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Soft soil improved by stone columns and/or ballast layer

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Iraq has the oldest railway network in the region. Currently the Iraqi railway company is rehabilitating the existing network in different regions and planning to expand it for the future. About 30–40% of the total length of the network is located in the southern part of Iraq, passing over soft, saturated, sedimentary deposits and in many areas close to the marshland. Stability and settlement are the major challenges to the safety and serviceability of rail tracks in these areas, thus ground improvement is essential to achieve the required level of performance. The paper presents results of tests of three treatment patterns. The first investigates the presence of a ballast embankment overlying a bed of soft, saturated clay. The second pattern focuses on the improvements achieved in load-carrying capacity and settlement as a result of eight stone columns added at an area replacement ratio of 0·196. The third pattern investigates the improvements achieved when patterns one and two are combined. In all tests, the ballast model embankment is loaded gradually by stress increments up to failure and stress deformation measurements are recorded and analysed in terms of bearing improvement ratio and settlement reduction ratio. Optimum outcomes are deducted from the third pattern, revealing bearing improvement ratio of 2·3 and settlement reduction ratio of 0·17.

Notation

B	width of footing
c_c	compression index
c_r	swelling index
c_u	undrained shear strength (kPa)
c_v	coefficient of consolidation (m^2/s)
e_0	initial void ratio
H	height of ballast
H/B	ballast ratio (height of ballast/width of footing)
k	coefficient of permeability (m/s)
m_v	coefficient of volume change (m^2/kN)
q	applied stress
q/c_u	bearing ratio
q_t/q_u	bearing improvement ratio (applied stress on treated soil/applied stress on untreated soil)
S/B	settlement ratio (settlement/width of footing)
S_t/S_u	settlement reduction ratio (settlement of treated soil/settlement of untreated soil)
W_{ci}	initial water content (%)
γ_{dry}	dry unit weight (kN/m^3)

1. Introduction

Soft saturated soils exist in the middle and southern parts of Iraq, concentrated along the alluvial plain through which the Tigris and Euphrates rivers flow. The alluvial plain begins north of Baghdad and extends to the Arabian Gulf, with a fair amount of marshland. Random data collected from different site investigation reports demonstrated values of undrained shear strength less than 20 kN/m^2 and compression indices as high as 0·3. The texture of these soils consists of fine silty clay loams, and silty clay with clay fraction between 50 and 70%. These constituents with a high water table revealed fair to poor soft deposits. Figure 1 demonstrates the distribution of soft deposits in Iraq.

Iraq is expecting to have a rapid development mission related to the extension and rehabilitation of the present railway network; about 30–40% of the network passes through the middle and southern parts, and sometimes comes closer to or crosses the marshland. Most likely ballast embankments are founded on soft ground to provide an adequate safety and reasonable level of performance. According to the design requirements of the new

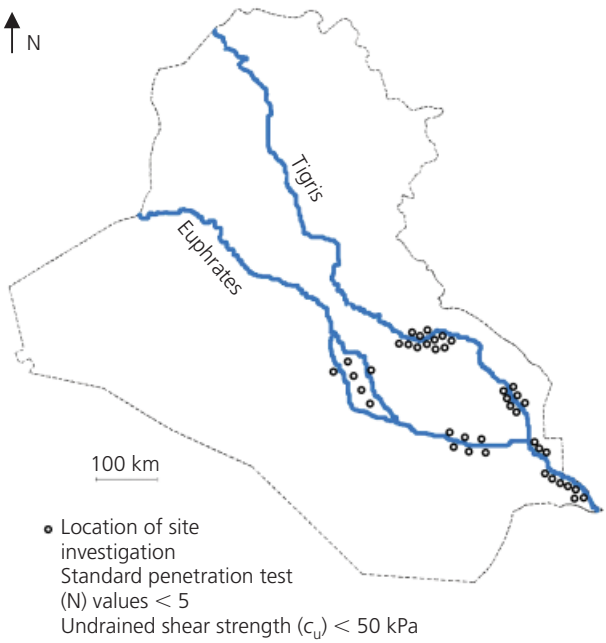

Figure 1. Location of the soft deposits in southern Iraq

track lines and/or upgrading the normal existing lines to account for high-speed trains, stone columns in combination with the ballast embankment are proposed as an adequate technique that will provide flexible construction ability in the treatment of different soils.

