FGS, MIQ, MCS, MIHT

This paper investigates a number of issues significant to the evaluation of the sustainability of vibro stone columns and, in so doing, discusses how a carbon dioxide emissions estimation tool could be developed for this area of ground improvement. It involves consideration of the use of recycled/secondary aggregates and how this choice affects the column design process, in terms of the durability of constructed works and the amounts of aggregates required. The paper concludes that assessing the sustainability of vibro stone column construction, in carbon dioxide emissions terms, is a complex process not based simply on whether recycled/secondary aggregates themselves are more 'environmentally friendly' than primary aggregates. Instead, it requires care on the part of ground improvement engineers, who must balance geographical variation of aggregate sources against the amount of aggregate used in the vibro stone column construction. However, it is noted that the use of carbon dioxide provides a limited view of sustainable performance, and that the continued development of emerging assessment tools could help capture some of the true complexity of the decision-making process.

1. INTRODUCTION

There are many drivers to enhance the sustainable credentials of the construction industry and recently there have been a number of studies illustrating how greater use of ground improvement techniques in the construction process can help significantly in this regard (Spaulding *et al.*, 2008a, b). This is particularly important as the greatest gains in sustainability terms are to be made at the start of a project where the impacts are greatest, commonly occurring during ground and geotechnical works (Jefferis, 2005).

Of the ground improvements methods available, the vibro-stone column (VSC) technique is one of the most commonly employed in the UK as well as across the rest of the world, since its introduction in the 1950s and 1960s (Serridge, 2005). The basic techniques associated with VSCs are described in detail in many texts (e.g. Mitchell and Jardine, 2002) but in essence involve the use of a large vibrating poker to form a series of columns of compacted stone within weak ground to produce 'reinforcing elements'. Three principal techniques can be used to introduce the stone into the ground, namely: wet-top feed, dry top-feed, and dry-bottom feed. Of these the dry top-feed and bottom-feed methods are the most commonly used in the UK, with the dry-bottom feed technique having largely superseded the wet-top feed method on environmental grounds (Serridge, 2005; Egan *et al.*, 2009).

In recent years there has been a growing body of research that has started to examine sustainability in ground improvement directly, highlighting the gains that can be made (e.g. Rogers *et al.*, 2009). However, much of the work focused on VSCs has only discussed implied sustainability savings through the use of recycled or secondary aggregates (e.g. Serridge, 2005; Tranter *et al.*, 2008). Although, some useful data are emerging in relation to behavioural characteristics of recycled or secondary aggregates in the context of VSCs, for example shear strength (e.g. McKelvey *et al.*, 2002; Slocombe, 2003). Unfortunately, this work has not conducted more vigorous evaluations using a common benchmark to compare true sustainability costs directly. Thus this present paper aims to highlight methods that can be used to undertake broader sustainability assessments as part of the decision-making process in VSC design, highlighting a number of key limitations that exist with such assessments. Although, this paper draws primarily from UK practice and conditions, the approaches reported within have applications in other parts of the world.

2. REQUIREMENTS FOR VSC

VSCs are designed to provide a stone column–soil composite, with improved stiffness and therefore bearing capacity and settlement characteristics. This is achieved by interactions between the compacted stone within the column and the surrounding soils, whereby compression within the column causes bulging of the column material, which is ultimately resisted by the lateral restraint developed in the surrounding soil. To achieve this, stone aggregate within the column must have sufficient shearing resistance and the stone particles themselves must be sufficiently strong to withstand individual stress concentrations.

Design of the columns in the UK typically considers: ultimate carrying capacity (Hughes and Withers, 1974); stress distributions within the columns (Baumann and Bauer, 1974) and appropriate settlement reduction factors, for example Priebe (1995), and a good review of these approaches has been

Appendix 3.1

Conference Proceedings Paper:

Serridge, C.J. (2008) Site characterization and Ground Improvement for embankment construction over soft ground, Paper 118, *In Foundations: Proceedings of the Second BGA International Conference on Foundations, ICOF 2008,* Brown, M.J., Bransby, M.F., Brennan, A.J. and Knappett, J.A. (Editors), pp 1403-1414, IHS Press, 2008, EP93.

Note: For copyright reasons only the front page of the paper is attached. The Publisher should be approached for a full copy of the paper.

118 SITE CHARACTERIZATION AND GROUND IMPROVEMENT FOR EMBANKMENT CONSTRUCTION OVER SOFT GROUND

C. J. SERRIDGE
Pennine Vibropiling Limited, Bacup, Lancashire OL13 9RW, UK
colin.serridge@pennine.co.uk

SUMMARY: It is not always possible to permit sufficient time within construction programmes to allow settlement to occur for new embankments, particularly over soft ground, without some form of ground improvement. The soil profile through which the ground improvement is installed has a significant influence on performance. A more extensive site and ground investigation programme is therefore required than for conventional foundation solutions. With highway embankment projects increasingly encroaching on marginal soft clay sites, both within the UK and Ireland, the importance of appropriate engineering geological and geomorphological input in site "geo-characterization" and development of the "ground model" is discussed, together with developments in cone penetration testing (CPT). In addition to satisfactory site geo-characterization, important steps in achieving successful ground improvement implementation and performance include: an appropriate design and review process; consideration of the interfacing of different ground improvement techniques; preliminary trials; testing, monitoring and quality control.
Keywords: CPT, embankments, geo-characterization, ground improvement, ground model, site investigation, soft ground.

INTRODUCTION

Over recent years, both highway embankment widening and new highway embankment construction schemes have increasingly encroached on (Quaternary) marginal soft clay sites, both within the UK and Ireland. Consequently it is not always possible to permit sufficient time within the construction programme/s to allow settlement to occur for new embankments without some form of ground improvement. Fast-track construction programmes hence require an innovative and flexible (design) approach.

For highway schemes, the approach(es) to bridges (or the transition between rigid abutments and the consolidating soil behind the abutment(s)), presents a challenge for the geotechnical engineer who is exploring cost-effective ground improvement solutions, and whose tasks are to ensure a smooth settlement profile and vehicle ride quality, and to reduce maintenance. Based upon recent ground improvement experiences on highway

Foundations: Proceedings of the Second BGA International Conference on Foundations, ICOF2008. Brown M. J., Bransby M. F., Brennan A. J. and Knappett J. A. (Editors). IHS BRE Press, 2008. EP93, ISBN 978-1-84806-044-9. *www.ihsbrepress.com*

Appendix 4.1

Prediction of ultimate carrying capacity of stone columns (after Hughes and Withers, 1974) and factor of safety against column bulging, (together with load carried by stone column (Pc), after Baumann and Bauer, 1974), for second load increment applied to (Bothkennar) trial footings (described as 'Pad' footings in calculations, i.e. Pad 1 = Trial footing 1 etc..), together with explanatory notes.

Factor of Safety Against Bulging Failure – Cohesive Soils

Calculation of Ultimate Load Carrying Capacity of Stone Column (Q_{ult})

Hughes and Withers (1974) undertook a series of laboratory tests on sand columns in a clay material. They observed that during loading the sand column developed end bearing pressures and adhesive frictional stresses, which are observed in conventional concrete piles. In addition it was also observed that the column bulges, with the bulging being supported by the lateral support of the soils. Hughes and Withers concluded that the load capacity of an isolated stone columns was a function of the lateral support provided by the soil in the bulging zone, which was calculated to be between 1.0 and 2.0 column diameters below the loaded level.

From pressuremeter theory, Gibson and Anderson 1961 showed that the limiting radial stress is given by:

$\sigma_{rl} = \sigma_{ro} + 4C_u + \mu$...1

where

σ_{rl} - Limiting radial stress

σ_{ro} - In situ radial stress

C_u - Cohesion of soil in bulging zone

μ - pore water pressure

(this is taken as zero since the presence of stone columns allows pore pressure dissipation)

At the point where the stone column is failing due to bulging:

$\sigma_v = K_p + \sigma_{ro}$…………………………………………………………………..2

where

σ_v – Vertical Effective Stress

K_p – Coefficient of passive Earth Pressure

$= (1+\sin\phi)/(1-\sin\phi)$

σ_{ro} - In situ radial stress (at failure = σ_{rl})

Therefore, combining equations 1 and 2, we get:

$\sigma_v = [(1+\sin\phi)/(1-\sin\phi)][\sigma_{ro} + 4C_u]$

Since the in situ lateral stress (σ_{ro}) is given by:

$\sigma_{ro} = K(\gamma h + p)$

where

K - coefficient of Earth Pressure (at rest value taken, K=1)

γ – soil density

h – critical depth

p – foundation loading.

Therefore the ultimate load of the column is given by:

$\sigma_v = [(1+\sin\phi)/(1-\sin\phi)][\gamma h + p + 4C_u]$

Calculation of Column Load (P_c)

Where stone columns are installed they are generally a magnitude stiffer than the surrounding soils, and by principles of stress share the stone columns will take a greater proportion of the load which is defined by Baumann and Bauer (1974) in the following expressions:

$$\frac{P_c}{P_s} = \frac{1+2\left(\frac{E_s}{E_c}\right).k_s.\ln.\left(\frac{a}{r_o}\right)}{2.\left(\frac{E_s}{E_c}\right)k_c.\ln\left(\frac{a}{r_o}\right)}$$

$$P_o.A_o = P_c.A_c + P_s.A_s$$

Where,

P_o = Imposed load from foundation
P_c = Stress on stone column
P_s = Stress on soil
A_o = Unit area per stone column
A_s = Cross sectional area of stone column
A_c = Cross section area of treated soil
E_c = Modulus of deformation for stone column aggregate
E_s = modulus of deformation for soil
k_s = Earth pressure co-efficient for column
k_c =Earth pressure co-efficient for soil
r_o = stone column radius
$a = (Ao/\Pi)^{0.5}$

From the above equations the load carried by the column (P_c) can be determined.

Calculating the Factor of Safety Against Bulging Failure

The factor of safety against bulging failure is determined by the following:

$FOS = Q_{ult}/P_c$

PREDICTION OF ULTIMATE CARRYING CAPACITY OF STONE COLUMNS
AND FACTOR OF SAFETY AGAINST BULGING FAILURE

PAD 1

DESIGN PARAMETERS:

Nc=	42.50	degrees	Foundation Po =	72.00	kN/m^2
Y=	17.50	kN/m^3	Foundation Ao =	1.13	m^2
h =	1.50	m			
c =	20.00	kN/m^2			
p =	0.00	kN/m^2	Es/Ec =	0.08	
			ro =	0.37	m
			Ks =	1.50	
			Kc =	0.60	

After Hughes and Withers (1974) Qult = 548.79 kN/m^2

Using Baumann and Bauer (1974)

FOUNDATION

Pc/Ps =	24.07	
Pc =	177.19	kN/m^2
Ps =	7.36	kN/m^2

Factor of safety = 3.10

PREDICTION OF ULTIMATE CARRYING CAPACITY OF STONE COLUMNS
AND FACTOR OF SAFETY AGAINST BULGING FAILURE

PAD 2

DESIGN PARAMETERS:

Nc=	42.50	degrees	Foundation Po =	67.10	kN/m^2
Y=	17.50	kN/m^3	Foundation Ao =	1.50	m^2
h =	1.50	m			
c =	20.00	kN/m^2			
p =	0.00	kN/m^2	Es/Ec =	0.08	
			ro =	0.37	m
			Ks =	1.50	
			Kc =	0.60	

After Hughes and Withers (1974) Qult = 548.79 kN/m^2

Using Baumann and Bauer (1974)

FOUNDATION

Pc/Ps =	19.18	
Pc =	207.15	kN/m^2
Ps =	10.80	kN/m^2

Factor of safety = 2.65

PREDICTION OF ULTIMATE CARRYING CAPACITY OF STONE COLUMNS
AND FACTOR OF SAFETY AGAINST BULGING FAILURE

PAD 3

DESIGN PARAMETERS:

Nc=	42.50	degrees	Foundation Po =	67.80	kN/m^2
Y=	17.50	kN/m^3	Foundation Ao =	1.13	m^2
h =	1.50	m			
c =	20.00	kN/m^2			
p =	0.00	kN/m^2	Es/Ec =	0.08	
			ro =	0.37	m
			Ks =	1.50	
			Kc =	0.60	

After Hughes and Withers (1974) Qult = 548.79 kN/m^2

Using Baumann and Bauer (1974)

FOUNDATION

Pc/Ps =	24.07	
Pc =	166.85	kN/m^2
Ps =	6.93	kN/m^2

Factor of safety = 3.29

Figure A4.1.1: (a) Prediction of ultimate carrying capacity of stone columns and factor of safety against bulging failure for trial footings (pads) 1-3.

