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Ground improvement techniques for railway embankments

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A high-speed railway project for trains of speeds of up to 160 km/h is currently being constructed between Rawang and Bidor (110 km long) in Peninsular Malaysia. The ground improvement methods adopted in the project are vibro-replacement with stone columns, dry deep soil mixing (cement columns), geogrid-reinforced piled embankments with individual pile caps and removal/replacement works. This paper provides a detailed insight into the design and implementation of vibro-replacement and the deep soil mixing treatment methods used in the project. The use of plate bearing tests and field instrumentation to monitor the performance of the stone columns and soil mixing ground treatment methods is also discussed. This paper also provides a brief overview of other treatment methods implemented in this high-speed railway project such as a pile embankment with geogrids and removal/replacement works.

NOTATION

A_{col}/A	area ratio
A_{col}	area of column
a	area replacement ratio calculated as $a = (A_{col}/s^2)$ for square grid pattern of spacing, s
C	cohesion of the composite system
C_u	undrained shear strength of the soil
$C_{d\ comp}$	drained cohesion of composite soil
$C_{u\ comp}$	undrained shear strength of composite soil
$c_{creep\ col}$	creep stress of the column
$c_{d\ col}$	drained cohesion of columns
$c_{d\ soil}$	drained cohesion of in situ soil
c_h	coefficient of consolidation for horizontal flow
$c_{u\ col}$	undrained cohesion of the column
$c_{u\ soil}$	undrained cohesion of the in situ soil
D_c	constrained moduli of columns
D_s	constrained moduli of soil
d	diameter of column
d_e	diameter of the equivalent soil cylinder
E_{col}	Young's modulus of the columns
E_{comp}	Young's modulus of composite soil
E_{soil}	Young's modulus of the in situ soil
m'	proportional load on the column
m_c	constant
m_E	constant
n_2	final improvement factor
P	peak angle of shear stress
$P_{all\ coll}$	allowable load on the column
s	spacing of the columns in square grid pattern
T	shear stress
T_h	time factor for consolidation by horizontal drainage
t	time
U	degree of consolidation
U_s	degree of settlement
U_p	average degree of pore pressure dissipation
v	volume of the element
δ_{imp}	imposed stress due to dead load and live load on top of the ground surface
γ_c	unit weight of column
γ_f	unit weight of fill
ε	allowable strain (ultimate)
$\sigma_{all\ col}$	allowable creep stress
σ_{fcol}	failure stress
$\sigma_{creep\ col}$	creep stress of column
σ_h	horizontal stress calculated at the top of soft layer
σ'_{vert}	vertical stress
$\tau_{d\ col}$	undrained shear stress of the column
$\tau_{u\ col}$	undrained shear stress of the column
ϕ'	friction angle of the composite system
ϕ_c	friction angle of column
$\phi_{d\ col}$	drained angle of friction of the column
$\phi_{d\ comp}$	drained angle of friction of composite soil
$\phi_{u\ comp}$	undrained angle of friction of composite soil
$\phi_{d\ soil}$	drained angle of friction of the in situ soil
ϕ_s	friction angle of the soil layer
$\phi_{u\ col}$	undrained angle of friction of columns
$\phi_{u\ soil}$	undrained angle of friction of in situ oil

1. INTRODUCTION

The electrified high-speed railway project runs between Rawang in the state of Selangor and Bidor in the state of Perak in Peninsular Malaysia over a total length of 110 km. Figure 1 indicates the location of the project site in Peninsular Malaysia. The geotechnical design of the project includes ground improvement of the existing foundation to sustain the imposed dead and traffic loads for train speeds of up to 160 km/h. The client's design requirements are a maximum post-construction settlement of 25 mm in six months and a differential settlement of 10 mm over a chord spanning 10 m. In addition, the degree of consolidation to be achieved is not to be lower than 85–90%. The required minimum long-term factor of

