IRAQI GYPSEOUS SOIL STABILIZED BY ORDINARY AND ENCASED STONE COLUMNS

Prof. Dr. Hussein H. Karim
Professor, Building and Construction Engineering Department, University of Technology, Baghdad-Iraq

Asst. Prof. Dr. Zeena W. Samuel
Assistant Professor, Building and Construction Engineering Department, University of Technology, Baghdad-Iraq

Huda K. Karim
M.Sc. in Geotechnical Engineering, Building and Construction Engineering Department, University of Technology, Baghdad-Iraq

ABSTRACT

Several model tests were performed to improve gypseous soil from Iraq with ordinary (common) and encased stone columns under static loading. The model tests performed on untreated soil were taken as a reference to the proposed improvement techniques, for which the stress ratio \((q_u/y_B)\) values equal 0.75. The mode of failure seems to be local shear failure type which gradually changes towards the general shear when using ordinary and encased stone columns. For ordinary stone column (OSC), the stress ratio \((q_u/y_B)\) increases to 1.17 at failure with increasing settlement ratio due to stiffness of stone column compared to untreated (normal) soil. The bearing improvement ratio \((q_t/q_{unt})\) shows a peak value at failure \((S/B=0.537)\), then drops down and remains nearly constant with increasing \(S/B\) ratio. This difference in behavior may be attributed to the gradual changes in the mode of failure. The settlement reduction ratio decreases to 0.48 at failure with increasing stress ratio \((q_u/y_B)\). It is found that there is slight increase in bearing capacity to 1.25 for soil treated with encased stone column (ESC) compared to soil treated with ordinary stone column. Also, the bearing improvement ratio \((q_t/q_{unt})\) shows a slight increase to about 1.66 at failure. Finally, the settlement reduction ratio \((S_t/S_{unt})\) at failure was 0.5 without any significant effect observed between ordinary and encased stone columns.

Key words: Gypseous Soil, Soil Reinforcement, Ordinary and Encased Stone Columns, Bearing Capacity, Settlement, Foundation.


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1. INTRODUCTION
Collapsible soils are those susceptible to large volumetric strains when they become saturated, these soils may be defined as any unsaturated soil that their particles are subjected to a substantial reorganization and accompanied with vast loss in volume after wetting with or without extra loading (Ayadat and Hanna, 2005). Different attempts were used to improve this type of soils, where a variety of reinforcing materials (natural materials, metallic strips and geosynthetics) were used for reinforcement applications. Some researchers used kerosene, lime, bentonite and kaolinite, sodium silicate solution, emulsified asphalt, cement, asphalt emulsion, cut-back asphalt, fuel oil and etc. Sufficient reduction has been noticed in compressibility characteristics and increase in the shear strength of gypseous soil upon treatment with these additives. All these materials were considered as a good solution to treat the problems of collapsible soil and to decrease the volume changes associated with this soil (Karim et al., 2012 and 2013).

New pattern of gypsous soil improvement is introduced by using a geogrid or stone columns as soil reinforcement. Al-Ameery (2003) represented an attempt for controlling the collapsibility of gypseous soil by reinforcing it with traditional and modified stone column using different percentages of sand, cement and asphalt with crushed stone. Salih (2003) investigated the possibility of using stone columns stabilized with liquid asphalt and lime to control the collapsibility of gypseous soils due to wetting. Both above model tests revealed an encouraging reduction in settlement due to the presence of stone columns. Mirza (2003) investigates the possibility of controlling the collapsibility of gypseous soil using the modified stone column by asphalt and cement technique. Four field model footings 1.25*1.25 m were constructed and placed on a selected site. Two of the model footings were placed directly on the natural ground and two on ground treated with four stone columns. The settlement of the four footings was recorded continuously during the 90 days, and comparison between them was made. The stabilized stone columns have successfully reduced the collapsibility of gypseous soil by 56 and 53% at 32 and 45kPa applied pressure respectively. Ayadat et al. (2008) also discussed the failure process of the stone column inserted in collapsible soil after wetting. This study reported that the stone columns have failed to strengthen loose fill that displays collapse behavior through the loss of the lateral confinement of the fill. Araujo et al. (2009) inspected the behavior of stone columns that are encased with geogrid and geotextile embedded in porous collapsible fine-grained soil by implementing field (in-situ) load tests. This study concluded that, encasing the sand column increases load capacity. Al-Obaidy (2016) investigated the possibility of using encased stone columns in ground improvement for Iraqi locations. It is found that Iraqi soils can be treated through encased stone columns as soils with similar collapsibility have been shown to be treated successfully through increases in the load capacity and reductions in the settlements.