PREDICTION OF ULTIMATE CARRYING CAPACITY OF STONE COLUMNS
AND FACTOR OF SAFETY AGAINST BULGING FAILURE

PAD 4

DESIGN PARAMETERS:

Nc=	42.50	degrees	Foundation Po =	71.10	kN/m^2
Y=	17.50	kN/m^3	Foundation Ao =	1.13	m^2
h =	1.50	m			
c =	20.00	kN/m^2			
p =	0.00	kN/m^2	Es/Ec =	0.08	
			ro =	0.37	m
			Ks =	1.50	
			Kc =	0.60	

After Hughes and Withers (1974) Qult = 548.79 kN/m^2

Using Baumann and Bauer (1974)

FOUNDATION

Pc/Ps = 24.07
Pc = 174.98 kN/m^2
Ps = 7.27 kN/m^2

Factor of safety = 3.14

PREDICTION OF ULTIMATE CARRYING CAPACITY OF STONE COLUMNS
AND FACTOR OF SAFETY AGAINST BULGING FAILURE

PAD 5

DESIGN PARAMETERS:

Nc=	42.50	degrees	Foundation Po =	67.80	kN/m^2
Y=	17.50	kN/m^3	Foundation Ao =	1.13	m^2
h =	1.50	m			
c =	20.00	kN/m^2			
p =	0.00	kN/m^2	Es/Ec =	0.08	
			ro =	0.37	m
			Ks =	1.50	
			Kc =	0.60	

After Hughes and Withers (1974) Qult = 548.79 kN/m^2

Using Baumann and Bauer (1974)

FOUNDATION

Pc/Ps = 24.07
Pc = 166.85 kN/m^2
Ps = 6.93 kN/m^2

Factor of safety = 3.29

PREDICTION OF ULTIMATE CARRYING CAPACITY OF STONE COLUMNS
AND FACTOR OF SAFETY AGAINST BULGING FAILURE

PAD 6

DESIGN PARAMETERS:

Nc=	42.50	degrees	Foundation Po =	69.60	kN/m^2
Y=	17.50	kN/m^3	Foundation Ao =	1.13	m^2
h =	1.50	m			
c =	20.00	kN/m^2			
p =	0.00	kN/m^2	Es/Ec =	0.08	
			ro =	0.37	m
			Ks =	1.50	
			Kc =	0.60	

After Hughes and Withers (1974) Qult = 548.79 kN/m^2

Using Baumann and Bauer (1974)

FOUNDATION

Pc/Ps = 24.07
Pc = 171.28 kN/m^2
Ps = 7.12 kN/m^2

Factor of safety = 3.20

Figure A4.1.1: (b) Prediction of ultimate carrying capacity of stone columns and factor of safety against bulging failure for trial footings (pads) 4-6.

PREDICTION OF ULTIMATE CARRYING CAPACITY OF STONE COLUMNS
AND FACTOR OF SAFETY AGAINST BULGING FAILURE

PAD 7

DESIGN PARAMETERS:

Nc=	42.50	degrees	Foundation Po =	67.00	kN/m^2
Y=	17.50	kN/m^3	Foundation Ao =	1.13	m^2
h =	1.50	m			
c =	20.00	kN/m^2			
p =	0.00	kN/m^2	Es/Ec =	0.08	
			ro =	0.37	m
			Ks =	1.50	
			Kc =	0.60	

After Hughes and Withers (1974) Qult = 548.79 kN/m^2

Using Baumann and Bauer (1974)

FOUNDATION

Pc/Ps =	24.07	
Pc =	164.89	kN/m^2
Ps =	6.85	kN/m^2

Factor of safety = 3.33

Notation

A_O	:	Total foundation area per stone column
Nc	:	Friction angle of stone column material
γ	:	Bulk unit weight of the soil
h	:	Critical depth at which bulging failure of the stone column occurs
c	:	Undrained shear strength of the soil surrounding the stone column
P	:	Additional load imposed on soil surrounding the stone column by foundations and/or floor load.
E_s	:	Modulus of deformation of the soil
E_c	:	Modulus of deformation of the stone column
P_o	:	Average imposed stress on the foundation
R_o	:	Original radius of the stone column
K_s	:	Coefficient of earth pressure for the soil
K_c	:	Coefficient of earth pressure for stone column
Q_{ult}	:	Ultimate carrying capacity of stone column
P_c	:	Average stress in the stone column at foundation level.
P_s	:	Average stress in the unmodified soil between stone columns

Figure A4.1.1: (c) Prediction of ultimate carrying capacity of stone columns and factor of safety against bulging failure for trial footing (pad) 7.

Appendix 4.2

Prediction of ultimate carrying capacity of stone columns and factor of safety against column bulging (after Hughes and Withers, 1974), utilising updated spreadsheet, for average applied second load increment in Bothkennar field trials (1.5 m column spacing beneath trial footing 6). Load carried by column (Pc) has also been calculated based upon Baumann and Bauer (1974).

Determination of Factor of Safety Against Bulging Failure

Contract Name:	Bothkennar field trials - Trial footing 6 (1.5 m column spacings)
Contract Number:	Bauer
Calculation Reference:	FOS against column bulging in soft Carse Clay

Design Details

P_o =	70	kN/m^2	= Imposed foundation load
A_o =	1.13	m^2	= area per stone column
r_o =	0.35	m	= column radius
d =	1.2	m	= foundation depth
P =	0		= Imposed loading around foundation

Calculation of Stone Column Capacity

K_s =	1.5		= Coeficient of Earth Pressure for soil
K_c =	0.6		= Coeficient forEarth Pressure for column
E_c =	50	MPa	= Modulus of Stone Column
E_s =	4	MPa	= Modulus of Soil
A_c =	0.385	m^2	= Cross sectional area of stone column
A_s =	0.745	m^2	= Cross sectional area of soil in A_o
ϕ_c =	42.5	°	= angle of friction of stone column material
Pc/Ps =	21.84		

P_c = 188.8 kPa

Calculation of Ultimate Load Capacity of Stone Column

ϕ_c	42.5		= Friction angle of Stone Column
C_u	20	kPa	= shear strength of soil
γ_s	17	kN/m^3	= Unit Weight of soil
h	1.5	m	= critical depth
			= d + 3*r_o

Qult = 544.9 kPa

Factor of Safety Against Bulging Failure

$$FOS = \frac{Q_{ult}}{P_c} = \mathbf{2.886}$$

Calculation Sheet Page 1 of 1

Appendix 5.1

Building Research Establishment (BRE) Large triaxial test apparatus and test results

Figure A5.1.1: (a) BRE large diameter triaxial cell apparatus.

BRE LARGE DIAMETER TRIAXIAL SAMPLE DATA

Sample origin - Bothkennar stone columns
Sample identification -
Test No. - 1
Date - 06/02/99

ram constant	=	1.809 cl/mm
top cap weight	=	6.81 kg.

ORIGINAL SAMPLE PROPERTIES

Sample preparation details - recovered from site stockpile, as delivered
Method of construction - 5 layers, light compaction to avoid particle breakdown

original hiegth =	544.5 mm	original moisture content =	5.5 %
original diam. =	228 mm	bulk sample weight =	38.13 kg.
original area =	0.040828 m2	prelim. dry sample weight =	36.142 kg.
original vol. =	0.022231 m3	original bulk density =	1.715 Mg/m3
		prelim. dry density =	1.626 Mg/m3
		original unit weight =	16.826 kN/m3
		prelim. dry unit weight =	15.949 kN/m3
		sample dry density =	#VALUE! Mg/m3
		sample dry unit weight =	#VALUE! kN/m3

TEST PROCEDURE

Drained/Undrained (U/D) - D
proving ring/internal load cell (p/l) - p/l
nominal cell pressure - 120
sample strain rate 0.4 mm/min.

CONSOLIDATION		POST CONSOLIDATION	
cell pressure =	125 kN/m2	ram travel =	3.3 mm
back pressure =	0 kN/m2	ram constant =	1.87 cl/mm
initial PWP =	3.4 kN/m2	total vol.change =	897 cl
final PWP =	2 kN/m2	corr.for cell exp= (see charts)	34 cl
change in length =	3.3 mm	actual vol.change=	400 cl
axial strain =	0.006061	actual vol.strain=	0.017993
sample height =	541.2 mm	sample volume =	0.021831 m3
sample area =	0.040338 m2		

SHEAR STAGE (see test data at failure on spreadsheet)

1/2 height at beginning of shear + 0.135m	=	0.406 m
pore pressure reading at failure	=	5 kN/m2
cell pressure reading at failure	=	125 kN/m2
effective confining pressure at failure	=	120.51 kN/m2
load cell reading at failure	=	27.5 kN
sample height at failure	=	0.4552 m
sample area at failure	=	0.043065 m2
weight of top cap	=	6.81 kg.
submerged sample unit weight	=	7.015916 kN/m3
effective deviator stress	=	640.2 kN/m2
effectve stress ratio at failure	=	6.3
angle of effective shear resistence	=	46.6 degrees

FINAL SAMPLE PROPERTIES

shear disspl'nt =	93.8 mm	bulk sample weight =	38.77 kg.
final height =	447.4 mm	moisture content =	? %
stain at end of shear stage =	17.33186 %	dry sample weight =	? kg.

Figure A5.1.1: (b) BRE large diameter triaxial laboratory test data extract.

Appendix 5.2

Laboratory test data for Bothkennar ‘crust’:

- Consolidation test data
- Plasticity indices

BS1377 : Part 5 : Clause 3 : 1990
One Dimensional Consolidation Test

PROJECT NUMBER: GEO / 1430
PROJECT NAME: BOTHKENNAR CRUST

Borehole No:
Sample No: 1
Depth: 0.60

Description: Firm brown very silty CLAY with pockets of weakly cemented rust-brown silt and a single fossilised root concretion (8mm dia).

