Performance of Reinforced Soil Wall Supported on Stone Columns

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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 soft 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.

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 soft 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.

Installation of stone columns is one of the most common techniques that can be adopted for improvement of soft, 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
- improve 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

The layout and elevation of the reinforced soil wall in the proposed development are shown in Figs. 1 and 2 respectively.
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 Fig. 3. 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 of the boreholes, MP and CPTu along the reinforced soil wall are shown in Fig. 4. As can be seen from Fig. 4, these profiles show very soft 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 and presented in Fig. 5. Fig. 6 shows the contour of the original topography at the site with the location of the reinforced soil wall. As can be seen from Fig. 6, the expected flow of surface runoff is towards the natural valley area. 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.

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 adopted for the Finite Element Analyses.
Table 1: Values of Soil Parameters Adopted

<table>
<thead>
<tr>
<th>Soil Description</th>
<th>Soil Model</th>
<th>Soil Model</th>
<th>Average SPT-N</th>
<th>Bulk Density $\gamma_b$ (kN/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stone Column MC</td>
<td>Drained</td>
<td>-</td>
<td>22</td>
<td></td>
</tr>
<tr>
<td>Crusher Run MC</td>
<td>Drained</td>
<td>-</td>
<td>22</td>
<td></td>
</tr>
<tr>
<td>RS Wall Backfill</td>
<td>MC</td>
<td>Drained</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td>Compacted Backfill</td>
<td>HS</td>
<td>Undrained</td>
<td>8</td>
<td>20</td>
</tr>
<tr>
<td>Soft Clay 1 HS</td>
<td>Drained</td>
<td>-</td>
<td>4</td>
<td>15</td>
</tr>
<tr>
<td>Soft Clay 2 HS</td>
<td>Drained</td>
<td>-</td>
<td>7</td>
<td>15</td>
</tr>
<tr>
<td>Medium Stiff Soil</td>
<td>HS</td>
<td>Drained</td>
<td>10</td>
<td>19</td>
</tr>
<tr>
<td>Bedrock NP</td>
<td>Non Porous</td>
<td>-</td>
<td>-</td>
<td>22</td>
</tr>
<tr>
<td>Pavement $\nu = 0.2$ LE</td>
<td>Non Porous</td>
<td>-</td>
<td>-</td>
<td>22</td>
</tr>
</tbody>
</table>

3.1 Bulging and General Shear

As mentioned in Section 3, stone columns of 1m diameter with a 2m centre to centre spacing have been proposed and designed as the supporting foundation. An independent check and review of the adequacy of the overall design was carried out by the authors. The design by the specialist contractor only used Priebe’s (1995) method to check the settlement of the subsoils treated with stone columns (Item 6 above). There were no calculations using other methods to check and determine Items 1 to 5 that were not adequately covered in Priebe’s method. It is quite common that the design of stone columns on checking the settlement only using Priebe’s method. However, such design practice should be discouraged and complemented with using more refined analysis to examine all aspects of possible failure mechanisms. Table 2 lists some of the methodologies available for bulging and general shear failure checks.

Table 2 Methods for Estimation of Ultimate Bearing Capacity of Stone Columns

<table>
<thead>
<tr>
<th>Mode of Failure</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulging</td>
<td>Greenwood (1970); Vesic (1972); Datye &amp; Nagaraju (1975); Hughes &amp; Withers (1974); Madhav et al. (1979).</td>
</tr>
<tr>
<td>General Shear</td>
<td>Madhav &amp; Vitkar (1978); Wong (1975); Barksdale &amp; Bachus (1983).</td>
</tr>
</tbody>
</table>

Independent check on the adequacy of bulging and general shear was carried out. 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.