The main purpose of the work implemented in this study is to conduct a series of model tests dealing with the ability of soil improvement techniques to stabilize the gypseous soil by ordinary (common) and encased stone columns and investigating the settlement reduction and the bearing capacity improvement in the entire system.

2. MATERIALS USED AND SOIL CHARACTERIZATION

2.1. Soil Used
The soil used in this study is a disturbed natural gypseous soil, samples were brought from "Ain Al-Tamur in KarbalaProvince" with a gypsum content of 30%. The soil was taken from a depth 5.0 m below the natural ground level. Figure 1 shows the grain size distribution for the soil used, while Table 1 summaries the properties of natural soil used in this study. All chemical tests were conducted with the assistance of laboratories of the “State Company of Geological Survey and Mining”. The results are presented in Table 2. Figure 2 shows X-ray diffraction for the soil used, it can be seen that soil consists of quartz (SiO$_2$), calcite (CaCO$_3$), feldspar, gypsum and palygorskite.
Prof. Dr. Hussein H. Karim, Asst. Prof. Dr. Zeena W. Samueel and Huda K. Karim

Figure 1 Grain size distribution of the soil used.

Table 1 Physical properties of the soil used.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grain size analysis</td>
<td></td>
</tr>
<tr>
<td>Effective size, $D_{10}$ (mm)</td>
<td>0.11</td>
</tr>
<tr>
<td>$D_{30}$ (mm)</td>
<td>0.24</td>
</tr>
<tr>
<td>$D_{60}$ (mm)</td>
<td>0.55</td>
</tr>
<tr>
<td>Coefficient of uniformity, $(C_u)$</td>
<td>5.0</td>
</tr>
<tr>
<td>Coefficient of curvature, $(C_c)$</td>
<td>0.95</td>
</tr>
<tr>
<td>Classification (USCS)</td>
<td>SP</td>
</tr>
<tr>
<td>Dry unit weights</td>
<td></td>
</tr>
<tr>
<td>Maximum dry unit weight (kN/m$^3$), $\gamma_d (max)$</td>
<td>14.4</td>
</tr>
<tr>
<td>Minimum dry unit weight (kN/m$^3$), $\gamma_d (min)$</td>
<td>12.34</td>
</tr>
<tr>
<td>Test dry unit weight (kN/m$^3$), $\gamma_d (test)$</td>
<td>12.83</td>
</tr>
<tr>
<td>Relative density, (R.D %)</td>
<td>38%</td>
</tr>
<tr>
<td>Void ratio</td>
<td></td>
</tr>
<tr>
<td>Maximum void ratio, $e_{max}$</td>
<td>0.88</td>
</tr>
<tr>
<td>Minimum void ratio, $e_{min}$</td>
<td>0.61</td>
</tr>
<tr>
<td>Specific gravity, $(G_s)$</td>
<td>2.32</td>
</tr>
<tr>
<td>Initial water content(%)</td>
<td>6</td>
</tr>
<tr>
<td>Angle of internal friction before soaking ($\Phi^*$)</td>
<td>39</td>
</tr>
<tr>
<td>Angle of internal friction after soaking ($\Phi^*$)</td>
<td>27</td>
</tr>
</tbody>
</table>