Diameter of specimen (mm)	76.45
Area of specimen (A) (mm)	4590.34
Initial height of specimen (Ho) (	19.05
Volume of specimen (cm³)	87.446
Dial gauge factor (mm/div)	0.001

Initial weight of specimen + ring (g)	254.90
Weight of ring (g)	89.83
Initial moisture content of specimen (%)	34.10
Initial wet density (Mg/m³)	1.888
Initial dry density (Mg/m³)	1.408
Particle density (Mg/m³)	2.68
Height of solids (Hs) (mm)	10.006
Initial degree of saturation (based on trimmings)	101.11
Initial degree of saturation (based on specimen)	97.92

Final weight of specimen + ring (g)	251.89
Final dry weight of specimen + ring (g)	213.92
Initial moisture content based on specimen	33.024
Final moisture content based on specimen	30.599
Height of voids (Hv) (mm)	9.044

Input data into yellow cells only

Calculation cells - DO NOT TOUC

VOIDS RATIO									COMPRESSIBILITY			COEFFICIENT OF CONSOLIDATION				
Increment No	Pressure (kPa)	Displacement Readings Initial	Displacement Readings Final	Cumulative compression (mm)	Machine correction (divs)	Cumulative correction (mm)	Consolidated height H=Ho-(DH-y)	Voids ratio e=H-Hs / Hs	Incremental Height change dH	Pressure change dp	mv=dH*1000 / H1 dp	t50 min	t90 min	H=(H1+H2) / 2	cv= 0.026H-² / t50	cv= 0.111H-² / t90
0	0	-	-	0	0	0	19.050	0.904	0	0	-	-	-	-	-	-
1	10	5476	5339	0.1370	0	0	18.913	0.890	0.137	10	0.719	2.75		18.982	3.41	
2	20	5339	5230	0.2460	0	0	18.804	0.879	0.109	10	0.576	7		18.859	1.32	
3	40	5230	5042	0.4340	0	0	18.616	0.860	0.188	20	0.500	6		18.710	1.52	
4	80	5042	4710	0.7660	0	0	18.284	0.827	0.332	40	0.446	4		18.450	2.21	
5	160	4710	4289	1.1870	0	0	17.863	0.785	0.421	80	0.288	5.3		18.074	1.60	
6		4289														
7																
8																

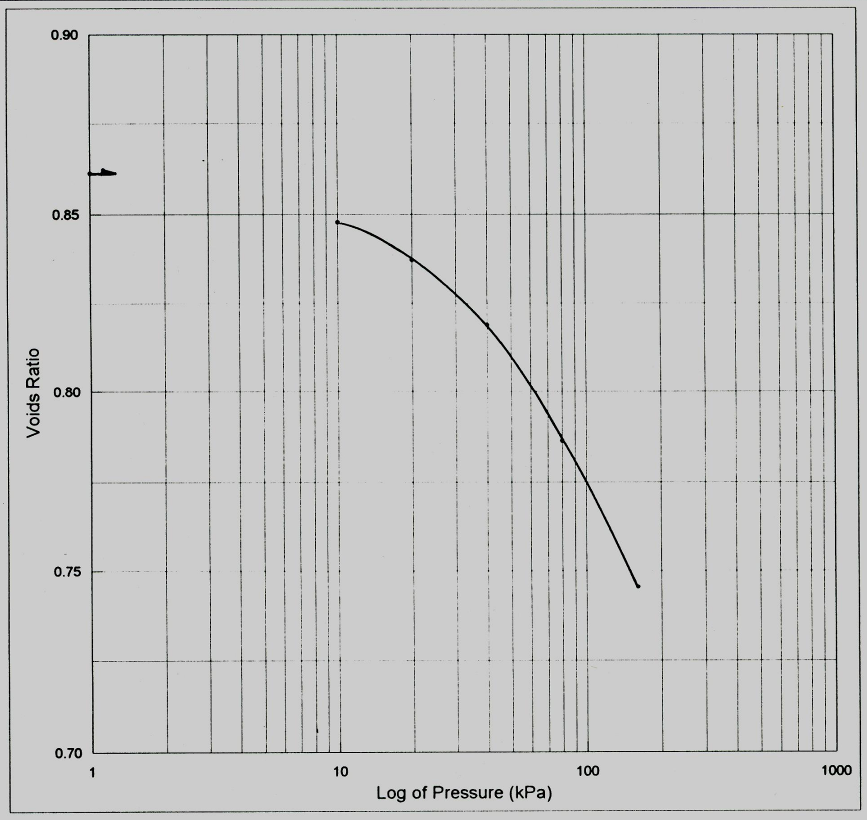
Figure A5.2.1: (a) Laboratory data from one – dimensional consolidation test on sample of Bothkennar crust.

BS1377 : Part 5 : Clause 3 : 1990

Determination of One Dimensional Consolidation.

Sample No: 1
Depth: 0.60

Description:
Firm brown very silty CLAY with pockets of weakly cemented rust-brown silt and a single fossilised root concretion (8mm dia).



Initial Moisture Content 34 | Initial Voids Ratio 0.861 | Bulk Density 1.89

Final Moisture Content 31 | Final Voids Ratio 0.745 | Dry Density 1.41

Specimen Size 76.45 dia * 19.05 high | Particle Density 2.62 (Calc)

Pressure Range (kPa)	Mv (m^2/MN)	Cv (m^2/yr)	Voids Ratio
0 - 10	0.719	3.41	0.848
10 - 20	0.576	1.32	0.837
20 - 40	0.500	1.52	0.819
40 - 80	0.446	2.21	0.786
80 - 160	0.288	1.60	0.745

Figure A5.2:1 (b) Plot of one-dimensional consolidation test result on Bothkennar crust.

BS1377 : Part 5 : Clause 3 : 1990
One Dimensional Consolidation Test

PROJECT NUMBER: GEO / 1430
PROJECT NAME: BOTHKENNAR CRUST

Sample No: 2
Depth: 0.60

Description: Firm brown very silty CLAY with pockets of weakly cemented rust-brown silt.

Diameter of specimen	(mm)	75.00
Area of specimen (A)	(mm)	4417.86
Initial height of specimen (Ho)	(	19
Volume of specimen (cm³)		83.939
Dial gauge factor	(mm/div)	0.001

Initial weight of specimen + ring (g)	283.70
Weight of ring (g)	118.78
Initial moisture content of specimen (%)	34.60
Initial wet density (Mg/m³)	1.965
Initial dry density (Mg/m³)	1.460
Particle density (Mg/m³)	2.68
Height of solids (Hs) (mm)	10.349
Initial degree of saturation (based on trimmings)	110.92
Initial degree of saturation (based on specimen)	96.87

Final weight of specimen + ring (g)	276.03
Final dry weight of specimen + ring (g)	245.43
Initial moisture content based on specimen	30.217
Final moisture content based on specimen	24.161
Height of voids (Hv) (mm)	8.651

Input data into yellow cells only

Calculation cells - DO NOT TOUCI

VOIDS RATIO									COMPRESSIBILITY			COEFFICIENT OF CONSOLIDATION				
Increment No	Pressure (kPa)	Displacement Readings Initial	Displacement Readings Final	Cumulative compression (mm)	Machine correction (divs)	Cumulative correction (mm)	Consolidated height H=Ho-(DH-y)	Voids ratio e=H-Hs / Hs	Incremental Height change dH	Pressure change dp	mv=dH*1000 / H1 dp	t50 min	t90 min	H=(H1+H2) / 2	cv= 0.026H-² / t50	cv= 0.111H-² / t90
0	0	-	-	0	0	0	19.000	0.836	0	0	-	-	-	-	-	-
1	10	9285	9148	0.1370	0	0	18.863	0.823	0.137	10	0.721	2.34		18.932	3.98	
2	20	9148	9036	0.2490	0	0	18.751	0.812	0.112	10	0.594	3		18.807	3.07	
3	40	9036	8886	0.3990	0	0	18.601	0.797	0.150	20	0.400	3		18.676	3.02	
4	80	8886	8644	0.6410	0	0	18.359	0.774	0.242	40	0.325	2.12		18.480	4.19	
5	160	8644	8283	1.0020	0	0	17.998	0.739	0.361	80	0.246	2.56		18.179	3.36	
6	320	8283	7841	1.4440	0	0	17.556	0.696	0.442	160	0.153	3.31		17.777	2.48	
7	640	7841	7247	2.0380	0	0	16.962	0.639	0.594	320	0.108	3		17.259	2.58	
8	1280	7247	6562	2.7230	0	0	16.277	0.573	0.685	640	0.063	2		16.620	3.59	
9		6562														
10																
11																

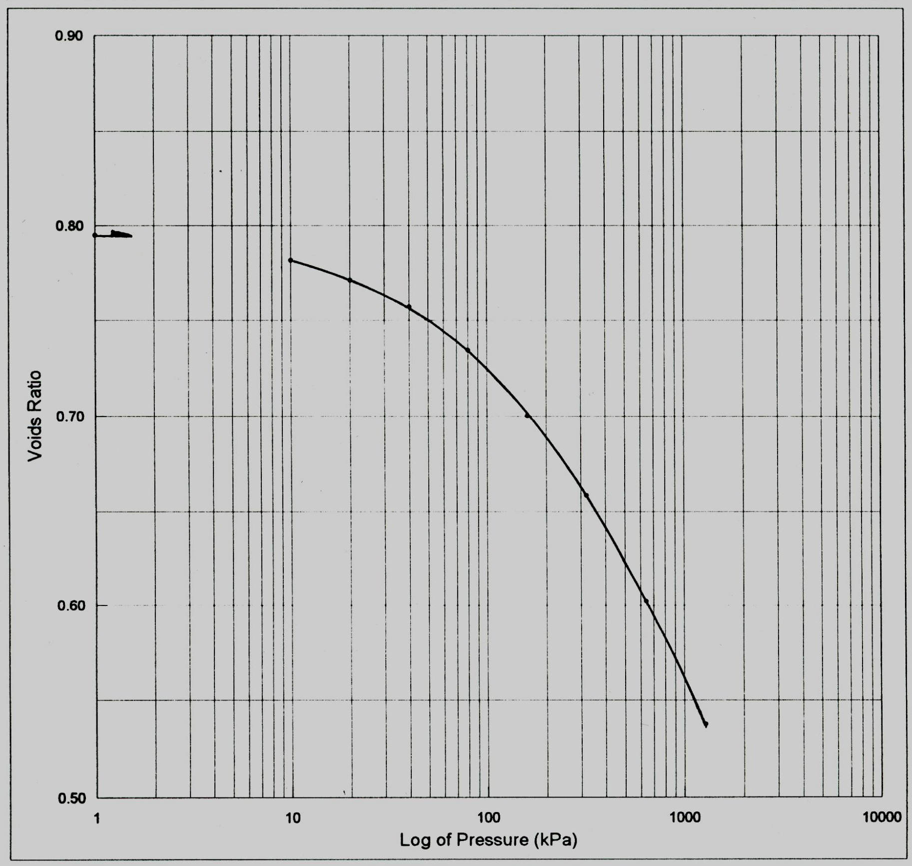
Figure A5.2.2: (a) Laboratory data from one –dimensional consolidation test on sample of Bothkennar crust.

BS1377 : Part 5 : Clause 3 : 1990

Determination of One Dimensional Consolidation.

Sample No: 2
Depth: 0.60

Description:
Firm brown very silty CLAY with pockets of weakly cemented rust-brown silt.



