The paper was published in the proceedings of the 11th Australia New Zealand Conference on Geomechanics and was edited by Prof. Guillermo Narsilio, Prof. Arul Arulrajah and Prof. Jayantha Kodikara. The conference was held in Melbourne, Australia, 15-18 July 2012.

Performance of Reinforced Soil Wall Supported by Stone Column

S. K. Tan[1], MIEAust and S. S. Liew[2], MIEM

[1]WorleyParsons Pty Ltd, Level 7, 250 St Georges Tce, WA 6000, Perth, Australia.; PH (08) 6311 6109; FAX (08) 9278 8110; email: sukwong.tan@worleyparsons.com
[2]G&P Geotechnics Sdn Bhd, 39-5 Jalan 3/146, Bandar Tasik Selatan, 57000 Kuala Lumpur, Malaysia.; PH (03) 9059 5396 ; FAX (03) 9059 5869; email: ssliew@gnpgeo.com.my

ABSTRACT

A 10m high reinforced soil (RS) wall has been designed to retain an access road and also the building platform for a mixed development located over a valley terrain. Within the lower part of the valley where the wall is located, the unfavourable ground conditions consisting of an approximately 12m thick compressible alluvial deposits overlying the stiff granitic residual formation was revealed during the subsurface investigation. In order to support the high wall vertically and laterally, stone columns were adopted as a ground treatment due to its economy reason. This paper demonstrates the design aspects of the stone column treatment as a composite treatment for the unfavourable ground conditions, the construction QA/QC measures and the verification of the performance via a comprehensive instrumentation scheme. From the instrumentation results, the inclinometer and extensometer installed at the edge of the wall shows minor lateral squeezing and settlement of the subsoil within the influence depth of the RS wall. However, as an overall performance, the deformation of the wall as a result of the lateral and settlement movements of the supporting ground is satisfactory indicating the effectiveness and economy of this proposed solution.

Keywords: reinforced soil wall, stone column, ground improvement

1 INTRODUCTION

The proposed development consists of four (4) blocks of high- rise condominium and office blocks with an adjoining multi storey carpark structure over about 5 acres of land. The building platform and access road to the entrance of the development necessitates the construction of a 10m high reinforced soil wall located over a valley terrain with deposition of compressible alluvial soils. The length of the proposed reinforced soil wall is approximately 180m. In order to support the high wall with vertical and lateral stability in the unfavourable ground conditions, stone columns as a ground treatment have been proposed and adopted as the foundation of the reinforced soil wall due to its technical and economical reasons.

The elevation of the reinforced soil wall in the proposed development are shown in Figure 1.

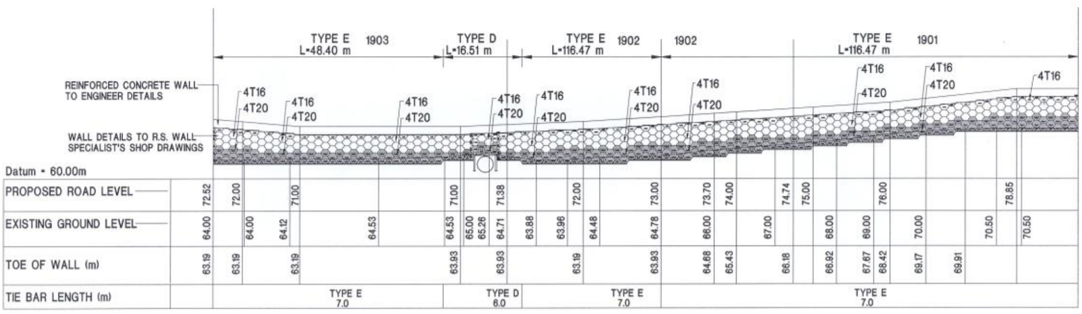


Figure 1. RS Wall Elevation

Installation of stone columns is one of the most common techniques that can be adopted for improvement on compressible soils. Stone columns provide the following primary functions:

- reinforcement of weak subsoil
- drainage for dissipation of excess pore pressure generated in subsoil under loading
- improving strength and deformation properties of soil in post installation