Table 2 Chemical properties of the soil used.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gypsum content (%)</td>
<td>30</td>
</tr>
<tr>
<td>Total sulphate content, (SO$_3$ %)</td>
<td>14</td>
</tr>
<tr>
<td>Total soluble salts, (T.S.S%)</td>
<td>20.90</td>
</tr>
<tr>
<td>pH value</td>
<td>8.20</td>
</tr>
<tr>
<td>X-ray diffraction</td>
<td>Gypsum, quartz, calcite, feldspar, and palygorskite</td>
</tr>
</tbody>
</table>
2.2. Crushed Stone Used
The crushed stone materials were obtained from a private mosaic factory. It was produced as a result of crushing big stones, which were brought from Panjawin city in northern Iraq. The crushed stone is of white color with angular shapes. Particles size distribution is shown in Figure 3. The crushed stone has uniform size with poorly graded gradation, and the details are listed in Table 3.
Table 3 Characterization of the crushed stone used.

<table>
<thead>
<tr>
<th>Property</th>
<th>Index value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. dry unit weight (kN/m³)</td>
<td>15.2</td>
</tr>
<tr>
<td>Min. dry unit weight (kN/m³)</td>
<td>14.3</td>
</tr>
<tr>
<td>Dry unit weight (kN/m³) at R.D =50%</td>
<td>14.7</td>
</tr>
<tr>
<td>$D_{10}$ (mm)</td>
<td>2.9</td>
</tr>
<tr>
<td>$D_{20}$ (mm)</td>
<td>4.8</td>
</tr>
<tr>
<td>$D_{60}$ (mm)</td>
<td>6.0</td>
</tr>
<tr>
<td>Specific gravity ($G_s$)</td>
<td>2.64</td>
</tr>
<tr>
<td>Coefficient of uniformity ($C_u$)</td>
<td>2.0</td>
</tr>
<tr>
<td>Coefficient of curvature ($C_c$)</td>
<td>1.32</td>
</tr>
<tr>
<td>Relative density (R.D %)</td>
<td>50</td>
</tr>
<tr>
<td>Angle of internal friction ($\phi^\circ$) at R.D =50%</td>
<td>41</td>
</tr>
</tbody>
</table>

2.3. Geogrid Used
The geogrid material was used almost in all tests, and manufactured by Al-Latifyia Factory for plastic mesh and having the engineering properties shown in Table 4 as provided by the manufacturing company. Figure 4 displays the geogrid reinforcement used.

Table 4 Engineering properties of geogrid used.

a. Dimensional properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Method</th>
<th>Unit</th>
<th>Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aperture size</td>
<td>ISO 9864</td>
<td>mm × mm</td>
<td>6 × 6</td>
</tr>
<tr>
<td>Mass per unit area</td>
<td></td>
<td>g/m²</td>
<td>363</td>
</tr>
<tr>
<td>Roll width</td>
<td></td>
<td>m</td>
<td>1</td>
</tr>
<tr>
<td>Roll length</td>
<td></td>
<td>m</td>
<td>30</td>
</tr>
</tbody>
</table>

b. Technical properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Method</th>
<th>Unit per (m) length</th>
<th>Data*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength at 2 %</td>
<td>ISO10319</td>
<td>kN/m</td>
<td>4.3</td>
</tr>
<tr>
<td>Tensile strength at 5 %</td>
<td></td>
<td>kN/m</td>
<td>7.7</td>
</tr>
<tr>
<td>Peak tensile strength</td>
<td></td>
<td>kN/m</td>
<td>13.5</td>
</tr>
<tr>
<td>Yield point elongation</td>
<td></td>
<td>%</td>
<td>20.0</td>
</tr>
</tbody>
</table>

* Determined in accordance to Saudi Arabian Standard Organization (SASO) Procedures.