2. Stone columns underneath embankments

The majority of previous research works on stone columns underneath embankments were carried out through field tests and trail embankments. Greenwood (1975) carried out a model test for a road embankment using stone columns of 1·2 m diameter and 6 m long, penetrating a soft soil layer 3–6 m thick. The reduction in settlement was only 15% compared to the untreated soil. McKenna *et al.* (1975) observed no improvement in the settlement of a soft layer 27·5 m thick underneath a 7·9 m high embankment when reinforced with stone columns, 0·9 m in diameter and 11·3 m long. Their findings were attributed to the large size of the crushed stones used in the construction of the stone columns. Vautrain (1977) studied the performance of an 8·5 m abutment constructed over 11 m thick soft clay with undrained shear strength between 15 and 50 kPa. The soft layer was reinforced with stone columns 1 m in diameter and 11 m long, demonstrating a stress concentration ratio between 2·5 and 2·7. Goughnour and Bayuk (1979) investigated the behaviour of a 10·7 m high embankment of an interchange ramp overlying a soft, saturated layer 3–5 m thick, with undrained shear strength 18 kPa. The installed stone columns were 1·1 m in diameter and 6·4 m long, constructed in a triangular pattern with spacing between 1·8 and 2·4 m. The observed results showed reasonable prediction of ultimate settlement. Results for a full-scale embankment 2·4 m high were presented by Bergado *et al.* (1987); the earth embankment was constructed over a layer of soft clay that was 8–9 m thick, reinforced with stone columns 0·3 m in diameter and 8 m long. The analysis of the results revealed a bearing improvement ratio of 4 and settlement reduction ratio of 0·35. Drescher and Fritz (1989) reported results of a full-scale railway embankment 9 m high constructed on an 8 m thick layer of soft soil with undrained shear strength of 15 kPa. The soft soil was reinforced with stone columns 1 m in diameter and 10 m long, constructed in a triangular grid, with the spacing varying between 1 and 2·5 m. The observed settlement reduction ratio was found to be 0·28. Raju (1997) studied the behaviour of a 10 m high embankment supported on very soft soil of undrained shear strength 10 kPa. Stone columns of 1–1·2 m diameter and length between 15 and 17 m were installed using the vibro-replacement technique. The analysis revealed a settlement reduction ratio of 0·25–0·4 depending on the geometry of the stone columns. The stress distribution below the stone columns supporting an embankment was investigated by Kirsch and Sondermann (2003), who demonstrated a comparison between numerical analysis and some analytical approaches.

A trial highway embankment of height varying between 4·5 and 6 m, overlying a 14 m thick layer of soft soil, with undrained shear strength increasing with depth from 8·5 to 25 kPa, was studied by Oh (2006). The embankment was divided into three sections: two were reinforced with stone columns, 0·5 m diameter and 16 m long, with spacing 2 and 3 m respectively, and the third was left untreated. The observed settlement reduction ratio was found to be 0·83. Filz and Navin (2006) developed a new numerical stress–strain analysis to evaluate the stability of highway embankment supported by columns installed by a deep mixing method. They recommended that the proposed analysis can be used for any type of columns provided that they are strong in compression and weak in bending and tension.

Arulrajah *et al.* (2009) investigated the behaviour of high-speed railway embankments with heights ranging from 1 to 12 m and top widths of 14·9–24·9 m, with side slopes of 1V:2H. The soils encountered on the project site were highly variable mixtures of very soft silts and clays extending in depth to 30 m below natural ground level. Stone columns and dry deep mixing cement columns were proposed for the treatment of the soft layer. Both techniques were evaluated through a number of load tests, and settlement plates indicated that the stringent performance requirements were met.

The above review demonstrates that the efficiency of the stone columns technique in improving bearing capacity and controlling settlement is influenced by many factors, but primarily field conditions, method of construction, spacing of columns and grid pattern.