Initial Moisture Content	35	Initial Voids Ratio	0.795	Bulk Density	1.96
Final Moisture Content	24	Final Voids Ratio	0.700	Dry Density	1.46
Specimen Size	75.00 dia * 19.00 high			Particle Density	2.62 (Calc)

Pressure Range (kPa)	Mv (m^2/MN)	Cv (m^2/yr)	Voids Ratio
0 - 10	0.721	3.98	0.782
10 - 20	0.594	3.07	0.771
20 - 40	0.400	3.02	0.757
40 - 80	0.325	4.19	0.734
80 - 160	0.246	3.36	0.700
160 - 320	0.153	2.48	0.658
320 - 640	0.106	2.58	0.602
640 - 1280	0.063	3.59	0.538

Figure A5.2.2: (b) Plot of one-dimensional consolidation test result on Bothkennar crust.

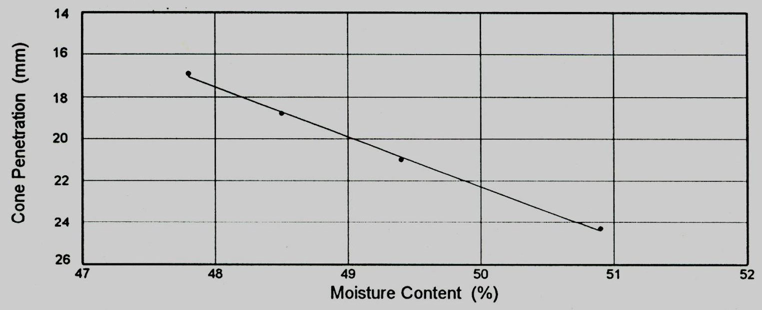
BS1377 : Part 2 : 1990

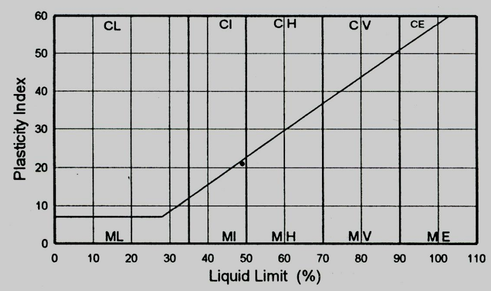
Liquid and Plastic Limits

Sample Number: 2 Depth (m): 0.60	Description: Brown very silty CLAY with pockets of rust-brown silt.

BS1377 : Part 2 : Clause 4.3 : 1990 Determination of Liquid Limit (Cone Penetrometer Method)
BS1377 : Part 2 : Clause 5 : 1990 Determination of Plastic Limit and Plasticity Index

Preparation: Specimen as received





Results:

Natural moisture content:	33 %
Percentage passing 425 μm sieve:	100 %
Liquid Limit:	49 %
Plastic Limit:	28 %
Plasticity Index:	21
Equivalent moisture content of material passing 425 μm sieve:	33 %
Liquidity Index:	0.24

Figure A5.2.3: Results of plasticity indices determination for sample of Bothkennar crust.

Appendix 6.1

Field trial stone column installation records:

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	21	
Ground level	2.786	m. AOD
Fill level	2.786	m. AOD (2)
lequired column base depth	6.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	5.7	m. (1) = (o)-(m)
Poker penetration level	-2.914	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	8	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Comments
17/09/97	09:04	0	0	2.786	Bucket and start
17/09/97	09:05	1	5.7	-2.914	Penetration depth
17/09/97	09:06	2	6.2	-3.414	Penetration with stone
17/09/97	09:07	3	6.2	-3.414	Bucket
17/09/97	09:08	4	6	-3.214	
17/09/97	09:10	6	5.5	-2.714	Bucket
17/09/97	09:12	8			Bucket
17/09/97	09:15	11	3	-0.214	Bucket
17/09/97	09:17	13	3	-0.214	Bucket
17/09/97	09:19	15	2.5	0.286	Bucket
17/09/97	09:20	16			Bucket
17/09/97	09:21	17	0	2.786	Finished
					Average column diameter

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	22	
Ground level	2.776	m. AOD
Fill level	2.776	m. AOD (2)
lequired column base depth	6.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	5.7	m. (1) = (o)-(m)
Poker penetration level	-2.924	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	7	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Comments
17/09/97	08:45	0	0	2.776	
17/09/97	08:47	2	0	2.776	Bucket and start
17/09/97	08:49	4	5.7	-2.924	penetration depth
17/09/97	08:50	5	6.2	-3.424	penetration with stone
17/09/97	08:51	6	6.2	-3.424	Bucket
17/09/97	08:52	7	6	-3.224	
17/09/97	08:53	8	5.5	-2.724	Bucket
17/09/97	08:55	10	5	-2.224	Bucket
17/09/97	08:56	11	4.5	-1.724	
17/09/97	08:57	12	4	-1.224	Bucket
17/09/97	08:59	14	3	-0.224	
17/09/97	09:00	15	2.5	0.276	Bucket
17/09/97	09:00	15	2	0.776	
17/09/97	09:02	17	0	2.776	Bucket and then finished
					Average column diameter

Figure A6.1.1 : Stone column installation record for column 21 and 22.

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	23	
Ground level	2.779	m. AOD
Fill level	2.779	m. AOD (2)
lequired column base depth	6.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	5.7	m. (1) = (o)-(m)
Poker penetration level	-2.921	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	6	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Comments
16/09/97	17:51	0	0	2.779	Bucket
16/09/97	17:52	1	5.7	-2.921	Penetration depth
16/09/97	17:53	2	6.2	-3.421	Penetration with stone
16/09/97	17:54	3	6	-3.221	Bucket
16/09/97	17:56	5	6	-3.221	
16/09/97	17:57	6	5.5	-2.721	Bucket
16/09/97	17:58	7	5	-2.221	
16/09/97	17:59	8	4	-1.221	Bucket
16/09/97	18:01	10	2.5	0.279	Bucket
16/09/97	18:03	12			Bucket
16/09/97	18:05	14	0	2.779	Finished
					Average column diameter

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	24	
Ground level	2.771	m. AOD
Fill level	2.771	m. AOD (2)
lequired column base depth	6.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	5.7	m. (1) = (o)-(m)
Poker penetration level	-2.929	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	7	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Comments
16/09/97	17:32	0	0	2.771	Bucket
16/09/97	17:35	3	5.7	-2.929	Penetration depth
16/09/97	17:35	3	6.2	-3.429	Penetration with stone
16/09/97	17:36	4	6	-3.229	Bucket
16/09/97	17:38	6	5.5	-2.729	
16/09/97	17:39	7	5.5	-2.729	Bucket
16/09/97	17:41	9	5	-2.229	
16/09/97	17:42	10	4.5	-1.729	Bucket
16/09/97	17:44	12	4	-1.229	Bucket
16/09/97	17:46	14	3.5	-0.729	Bucket
16/09/97	17:47	15	3	-0.229	
16/09/97	17:48	16	2	0.771	Bucket
16/09/97	17:49	17	2	0.771	
16/09/97	17:50	18	0	2.771	Finished
					Average column diameter

Figure A6.1.2: Stone column installation record for columns 23 and 24.

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	25	
Ground level	2.705	m. AOD
Fill level	2.705	m. AOD (2)
Required column base depth	6.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	5.7	m. (1) = (o)-(m)
Poker penetration level	-2.995	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	7	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Comments
16/09/97	17:17	0	0	2.705	Bucket
16/09/97	17:18	1	5.7	-2.995	Penetration depth
16/09/97	17:20	3	6	-3.295	Bucket and penetration with stone
16/09/97	17:21	4	5.5	-2.795	Bucket
16/09/97	17:22	5	5.5	-2.795	
16/09/97	17:23	6	5.5	-2.795	Bucket
16/09/97	17:24	7	5	-2.295	
16/09/97	17:25	8	4	-1.295	
16/09/97	17:26	9	3.5	-0.795	Bucket
16/09/97	17:27	10	3	-0.295	Bucket
16/09/97	17:28	11	2.5	0.205	
16/09/97	17:29	12	2	0.705	Bucket
16/09/97	17:30	13	0	2.705	Finished
					Average column diameter

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	26	
Ground level	2.686	m. AOD
Fill level	2.686	m. AOD (2)
Required column base depth	6.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	5.7	m. (1) = (o)-(m)
Poker penetration level	-3.014	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	6	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Comments
16/09/97	17:01	0	0	2.686	Bucket
16/09/97	17:03	2	5.7	-3.014	Penetration depth
16/09/97	17:04	3	6.2	-3.514	Bucket and penetration with stone
16/09/97	17:05	4	6.2	-3.514	
16/09/97	17:06	5	6	-3.314	Bucket
16/09/97	17:08	7	5	-2.314	
16/09/97	17:09	8	3	-0.314	Bucket
16/09/97	17:10	9	3	-0.314	
16/09/97	17:12	11	2.5	0.186	Bucket
16/09/97	17:13	12	2.5	0.186	
16/09/97	17:14	13	1.5	1.186	Bucket
16/09/97	17:16	15	0	2.686	Finished

Figure A6.1.3: Stone column installation record for columns 25 and 26.

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	27	
Ground level	2.683	m. AOD
Fill level	2.683	m. AOD (2)
Required column base depth	6.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	5.7	m. (1) = (o)-(m)
Poker penetration level	-3.017	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	6	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Comments
16/09/97	16:45	0	0	2.683	Bucket
16/09/97	16:47	2	5.7	-3.017	Penetration depth
16/09/97	16:48	3	6	-3.317	Penetration with stone
16/09/97	16:49	4	6	-3.317	Bucket
16/09/97	16:51	6	6	-3.317	Bucket
16/09/97	16:52	7	5.5	-2.817	
16/09/97	16:53	8	4	-1.317	Bucket
16/09/97	16:54	9	3	-0.317	
16/09/97	16:55	10	3	-0.317	Bucket
16/09/97	16:58	13	2.5	0.183	Bucket
16/09/97	17:00	15	0	2.683	Finished

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Associated Instrumentation
Mini cell No. 10 - Vert. stress @ 4.5m beneath col 28

Cell zero	1.14	Bar
Box zero	0.11	Bar

Column No.	28	
Ground level	2.695	m. AOD
Fill level	2.695	m. AOD (2)
Required column base depth	4.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	3.7	m. (1) = (o)-(m)
Poker penetration level	-1.005	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	5	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Cell 10 Glotzl box reading (Bar)	Cell 10 Stress (kPa)	Comments
16/09/97	16:13	0	0	2.695	1.9	76	Bucket
16/09/97	16:15	2	3.7	-1.005	2.35	121	penetration depth
16/09/97	16:16		4.2	-1.505	2.5	136	Penetration with stone
16/09/97	16:16	3	4	-1.305			Bucket
16/09/97	16:17	4	4	-1.305			
16/09/97	16:17	4	3.5	-0.805			
16/09/97	16:18	5	3.5	-0.805			Bucket
16/09/97	16:19	6	3	-0.305			
16/09/97	16:20	7	2	0.695			Bucket
16/09/97	16:21	8	1.5	1.195			Bucket
16/09/97	16:22	9	1	1.695	2.35	121	Finished
16/09/97							Cell showed reaction at 2m
16/09/97					2.4	126	When pushing on top part
16/09/97	16:23	10	0	2.695	2.35	121	Steady after
16/09/97	18:15	122	installed		2.05	91	installed
22/09/97	10:30	8297			1.95	81	during excavation
22/09/97	12:00	8387			1.95	81	post excavation
23/09/97	16:00	10067			1.95	81	wet blinding
24/09/97	16:00	11507			1.96	82	wet concrete
15/10/97	15:10	41697			2	86	

Figure A6.1.4: Stone column installation record for columns 27 and 28.

