Sloping Rubble Marine Seawalls Founded on Soft Soil Improved by Stone Columns

Wilhelm S. Degen, Betterground (HK) Ltd., Hong Kong, +852 3628 3818, wdegen@mac.com
Chengbin Miao, Betterground (HK) Ltd., Hong Kong, +852 3628 3818, c.miao@betterground.hk
Eleonora Di Mario, Tony Gee & Partners LLP, formerly Benaim Ltd. (an AECOM company), Hong Kong, +852 3628 3818, eleonora.di-mario@tonygee.com
Matthew G. Mills, Benaim Ltd. (an AECOM company), Hong Kong, +852 2410 3825, matt.mills@aecom.com
Colin B. Turnbull, Benaim Ltd. (an AECOM company), Hong Kong, +852 2410 3828, colin.turnbull@aecom.com

ABSTRACT: In the past decade, the ‘zero-dredge’ philosophy has become the preferred option for the formation of marine seawalls and reclamations, compared to the partially or fully dredged options, as this solution presents several environmental advantages. Recent relevant project experience is presented herein in order to highlight key design principles and construction issues central to the successful construction and performance of a sloping rubble seawall founded on soft soil improved by stone columns. If designed and installed with proper quality control, correct treatment intensity and sensible construction staging, stone columns are a safe and cost effective method for seawall foundations. This paper provides guidelines with regards to the design, construction and quality control for the correct implementation of stone columns as a foundation element for sloping rubble marine seawalls.

INTRODUCTION

Benefits of sloping marine seawalls founded on stone columns

To suit the current environmental requirements in Hong Kong, recent projects are tending to favour zero-dredge solutions, such as the Hong Kong Boundary Crossing Facility and the forthcoming Three-Runway System. The installation of stone columns allows the underlying deposits, generally consisting of very soft to soft clay and silt (soft soil), to improve in strength to the extent that the seawall can be directly placed on the seabed. This negates the need to dredge the soft soil and, therefore, this ground improvement technique represents an ideal solution for zero-dredge projects.

Stone columns can also provide several other beneficial effects, including: accelerated drainage, reduced settlement, increased slope stability and soil reinforcement in the event of an earthquake. Stone columns, in contrast to competing cementitious methods, are a very ductile foundation method. This ductile behaviour is part of the design concept of stone columns in soft soil. Past projects located in Hong Kong and elsewhere, with an improved soft soil layer thickness of over 15m, have shown that stone columns can remain fully functional in terms of both drainage and horizontal reinforcement, even after settlements of over 3m and lateral displacements of up to 1m have occurred.

SEAWALL DESIGN PRINCIPLES

General design considerations

A critical design aspect for any seawall is the evaluation of its deformation behaviour (combined vertical and lateral displacements) and stability under horizontal loads induced by reclamation fill placement, including, in some cases, subsequent surcharge placement. This is particularly relevant when using a non-dredged approach where *in-situ* soft soil remains in place.

With regards to seawall construction, it is important to ensure that the *in-situ* soft soil provides sufficient horizontal confinement to the stone columns in order to ensure the stability in each stage. Such confinement is achieved through the consolidation of the soft soil under the overburden vertical load induced by the seawall, during and post seawall construction. This can be achieved by incorporating suitable ‘pause periods’. These pause periods should be specified at, both, the end of each filling stage during seawall construction and prior to proceeding with the fill placement for the reclamation. Further consideration about the evaluation of the required overburden pressure is provided in the following section “*Mechanism of interaction between soft soil and stone columns*”.

In the evaluation of the required pause periods, the designer should consider that a zone of reduced column permeability may develop at its perimeter due to mixing of the disturbed soft soil with the column material, when adopting the dry bottom feed method for stone column construction. A recent study (Weber et al., 2009) states that this ‘smear zone’ should be taken into account in the consolidation analysis, as smearing has a dominating effect on the consolidation behaviour of the improved soft soil. When an uncoupled consolidation analysis is performed, in terms of stability and consolidation, the smear zone may be accounted for in the consolidation assessment by reducing the diameter of the stone columns by 5% (Kirsch and Kirsch, 2010).

Within the main reclamation area behind the seawall, experience on various projects shows that the construction sequence and the methodology of fill placement have a significant influence on the horizontal displacement of the seawall. The designer is therefore recommended to plan a staged fill placement sequence, similar to that specified for the seawall construction, to ensure that the soft soil beneath the reclamation area is allowed to progressively consolidate and gain sufficient shear strength during construction. With regards to fill placement, it is not recommended to progressively fill perpendicular to the seawall as this practice will cause excessive lateral displacements of the seawall, i.e. filling should be progressed at as an oblique angle as practical to the seawall.

Recommended replacement factor for stone columns in soft soil

The first author has been involved in numerous stone column projects over the last 30 years. According to his experience, the most common application for stone columns has been in silty sands and sandy silts, with a design focus on liquefaction prevention, rather than slope stability. These projects have generally employed replacement factors (RF) (also known as ‘area’ or ‘replacement’ ratio) ranging between 8% and 20%, with the majority between 10% and 15%.

It should be noted that RFs in soft soil are generally higher when slope stabilisation is the primary objective. Although the range is similar, generally between 8% and 30%, the majority of such projects employ a RF between 15% and 25%. This correlates well with guidance given by Barksdale (1983).

Although projects implementing RFs below 15% may be successfully designed and constructed, they may also carry higher risks due to unforeseen challenges. It is therefore recommended that a RF>15% is employed for stone columns installed in soft soil. An example of installation grid pattern and relevant equations for the calculation of RF is given in Fig. 1 in Addendum 1.