In the present work, the bearing capacity and compressibility of soft soil reinforced with stone columns underneath a ballast model embankment of variable height are investigated and the

improvements are expressed in terms of bearing ratio and settlement ratio of the treated soil compared to the untreated soil.

3. Materials used

Two types of material were used: the first was a brown clayey soil consisting of 3·3% sand, 31·7% silt and 65% clay. The liquid limit and the plasticity index are 35% and 16·5 respectively, revealing a material classified as CL according to the Unified Soil Classification System.

The crushed stone is produced by crushing big stones brought from Penjewen city, located north of Iraq. The crushed stone is poorly graded with $D_{10} = 4{\cdot}66$ mm, $D_{30} = 5{\cdot}0$ mm and $D_{60} = 5{\cdot}12$ mm. The coefficient of uniformity and coefficient of curvature are 1·02 and 1·05, respectively. The specific gravity of the crushed stone is 2·64 and the angle of internal friction is 42 at relative density of 71%, corresponding to dry unit weight of 15 kN/m^3, which was used in preparing the stone columns.

4. Model preparation and testing

Beds of fully saturated soil were prepared at undrained shear strength of 9 kPa, representing the lowest value of shear strength of soil, close to that in the marshland. This value was achieved after several trials of natural drying and mixing with continuous measurements of undrained shear strength. The soil was placed in ten layers inside a steel container of dimensions 1000 mm × 400 mm × 700 mm. Each layer was tamped gently with a wooden tamper 75 mm × 75 mm to remove any entrapped air. After completing the final layer, the top surface was scraped, levelled and covered with a polythene sheet, and a wooden board of the same size was placed on top with 5 kPa seating pressure. The bed of soil was left for a period of 2 d to regain its strength by self-weight consolidation. Table 1 illustrates the undrained shear strength and compressibility characteristics of the prepared bed of soil. The stone columns were constructed following the same procedure applied by Rahil (2007). All stone columns have a diameter of 50 mm, length-to-diameter ratio, $L/D = 6$, with spacing of two times the diameter. A steel template was used to fix the location of the eight stone columns and control their vertical alignment. A hollow, plastic, polyvinyl chloride (PVC) pipe with external diameter of 50 mm was pushed down into the bed to the predetermined depth and the soil inside was carefully removed by a hand auger. The crushed stone was poured into the hole in layers and compacted gently by tamping rod. After pouring all the specific amount of crushed stone, the full depth of the hole was filled with stone at dry unit weight of 15 kN/m^3; Figures 2 and 3 illustrate the steps of construction of the stone columns.

Undrained shear strength: kPa	9
Initial water content, W_{ci}: %	26·8
Initial void ratio, e_0	0·72
Compression index, c_c	0·232
Swelling index, c_r	0·033
Avg. coefficient of volume change, m_v: m^2/kN	$7{\cdot}8 \times 10^{-4}$
Coefficient of consolidation, c_v: m^2/s	$6{\cdot}3 \times 10^{-8}$
Coefficient of permeability, k: m/s	$4{\cdot}7 \times 10^{-10}$
Dry unit weight, γ_{dry}: kN/m^3	15·6

Table 1. Strength and compressibility characteristics of soil used

Following the completion of the construction of the stone columns, crushed stone was spread carefully in the form of layers, each 25 mm thick. Each layer was compacted gently to attain a placement unit weight of 15 kN/m^3, corresponding to a relative density of 71%. Two heights, 50 mm and 100 mm, were selected for the ballast layer, which took the shape of a model embankment with crest width 300 mm.

The whole assembly was moved and fixed in position inside a loading rig so that the centre of the model footing, 200 mm wide and 400 mm long, coincided with the centre of the bed of soil. The footing was then lowered and brought in contact with the