Fig. 7 (a) Stresses on stone column (b) Comparison of different methods (after Madhav & Miura, 1994)

Most of the methods listed in Table 2 are reproduced in a graph by Madhav & Miura (1994) together with their proposed method as shown in 7. It is observed that there is a large range of possible ultimate bearing capacity when using different methods and this will cause confusion and indicate larger uncertainties in the assessment to the design engineer. Therefore, load tests and instrumentation monitoring had been carried out to verify the design performance. The load tests and instrumentation monitoring results would be presented in Sections 4 and 6 respectively.
3.2 Stress Distribution between Stone Column and Cohesive Soil

When a load is applied on the composite ground, studies (e.g. Greenwood, 1970; Aboshi et al., 1979; Goughnour & Bayuk, 1979; Balaam et al. 1977) indicated that high stress is normally attracted in the relatively stiff granular pile while the remaining load would be transferred to the surrounding less stiffer clayey soil. Bergado et al. (1991) further described on the stress distribution between the granular piles and clay. The distribution of vertical stress within a unit cell is expressed by a stress concentration factor defined as:

\[
n = \frac{\sigma_s}{\sigma_c}
\]

\[
\sigma_s = \frac{n \sigma}{1 + (n - 1) a_s} = \mu_s \sigma
\]

\[
\sigma_c = \frac{\sigma}{1 + (n - 1) a_s} = \mu_c \sigma
\]

where:
- \( n \) = stress concentration factor
- \( \sigma \) = average stress over unit cell area
- \( \sigma_s \) = stress in the granular pile (stone column)
- \( \sigma_c \) = stress surrounding cohesive soil.
- \( \mu_s \) = ratio of stress in the pile
- \( \mu_c \) = ratio of stress in the clay
- \( a_s \) = area replacement ratio

Based on Eqs. (1), (2) and (3), the stress in the stone column and cohesive soil can be determined and subsequently used to check the Factor of Safety against bearing failure.

3.3 Bearing Capacity of Subsoil and Stone Column

The ultimate bearing capacity for a single isolated pile given by Bergado et al. (1991) is expressed as follows:

\[
q_{ul} = c x N'c
\]

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 soft Bangkok clay.

The bearing capacity of the stone column and soft cohesive soil are calculated separately to check against the stress within the subsoil and stone columns respectively. In this case study, the estimated subsoil undrained shear strength is 60kPa. Adopting a \( N'c \) value of 16, the ultimate bearing capacity of stone column is 960kPa. The ultimate bearing capacities of subsoil and stone columns are subsequently checked against the stresses induced in subsoil and stone columns respectively. Stress distribution between the subsoil and the stone column has been elaborated in Section 3.2.

Verification test was also carried out to ascertain the bearing capacity of stone column. This will be elaborated in Section 4.

3.4 Global Stability of RS Wall

As the RS Wall is a proprietary retaining wall system, the internal stability of the RS Wall was designed by the proprietary specialist. The global stability of the RS Wall supported by stone column was checked using average shear strength method. The average shear strength method is widely used in stability analysis for sand compaction piles (Aboshi et al., 1979; Barksdale, 1981). The FOS of the RS wall is found to be adequate.

3.5 Overall Ground Settlement After Improvement

As mentioned in Section 3.1, the specialist contractor used Priebe’s (1995) methods to check on the settlement of the subsoils treated with stone columns. The estimated settlement before improvement is approximately 650mm while the estimated settlement after improvement is in the range of 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 and Fig. 8 present the specification and allowable grading envelopes 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 soft alluvial deposits. A typical quality control printout from the recording device showing the plots of pressure vs time and depth vs time is shown in Fig. 9.

<table>
<thead>
<tr>
<th>Test Standard</th>
<th>Criteria</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushing Value</td>
<td>BS 882:1992</td>
<td>&lt;30%</td>
</tr>
<tr>
<td>Los Angeles Abrasion</td>
<td>ASTM C131</td>
<td>Max loss of 40% at 500 revolutions</td>
</tr>
<tr>
<td>Flakiness Index</td>
<td>BS 882:1992</td>
<td>&lt;30%</td>
</tr>
<tr>
<td>Sulphate Soundness</td>
<td>ASTM C88</td>
<td>&lt;12%</td>
</tr>
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</table>

Sometime, in-situ field test, such as Standard Penetration Test (SPT), Piezocone Penetration Test (CPT), Dynamic Penetration Test (DPT) may be proposed to verify the quality of installation. But these tests are not a good indication of the effectiveness of stone column treated composite ground as the stone column treated ground has huge variation in the soil consistency. Therefore it is generally not recommended to carry out such in-situ field tests. Instead, the 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 square grid pattern with 2m centre to centre column spacing.