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	29	
Ground level	2.721	m. AOD
Fill level	2.721	m. AOD (2)
Iequired column base depth	4.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	3.7	m. (1) = (o)-(m)
Poker penetration level	-0.979	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	4	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Comments
16/09/97	15:54	0	0	2.721	Bucket
16/09/97	15:56	2	3.7	-0.979	Penetration depth
16/09/97	15:57	3	0	2.721	Poker withdrawn to clear b lockage
16/09/97	16:04	10	4.2	-1.479	
16/09/97	16:06	12	4.1	-1.379	Bucket
16/09/97	16:08	14	3.5	-0.779	
16/09/97	16:09	15	3	-0.279	Bucket
16/09/97	16:10	16	2.5	0.221	
16/09/97	16:10	16	2	0.721	Bucket
16/09/97	16:11	17	2	0.721	building top part
16/09/97	16:12	18	0	2.721	Finished

Figure A6.1.5: Stone column installation record for column 29.

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	31	
Ground level	2.741	m. AOD
Fill level	2.741	m. AOD (2)
Required column base depth	6.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	5.7	m. (1) = (o)-(m)
Poker penetration level	-2.959	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	7	

Associated instrumentation

Inclinometer system

Section Extension	Length
2	0.4
3	0.4
4	0.4
1	0.5
5	0.8

Piezometer No. 8 - Pore water pressure @ 1.0m

Cell zero	0.15	Bar
Box zero	0.11	Bar
Cell elevation	1.741	m. AOD

Tape Taught Rod Vertical

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Piezo 8 Glotzl box reading (Bar)	Piezo 8 Stress elev. m AOD (-26)	Inclin section 2	Inclin section 3	Inclin section 4	Inclin section 1	Inclin section 5	Tape distance (m)	Measured angle (Deg)	Horizontal distance (m)	Comments
16/09/97	15:03	0	0	2.741	0.15	1.74	-1106	5020	-1276	-8587	3330	6.475			Bucket
16/09/97	15:05	2	1.5	1.241	0.5	5.31									
16/09/97	15:07	4	0	2.741	0.22	2.45									Poker out to clear blockage
16/09/97	15:09	6	5.7	-2.959											Penetration depth
16/09/97	15:10	7	6	-3.259											Penetration with stone
16/09/97	15:11	8	5.5	-2.759											Bucket
16/09/97	15:13	10													Bucket
16/09/97	15:14	11	5	-2.259	0.33	3.58									Bucket
16/09/97	15:19	16	4	-1.259	0.33	3.58									Bucket
16/09/97	15:20	17													Bucket
16/09/97	15:22	19	2	0.741	0.44	4.70									Bucket
16/09/97	15:23	20													
16/09/97	15:24	21	0	2.741											Finished
16/09/97															
16/09/97	15:30	27		installed	0.44	4.70									
16/09/97															
16/09/97	16:00	57		installed			-16815	10827	-16948	4494	662	6.295			
16/09/97	18:15	192		installed	0.24	2.66									
															post col construction
22/09/97	10:30	8367			0.24	2.66									
22/09/97	16:30	8727			0.21	2.35									
23/09/97	16:30	10167			0.21	2.35	-16035	11585	-16901	3658	-445				post excavation
24/09/97	15:10	11527			0.2	2.25	-15995	11510	-16854	3649	-410	6.333			post concrete
15/10/97	15:10	41767			0.16	1.84									
04/11/97	10:50	70307			0.15	1.74									

Figure A6.1.6: Stone column installation record for column 31.

Bothkennar Soft Soil Vibro Trial

Associated instrumentation
Mini cell No. 27 - Vert. stress @ 8.5m

Cell zero	0.67	Bar
Box zero	0.11	Bar

Vibro stone column installation

Column No.	32	
Ground level	2.751	m. AOD
Fill level	2.751	m. AOD (2)
Required column base depth	8.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	7.7	m. (1) = (o)-(m)
Poker penetration level	-4.949	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	5	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Cell 27 Glotzl box reading (Bar)	Cell 27 Stress (kPa)	Comments
16/09/97	13:51	0		2.751	1.7	103	Bucket
16/09/97			7.7	-4.949	2.2	142	1.85 after first penetration
16/09/97			8.2	-5.449	2.7	192	firts penetration with stone
16/09/97	13:56	5					Bucket
16/09/97	13:57	6	7.7	-4.949	2.9	212	
16/09/97	13:59	8	6	-3.249			Bucket
16/09/97	14:01	10	4	-1.249			
16/09/97	14:02	11	3.5	-0.749	2.8	202	Responding during push
16/09/97	14:02	11					Bucket
16/09/97	14:03	12	3.5	-0.749			Bucket
16/09/97	14:05	14	0	2.751	2.5	172	Finished
16/09/97							Steady while building top
16/09/97	18:15	264	installed		2.25	147	Installed
22/09/97	10:30	8439			1.85	107	pre-excavation
22/09/97	16:30	8799			1.86	108	post-excavation
23/09/97	16:30	10239			1.78	100	wet blinding
24/09/97	15:10	11599			1.76	98	wet concrete
15/10/97	15:10	41839			1.64	86	

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	33	
Ground level	2.725	m. AOD
Fill level	2.725	m. AOD (2)
Required column base depth	8.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	7.7	m. (1) = (o)-(m)
Poker penetration level	-4.975	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	8	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Comments
16/09/97	13:22	0	0	2.725	Bucket
16/09/97	13:25	3	7.7	-4.975	Penetration Depth
16/09/97	13:26	4	8.2	-5.476	Depth with stone
16/09/97	13:27	5	7.2	-4.475	Bucket
16/09/97	13:29	7	7	-4.275	
16/09/97	13:30	8	7	-4.275	Bucket
16/09/97	13:32	10	7	-4.275	
16/09/97	13:33	11	6.5	-3.775	
16/09/97	13:34	12	6	-3.275	
16/09/97	13:35	13	6	-3.275	Bucket
16/09/97	13:37	15	5.5	-2.775	
16/09/97	13:38	16	5.5	-2.775	Bucket
16/09/97	13:41	19	5	-2.275	
16/09/97	13:43	21	3.5	-0.775	Bucket
16/09/97	13:45	23	2	0.725	Bucket
16/09/97	13:48	26	1	1.725	Bucket
16/09/97	13:50	28	0	2.725	Finished

Figure A6.1.7: Stone column installation record for columns 32 and 33.

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	34	
Ground level	2.731	m. AOD
Fill level	2.731	m. AOD (2)
Required column base depth	6.9	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	6.4	m. (1) = (o)-(m)
Poker penetration level	-3.669	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	7	

Associated instrumentation

Mini cell No. 7 - Vert. stress @ 7.2m				Mini cell No. 109 - Horiz. stress @ 1.7m		
Cell elevation	-4.469	m. AOD		Cell elevation	1.031	m. AOD
Cell zero	1.48	Bar		Cell zero	1.04	Bar
Box zero	0.56	Bar		Box zero	0.11	Bar
Mini cell No. 24 - Horiz. stress @ 2.3m				Piezometer No. 9 - Pore water pressure @ 2.3m		
Cell elevation	0.431	m. AOD		Cell elevation	0.431	m. AOD
Cell zero	1.39	Bar		Cell zero	0.04	Bar
Box zero	0.56	Bar		Box zero	0.11	Bar

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Cell 7 Glotzl box reading (Bar)	Cell 7 Stress elev. m AOD	Cell 24 Glotzl box reading (Bar)	Cell 24 Stress elev. m AOD	Cell 109 Glotzl box reading (Bar)	Cell 109 Stress elev. m AOD	Peizo 9 Glotzl box reading (Bar)	Piezo 9 Stress elev. m AOD	Comments
						-204	2	-195 2		-115		-15	
16/09/97	12:40	0	0	2.731	3.15	7.97	1.95	1.65	1.25	3.17	0.4	4.10	
16/09/97	12:41	1							1.25	3.17			Start and Bucket
16/09/97	12:45		6	-3.269	3.41	10.62							
16/09/97					3.6	12.55							Max Penetration
16/09/97					4	16.63							
16/09/97	12:46	6			3.5	11.54							On Withdrawal
16/09/97	12:47	7	6.4	-3.669	3.7	13.57							Bucket
16/09/97	12:49	9			4.2	18.67					0.6	6.14	
16/09/97	12:50	10	6	-3.269									
16/09/97	12:50	10							1.37	4.39			Bucket Air Fracture
16/09/97	12:53	13	5.5	-2.769	4.2	18.67							
16/09/97	12:54	14											Bucket
16/09/97	12:54	14	4.5	-1.769	4.2	18.67			1.14	2.05			
16/09/97	12:56	16	4.5	-1.769			1.8	0.02					Bucket
16/09/97	12:58	18	2	0.731					1.55	6.23			Bucket
16/09/97	13:00	20	1.5	1.231									Bucket
16/09/97					3.85	15.10							top of col constrn, cell 7 still reacting
16/09/97	13:02	22	0	2.731									Finished
16/09/97													
16/09/97													
16/09/97	13:09	29	installed		3.83	14.90	1.94	6.04	1.5	5.72			ell 24 re charged new zero .11 rdg 1.9
16/09/97	18:15	335	installed		2.85	9.50	1.75	4.10	1.34	4.09	0.6	6.14	
22/09/97	10:30	8510			2.8	8.99	1.66	3.18	1.24	3.07	0.39	4.00	pre-excavation
22/09/97	16:30	8870			2.84	9.39	1.65	3.08	1.23	2.97	0.39	4.00	post-excavation
23/09/97	16:30	10310			2.91	10.11	1.66	3.18	1.23	2.97	0.38	3.90	wet blinding
24/09/97	15:10	11670			2.84	9.39	1.65	3.08	1.26	3.27	0.38	3.90	wet concrete
15/10/97	15:10	41910			2.65	7.46	1.61	2.67	1.28	3.48	0.31	3.18	
03/11/97	11:00	69020			2.65	7.46	1.59	2.47	1.27	3.38	0.29	2.98	Pre-loading

Figure A6.1.8: Stone column installation record for column 34.