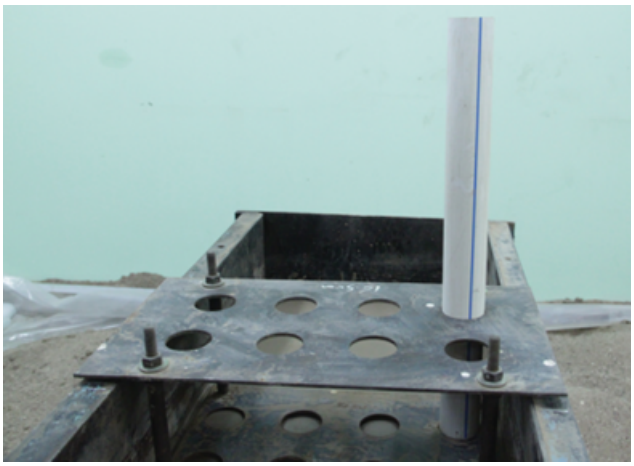

Figure 2. Steps of construction of stone columns

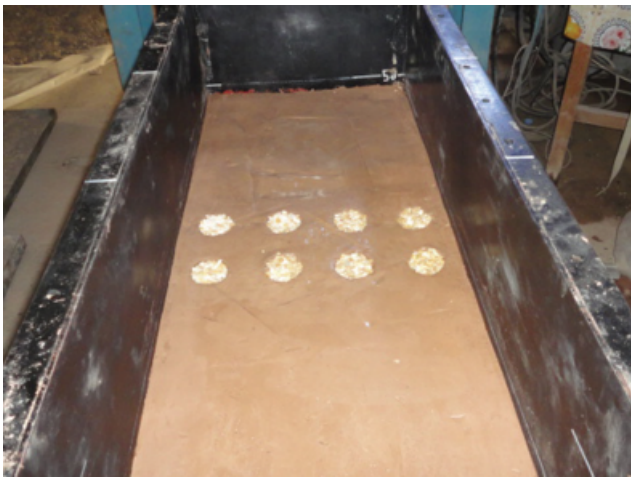

Figure 3. Top view of completed stone columns

crest of the model embankment and linear position displacement transducers (LPDTs) were fixed in position. Monotonic stress was then gradually applied and settlement against applied stress was recorded continuously up to failure.

5. Presentation and discussion of model test results

Prior to the discussion of the results, Figure 4 illustrates a section through a typical model test, clarifying all notation used in this paper. The discussion is divided into two sections: the first is devoted to results concerning improvements in bearing capacity due to the presence of the ballast layer alone, presence of stone columns alone and finally to the presence of both ballast layer and stone columns. The increase in bearing capacity is determined using the term 'bearing improvement ratio', defined as the ratio of the bearing ratio, q/c_u, of the treated soil to that of the untreated soil; this ratio is simply given the notation q_t/q_u. The second section of the discussion is devoted to the reduction in settlement gained by each of the three treatment patterns. The term 'settlement reduction ratio' is defined as the ratio of the settlement of the treated soil to the settlement of the untreated soil and represented by the notation S_t/S_u.

5.1 Bearing ratio and bearing improvement ratio

The ballast material in railway constructions is used primarily to distribute the stresses generated from the weight of the train to the subgrade soil. It also enables water to drain and provides an adequate platform to facilitate track maintenance. In the present paper, the bearing ratio and bearing improvement ratio of each treatment pattern are discussed separately and the results are compared with several design methods.

The first treatment pattern focuses on the influence of two ballast ratios $H/B = 0{\cdot}25$ and $0{\cdot}5$ on the stress-carrying capacity. The bearing ratio plotted against settlement ratio for $H/B = 0{\cdot}25$, shown in Figure 5, demonstrates marginal increase over the untreated soil. Both curves exhibited a failure pattern close to a punching mode, revealing bearing ratios of 5 and 5·5 at failure