Mechanism of interaction between soft soil and stone columns

To describe the distribution of the overburden vertical load induced by the seawall onto the *in-situ* soft soil and stone columns below, a stress concentration factor (n) is introduced (e.g. Priebe, 1995). The n factor is defined as the ratio between the stress in the stone columns over the stress into the soft soil. The combined shear resistance of the soft soil and stone columns can be analysed by considering a ‘stone

column unit cell' (Barksdale and Bachus, 1983) beneath the footprint of the seawall, as shown below in Fig. 2 in Addendum 1.
The combined shear resistance of the unit cell along a potential failure surface can be expressed as shown in equation 1:

$$\tau_{uc} = (\tau_s \cdot A_s + \tau_c \cdot A_c)/A_{uc} = (c_u \cdot A_s + \sigma'_c \cdot \cos^2\delta \cdot \tan\varphi'_c \cdot A_c)/A_{uc} \qquad [1]$$

where, τ_{uc} = shear resistance of composite system, τ_s = shear resistance of soil, A_s = area of soil, τ_c = shear resistance of stone column, A_c = area of stone column, A_{uc} = area of composite system, c_u = undrained shear strength of soil, σ'_c = effective vertical stress in stone column, φ'_c = friction angle of stone column material, and δ = angle of assumed failure cone in stone column.

Although the vertical effective stress in the soft soil (σ'_s) is not taken into account in the above equation, there is generally a linear correlation between the undrained shear strength and the effective vertical stress. Therefore, the contribution from the soft soil ($c_u \cdot A_s$) to the unit cell resistance can be directly correlated to the proportion of overburden stress transmitted to the soft soil. This contribution tends to decrease as *n* increases. Conversely, the contribution to the unit cell resistance from the stone columns ($\sigma'_c \cdot \cos^2\delta \cdot \tan\varphi'_c \cdot A_c$) will tend to increase as *n* increases. The combined shear resistance of the unit cell generally increases less than linearly as *n* increases. A typical trend of unit cell combined shear resistance for a replacement factor of 17% is shown below in Graph 2.1 (see Fig. 3 in Addendum 1).

The response of the seawall in terms of horizontal displacements has been studied by implementing a 3-dimensional finite element model with commercially available software (PLAXIS 3D). In this model, the overburden from the seawall, reclamation fill and its surcharge has been represented by surface loads. The variability of *n* has therefore been modelled by varying the ratio between the surface load applied on the stone columns and soft soil.

The behaviour under lateral load is predominantly governed by the mechanical properties of the clay and these mechanical properties are directly affected by *n*, as shown in Equation 1. The results of the sensitivity analysis (see Graph 2.2 in Fig. 3 in Addendum 1), correlate well with this understanding and show that the horizontal displacements of the seawall and underlying improved soft soil increase more than linearly as *n* increases.

An example of both the model geometry and resultant horizontal displacement contour plan are shown in Fig. 4 in Addendum 1. The contour plan shows that the largest horizontal displacements occur approximately within the top 10 to 15m below the seawall footprint.

These results highlight the significant influence that *n* can have on the behaviour of the seawall. In practice, *n* may vary depending on various factors including, but not limited to: 1) the stone column replacement ratio, 2) the ratio between stone column length and diameter (L/D), 3) the ratio between the stiffness of the stone column and soft soil, 4) the properties of geotextiles used i.e. at the base of the seawall, and 5) the degree of consolidation of the soft soil (Castro and Sagaseta, 2009).

Unduly large horizontal displacements of the seawall during the initial filling stages may lead to integrity loss of the stone columns and/or excessive shear strain within the soft soil. This, in turn, may cause the undrained shear strength of the soft soil to reduce to its residual minimum value and adversely affect the overall stability performance. Therefore, it is recommended that the designer carefully consider the consequences of implementing techniques where high *n* values may be expected, which could lead to a significant adverse impact on the horizontal displacements of a laterally loaded seawall.

Design recommendations for enhancing stone column performance

Generally, the top 5 to 10m of the soft soil show very low undrained shear strength values (c_u = 0 to 10kPa), limiting the lateral confinement provided to the stone columns during installation and during early stages of construction. To increase the confinement of the stone columns in the early stages of construction, the designer should consider pre-consolidation of the top 5 to 10m of soft soil by means of Pre-fabricated Vertical Drains (PVDs) combined with a preload of approximately 2 to 3m of aggregate fill. The pre-consolidation phase should aim to increase the undrained shear strength of the soft soil by approximately 5kPa, providing an improved layer with an undrained shear strength ranging from 5 to 15kPa prior to the stone columns installation.

The designer should also consider recommending the installation of stone columns in two phases, as shown above in Fig. 1 in Addendum 1. Phase one should have double spaced columns and then phase two would 'fill in the gaps'. The stone columns installed during the first stage will provide additional lateral confinement during the second stage of stone column installation, beneficially improving the quality of the secondary stone columns.

Based on the results shown in section *"Mechanism of interaction between soft soil and stone columns"*, the largest horizontal displacements below the seawall footprint are expected to occur approximately within the top 10 to 15m below the top surface of the soft soil. Further to pre-consolidation, it is recommended that the designer consider the installation of stone columns with a variable diameter, i.e. a larger diameter in the upper part of the soft soil layer to increase the combined stiffness of the soil where these larger displacements can be expected. Within the same stone column, experience shows that the variation can be as large as 50% i.e. 1.5m diameter in very soft layers to 75cm in the competent layers.

STONE COLUMNS BEST PRACTICE

Determination of treatment depth and refusal depth

The design depth of a stone column installation must be sufficient to:

a) Carry vertical loads to a lower bearing layer such as to minimize the static settlement of the stone column to acceptable levels;
b) Reinforce all layers through which a critical slope failure might otherwise occur; and
c) Avoid negative surface effects from soil liquefaction of any layer below the treatment depth.