Table 3: Stone Aggregate Specification for Wet Method

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Fig. 10 shows load settlement behaviour of a 1m diameter stone column. As shown in Section 3.3, the ultimate capacity of stone column 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, as can be seen from Fig. 10, the maximum settlement of stone column is approximately 110mm. 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.

The proposed ground treatment area and typical cross section are shown in Figs. 11 and 12 respectively.

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 the followings:
• Displacement settlement marker
• Inclinometer at CH84.6
• Extensometer at CH84.6

Fig. 13 shows the instrumentation layout while Figs. 14 and 15 show the details of instrumentation for displacement markers on the RS wall panels and inclinometer with 3-level magnetic extensometers installed in front of the RS wall. Weekly readings were taken to monitor the changes in the profile during the construction of the RS wall for a period of 2 months. A final reading was taken at the 4th month to monitor the rate of movement. The following subsection discusses the monitoring results in further detail.

6 MONITORING INSTRUMENTATION SCHEME

6.1 Settlement Profile

As mentioned in Section 3, the allowable settlement is anticipated to be in the range of 250mm to 280mm. As can be seen from Fig. 16, RS wall has settled approximately 115mm between 29th July 2004 and 30th November 2004 over a period of 4 months. Despite the relatively significant amount of settlement, the magnitude of settlement is still far lower than the expected settlement envisaged in the design calculation. In addition, the rate of settlement at two most critical sections (CH33 and CH113) has stabilized to approximately 0.2mm/day at the end of year 2004 (see Fig. 18). This shows that the stone column is effective in facilitating the drainage within the soft clay since the rate of settlement seems to have stabilised within 6 weeks after the completion of RS wall. As can be seen from Fig. 17, the maximum distortional movement is 1:178, which is still within the allowable distortion of 1:100. 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.

6.2 Inclinometer

As can be seen from Figs. 19 and 20, the maximum lateral displacement in the major axis (A-A) and minor axis (B-B) are approximately 33mm and 8mm respectively at the depth of 4m below ground level. The rate of lateral displacement in the A-A axis and B-B axis are 0.1mm/day and 0.025mm/day (see Figs. 19 and 20 respectively) 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.

6.3 Magnetic Extensometer

It was observed from Fig. 23 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 readings 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.

Fig. 16  RS wall profile and settlement displacement marker profile with time

Fig. 17  Distortional movement at last monitoring reading

Fig. 18  Rate of settlement for displacement marker with time

Fig. 19  Lateral displacement profile at major axis (A-A) with time

Fig. 20  Lateral displacement profile at minor axis (B-B) with time

Fig. 21  Rate of Displacement for Inclinometer at Major Axis (A-A)

Fig. 22  Rate of displacement for inclinometer at minor axis (B-B)
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 ($\rho_t$) and lateral displacement ($\delta$) by Matsuo (1977) is adopted. The diagram is presented in Fig. 24. 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.

Based on the design, the allowable settlement is anticipated to be 300mm. Therefore, the allowable lateral displacement shall not be exceeding 80mm based on a FOS of 1.5 in the worst possible scenario. The final monitoring results (measured settlement and lateral displacement) indicate that the FOS of the RS wall is still within the green zone.

7 CONCLUSIONS AND RECOMMENDATIONS

Based on the above discussions, ground treatment using stone columns to support 10m high RS wall on 12m thick soft compressible alluvial deposits has been successfully installed and proven effective with the consideration of economy.

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