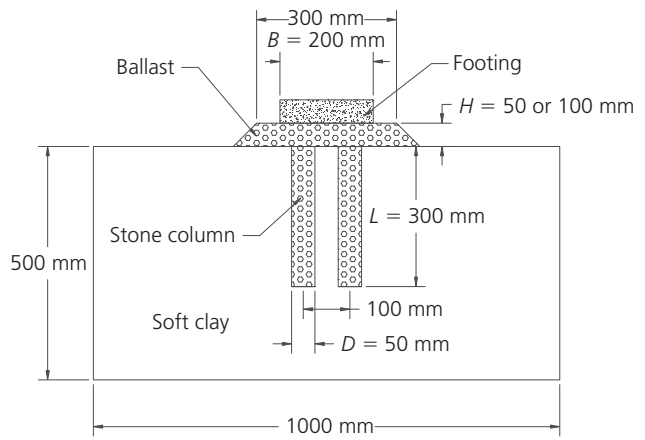

Figure 4. Section through a typical model

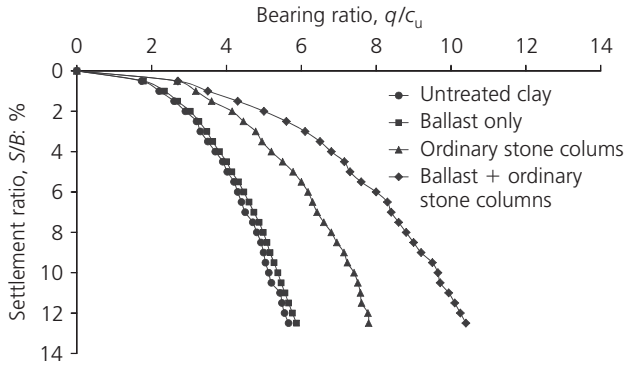

Figure 5. Bearing ratio plotted against settlement ratio – $H/B = 0{\cdot}25$

and a marginal increase in bearing improvement ratio, less than 1·1, as shown later in Figure 7. Although this ballast layer underneath the footing is stiffer than the soft soil, the shallow thickness is not sufficient to host a significant amount of the applied stress. When the ballast ratio H/B was increased to 0·5 (Figure 6), appreciable increase in bearing ratio was observed, generating a bearing improvement ratio of 1·3 at failure; see Figure 8 later in this section. This significant increase is a common finding because most of the slip planes generated during loading or at failure are hosted by the ballast layer.

The second treatment pattern is stone columns without ballast layer. The mechanism of stress distribution in soil reinforced with stone columns differs greatly from that of the ballast layer. The applied stress is concentrated more on the stiffer stone columns as rigid inclusions compared to the surrounding soil, generating a state of uneven stress distribution. The stone columns may also experience a bulging pattern underneath the loaded area depending on the stiffness of the columns and the surrounding soil.

Figures 5 and 6 demonstrate an increase in settlement ratio with increasing bearing ratio, exhibiting a shape of curve between

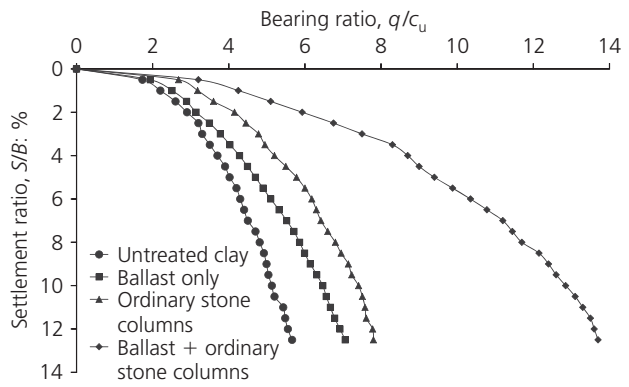

Figure 6. Bearing ratio plotted against settlement ratio – $H/B = 0{\cdot}50$

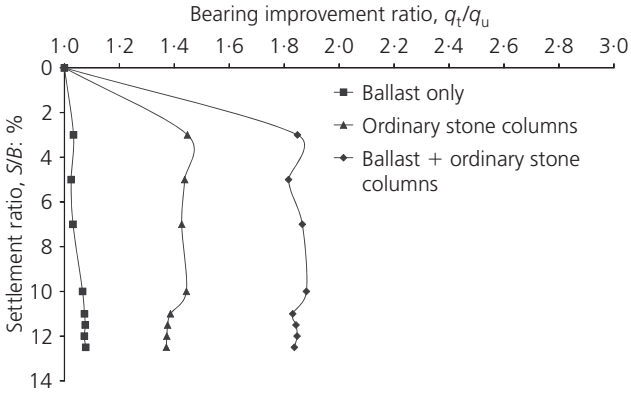

Figure 7. Bearing improvement ratio plotted against settlement ratio – $H/B = 0{\cdot}25$