- increasing unit weight of in situ soil and acting as a strong stiff elements carrying higher imposed stresses

2 SITE CONDITION OF CASE STUDY

2.1 Subsoil Conditions

Subsurface investigation (S.I.) was carried out to establish the subsurface conditions for the mixed development. The S.I layout is presented in Figure 2. However, only the relevant S.I field works along the proposed reinforced soil wall were selected to present as follows:

- Four (4) exploratory boreholes
- Total of Seventy Seven (77) Mackintosh Probe (MP) Test
  (i) 60 for determination of stone column extent (not shown in the layout)
  (ii)17 along E-wall
- Four (4) Piezocone Penetration Tests (CPTu)

Based on the above field works carried out, the overburden materials of this area mainly consist of sandy silt and sandy clay. The profiles interpreted from the boreholes, MPs and CPTus along the reinforced soil wall show compressible sandy clay material with thickness varying between 5m and 12m, in which settlement and bearing instability of the RS wall can be expected. Due to the varying thickness of compressible subsoil, the geotechnical soil model is simplified for analysis purposes. 9.5m and 3.0m thick clay layers (Layer 1 and Layer 2) with underlying bedrock has been adopted as the design soil profile. Due to the contour of the original topography, surface runoff is expected to flow towards the natural valley where the reinforced soil wall is located. This area is within the proximity of the previous water stream before channelisation and therefore weak deposits are not uncommon.

Some of the MPs were carried out to determine the extent of ground treatment using stone columns while CPTu were carried out to determine the continuous strength profile and coefficient of consolidation to estimate settlement with and without stone columns.

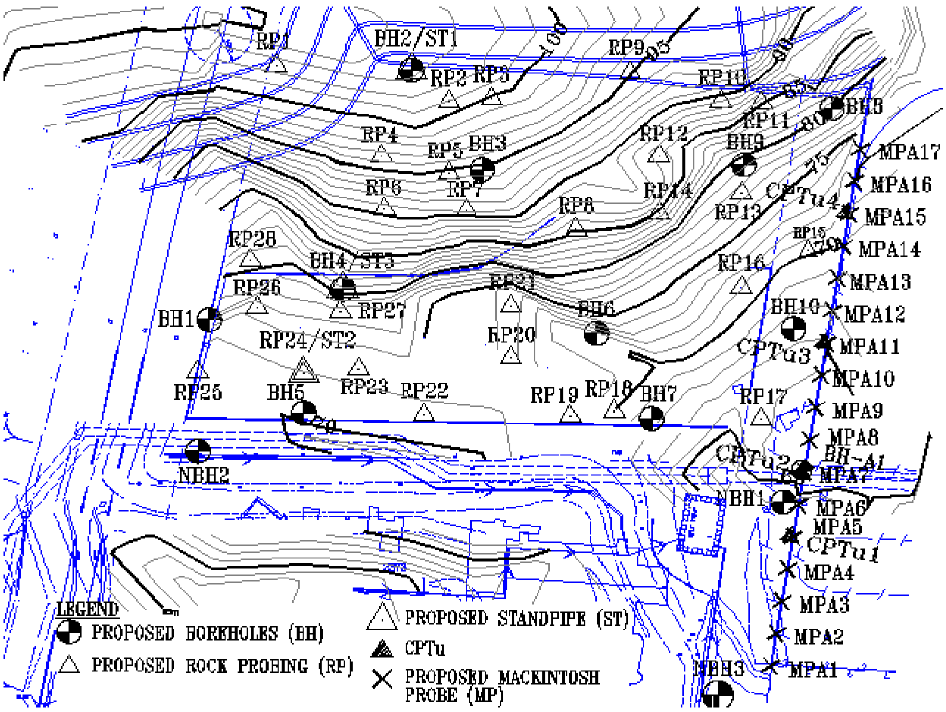


Figure 2. Layout of Exploration Works

2.2 Soil Parameters

Based on field tests (e.g. MP and CPTu) and laboratory tests (e.g. Isotropically Consolidation Undrained (C.I.U) tests, oedometer tests), a summary of the interpreted soil parameters is shown in Table 1 and is adopted for Finite Element Analyses (FEA).