The CPT is a very efficient tool to assist in the evaluation of the depth of treatment required in order to meet design targets. However, CPTs can only be undertaken at a limited number of locations and therefore the base of the soft soil must be inferred via Kriging interpolation or similar. The soil boundary between CPT locations will, almost, never follow this inferred line, which can result in areas of competent soil above and, worse, areas of soft soil below the target refusal level (see Fig. 5 in Addendum 1). Therefore, a too strict adherence to a CPT evaluated treatment depth limit can cause challenges on site. To compensate, feedback on soil stiffness given by the vibroprobe during penetration must be considered. It is recommended that the Engineer provides provision within the project specification to allow for the termination criteria to be modified given the actual subsurface ground conditions encountered on site during construction, i.e. a performance based specification.

In a recent project, a suitable termination criterion in a clayey alluvial soil was assessed by comparing the cone resistance data from a large sample of CPT locations with the actual depth of refusal for their adjacent stone columns. A typical comparison report is given in Fig. 6 in Addendum 1, which shows that all of the uncorrected cone resistance (q_c) values at refusal depth were much higher (over 2MPa) than the required 1.5MPa, detailed in the project specification.

The resistance of the soil can be indirectly measured by the amount of effort (measured by ampere) required for the vibroprobe equipment to progress over a certain depth. For this project, the Contractor utilised this correlation to propose and successfully implement the following guideline for refusal depth:

"Penetration beyond the CPT based contract toe level will be carried out as far as practical. However, due to the capacity of the equipment, the stone column will be, in any case, terminated at a level determined from three trials of penetration that yield less than 20cm advancement while observing an increase in the amperes measured up to a maximum of 1.75 times the amperes measured under free hanging conditions."

The implementation of this guideline is shown in a typical quality control report below (see Fig. 7 in Addendum 1).

Quality control of stone column

By their nature, offshore stone columns are difficult to inspect visually or evaluate with load testing and, therefore, digital data recording of key installation parameters is crucial. It is recommended that the quality control system should contain, as a minimum, the following features:

1. Log of the stone column installation taken at one second intervals;
2. Log of ampere and depth over time;
3. Log of air pressure in the stone placement tube; and
4. Shape of the stone column plotted over depth.

A comparison between the quality control reports for a stone column with an installation defect (bottleneck) at 9m depth is shown below, where readings are taken at one second intervals (Fig. 8 in Addendum 1) and 20 second intervals (Fig. 9 in Addendum 1). The shapes of the stone columns are given over depth on the right side of the figures.

These figures highlight the importance of specifying an appropriate frequency of logging steps. The defect can be easily seen in both the graph and shape plot when the readings are taken at one second intervals. However, it can be seen that this defect is lost in the 'noise' created by the infrequency of the 20 second interval logging. More details on offshore quality control equipment and procedures are given in Degen (2014).

The pressure in the gravel tube is an important aspect to consider when installing stone columns. As the lower sections of the stone columns are likely to be subject to significant pressures, the pressure in the gravel tube must be maintained at a high enough level to ensure adequate flow of stones. Any defect in the telescoped rig which results in pressure release will lead to an imperfection in the lower end of the columns. As such, measurement of the pressure in the gravel tube (common practice in deep offshore works) is essential, as it acts as the sole quality check for any such issue.

IN-SITU TESTING, INTRUMENTATION AND MONITORING

Observational approach

The behaviour of a seawall founded on improved soft soil is complex and can be difficult to predict. It is therefore recommended that an observational approach is adopted for the design and construction (as suggested in BS EN 1997-1:2004). The designer may consider introducing appropriate ‘check points’ in-between construction stages, where the behaviour of the seawall, the stone columns and the soft soil should be reviewed. To support these reviews, appropriate instrumentation should be installed to help assess whether the performance of the seawall is consistent with the theoretical models adopted in the design.

When adopting the observational approach, the designer should be contracted to compare instrumentation readings to pre-set design trigger levels (Alert, Action, Alarm levels) for each construction design stage. This process enables the designer to assess the performance of the seawall and allows appropriate remedial measures to be recommended and implemented in a timely manner.

Instrumentation and monitoring

When an observational approach is adopted, validation of the design assumptions relating to the stability and displacement of the seawall is considered critical when assessing whether to proceed with subsequent construction design stages. An instrumentation and monitoring plan comprising a range of geotechnical instrumentation, together with *in-situ* testing, is therefore required to adequately monitor the behaviour of the seawall during reclamation fill and surcharge placement. The gain in undrained shear strength is a significant indicator of the degree of consolidation that has occurred and whether or not the foundation soil has improved sufficiently to maintain seawall stability. It is recommended that verification of the undrained shear strength of the soft soil should be undertaken by direct measurement utilising in-situ Vane Shear Tests (VSTs), or similar. Experience on previous projects shows that VSTs can be successfully carried out between the stone columns, without significant issues to the contractor. In the reclamation area treated with PVDs, CPTs can also be adopted for the evaluation of the shear strength gain. Suitable calibration field tests should first be undertaken, particularly in order to select an appropriate cone factor (N_k) for the interpretation of the CPTs.

A typical layout for monitoring instrumentation is shown in Fig. 10 in Addendum 1. Sub-surface settlement marker plates and extensometers may be installed to monitor the settlement of the seawall and the reclamation. Within the seawall footprint, the interpretation of the settlement data may be used as an indirect method to evaluate both the degree of consolidation in the soft soil and the stress concentration factor.