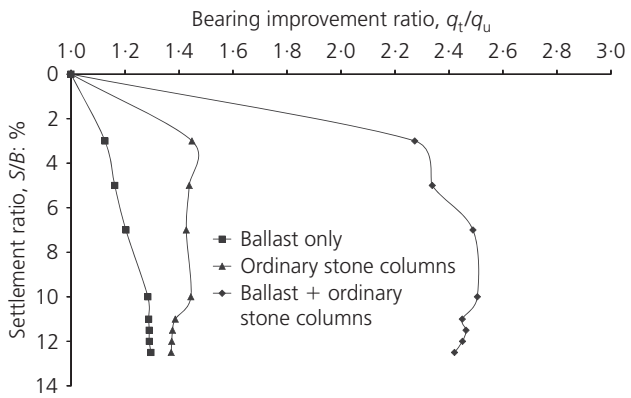

Figure 8. Bearing improvement ratio plotted against settlement ratio – $H/B = 0{\cdot}50$

local and general shear modes. This is primarily due to the increase in stiffness of the composite soil revealing a bearing ratio of 7·3 at failure. For comparison purposes, the bearing ratio at failure obtained from the model tests is compared with corresponding values calculated from several analytical equations shown in Table 2. A stress concentration factor of 3 is assumed whenever needed, as recommended by most references. According to Greenwood (1970), the column material gets compressed axially and expands laterally and the surrounding soil is expected to mobilise the passive pressure condition. Based on this approach, the calculated bearing ratio is 6·2, which is lower than the value of 7·3 obtained from model tests. The cavity expansion theory presented by Vesic (1972) was applied to derive the ultimate stress of a group of stone columns, and revealed bearing ratio at failure equal to 8·8. Hughes *et al.* (1975) stated that the ultimate load is governed by bulging failure of the column in the upper zone, which is controlled by the limiting radial restraint of the soil on the bulging. The analysis revealed a bearing ratio of 10·2 at failure. On the other hand, Madhav and Vitkar (1978) presented a bearing capacity equation of plain strain conditions where the load is applied on both stone columns and the surrounding soil, and accordingly the bearing ratio at failure is 8·3. Sowers (1979) estimated the ultimate carrying capacity of long, rigid footing resting on cohesive soil reinforced with stone columns by approximating the failure surface by two straight rupture lines, demonstrating a bearing ratio of 10·2. Bouassida *et al.* (1995) used the stress field exhibited in static solution of a composite cell model and developed a general bearing capacity of a raft foundation resting on a soil reinforced with stone columns. The method revealed a bearing ratio of 5·2. It can be seen from Table 2 that the bearing ratio obtained from the model tests is within the range of the different proposed equations and the simplest approach proposed by Madhav and Vitkar (1978) provided the closest results to the model.

Reference	q/c_u at failure
Present study	7·3
Greenwood (1970)	6·2
Vesic (1972)	8·8
Hughes *et al.* (1975)	10·2
Madhav and Vitkar (1978)	8·3
Sowers (1979)	10·2
Bouassida *et al.* (1995)	5·2

Table 2. Comparison of bearing ratio at failure of stone columns

The bearing improvement ratio plotted against settlement ratio, Figures 7 and 8, exhibits a peak value of 1·45 at $S/B = 3\%$. This peak is generated as the result of bulging of stone columns, followed by a gradual drop with increasing settlement ratio, reaching an ultimate bearing improvement ratio of 1·4 at failure ($S/B = 10\%$).

The third pattern of improvement consists of ballast embankment overlying a bed of soil reinforced with stone columns. The applied stress is distributed first through the ballast layer then transferred to the reinforced bed of soil. The mechanism of stress distribution in the ballast layer can be approximated by the 2:1 method, assuming the ballast layer as an elastic material. The stress reaching the interface surface between the ballast and the soft soil is partly concentrated on the stone columns and the rest is distributed on the soil surrounding the columns. This share of stress depends on the relative stiffness between the stone columns and the surrounding soil. Based on the 2:1 method, the two ballast ratios $H/B = 0{\cdot}25$ and 0·5 will distribute the applied stress along contact surfaces of 250 mm × 450 mm and 300 mm × 500 mm respectively. The difference between the two heights of the ballast embankment is clearly seen in terms of bearing ratio in Figures 5 and 6. At low bearing ratio $\leqslant 3$, no significant effect of ballast depth ratio H/B on settlement ratio was observed and the two ballast ratios generated approximately the same settlement ratio. The discrepancy between the two became more significant when the bearing ratio was gradually increased and reached its maximum value at failure.