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	35	
Ground level	2.736	m. AOD
Fill level	2.736	m. AOD (2)
lequired column base depth	6.9	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	6.4	m. (1) = (o)-(m)
Poker penetration level	-3.664	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	7	

Piezometer No. 10 - Pore water pressure @ 1.7m		
Cell elevation	1.036	m
Cell zero	0.17	Ba
Box zero	0.11	Ba

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Peizo 10 Glotzl box reading (Bar)	Piezo 10 Stress (kPa) -28	Comments
16/09/97	11:59	0	0	2.736	0.21	3.18	
16/09/97	12:00	1	0				Bucket
16/09/97	12:01	2	1		0.3	4.09	
16/09/97			2.5		0.8	9.19	
16/09/97	12:02	3	5.5		0.45	5.62	
16/09/97	12:03	4					Bucket
16/09/97	12:04	5	6.9		0.5	6.13	Very responsive while building column
16/09/97	12:05	6	6.5				
16/09/97	12:06	7	6.5				Bucket
16/09/97	12:08	9	5.5				
16/09/97	12:09	10	5.3				
16/09/97	12:10	11	5		0.6	7.15	Bucket
16/09/97	12:12	13	3				Bucket
16/09/97	12:15	16	2.5		0.5	6.13	Pore pressure NIL while retracting near top
16/09/97	12:17	18			0.45	5.62	No Activity due to blockage
16/09/97							One load wasted while clearing block
16/09/97							
16/09/97							
16/09/97	12:20	21					Bucket and restart
16/09/97	12:22	23	3.5				
16/09/97	12:23	24					Bucket
16/09/97	12:24	25	2.5				
16/09/97	12:25	26	2.5				Bucket
16/09/97	12:26	27	2				
16/09/97	12:26	27	1.7		0.7	8.17	Compacting stone at instrument level
16/09/97	12:27	28	0				Finished
16/09/97					0.8	9.19	Compacting of stone above Piezo
16/09/97	12:29	30	installed		0.5	6.13	Residual pressure before moving away
16/09/97	18:15	376	installed		0.37	4.81	
22/09/97	10:30	8551			0.23	3.38	
22/09/97	16:30	8911			0.26	3.69	
23/09/97	16:30	10351			0.26	3.69	
24/09/97	15:10	11711			0.25	3.58	
15/10/97	15:10	41951			0.23	3.38	
03/11/97	11:00	69061			0.21	3.18	Pre-loading

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	36	
Ground level	2.712	m. AOD
Fill level	2.712	m. AOD (2)
ıquired column base depth	6.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	5.7	m. (1) = (o)-(m)
Poker penetration level	-2.988	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	6	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Comments
16/09/97	10:46	0	0	2.712	Bucket
16/09/97	10:49	3	6.2	-3.488	
16/09/97	10:50	4	6.2	-3.488	Bucket
16/09/97	10:53	7	5.5	-2.788	Bucket
16/09/97	10:54	8	4.5	-1.788	
16/09/97			5	-2.288	
16/09/97	10:55	9	4.5	-1.788	
16/09/97	10:56	10	3	-0.288	Bucket
16/09/97	10:58	12	2.5	0.212	
16/09/97	10:58	12	3	-0.288	Bucket
16/09/97	10:59	13	3	-0.288	
16/09/97	11:00	14	2.5	0.212	Bucket
16/09/97	11:01	15	2	0.712	
16/09/97	11:01	15	1.5	1.212	
16/09/97	11:03	17	0	2.712	Finished

Figure A6.1.9: Stone column installation record for columns 35 and 36.

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	38	
Ground level	2.731	m. AOD
Fill level	2.731	m. AOD (2)
lequired column base depth	6.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	5.7	m. (1) = (o)-(m)
Poker penetration level	-2.969	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	6	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Comments
17/09/97	09:44	0	0	2.731	Bucket
17/09/97	09:45	1	0	2.731	Start
17/09/97	09:46	2	5.7	-2.969	Penetration depth
17/09/97	09:46	2	6.2	-3.469	Penetration with stone
17/09/97	09:47	3			Bucket
17/09/97	09:49	5	5.5	-2.769	Bucket
17/09/97	09:50	6	5.5	-2.769	
17/09/97	09:51	7			Bucket
17/09/97	09:52	8	4	-1.269	Bucket
17/09/97	09:54	10	2.5	0.231	Bucket
17/09/97	09:55	11	0	2.731	Finished

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	39	
Ground level	2.736	m. AOD
Fill level	2.736	m. AOD (2)
quired column base depth	6.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	5.7	m. (1) = (o)-(m)
Poker penetration level	-2.964	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	7	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Comments
17/09/97	09:25	0	0	2.736	Bucket
17/09/97	09:27	2	5.7	-2.964	Penetration depth
17/09/97	09:28	3	6.2	-3.464	Penetration with stone
17/09/97	09:30	5	6.2	-3.464	Bucket
17/09/97	09:31	6	6	-3.264	
17/09/97	09:32	7	6.2	-3.464	Bucket
17/09/97	09:34	9	5.5	-2.764	Bucket
17/09/97	09:36	11	5	-2.264	
17/09/97	09:38	13	4	-1.264	Bucket
17/09/97	09:39	14	3	-0.264	
17/09/97	09:40	15	3	-0.264	Bucket
17/09/97	09:42	17	2	0.736	Bucket
17/09/97	09:44	19	0	2.736	Finished

Figure A6.1.10: Stone column installation record for columns 38 and 39.

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	45	
Ground level	2.736	m. AOD
Fill level	2.736	m. AOD (2)
lequired column base depth	6.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	5.7	m. (1) = (o)-(m)
Poker penetration level	-2.964	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	8	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Comments
18/09/97	11:03	0		2.736	Start +Bucket
18/09/97	11:05	2	5.7	-2.964	Penetration depth
18/09/97	11:06	3	6.2	-3.464	Penetration with stone + Bucket
18/09/97	11:10	7	5.5	-2.764	Bucket
18/09/97	11:12	9	4	-1.264	Bucket
18/09/97	11:14	11	4	-1.264	Bucket
18/09/97	11:16	13	3.5	-0.764	
18/09/97	11:17	14	3.25	-0.514	Bucket
18/09/97	11:20	17	3	-0.264	Bucket
18/09/97	11:22	19	2.5	0.236	Bucket
18/09/97	11:24	21	0	2.736	Finish
					Average diameter

Bothkennar Soft Soil Vibro Trial

Vibro stone column installation

Column No.	46	
Ground level	2.736	m. AOD
Fill level	2.736	m. AOD (2)
lequired column base depth	6.2	m. (0)
Penetration reduction	-0.5	m. (m)
Poker penetration depth	5.7	m. (1) = (o)-(m)
Poker penetration level	-2.964	m. AOD = AOD(2) - (1)
Loader bucket volume:	0.68	cub.m
No of buckets:	5	

Date	Time (hrs/min/secs)	ELAPSED TIME (MINS)	Poker penetration depth (m)	Poker penetration (m.AOD)	Comments
18/09/97	12:19	0	0	2.736	Start + Bucket
18/09/97	12:20	1	5.7	-2.964	Penetration depth
18/09/97	12:20	1	6.2	-3.464	penetration with stone
18/09/97	12:22	3	6	-3.264	Bucket
18/09/97	12:25	6	5.5	-2.764	Bucket
18/09/97	12:27	8	5	-2.264	
18/09/97	12:28	9	4.5	-1.764	Bucket
18/09/97	12:31	12	3	-0.264	Bucket
18/09/97	12:32	13	0	2.736	Finished
					Average diameter

Figure A6.1.15: Stone column installation record for columns 45 and 46

Appendix 7.1

Conference Proceedings Paper:

Serridge, C.J. and Sarsby, R.W. (2008) A review of field trials investigating the performance of partial depth vibro stone columns in a deep soft clay deposit. *In Proceedings of Conference on Geotechnics of soft soils – Focus on Ground Improvement*, pp. 293-298 Strathclyde University, June, 2008. Karstunen and Leoni (eds), 2009, Taylor and Francis, London, (cover sheet).

Note: For copyright reasons only the front page of the paper is attached. The Publisher should be approached for a full copy of the paper.

Geotechnics of Soft Soils – Focus on Ground Improvement – Karstunen & Leoni (eds)

A review of field trials investigating the performance of partial depth vibro stone columns in a deep soft clay deposit

C.J. Serridge
School of Engineering and the Built Environment, University of Wolverhampton, Wolverhampton, UK

R.W. Sarsby
School of Engineering and the Built Environment, University of Wolverhampton, Wolverhampton, UK

ABSTRACT: Useful laboratory based research and numerical modelling has been undertaken on vibro-stone columns in soft clay soils over the last 10–15 years. However, there exists the inherent issue, respectively, of scale effects and of being able to compare such results with actual field data. The main aims of field trials carried out at the (former) Bothkennar soft clay research site in Scotland, were to investigate ground response to installation of partial depth ("floating") vibro-stone columns, by the dry bottom feed method, in a deep sensitive soft clay deposit, together with the effects of stone column length, spacing and a thin surface "crust" on the behaviour of trial foundations/footings constructed over the installed trial stone columns. The data obtained has important implications with regard to the way we approach both stone column design and installation for partial depth vibro stone columns in deep soft clay deposits, and highlights the importance of field trials to calibrate numerical models, particularly in view of the complex field behaviour observed.

1 INTRODUCTION

Vibro-stone column techniques are increasingly being considered for the development of marginal sites with deep soft clay deposits and, where some settlement can be tolerated, to provide an economic alternative solution to the traditional approach of deep foundation piles.

The limited published field data relating to vibro stone columns (VSC's) in deep soft clay soils have focussed on the wet top-feed method, with little if any field data existing for the dry bottom feed method. Field trials are an important pre-requisite to any new vibro stone column applications or systems being proposed in difficult ground. In order to address these issues instrumented field trials were undertaken at the former Bothkennar soft clay research site in Scotland (Watts & Serridge, 2000). The field trials at Bothkennar have yielded significant data and which has important implications with regard to the way we approach both vibro stone column design and installation, for partial depth ("floating") stone columns beneath shallow footings in deep soft sensitive clay profiles. This forms the basis of ongoing research which is reviewed in this paper.

2 BOTHKENNAR SITE

The (former) Bothkennar soft clay research site is located on former intertidal mudflats adjacent to the south bank of the Forth estuary, about 1 km south of the Kincardine Bridge and approximately midway between Glasgow and Edinburgh in central Scotland. The site is low-lying and level and has an elevation of between 2.5 m and 3.0 m AOD and is within the tidal range of the estuary, which was around 5.0 m, with mean high water spring tide being + 2.86 m AOD at the time of the field trials.

3 SITE GEO-CHARACTERIZATION

Because of its research applications the Bothkennar site has been subject to a number of comprehensive site investigations and the site geo-characterization of the soft clay deposit is well documented (e.g. Institution of Civil Engineers, 1992; Nash et al., 1992; Hight et al., 1992). This was clearly of assistance in the selection of soil geotechnical parameters for the field trial(s). Only the main characteristics are therefore summarized here.

The Bothkennar soft clay research site lies within the outcrop of the Holocene raised estuarine deposits locally termed "carse clays" which occur widely at the head of the Forth estuary (Barras & Paul, 2000).

At the location of the trials, a normally to lightly over-consolidated clay profile is present with a firm to stiff "desiccated crust", about 1.1 m thick. This is underlain by a thin (approx. 0.3 m) band of shells

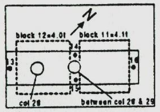
Appendix 7.2

Spreadsheet data extracts relating to instrumentation responses and settlement responses to load application to trial footings.