Vibrating wire piezometers may be installed at different depths to monitor the pore water dissipation during the construction sequence. It should be noted, however, that the pore water pressure measured at the piezometric cell location is highly influenced by the radial distance of the cell from the adjacent drainage elements (i.e. stone columns and/or PVDs). In practice, verticality errors in both the instrumentation and drainage elements, along with the accumulated horizontal displacement of the soil mass, will likely result in this distance differing from the theoretical assumption. As these errors are difficult to quantify, and could significantly affect the evaluation of the consolidation process, it is not recommended to solely rely on piezometric data to control the rate of filling, the length of the pause periods or assess stability. Rather, this evaluation may be used as a supplemental check for the *in-situ* testing to demonstrate that the dissipation of pore pressures is in accordance with that predicted from the design models and to compare their response to the other instrumentation and testing.

Based on project experience, inclinometers are generally preferred over surface movement markers for the monitoring of horizontal displacements of the seawall as the maximum horizontal displacement of the seawall was generally recorded at the seabed level, i.e. below the installation depth of surface movement markers. It is also recommended to combine the inclinometers with extensometers as these provide a reliable method to define settlements per stratum. It should also be noted that surface movement markers are often so significantly disturbed by site construction activities that readings become unreliable. A summary of typical instrumentation, testing and monitoring schedule is provided below in Table 1.

Table 1. Typical instrumentation, testing and monitoring schedule

Type of Testing/ Instrument	Typical Testing/ Installation Level	Purpose of Testing/Monitoring	Typical Monitoring Frequency
In-situ Vane Shear Test	1m below seabed level and every 3m to the base of the soft soil layer	Verify strengths within the soft soil at the end of prescribed pause periods, prior to further placement of fill	Check points prescribed by the designer
Inclinometer/ Extensometer	From top of fill level to 5m (min.)within the competent soil stratum	Compare the performance of the seawall in terms of displacements with design prediction	Twice per week
Sub-surface Movement Marker Plates	Top of stone blanket and at intermediate fill levels	Validate the assumptions for the consolidation process prediction	Twice per week
Double Tip Vibrating Wire Piezometer	Piezometric cells equally spaced within soft soil layer	Evaluate the overall performance of the drainage system	Twice per week

RISK MANAGEMENT – MITIGATION AND REMEDIAL MEASURES

As with any engineering project, it is important to capture as many of the potential risks as early as possible in order to manage them effectively. This is particularly pertinent when undertaking work in soft soil, where the behaviour of the seawall can be difficult to predict and large deformations may be expected.

The degree of risk associated with the geotechnical hazards should be assessed by the designer and included in a project risk register. In order to reduce risks to acceptable levels, this risk register should highlight possible mitigation measures to be implemented before and/or during construction. The designer may also consider providing information on possible remedial measures that may be adopted if the monitoring results or overall performance of the work deviate from design predictions. Some of the key hazards that should be considered in the risk assessment, together with typical mitigation and remedial measures, are summarised below in Table 2.

Table 2. Hazards and Mitigation and Remedial Measures

Hazard	Mitigation and Remedial Measures
Unforeseen ground conditions	i. Ensure adequate ground investigation is undertaken prior to construction and recommend additional site investigation, as necessary.
Monitoring and/or *in-situ* testing deviates from the specification	i. Ensure instrumentation and *in-situ* testing is fully detailed in drawings. ii. Advise contractor of the potential impact and risks of not installing the instrumentation and undertaking the monitoring and *in-situ* testing as per the design drawings and schedules. iii. Promptly recommend replacement of instruments when their serviceability limit is approached i.e. by deformations induced by the surrounding soil.
Inconsistent *in-situ* testing and/or monitoring results	i. Compare results from different instruments/tests in nearby locations in order to establish which instruments/tests should be considered unreliable. ii. Undertake additional *in-situ* testing to validate anomalous results.
Construction not in accordance with the prescribed construction sequence	i. Ensure the construction sequence is adequately detailed in the drawings. ii. Advise contractor of the potential impact of not following the proposed construction sequence, notably liability and associated costs of failure or not satisfying limiting design criteria. iii. If discrepancies are identified, a detailed review of the impact on the work performance should be undertaken to decide whether it is considered appropriate to proceed to the next construction design stage and, if necessary, modify the construction sequence accordingly.
In-situ testing indicates that the improvement in the foundation soil is not in accordance with the design models	i. Detailed review of monitoring results and reanalysis to decide whether it is considered appropriate to proceed to the next construction design stage. ii. Increasing the duration of the surcharge/pause period(s) to allow the underlying soft soil to improve in strength before proceeding with the next construction design stage. iii. Increasing the surcharge fill heights to increase the effect of consolidation and improvement in shear strength.
Monitoring results are not in accordance with the design models, or results reach/exceed, the relevant pre-set instrumentation trigger (AAA) levels earlier than predicted	i. Installation of additional PVDs to enhance the consolidation process. ii. Provision of a remedial toe berm to provide an increased stabilising force. iii. Installation of additional ground improvement beyond the seawall to increase the stability of the edge of the reclamation.

CONCLUSION AND RECOMMENDATIONS

Recent projects show that sloping rubble marine seawalls can be reliably founded on soft soil improved by stone columns. To successfully design and construct this type of seawall, the following key elements should be considered:

Design

- Confinement of the stone columns – sufficient confinement of the stone columns must be achieved through pre-consolidation and a staged construction design sequence of the seawall.
- Pause periods – in order to achieve required strength gains in the soft soil, thorough assessment of the required pause periods should be incorporated into the design along with proper staging of the works.
- Filling placement – filling direction should be progressed at as an oblique angle as practical to the seawall to reduce the lateral deformation of the seawall.
- Soil smearing – smearing of the soil immediately surrounding the stone column must be accounted for in the design.
- Stress concentration factor – it is not recommended to use techniques which increase the stress concentration factor as the potential for large horizontal displacements also increases.
- Variable column diameter – the stone column diameter should increase in softer layers and reduce in more competent layers.