The bearing ratios at failure are 9·6 and 12·8 for $H/B = 0{\cdot}25$ and 0·5 respectively. These values are compared with calculated values obtained from two theories, Madhav and Vitkar (1978) and Sowers (1979), as shown in Table 3. Both theories showed very close results when compared to each other for both ballast ratios, but slightly conservative results when compared to the model test results.

Table 4 summarises the average values of bearing improvement ratio at failure obtained from the model tests for the three patterns of improvement.

5.2 Settlement reduction ratio

Most of the improvement techniques focus on the reduction in settlement of soft, saturated deposits as the most critical parameter in design requirements. The present paper defines the settlement reduction ratio as the ratio of settlement generated in a treated soil to the corresponding settlement of the untreated soil at the same bearing ratio. Similarly to the discussion on bearing improvement ratio, the three patterns are discussed separately and the settlement reduction ratios are compared with several analytical theories. The discussion of the settlement reduction of the different treatment cases is limited to bearing ratio $\leqslant 6$, which is the maximum of the range for the untreated soils.

In the first set of model tests, Figures 9 and 10, where only ballast embankment of $H/B = 0{\cdot}25$ and 0·5 were used underneath the footing, the maximum settlement reduction ratios are 0·80 and 0·6 for $H/B = 0{\cdot}25$ and $H/B = 0{\cdot}50$ respectively. If the design target is to reduce the settlement of the untreated soil by 40%, then a ballast depth ratio $H/B = 0{\cdot}5$ is recommended.

Reference	q/c_u at failure	
	$H/B = 0{\cdot}25$	$H/B = 0{\cdot}5$
Present study	9·6	12·8
Madhav and Vitkar (1978)	7·7	11·6
Sowers (1979)	7·4	11·7

Table 3. Comparison of bearing ratio at failure of stone columns and ballast layer

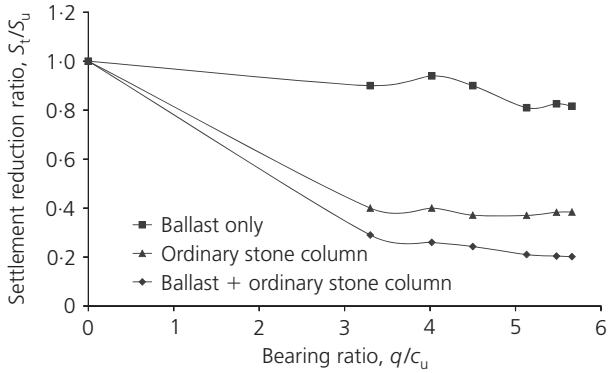

Figure 9. Bearing ratio plotted against settlement reduction ratio – $H/B = 0{\cdot}25$

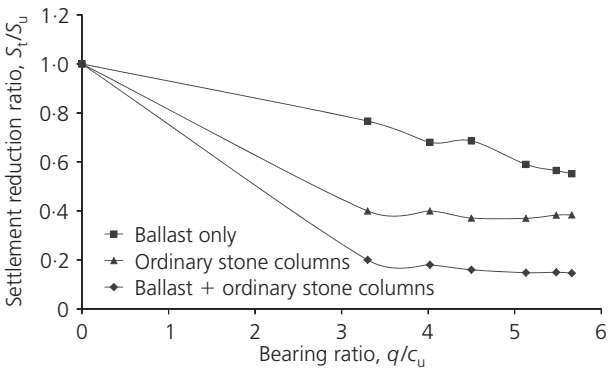

Figure 10. Bearing ratio plotted against settlement reduction ratio – $H/B = 0{\cdot}50$