Bothkennar Soft Soil Vibro Trial

TEST PAD BEHAVIOUR

PAD No: 3
COLUMNS: 2
COLUMN SPACING: 1.5 m
:PTH OF TREATMENT: 4.2 m
COLUMN No's: 28,29



PAD LENGTH: 3.14 m
PAD WIDTH: 0.765 m

BLOCK 11: 4.1 tonne
BLOCK 12: 4 tonne

TOTAL BLOCK WEIGHT: 8.1 tonne
FOUNDATION LINE LOAD: 25.3 kN/m

USING SITE DATUM

Associated instrumentation
Flatjack cell - Vert. stress on col 28

Cell zero 0.99 Bar
Box zero 0.11 Bar

Flatjack cell - Vert. stress between col 28 & 29

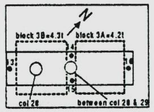
Cell zero 1 Bar
Box zero 0.11 Bar

Mini cell No. 10 - Vert. stress @ 4.5m beneath col 28

Cell zero 1.03 Bar
Box zero 0.11 Bar

DATE	TIME	ELAPSED TIME (MINS)	NETT APPLIED LOAD (kN/m^2)	LEVELLING PINS - LEVEL 13 (m AOD)	LEVELLING PINS - LEVEL 14 (m AOD)	LEVELLING PINS - LEVEL 15 (m AOD)	LEVELLING PINS - LEVEL 16 (m AOD)	LEVELLING PINS - SETTLEMENT 13 (mm)	LEVELLING PINS - SETTLEMENT 14 (mm)	LEVELLING PINS - SETTLEMENT 15 (mm)	LEVELLING PINS - SETTLEMENT 16 (mm)	INCREMENT AVERAGE SETTLEMENT (mm)	TOTAL TRANSVERSE TILT (mm)	TOTAL DIFFERENTIAL SETTLEMENT (mm)	Cell 10 Glotzl box reading (Bar)	Cell 10 Stress (kPa)	Col 28 Glotzl box reading (Bar)	Col 28 Stress (kPa)	Col 28/29 Glotzl box reading (Bar)	Col 28/29 Stress (kPa)	Stress ratio	Comments
03/11/97	10:30			2.52535	2.53100	2.53112	2.49656	0	0	0	0	0			1.99	96	1.13	14	1.13	13		
04/11/97	11:52	0		2.52609	2.53153	2.53167	2.49728	0.0	0.0	0.0	0.0	0.0	0	0	1.99	96	1.13	14	1.14	14		pre-loading
04/11/97	12:11	19	33.1	2.52428	2.52940	2.52938	2.49485	1.8	2.1	2.3	2.4	2.2	-0.16	-0.62	2.03	100	1.23	24	1.22	22	1.25	post-loading
04/11/97	15:30	218	33.1	2.52199	2.52739	2.52741	2.49301	4.1	4.1	4.3	4.3	4.2	-0.12	-0.17	2.04	101	1.25	26	1.22	22	1.50	
05/11/97	09:09	1277	33.1	2.52094	2.52609	2.52625	2.49185	5.1	5.4	5.4	5.4	5.4	0.02	-0.28								
05/11/97	15:15	1643	33.1												2.02	99	1.25	26	1.2	20	2.00	
06/11/97	10:30	2798	33.1												2.04	101	1.28	27	1.21	21	1.86	
06/11/97	12:30	2918	33.1	2.51957	2.52469	2.52477	2.49007	6.5	6.8	6.9	7.2	6.9	-0.06	-0.69								
07/11/97	10:30	4238	33.1												2.04	101	1.26	27	1.2	20	2.17	
07/11/97	11:30	4298	33.1	2.51937	2.52421	2.52446	2.48973	6.7	7.3	7.2	7.6	7.2	0.11	-0.83								
11/11/97	10:12	9980	33.1	2.51744	2.52231	2.52261	2.48757	8.6	9.2	9.1	9.7	9.2	0.16	-1.06								
11/11/97	13:00	10148	33.1												2.01	98	1.25	26	1.18	18	3.00	
10/12/97	10:00	51728	33.1	2.51366	2.51784	2.51816	2.48275	12.4	13.7	13.5	14.5	13.5	0.18	-2.1								
10/12/97	13:00	51908	33.1												2.07	104	1.26	27	1.2	20	2.17	
19/01/98	10:00	109328	33.1	2.51259	2.51666	2.51719	2.48136	13.5	14.9	14.5	15.9	14.7	0.39	-2.42								
19/01/98	13:00	109508	33.1												1.99	96	1.23	24	1.2	20	1.67	
01/04/98	12:00	213128	33.1	2.50947	2.51328	2.51372	2.47778	16.6	18.3	18.0	19.5	18.1	0.3	-2.88								
01/04/98	18:30	213518	33.1												2.11	108	1.27	28	1.2	20	2.33	
																					1.99	Average

3A 4.2 tonne
3B 4.3 tonne
TOTAL NEW BLOCK WEIGHT: 8.5 tonne
NEW FOUNDATION LINE LOAD: 51.9 kN/m



DATE	TIME	ELAPSED TIME (MINS)	NETT APPLIED LOAD (kN/m^2)	LEVELLING PINS - LEVEL 13 (m AOD)	LEVELLING PINS - LEVEL 14 (m AOD)	LEVELLING PINS - LEVEL 15 (m AOD)	LEVELLING PINS - LEVEL 16 (m AOD)	LEVELLING PINS - SETTLEMENT 13 (mm)	LEVELLING PINS - SETTLEMENT 14 (mm)	LEVELLING PINS - SETTLEMENT 15 (mm)	LEVELLING PINS - SETTLEMENT 16 (mm)	INCREMENT AVERAGE SETTLEMENT (mm)	TOTAL TRANSVERSE TILT (mm)	TOTAL DIFFERENTIAL SETTLEMENT (mm)	TOTAL AVERAGE SETTLEMENT (mm)	Cell 10 Glotzl box reading (Bar)	Cell 10 Stress (kPa)	Col 28 Glotzl box reading (Bar)	Col 28 Stress (kPa)	Col 28/29 Glotzl box reading (Bar)	Col 28/29 Stress (kPa)	Stress ratio	Comments
02/04/98	13:40	214668	33.1	2.50947	2.51328	2.51372	2.47778	0.0	0.0	0.0	0.0	0.0	0.3	-2.88	18.1								
02/04/98	13:40	214668	33.1													2.11	108	1.27	28	1.2	20	2.33	pre-loading
02/04/98	13:48	214676	67.8	2.50761	2.51114	2.51174	2.47569	1.9	2.1	2.0	2.1	2.0	0.46	-3.11	20.1								
02/04/98	13:45	214673	67.8													2.18	115	1.4	41	1.3	30	1.69	post-loading
02/04/98	17:43	214911	67.8	2.50626	2.50986	2.51057	2.47431	3.2	3.4	3.2	3.5	3.3	0.57	-3.14	21.4								
08/04/98	11:20	223168	67.8	2.50282	2.50586	2.50689	2.47000	6.6	7.4	6.8	7.8	7.2	0.89	-4.01	25.2								
08/04/98	15:30	223418	67.8													2.13	110	1.5	51	1.26	26	3.08	
18/05/98	11:19	280767	67.8	2.49683	2.49932	2.50069	2.46319	12.6	14.0	13.0	14.6	13.6	1.23	-4.83	31.6								
18/05/98	13:30	280898	67.8													2.13	110	1.56	57	1.26	26	3.58	post-loading
24/07/98	12:00	377288	67.8	2.49126	2.49337	2.49484	2.45744	18.2	19.9	18.9	20.3	19.3	1.33	-5.01	37.4								
24/07/98	12:00	377288	67.8													2.07	104	1.57	58	1.26	26	3.67	post-loading
07/09/98	19:00	442508	67.8	2.48983	2.49210	2.49351	2.45585	19.6	21.2	20.2	21.9	20.7	1.27	-5.17	38.8								
08/09/98	09:00	443348	67.8													2.1	107	1.61	62	1.28	28	3.43	post-loading
08/09/98	17:00	443828	0.0	2.49297	2.49589	2.49699	2.45929	16.5	17.4	16.7	18.5	17.3	0.96	-4.87	35.4								
08/09/98	18:00	443888	0.0													2.02	99	1.17	18	1.14	14	0.00	unloaded
																						3.09	Average

Figure A7.1.1: Trial footing 3. Response of instrumentation to load application.

Bothkennar Soft Soil Vibro Trial

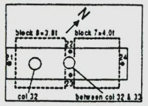
TEST PAD BEHAVIOUR

PAD No:	5	PAD LENGTH:	3.1 m
COLUMNS:	2	PAD WIDTH:	0.77 m
COLUMN SPACING:	1.5 m	BLOCK 7:	4 tonne
:PTH OF TREATMENT:	8.2 m	BLOCK 8:	3.8 tonne
COLUMN No's:	33,32	TOTAL BLOCK WEIGHT:	7.8 tonne
		FOUNDATION LINE LOAD:	24.7 kN/m

Associated instrumentation
Flatjack cell - Vert. stress on col 32
Cell zero 0.91 Bar
Box zero 0.11 Bar

Flatjack cell - Vert. stress between col 32 & 33
Cell zero 0.91 Bar
Box zero 0.11 Bar

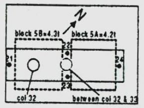
Mini cell No. 27 - Vert. stress @ 8.5m beneath col 32
Cell zero 0.56 Bar
Box zero 0.11 Bar



DATE	TIME	ELAPSED TIME (MINS)	NETT APPLIED LOAD (kN/m^2)	LEVELLING PINS - LEVEL 21 (m AOD)	22 (m AOD)	23 (m AOD)	24 (m AOD)	LEVELLING PINS - SETTLEMENT 21 (mm)	22 (mm)	23 (mm)	24 (mm)	INCREMENT AVERAGE SETTLEMENT (mm)	TOTAL TRANSVERSE TILT (mm)	TOTAL DIFFERENTIAL SETTLEMENT (mm)
03/11/97	11:00	0		2.55779	2.56155	2.55991	2.54736	0	0	0	0	0		
04/11/97	10:55	0		2.55847	2.56220	2.56035	2.54787	0.0	0.0	0.0	0.0	0.0	0	0
04/11/97	11:20	25	32.1	2.55658	2.56005	2.55851	2.54811	1.9	2.2	1.8	1.8	1.9	0.31	0.13
04/11/97	11:25	30	32.1											
04/11/97	12:20	85	32.1	2.55551	2.55902	2.55763	2.54534	3.0	3.2	2.7	2.5	2.8	0.46	0.43
04/11/97	15:29	274	32.1	2.55426	2.55761	2.55619	2.54419	4.2	4.6	4.2	3.7	4.2	0.43	0.53
05/11/97	09:20	1345	32.1	2.55312	2.55674	2.55540	2.54342	5.4	5.5	4.9	4.4	5.1	0.51	0.9
05/11/97	15:15	1700	32.1											
06/11/97	10:30	2855	32.1											
06/11/97	12:30	2975	32.1	2.55168	2.55519	2.55418	2.54175	6.8	7.0	6.2	6.1	6.5	0.84	0.67
07/11/97	10:30	4295	32.1											
07/11/97	12:30	4415	32.1	2.55117	2.55459	2.55362	2.54135	7.3	7.6	6.7	6.5	7.0	0.88	0.78
11/11/97	10:19	10044	32.1	2.54890	2.55251	2.55154	2.53933	9.6	9.7	8.8	8.5	9.2	0.88	1.03
11/11/97	13:00	10205	32.1											
10/12/97	10:00	51785	32.1	2.54322	2.54688	2.54608	2.53424	15.3	15.3	14.3	13.6	14.6	1.05	1.62
10/12/97	13:00	51965	32.1											
19/01/98	10:00	109385	32.1	2.54115	2.54471	2.54399	2.53180	17.3	17.5	16.4	16.1	16.8	1.13	1.25
10/12/97	13:00	51965	32.1											
01/04/98	12:00	213185	32.1	2.53639	2.54010	2.53938	2.52724	22.1	22.1	21.0	20.6	21.4	1.13	1.45
01/04/98	18:03	213548	32.1											