Stone Column Construction

- Logging of key parameters – during installation amperes (i.e. effort of vibroprobe equipment), depth, gravel tube pressure and gravel consumption should be logged as a minimum. The maximum interval in logging steps should be one second; otherwise defects in the columns may remain undetected.
- Refusal depth of stone columns – the refusal depth should not rely solely on pre-construction data, such as CPTs. It has been shown that refusal depths can be reliably determined with vibroprobe ampere readings, which correlate well with soil shear strength.

Observational Approach

- Observational approach – this method should be adopted for seawalls installed in soft soils.
- Instrumentation and Monitoring – an appropriate suite of instrumentation and monitoring should be specified, which must complement the original design assumptions such that they may be compared with the observed behaviour of the seawall and reclamation.
- *In-situ* testing – adequate testing to verify strengths should be undertaken within the soft soil at the end of prescribed pause periods.
- Pre-set displacement trigger (AAA) levels – the vertical and lateral displacement trigger levels should be evaluated prior to the works for the seawall and regularly checked against the monitored displacements during construction.

Risk Management

- Risk register – a risk register should be adopted as part of the design, with clearly identified mitigation and remedial measures that could be implemented in a timely manner, as required.

REFERENCES

Barksdale R.D. and Bachus R.C., 1983. Design and construction of stone columns. Federal Highway Administration, RD-83/026.

Castro J. and Sagaseta C., 2009. Influence of stone column deformation on surrounding soil consolidation. Geotechnics of Soft Soils – Focus on Ground Improvement. Taylor & Francis Group, London, pp. 333-338.

Kirsch K. and Kirsch F., 2010. Ground improvement by deep vibratory methods. Spon Press, London and New York, pp.131.

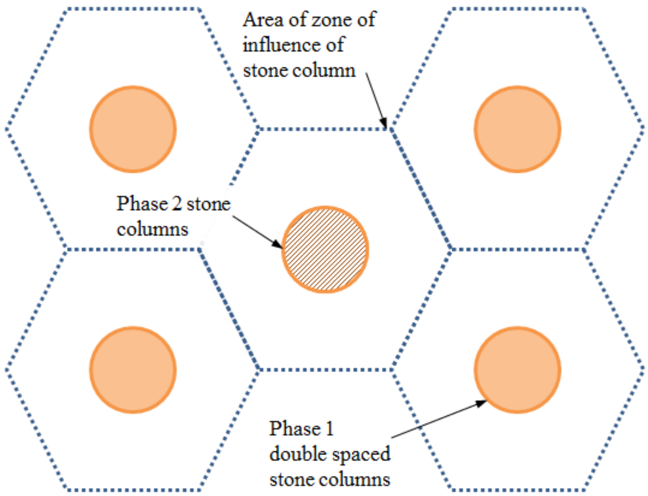
Weber T.M., Springman S.M., Gäb M., Racansky V. and Schweiger H.F., 2009. Numerical modelling of stone columns in soft clay under an embankment. In Karstunen & Leoni (eds), Geotechnics of soft soils – focus on ground improvement. Taylor & Francis Group, London, pp. 305-312.

Priebe, H.J., 1995. The design of vibro replacement. Ground Engineering, London, pp. 31-37.

Degen, W. 2014. Advances in equipment and quality control for offshore and onshore stone columns and vibro compaction. 18th Annual National Conf. on Geotech. Engineering, Jakarta, pp. 251- 255.

BS EN 1997-1:2004, 2004. Eurocode 7: Geotechnical design - Part 1: General rules. European Committee for Standardization, pp. 36.

ADDENDUM 1 – FIGURES



For a Triangular Grid:

$$RF = \frac{Ac}{As} = \left(\frac{\pi d^2}{2\sqrt{3}s^2}\right) \sim 0.91 \left(\frac{d^2}{s^2}\right)$$

For a Square Grid:

$$RF = \frac{Ac}{As} = \pi \left(\frac{r}{s}\right)^2$$

Where:

Ac = Area of stone column
As = Area of zone of influence
d = Column diameter
s = Grid spacing

Fig. 1 Example installation phasing and triangular grid pattern with relevant equations for calculation of RF