The second set of model tests was performed on the bed of soil reinforced with stone columns only. Bearing ratio plotted against settlement reduction ratio is shown in Figures 9 and 10. For the range of bearing ratios $\leqslant 3{\cdot}5$, a steep reduction in settlement ratio was recognised and a value of 0·4 was quoted at bearing ratio 3·5; this remained constant with increasing bearing ratio until the end of the test. This outcome proves that the efficiency of stone columns alone can reduce the settlement of the untreated soil by 60%. For comparison purposes and according to the boundary conditions and the properties of both materials, settle-

Present study	Bearing improvement ratio at failure					
	$H/B = 0$	$H/B = 0{\cdot}25$	$H/B = 0{\cdot}5$	Stone columns	Stone columns plus ballast layer, $H/B = 0{\cdot}25$	Stone columns plus ballast layer, $H/B = 0{\cdot}5$
Average	1·00	1·1	1·3	1·4	1·9	2·3

Table 4. Average values of bearing improvement ratios for the three patterns of improvement

ment reduction ratios calculated by several theories (Balaam and Booker, 1981; FHWA, 1983; Priebe, 1995) are presented in Table 5. All settlement reduction ratios quoted in the table are in good agreement with those obtained from the model tests, apart from the one suggested by FHWA (1983), which does not take into account the full properties of the materials.

The third set of model tests investigates the combined effect of ballast embankment and stone columns on settlement reduction ratio. In Figures 9 and 10 the behaviour of this set is not far from the second set where only stone columns were used. The settlement reduction ratios at bearing ratio 3·5 are about 0·3 and 0·2 for $H/B = 0{\cdot}25$ and 0·5 respectively. Exceeding this specific bearing ratio, the settlement reduction ratio continues to decrease gradually at a slow rate and ultimately reaches the values of 0·21 and 0·17 at failure for the respective ballast depths. It can be stated that the presence of the ballast layer becomes more effective in reducing settlement at bearing ratios close to failure. Thus, from the practical point of view, it is recommended to spread a well-compacted ballast layer over the surface of the reinforced bed of soil to distribute the applied stress evenly and minimise the settlement. The settlement reduction ratios at failure for the three patterns of improvement are summarised in Table 6.

6. Conclusions

The current paper has focused on the advantages gained by implementing three patterns of improvements on a soft deposit. The first is using a ballast embankment of thickness ratio $H/B = 0{\cdot}25$ and 0·5, the second is based on a group of eight stone columns with area replacement ratio 0·196 and the third is the implementation of incorporating both a ballast layer and stone columns. The bearing ratio and settlement ratio obtained from the model tests are in harmony with several analytical approaches present in the literature. The bearing improvement ratio and settlement reduction ratio at failure are summarised in the following points.

(*a*) The first pattern of improvement, where ballast ratios $H/B = 0{\cdot}25$ and 0·5 were placed underneath the footing, revealed bearing improvement ratios of 1·1 and 1·3 and settlement reduction ratios of 0·8 and 0·6 respectively.
(*b*) The second pattern of improvement consisted of eight stone columns with area replacement ratio 0·196. A maximum bearing improvement ratio of 1·4 and the corresponding settlement reduction ratio of 0·40 were obtained.
(*c*) The third pattern of improvement combined patterns one and two. The maximum values of bearing improvement ratio at failure were 1·9 and 2·3 and the average corresponding settlement reduction ratios were 0·21 and 0·17, for ballast ratios $H/B = 0{\cdot}25$ and 0·50 respectively.

Reference	Settlement reduction ratio, S_t/S_u
Present authors	0·40
Balaam and Booker (1981)	0·48
FHWA (1983)	0·71
Priebe (1995)	0·40

Table 5. Comparison of settlement reduction ratios at failure of stone columns

Present study	Settlement reduction ratio at failure					
	$H/B = 0$	$H/B = 0{\cdot}25$	$H/B = 0{\cdot}5$	Stone columns	Stone columns plus ballast layer, $H/B = 0{\cdot}25$	Stone columns plus ballast layer, $H/B = 0{\cdot}5$
Average	1·00	0·80	0·60	0·40	0·21	0·17

Table 6. Average values of settlement reduction ratios at failure of the three patterns of improvement

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