Table 1: Values of Soil Parameters Adopted

Soil Description	Soil Model	Bulk Density γ_b (kN/m^3)	E' (MPa)	c' (kPa)	ϕ'	c_v (m^2/yr)
Stone Column	MC - Drained	22	120	0	40	-
Crusher Run	MC – Drained	22	120	0	38	-
RS Wall Backfill	MC – Drained	20	20	0	36	-
Compacted Backfill	HS – Undrained	20	16	5	31	-
Clay Layer 1	HS – Drained	15	8	2	22	12
Clay Layer 2	HS – Drained	15	14	3	23	12
Medium Stiff Soil	HS - Drained	19	20	5	31	12
Bedrock	NP – Non Porous	22	540	250	30	-
Pavement (ν = 0.2)	LE – Non Porous	22	86	-	-	-

MC – Mohr Coulomb, HS – Hardening Soil, NP – Non Porous, LE – Linear Elastic

3 STONE COLUMN DESIGN

Based on the available subsoil profile information, stone columns of 1m diameter with a 2m centre to centre spacing have been proposed and designed as the supporting foundation with stone column length up to 10m.

In the design of the stone column, the following design considerations were taken into account :-

1. Bulging of individual stone columns
2. General shear of stone columns
3. Stress distribution between stone columns and cohesive soil
4. Bearing capacity of subsoil and stone columns respectively
5. Global stability of RS wall
6. Overall ground settlement after improvement

Table 2 presents a summary of design check based on the abovementioned design considerations.

Table 2: Summary of Stone Column Design Check

<table>
<tr><th>Aspect</th><th>References</th><th>Comment</th></tr>
<tr><td>Bulging</td><td>Greenwood (1970); Vesic (1972); Datye & Nagaraju (1975); Hughes and Withers (1974); Madhav et al. (1979).</td><td rowspan="2">• Since the ultimate bearing capacity of stone column is highly dependent on the subsoil strength, the gain in strength of the subsoil during the construction stage is taken into consideration when checking the adequacy of the bearing capacity of a stone column against bulging and general shear.
• The design by the specialist contractor only used Priebe's (1995) method to check the settlement of the subsoils treated with stone columns. It is recommended to use more refined analysis to examine all aspects of possible failure mechanisms.
• Due to large variation in the range of possible ultimate bearing capacity when using different methods, load instrumentation monitoring had been carried out to verify the design performance. Load tests and instrumentation monitoring results would be presented in Sections 4 and 5 respectively.</td></tr>
<tr><td>General Shear</td><td>Madhav & Vitkar (1978); Wong (1975); Barksdale and Bachus (1983).</td></tr>
<tr><td>Stress Distribution between stone columns and cohesive soil</td><td rowspan="2">Bergado et al (1991)</td><td>• The stress distribution between granular piles and clay has been determined and used to check the Factor of Safety against bearing failure.</td></tr>
<tr><td>Bearing Capacity</td><td>• The ultimate bearing capacity for a single isolated pile given by Bergado et al (1991) is expressed as follows:
$q_{ult} = c \times N'_c$ (1)
where
c = undrained shear strength of clayey subsoil (kPa)
N'_c = composite bearing capacity factor for the granular pile which ranges from 15 to 18 for Bangkok clay.
➢ Estimated average subsoil undrained shear strength = 60kPa
➢ Ultimate bearing capacity of stone column = 960kPa
Verification test was carried out to ascertain the bearing capacity of stone column.</td></tr>
<tr><td>Global Stability</td><td>Aboshi et al., 1979; Barksdale, 1981</td><td>• Internal stability of RS wall was checked by proprietary specialist.
• The global stability of the RS wall supported by stone column was checked by the authors using average shear strength method. The</td></tr>
</table>

Aspect	References	Comment
		average shear strength method is widely used in stability analysis for sand compaction piles. • The FOS of RS wall is found to be adequate.
Ground Settlement after Improvement	Priebe (1995)	• The specialist contractor used Priebe's (1995) methods to check on the settlement of the subsoil treated with stone columns. ➢ The estimated settlement before improvement ≈ 650mm ➢ The estimated settlement after improvement ≈ 250mm to 280mm.