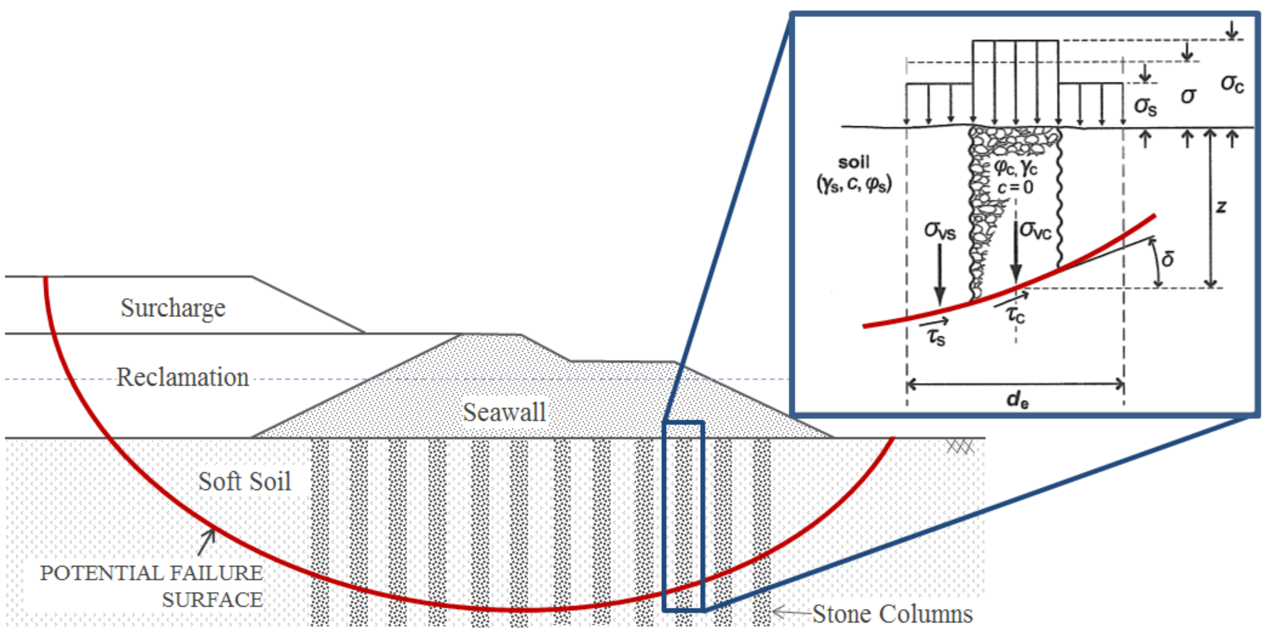


Fig. 2 Stone column unit cell concept (Barksdale and Bachus, 1983)

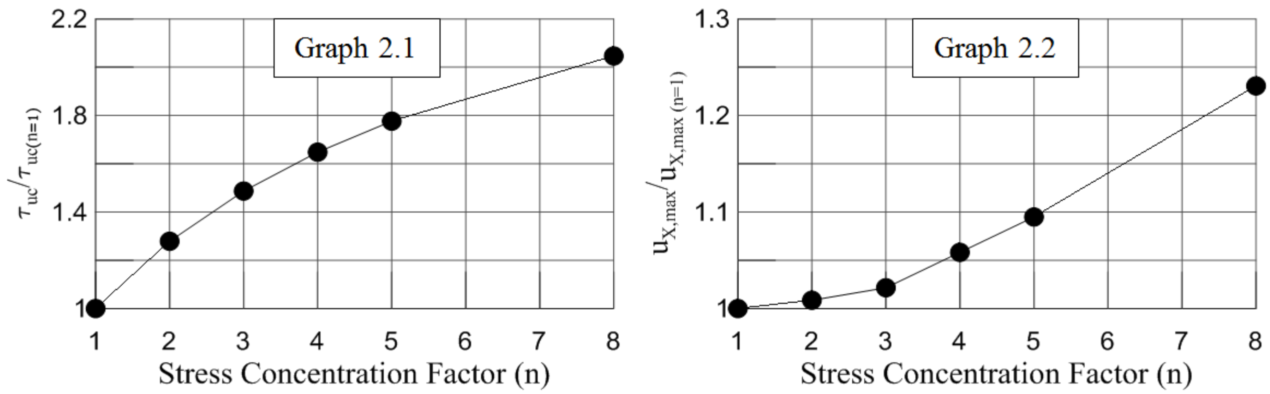


Fig. 3 Normalised unit cell strength (τ_{uc}) and maximum horizontal displacement ($U_{x,max}$) versus *n*

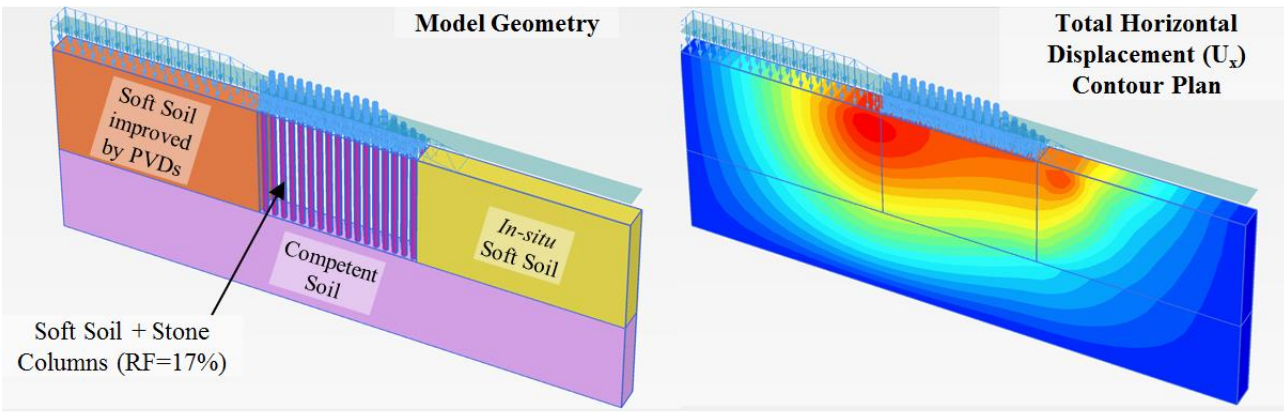


Fig. 4 Geometry of the PLAXIS 3D model and typical horizontal displacement (U_x) contour