Figure 4 The geogrid used

3. FORMULATION OF TEST SETUP
All the models were tested by using a setup described below. The soil tank model with dimensions of 60 cm×60 cm ×75 cm, and steel plate thickness of 6 mm (Figure 5) was used to host the soil bed. The soil tank model is placed below the steel loading frame with the axial loading system to transfer load (Figure
6). The stainless steel compression load cell (with maximum capacity of 5 ton, and rated output of $2 \pm 0.005$ mV/V) was utilized to measure the load (Figure 7). A digital weighting indicator with sensitivity is $0.2 \, \mu N/\text{Digit}$ (Figure 8) was used to record the amount of applied load. A rigid square steel base plate with dimensions of $200 \, \text{mm} \times 200 \, \text{mm} \times 10 \, \text{mm}$ was used to model footing placed on the model to transfer the load used in the current study.

3.1. Preparation the Bed of Soil

The natural soil consists of large aggregated lumps. This requires crushing, remixing and air-drying. The required weight of natural soil was mixed with specific amount of water depending on the density (wet) at which the soil bed is prepared. Placing the filter material at the base of the container to 5 cm in height, and then compacting it by a certain hammer to the required density of 13.6 kN/m³ that gives a relative density of the soil 38%. After that, a mesh is placed over the filter to prevent mixing it with the soil bed as shown in the Figure 9. The prepared soil was then spread into sub layers, each layer was 5cm thick and compacted uniformly to achieve as possible as the desired density. After completing the preparation of soil bed, the surface is leveled to get a flat plane. There after the model tank is soaked with water for approximately five days. The process of soaking involves lifting water from the base of the container to the top of soil surface and then the water is lowered again to the base of filter material. Then, the container is covered with a sheet of nylon to preserve the water content of soil as possible as constant as shown in Figures 10 and 11.
3.2. Installation of Stone Columns

To install the stone columns correctly in their proper locations, a special frame was manufactured. The frame consists of two plates connected together by four bolts. The distance between two plates is 150 mm. Each plate was perforated by four holes with a diameter of 50 mm (the same as the diameter of the stone column). The depth of each stone column was predetermined (corresponding to L/D=6). A PVC pipe with external diameter of 50 mm was inserted in the bed to the specific depth with the aid of the frame. Figure 12 demonstrates the procedure of the stone columns installation. To remove the soil inside the PVC pipe, a hand auger was used. After that, the PVC pipe was removed carefully. The crushed stones were put into the hole as layers, and then compacted gently by tamping rod. After putting all the specific amount of crushed stone, the full depth of the hole is filled with stones at a dry unit weight of 14.7 kN/m³. All stone columns have a diameter of 50 mm, length to diameter ratio (L/D) of 6 with a spacing of two times the diameter and area replacement ratio ($a_r$) equals to 0.785.
3.3. Installation of Encased Stone Columns
To install the encased stone columns, the same procedure followed in construction of the ordinary stone columns was conducted here. At first, four samples formed from geogrid tubes were prepared by warping up roll of the geogrid and sew by a nylon strings with the same diameter and length ratio of the ordinary stone column (L/D = 6). A hallow PVC plastic pipe with 50 mm external diameter was pushed down to the bed (i.e. to the specific depth) with the assistance of the frame. Then, a hand auger was used to remove the soil inside the PVC pipe. After that, the soil inside the PVC tube was removed carefully. Finally, the tube of geogrid was inserted into the stones as shown in Figure 13.

Figure 13 The method of installation of the encased stone columns.

4. RESULTS AND DISCUSSION

4.1. Untreated Model
One model test was performed on a bed of saturated gypseous soil. The footing was placed on the surface of the bed of soil and loaded gradually up to failure. The results of bearing ratio \(q_u/\gamma B\) versus settlement ratio \(S/B\) are shown in Figure 14. From this figure, it is obvious that the settlement ratio increases linearly with an increase of stress. This behavior was expected, where the increase in load would increase the rate of solution and cause softening of the soil resulting in loss of shear strength and increase in collapse settlement as proposed by Hussein (2012). The mode of failure seems to be local shear failure.