4 QUALITY ASSURANCE AND QUALITY CONTROL (QA/QC) DURING CONSTRUCTION

In order to obtain good construction quality of stone columns, the following considerations pertaining to the QA/QC have been taken into account:

- grading of durable stone aggregates to be within allowable grading envelope
- verification tests (plate load test)
- termination criteria of stone column installation

It was specified in the specification that the stones used shall be of clean, hard, durable and chemically inert natural materials so as to remain stable during column construction and working life in the anticipated ground water conditions. Table 3 presents the specification of stone aggregate respectively adopted by the authors.

According to the method statement provided by the specialist contractor, the compaction of the stones is deemed adequate if the hydraulic pressure in the vibratory probe rises to about **190 bars** as recorded by the recording device. This recommended pressure was verified against the proposed performance criteria at site during the construction of first / trial column by plate load test and was subsequently used as a basis for the construction of the subsequent stone columns. The quality control records during the installation of stone columns showed that columns with adequate compaction levels can be formed in alluvial deposits.

Table 3: Stone Aggregate Specification for Wet Method

Test	Standard	Criteria	Frequency
Crushing Value	BS 882:1992	<30%	1 test per 30,000 tonnes of aggregate
Los Angeles Abrasion	ASTM C131	Max loss of 40% at 500 revolutions	
Flakiness Index	BS 882:1992	<30%	
Sulphate Soundness	ASTM C88	<12%	

Plate bearing test was recommended to be carried out to directly verify the load bearing capacity and settlement behaviour of the stone column. The proposed ground treatment consists of a square grid pattern with 2m centre to centre column spacing.

As shown in Table 2, the ultimate capacity of stone columns is assessed to be 960 kPa. A plate size of 1m x 1m was used for the plate bearing test. As such, the stone column was supposedly loaded up to 960 kN. However, the actual test load imposed on the ground was 900kN due to site miscommunication. Nevertheless, the maximum settlement of stone column is approximately 110mm from the plate load test results. It is believed that the plate load test results will give a more optimistic settlement performance as the plate size is smaller than the effective equivalent treatment area for a stone column and the disperse effect of load into the treated ground will tend to under-stress the treated soil, thus leading to lesser settlement. The anticipated total settlement after treatment is in the range of 250mm to 280mm. As such, the performance of the stone column is considered satisfactory.

5 MONITORING INSTRUMENTATION SCHEME

The instrumentation programme was carried out with the following objectives:-
(i) to monitor the performance of the stone column.
(ii) to foresee any potential instability so that remedial works can be timely carried out if necessary.
The monitoring instrumentation scheme comprised of displacement settlement markers along RS wall panels and an inclinometer at CH84.6. The inclinometer has 3-level magnetic extensometers and was installed in front of the RS wall.

5.1 Vertical Settlement and Lateral Displacement Profiles

Figure 3 shows that RS Wall has settled approximately 115mm between 29th July 2004 and 30th November 2004 over a period of 4 months, which is still far lower than the expected settlement envisaged in the design calculation. This shows that the stone column is effective in facilitating the drainage within the clay since the rate of settlement seems to have stabilised within 6 weeks after the completion of RS wall. Since the settlement still occurred at the last monitoring date, it had been recommended to take another settlement reading to ensure that settlement trend has stabilized and does not pose further differential settlement and total settlement problems. However, this has yet to be carried out and is likely to be done at the end of the building construction. Nevertheless, the settlement is expected to have stabilised and the distortional movement to be within the allowable limit as there is no observable distress on the completed road finishes and utility services.