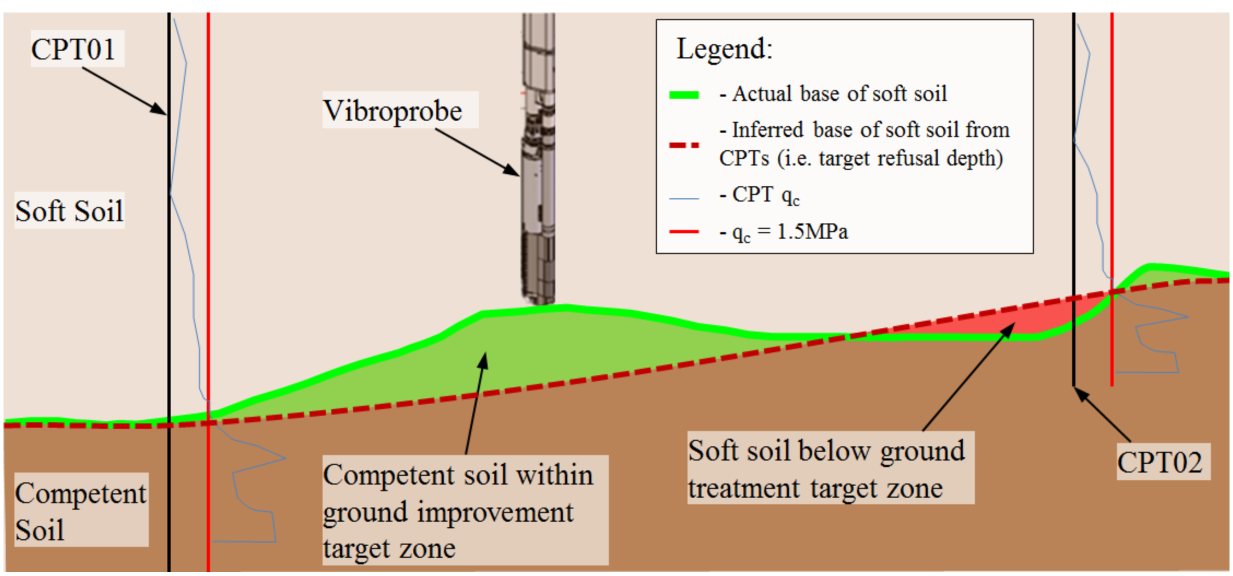


Fig. 5 Comparison of refusal depth in design and in practice

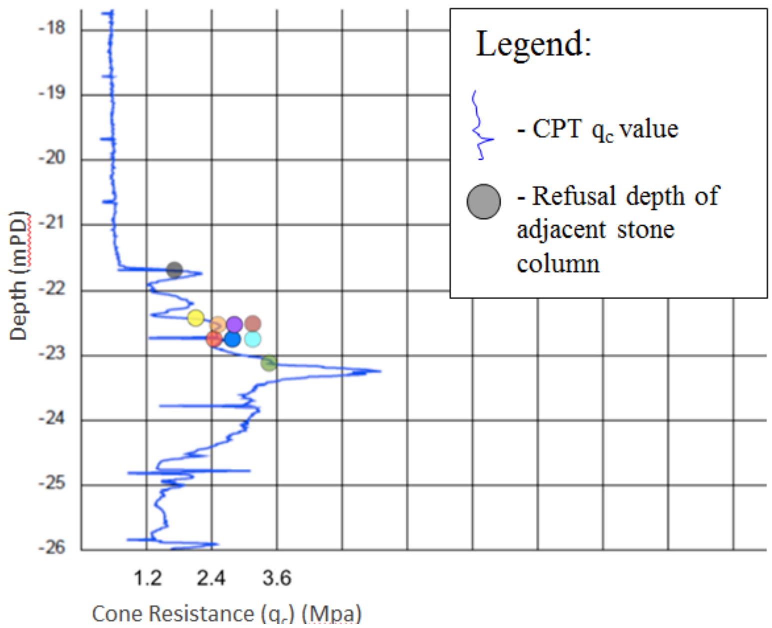


Fig. 6 CPT q_c plot against depth with actual refusal depths achieved in adjacent stone columns

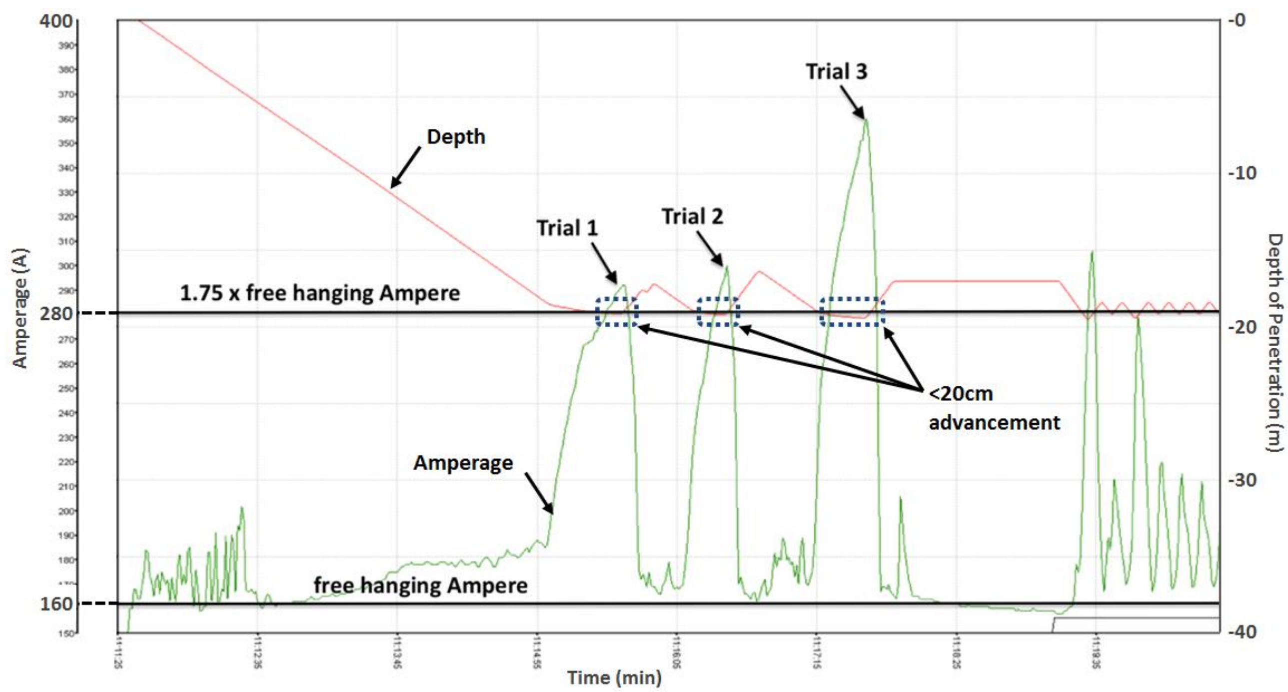


Fig. 7 Penetration stage in a typical quality control report for stone column installation