Cell 27 Glotzl box reading (Bar)	Cell 27 Stress (kPa)	Col 32 Glotzl box reading (Bar)	Col 32 Stress (kPa)	Col 32/33 Glotzl box reading (Bar)	Col 32/33 Stress (kPa)	Stress ratio	Comments
1.6	104	1.06	15	1.06	15		
1.61	105	1.06	15	1.06	15		pre-loading
1.61	105	1.22	31	1.16	25	1.60	post-loading
1.61	105	1.3	39	1.2	29	1.71	
1.6	104	1.35	44	1.14	23	3.62	
1.61	105	1.36	45	1.13	22	4.29	
1.61	105	1.36	45	1.13	22	4.29	
1.61	105	1.35	44	1.1	19	7.25	
1.59	103	1.33	42	1.15	24	3.00	
1.55	99	1.26	35	1.2	29	1.43	
1.52	96	1.31	40	1.15	24	2.78	
						3.33	Average

5A	4.2	tonne
5B	4.3	tonne
TOTAL NEW BLOCK WEIGHT:	8.5	tonne
NEW FOUNDATION LINE LOAD:	51.6	kN/m



USING SITE DATUM

DATE	TIME	ELAPSED TIME (MINS)	NETT APPLIED LOAD (kN/m^2)	LEVELLING PINS - LEVEL 21 (m AOD)	22 (m AOD)	23 (m AOD)	24 (m AOD)	LEVELLING PINS - SETTLEMENT 21 (mm)	22 (mm)	23 (mm)	24 (mm)	INCREMENT AVERAGE SETTLEMENT (mm)	TOTAL TRANSVERSE TILT (mm)	TOTAL DIFFERENTIAL SETTLEMENT (mm)	TOTAL AVERAGE SETTLEMENT (mm)
02/04/98	12:25	214650	32.1	2.53639	2.54010	2.53938	2.52724	0.0	0.0	0.0	0.0	0.0	1.13	1.45	21.4
02/04/98	12:25	214650	32.1												
02/04/98	12:40	214665	67.0	2.53481	2.53833	2.53815	2.52573	1.6	1.8	1.2	1.5	1.5	1.67	1.52	23.0
02/04/98	12:30	214655	67.0												
02/04/98	17:30	214955	67.0	2.53368	2.53702	2.53723	2.52464	2.7	3.1	2.2	2.6	2.6	2.06	1.56	24.1
08/04/98	12:30	223295	67.0	2.52995	2.53358	2.53455	2.52208	6.4	6.5	4.8	5.2	5.7	2.82	2.73	27.2
08/04/98	15:30	223475	67.0												
18/05/98	12:00	280865	67.0	2.52344	2.52727	2.52872	2.51632	13.0	12.8	10.7	10.9	11.8	3.3	3.48	33.3
18/05/98	13:30	280955	67.0												
24/07/98	09:20	377185	67.0	2.51713	2.52072	2.52275	2.51040	19.3	19.4	16.6	16.8	18.0	3.88	3.87	39.5
24/07/98	12:00	377345	67.0												
07/09/98	18:55	442560	67.0	2.51547	2.51909	2.52112	2.50896	20.9	21.0	18.3	18.3	19.6	3.88	4.09	41.1
08/09/98	09:00	443405	67.0												
08/09/98	16:25	443850	0.0	2.51861	2.52282	2.52419	2.51182	17.8	17.3	15.2	15.4	16.4	3.22	3.81	37.9
08/09/98	18:00	443945	0.0												

Cell 27 Glotzl box reading (Bar)	Cell 27 Stress (kPa)	Col 32 Glotzl box reading (Bar)	Col 32 Stress (kPa)	Col 32/33 Glotzl box reading (Bar)	Col 32/33 Stress (kPa)	Stress ratio	Comments
1.52	96	1.31	40	1.15	24	2.78	
1.54	98	1.54	63	1.25	34	2.53	post-loading
1.53	97	1.67	76	1.23	32	3.59	post-loading
1.53	97	1.73	82	1.22	31	4.19	post-loading
1.53	97	1.73	82	1.22	31	4.19	post-loading
1.48	92	1.77	86	1.22	31	4.44	post-loading
1.46	90	1.13	22	1.08	17	3.50	unloaded
						3.79	Average

Figure A7.1.2: Trial footing 5. Response of instrumentation to load application.

Bothkennar Soft Soil Vibro Trial

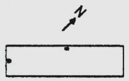
TEST PAD BEHAVIOUR

PAD No: 8
COLUMNS: 2
COLUMN SPACING: 1.5 m
DEPTH OF TREATMENT: N/A m
COLUMN No's: 38,39

PAD LENGTH: 3.05 m
PAD WIDTH: 0.75 m

BLOCK 1: 3.9 tonne
BLOCK 2: 4.1 tonne

TOTAL BLOCK WEIGHT: 8 tonne
FOUNDATION LINE LOAD: 25.7 kN/m



DATE	TIME	ELAPSED TIME (MINS)	NETT APPLIED LOAD (kN/m^2)	LEVELLING PINS - LEVEL 31 (m AOD)	32 (m AOD)	33 (m AOD)	34 (m AOD)	LEVELLING PINS - SETTLEMENT 31 (mm)	32 (mm)	33 (mm)	34 (mm)	AVERAGE SETTLEMENT (mm)	TRANSVERSE TILT (mm)	Comments
03/11/97	11.00	0		2.51102	2.50951	2.50928	2.50340	0	0	0	0	0		
04/11/97	10.03	0		2.51156	2.51007	2.50983	2.50394	0.0	0.0	0.0	0.0	0.0	0	pre-loading
04/11/97	10.19	16	34.3	2.50959	2.50725	2.50718	2.50095	2.0	2.8	2.7	3.0	2.6	0.17	post-loading
04/11/97	12.30	147	34.3	2.50834	2.50627	2.50617	2.50011	3.2	3.8	3.7	3.8	3.6	0.14	
04/11/97	15.12	309	34.3	2.50830	2.50619	2.50605	2.50009	3.3	3.9	3.8	3.8	3.7	0.1	
05/11/97	09.33	1410	34.3	2.50813	2.50593	2.50576	2.49979	3.4	4.1	4.1	4.1	3.9	0.07	
06/11/97	12.30	3027	34.3	2.50664	2.50465	2.50451	2.49854	4.9	5.4	5.3	5.4	5.3	0.1	
07/11/97	12.00	4437	34.3	2.50675	2.50484	2.50472	2.49865	4.8	5.2	5.1	5.3	5.1	0.12	
11/11/97	10.34	10111	34.3	2.50557	2.50389	2.50375	2.49795	6.0	6.2	6.1	6.0	6.1	0.1	
10/12/97	10.00	51837	34.3	2.50441	2.50321	2.50315	2.49810	7.1	6.9	6.7	5.8	6.6	0.18	
19/01/98	10.00	109437	34.3	2.50433	2.50356	2.50329	2.49863	7.2	6.5	6.5	5.3	6.4	-0.03	
01/04/98	12.00	213237	34.3	2.50220	2.50185	2.50164	2.49731	9.4	8.2	8.2	6.6	8.1	0.03	

8A: 4.2 tonne
8B: 4.5 tonne
TOTAL NEW BLOCK WEIGHT: 8.7 tonne
NEW FOUNDATION LINE LOAD: 53.7 kN/m

USING SITE DATUM

DATE	TIME	ELAPSED TIME (MINS)	NETT APPLIED LOAD (kN/m^2)	LEVELLING PINS - LEVEL 31 (m AOD)	32 (m AOD)	33 (m AOD)	34 (m AOD)	LEVELLING PINS - SETTLEMENT 31 (mm)	32 (mm)	33 (mm)	34 (mm)	AVERAGE SETTLEMENT (mm)	TRANSVERSE TILT (mm)	TOTAL AVERAGE SETTLEMENT (mm)	Comments
02/04/98	11.00	214617	34.3	2.50220	2.50185	2.50164	2.49731	0.0	0.0	0.0	0.0	0.0	0	8.1	
02/04/98	11.30	214647	71.6	2.50052	2.49937	2.49946	2.49449	1.7	2.5	2.2	2.8	2.3	0.3	10.4	pre-loading
02/04/98	17:03	214980	71.6	2.49315	2.49852	2.49822	2.49968	9.0	3.3	3.4	-2.4	3.4	-0.09	11.5	post-loading
08/04/98	13:00	223377	71.6	2.49819	2.49787	2.49695	2.49126	4.0	4.0	4.7	6.1	4.7	-0.71	12.8	post-loading
18/05/98	12:35	280952	71.6	2.49570	2.49338	2.49408	2.48787	6.5	8.5	7.6	9.4	8.0	0.91	16.1	post-loading
24/07/98	12:00	377397	71.6	2.49262	2.48997	2.49069	2.48469	9.6	11.9	10.9	12.6	11.3	0.93	19.4	post-loading
07/09/98	19:16	442633	71.6	2.49208	2.48931	2.49018	2.48417	10.1	12.5	11.5	13.1	11.8	1.08	19.9	post-loading
08/09/98	10:41	443558	71.6	2.49217	2.48916	2.49041	2.48426	10.0	12.7	11.2	13.1	11.8	1.456	19.9	pre 3rd loading

7A: 4.3 tonne
7B: 4.1 tonne
TOTAL NEW BLOCK WEIGHT: 8.4 tonne
NEW FOUNDATION LINE LOAD: 80.7 kN/m

DATE	TIME	ELAPSED TIME (MINS)	NETT APPLIED LOAD (kN/m^2)	LEVELLING PINS - LEVEL 31 (m AOD)	32 (m AOD)	33 (m AOD)	34 (m AOD)	LEVELLING PINS - SETTLEMENT 31 (mm)	32 (mm)	33 (mm)	34 (mm)	AVERAGE SETTLEMENT (mm)	TRANSVERSE TILT (mm)	TOTAL AVERAGE SETTLEMENT (mm)	Comments
08/09/98	10.41	443558	71.6	2.49217	2.48916	2.49041	2.48426	0.0	0.0	0.0	0.0	0.0	0	19.9	pre 3rd loading
08/09/98	10:51	443568	107.6	2.49046	2.48745	2.48835	2.48191	1.7	1.7	2.1	2.4	2.0	-0.346	21.8	post 3rd loading
08/09/98	15:11	443828	107.6	2.48902	2.48388	2.48448	2.48426	3.2	5.3	5.9	2.5	4.2	-0.646	24.1	post 3rd loading

4: 4 tonne
TOTAL NEW BLOCK WEIGHT: 4 tonne
NEW FOUNDATION LINE LOAD: 93.6 kN/m

DATE	TIME	ELAPSED TIME (MINS)	NETT APPLIED LOAD (kN/m^2)	LEVELLING PINS - LEVEL 31 (m AOD)	32 (m AOD)	33 (m AOD)	34 (m AOD)	LEVELLING PINS - SETTLEMENT 31 (mm)	32 (mm)	33 (mm)	34 (mm)	AVERAGE SETTLEMENT (mm)	TRANSVERSE TILT (mm)	TOTAL AVERAGE SETTLEMENT (mm)	Comments
08/09/98	15:11	443828	107.6	2.48902	2.48388	2.48448	2.48426	0.0	0.0	0.0	0.0	0.0	0	24.1	pre 4th loading
08/09/98	15:24	443841	124.8	2.48830	2.48070	2.48213	2.47140	0.7	3.2	2.3	12.9	4.8	0.83	28.8	post 4th loading
09/09/98	18:06	445443	124.8	2.47789	2.45711	2.45845	2.45502	11.1	26.8	26.0	29.2	23.3	0.74	47.4	post 4th loading
16/09/98	10:00	455037	124.8	2.46694	2.43612	2.43556	2.39985	22.1	47.8	48.9	84.4	50.8	-1.16	74.9	post 4th loading

Figure A7.1.3: Trial footing 8. Response of instrumentation to load application.