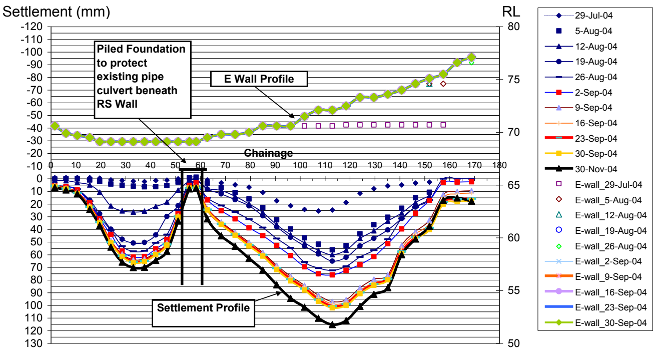


Figure 3. Settlement Displacement Marker Profile With Time

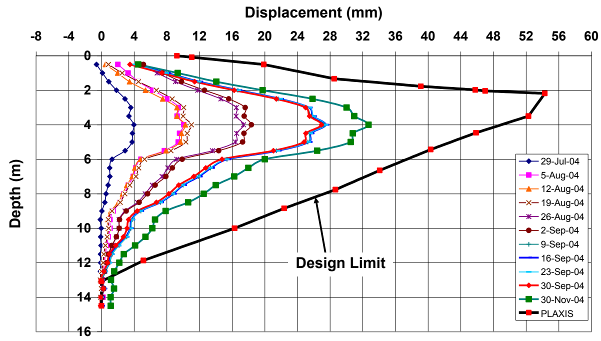


Figure 4. Lateral Displacement Profile at Major Axis (A-A) with Time

Figure 4 shows that the maximum lateral displacement in the major axis (A-A) is approximately 33mm at the depth of 4m below ground level. The rate of lateral displacement in the A-A axis is 0.1mm/day, indicating that the movement has stabilised. The inclinometer monitoring results are within the acceptable limit as interpreted from finite element analyses using PLAXIS. Therefore, it can be concluded that the stone column design is effective.

5.2 Magnetic Extensometer

It was observed from Figure 5 that the settlement at extensometer SM2 is larger than that of extensometer SM1. SM1 and SM2 are located approximately at the depth 0.9m and 3.8m below ground level respectively. The fact that SM1 settled less than extensometer SM2 is possibly due to localised upheaving of the soil mass above SM2. As can be seen from inclinometer reading in the major axis (A-A), there is a large lateral movement at the depth of 4m below ground level. This agrees with the hypothesis that localised upheaving occurred between the ground level and 4m below ground level. The upheaving explains the reason of SM1 settled less than SM2. Nevertheless, the results are within the acceptable limit as interpreted from the finite element analyses using PLAXIS.

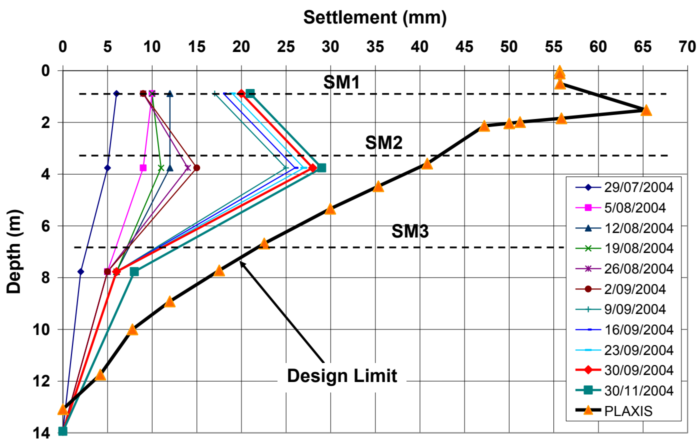
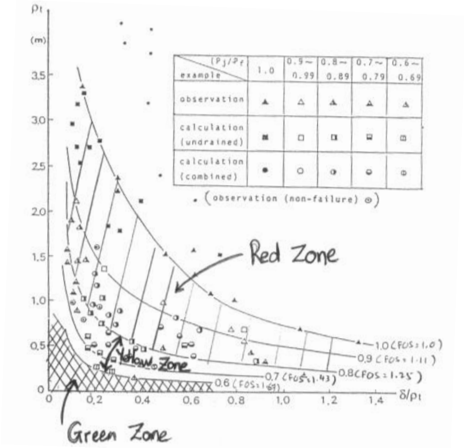