![Figure 14 Stress–settlement curve for a footing resting on untreated soil.](image)

4.2. Model Tests on Treated Gypseous Soil

4.2.1 Model test with ordinary stone column (OSC)
One model test was performed by treating with ordinary stone columns of type floating with L/D=6 and the area replacement ratio \(\alpha_r=0.196\).
Bearing ratio versus settlement ratio

Figure 15 demonstrates the relationship between bearing ratio \( (q_u/\gamma B) \) and settlement ratio \( (S/B) \) for the case of ordinary stone column. For comparison, the results of untreated soil and treated soil with ordinary stone columns are presented in this figure. Also, it can be stated that bearing ratio increases from 0.75 to 1.17 in case ordinary stone columns. Confinement, and thus stiffness of the stone, is provided by the lateral stress within the weak soil. When the vertical stress is applied on the ground surface, both the stones and the weak soil will move downward resulting in a substantial concentration of stress within the stone column which becomes stiffer than the surrounding soil.

Bearing improvement ratio versus settlement ratio

To evaluate the amount of improvement achieved by ordinary stone column, a bearing improvement ratio is introduced denoted by \( (q_u/\gamma B)_t \) for the treated soil divided by the corresponding \( (q_u/\gamma B)_{unt} \) for the untreated soil. The ratio is plotted against settlement ratio \( (S/B\%) \) as shown in Figure 16. Peak values of improvement ratio are indicated at around \( S/B\%=0.537\% \), then declines down and remains nearly steady with increasing settlement ratio. This behavior is due to the fact that the stone columns are stiffer than the surrounding soil. As model is loaded, the stress is transferred to the stone columns expressing these peak values then it is gradually transferred to the surrounding soil implied by the drop in the improvement ratio.

Settlement reduction ratio versus bearing ratio

Variation of settlement reduction ratio \( (S_t/S_{unt}) \) versus bearing ratio \( (q_u/\gamma B) \) for ordinary stone column is shown in Figure 17. Since the failure was defined as the applied stress that corresponds to \( (S/B=10\%) \), then the settlement reduction ratio was determined as \( S_t/S_{unt} \); where \( S_{unt} \) represents a settlement at a constant value of 10% of the footing diameter for the untreated soil. The \( S_t \) represents the settlement of the treated soil corresponding to the failure pressure of the untreated model. The results imply a decrease in settlement reduction ratio. The value of settlement reduction at failure was 0.48.

![Figure 15](attachment:figure15.png)

**Figure 15** Stress–settlement curve for a footing resting on gypseous soil treated by ordinary stone columns.
4.2.3 Model test with encased stone column (ESC)

One model test was prepared and tested using acylindrical geogrid surrounding the stone column. The encased stone columns were 50mm diameter, L/D = 6 and the area replacement ratio $a_r = 0.196$.

**Bearing ratio versus settlement ratio**

The variation of bearing ratio ($q_u/\gamma B$) versus settlement ratio (S/B%) for encased stone columns configuration is shown in Figures 18. Results of untreated soil and soil treated with both ordinary and encased stone columns are also presented in Figure 19 for comparison purpose. Based on the results, the relative density of the backfill stone particles seems the main factor which plays a major role in increasing the bearing capacity of soil treated with encased stone columns. Figure 19 demonstrates slight increase in bearing capacity for soil treated with ESC as compared with OSC of the same configuration as proposed by Al-Baiaty (2012). This may be due to the high resistance developed through the dense packing of the stone particles at relative density, and the extension of the effective diameter of stone columns (Shankar and Shroff, 1997). Figure 19 also shows a slight increase in bearing ratio ($q_u/\gamma B$) with increasing the settlement ratio (S/B%), starting from a value of bearing ratio 1.25 till the end of test. This may be due to the larger deformation (bulging) occurs at the center of the area.
Figure 18 Stress–settlement curve for a footing resting on gypseous soil treated by encased stone columns.