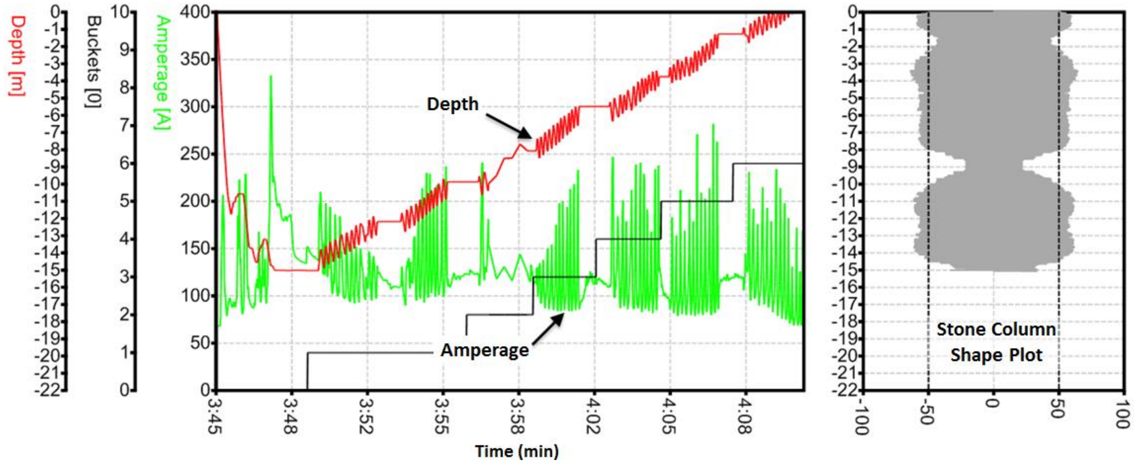


Fig. 8 Quality control report "record per second"

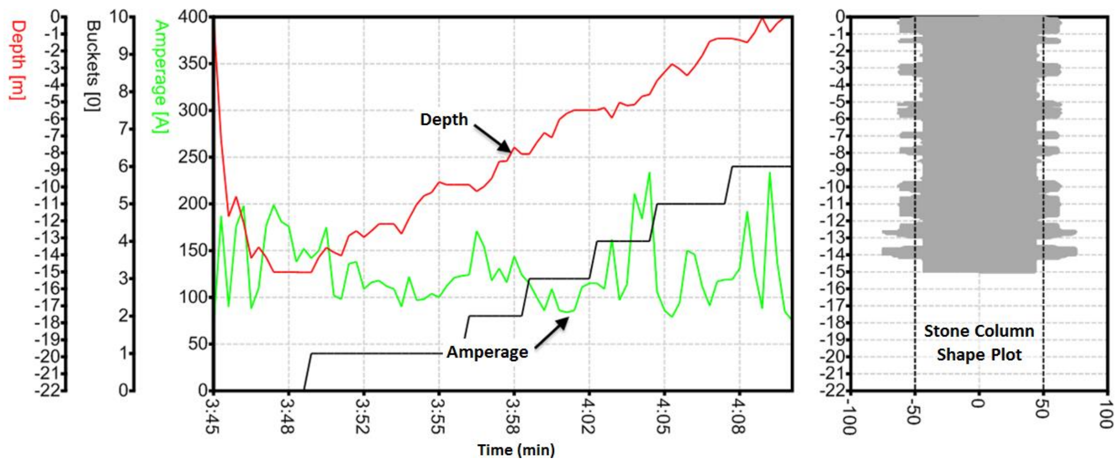


Fig. 9 Quality control reports "record per 20 seconds"

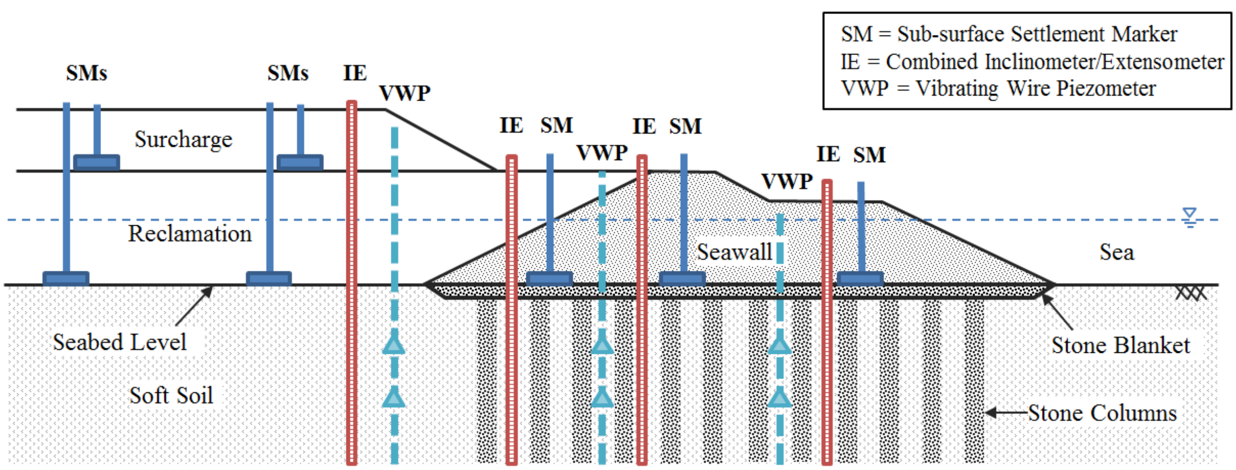


Fig. 10 Typical layout of monitoring instrumentation