Figure 5. Settlement Profile of Extensometer



$\delta \approx 33mm$
$\rho_t \approx 115mm$

Figure 6. (δ/ρ_t) Diagram with Inverse Factor of Safety (After Matsuo et al, 1977)

In order to monitor the performance of the stone column in stability aspect, a diagram showing the factor of safety of the embankment (in this case, reinforced soil wall) for any given settlement (p_t) and lateral displacement (δ) by Matsuo (1977) is adopted. The diagram is presented in Figure 6. The **green zone (**FOS>1.67) indicates no stability issues while the **red zone** (FOS<1.25) indicates that action is required to mitigate the stability problem. The **yellow zone** indicates the transition zone between the green zone and the red zone in which contingency measures should be in place for implementation should the monitoring results reach the red zone.

6 CONCLUSION

In summary, the following aspects should be considered when designing stone column to support heavy structures especially in unfavourable ground conditions:-

1. Design aspect (check on bulging, general shear, bearing capacity, global stability, settlement) rather than using the simplified method such as Priebe's method.
2. Quality Assurance and Quality Control during Construction (stone grading, verification tests, proper termination criteria).

Observational method via comprehensive instrumentation schemes at strategic locations coupled with Finite Element Analysis should be used to compare design predictions with field performance to verify the performance of the structure and ensure safety.

REFERENCES

Aboshi, H., Ichimoto, E., Enoki, M. and Harada, K. (1979). "*The Compozer- A Method to Improve Characteristics of Soft Clays by Inclusion of Large Diameter Sand Columns*." Proc. Intl. Conf. on Soil Reinforcement: Reinforced Earth and Other Techniques, *Vol. 1*, Paris, 211-216.

Barksdale, R.D. (1981). "*Site Improvement in Japan Using Sand Compaction Piles*." Georgia Institute of Technology, Atlanta.

Barksdale, R.D. and Bachus, R.C. (1983). "*Design and Construction of Stone Columns*." Report No. FHWA/RD-83/026, National Technical Information Service, Springfield, Virginia, USA.

Datye, K.R. and Nagaraju, S.S. (1981). "*Design Approach and Field Control for Stone Columns*." Proc. 10th Intl. Conf. On Soil Mech. Found. Eng'g., Stockholm.

D.T. Bergado, M.C. Alfaro and J.C. Chai (1991). "*The Granular Pile: Its Present State and Future Prospects for Improvement of Soft Bangkok Clay*." Geotechnical Engineering Vol. 22, 143-175.

Greenwood, D.A. (1970). *Mechanical Improvement of Soils Below Ground Surface*. Proc. Ground Eng'g. Conference, Institute of Civil Eng'g, 9-20.

Hughes, J.M.O. and Withers, N.J. (1974). "*Reinforcing soft cohesive soil with stone columns*." Ground Engineering. Vol. 7, No. 3, 42-49.

Madhav, M.R. and Vitkar, R.P. (1978). "*Strip Footing on Weak Clay Stabilized with Granular Trench or Pile*." Canadian Geotechnical Journal, Vol. 15, No. 4, 605-609.

Madhav, M.R., Iyengar, N.G.R., Vitkar, R.P. and Nandia, A. (1979). "*Increased Bearing Capacity and Reduced Settlements due to Inclusions in Soil*." Proc. Intl. Conf. On Soil Reinforcement : Reinforced Earth and Other Techniques, Vol. 2, 239-333.

Matsuo, M. and Kawamura, K. (1977). "*Diagram for Construction Control of Embankment on Soft Ground*. Soils and Foundations," Vol.22, No.4, 45-56

Priebe, H.J. (1995). "*The design of vibro replacement*." Ground Engineering, December, 31-37.

Wong, H.Y. (1975). "*Vibroflotation – Its Effect on Weak Cohesive Soils*." Civil Engineering (London), No. 82, 44-67.

Vesic, A.S. (1972). "*Expansion of Cavities in Infinite Soil Mass*." Journal of Soil Mech. And Found. Eng'g Div., ASCE, Vol. 98, No. SM3, 265-290.