Figure 19 Stress–settlement curve for a footing resting on gypseous soil treated by encased stone columns.

Bearing improvement ratio versus settlement ratio

The variation of bearing improvement ratio \( \left( \frac{q_t}{q_{unt}} \right) \) versus settlement ratio (S/B\%) for soil treated with encased stone columns at different configurations are shown in Figure20. The figure reflects the peak values of the improvement ratio as indicated at S/B of about 0.535\%, then an abrupt drop is followed which remains steady with the increase in settlement ratio.
Iraqi Gypseous Soil Stabilized by Ordinary and Encased Stone Columns

Figure 20 Bearing improvement ratio versus settlement ratio after soil improved by stone columns.

Settlement reduction ratio versus bearing ratio
To evaluate the amount of settlement reduction ratio achieved by the encasement over the untreated soil with the soil treated with ordinary stone columns, the settlement reduction ratio \( (S_t/S_{unt}) \) versus bearing ratio \( (q_t/\gamma B) \) is presented in Figure 21. In this figure, no significant effect is observed between ordinary stone columns (OSC) and encased stone column (ESC). This means that at up to this stress level no mobilization is achieved by the geogrid encasement.

Table 5 illustrates the bearing capacity ratio at failure \( (q_u/\gamma B) \), bearing improvement ratio \( (q_t/q_{unt}) \) at failure and settlement reduction ratio at failure \( (S_t/S_{unt}) \) for both ordinary and encased stone columns.

Table 5 Values of bearing capacity ratio \( (q_u/\gamma B) \), bearing improvement ratio \( (q_t/q_{unt}) \), and settlement reduction ratio \( (S_t/S_{unt}) \) at failure.

<table>
<thead>
<tr>
<th>Case</th>
<th>( q_u/\gamma B ) at failure</th>
<th>( q_t/q_{unt} ) at failure</th>
<th>( S_t/S_{unt} ) at failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Untreated</td>
<td>0.75</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Ordinary stone column</td>
<td>1.17</td>
<td>1.56</td>
<td>0.48</td>
</tr>
<tr>
<td>Encased stone column</td>
<td>1.25</td>
<td>1.66</td>
<td>0.50</td>
</tr>
</tbody>
</table>
5. CONCLUSIONS

Based on the analysis of the model tests performed on untreated and treated soils with ordinary and encased stone columns under static loading, the following conclusions points may be drawn:

1. The stress ratio for the untreated gypseous soil \( (q_u/yB) \) of 0.75 was taken as a reference for model tests of the proposed improvement techniques using ordinary and encased stone columns under static loading.
2. The mode of failure seems to be local shear failure type which gradually changes towards the general shear when using ordinary and encased stone columns.
3. For ordinary stone column (OSC), the stress ratio \( (q_u/yB) \) increases to 1.17 at failure with increasing settlement ratio compared to untreated soil due to stiffness of stone columns.
4. The bearing improvement ratio \( (q_t/q_{unt})\) for OSC shows a peak value at failure (S/B=0.537), then drops down and remains nearly constant with increasing S/B ratio, this difference in behavior may be due to the gradual changes in the mode of failure. While, the settlement reduction ratio decreases to 0.48 at failure with increasing stress ratio \( (q_u/yB) \).
5. For soil treated with encased stone column (ESC), it is found that there is slight increase in both bearing capacity (1.25) and bearing improvement ratio \( (q_t/q_{unt})\) to about 1.66 at failure compared to soil treated with ordinary stone column. While, the settlement reduction ratio at failure was 0.5 without any significant effect observed between ordinary and encased stone columns.

REFERENCES


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