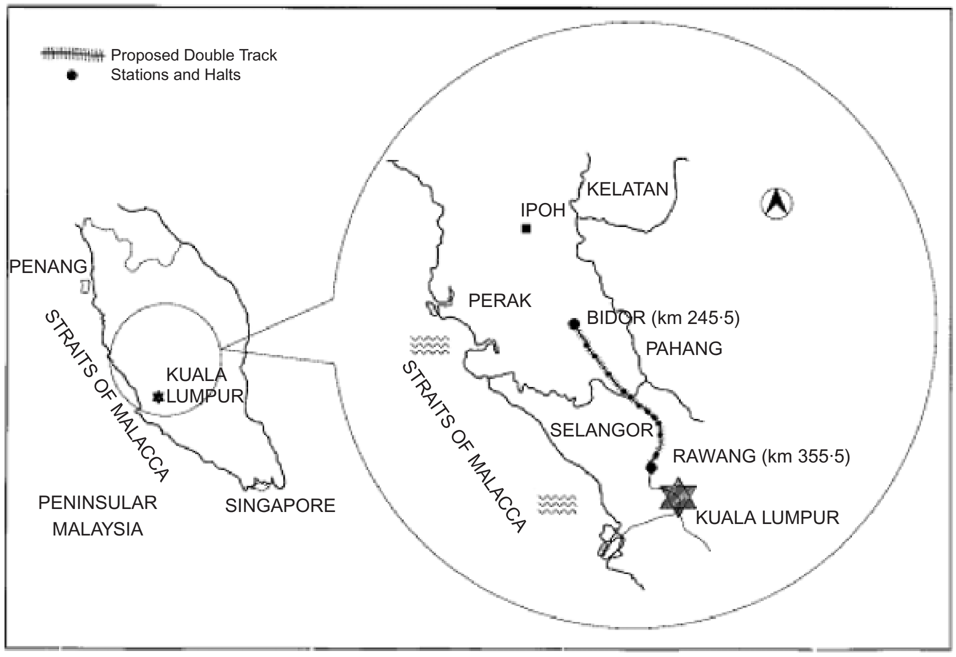

Figure 1. Location of project site in Peninsular Malaysia

safety for slope stability was 1·5. Due to the stringent settlement restrictions and the fast-track nature of the project, an array of ground improvement techniques had to be implemented in locations with soft soils or loose sands on which proposed high embankments were identified. Ground improvement was thus required to ensure adequate performance of the embankments in terms of settlement and slope stability as well as completion of the project within the required project duration.

This paper provides a detailed insight into the vibro replacement with stone columns and dry deep soil mixing treatment methods applied in the project. Vibro replacement with stone columns is a subsoil improvement method in which large-sized columns of coarse backfill material are installed in the soil by means of special depth vibrators. Dry deep soil mixing technology is a development of the lime–cement column method. This paper also briefly discusses piled embankments with geogrids and removal/replacement, which were also treatment methods adopted in this project.

The railway embankments in the project have heights ranging from 1 to 12 m. The top of the embankment has a minimum width of 14·9 m for embankments less than 10 m in height and a width of 24·9 m for embankments greater than 10 m in height. The side slopes of the embankments have gradients of $1V{:}2H$. Berms of 3 m width are provided on either side of embankments which were greater than 5 m in height. The soils encountered on the project site are highly variable mixtures of very soft silts and clays, as well as loose sands to depths of up to 30 m. Two approaches were needed for the treatment process due to construction constraints: (*a*) treatment of the full width of the embankment was required in locations where the new alignments needed the construction of two new tracks; (b) treatment at locations where a new track was to be first constructed while the existing live track was to be later rehabilitated. Treatment in the first stage would be for the width of the embankment under the new track only. Treatment for the second stage would include treatment under the proposed rehabilitated track which would be carried out once the train operations had been shifted to the new live track.

2. VIBRO-REPLACEMENT WITH STONE COLUMNS

Vibro-replacement with stone columns is a subsoil improvement method in which large-sized columns of coarse backfill material are installed in the soil by means of special depth vibrators. The stone columns and the intervening soils form an integrated foundation support system having low compressibility and improved load-bearing capacity. Vibro-replacement with stone columns allows for the treatment of a wide range of soils, from soft clays to loose sands, by forming reinforcing elements of low compressibility and high shear strength. In addition to improving strength and deformation properties, stone columns densify in situ soil, rapidly drain the generated excess pore water pressures, accelerate consolidation and minimise post-construction settlement. Normally the columns fully penetrate the weak layer with the result that the stone column and natural soil combination develops greatly enhanced bearing capacity and reduced compressibility characteristics. The method is an ideal solution for use in embankments as it negates the effect of a 'hard point'. The dry or wet method of installation can be used depending on the proximity to the existing railway track and water sources. The size of the vibrator is around 40 cm and penetration of the vibrator into the ground with water jetting will result in a hole of diameter 50–60 cm being created. An annular space is created between the vibrator and the hole through which the stone is fed to the compaction point. The up and down motion of the vibrator is used to laterally displace the stone into the ground and at the same time compact the stone column. This will result in the creation of the required diameter of column. Figure 2 presents

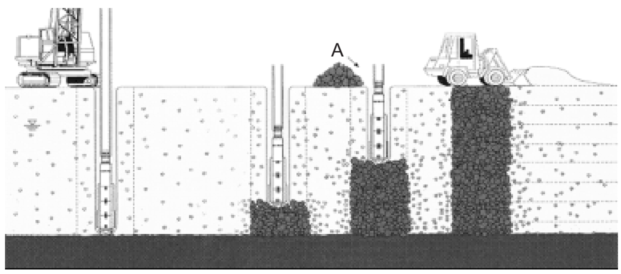

Figure 2. Schematic illustrating the stone column installation process (courtesy of Keller)

a schematic diagram illustrating the installation process of stone columns.

2.1. Stone column design methodology

The following idealised conditions are assumed in the design: the column is based on a rigid layer; the column material is incompressible; the design considers the group effect of the columns and the contribution of the attributable soil surrounding the columns; column material shears from the beginning whereas the surrounding soil reacts elastically.

2.1.1. Settlement. Settlement under the embankment loads was calculated using the Priebe method.[1] This method gives consideration for improvement, overburden and compatibility control with the use of the various improvement factors. The process is repeated for each of the various soil layers. The reader is referred to Priebe,[1] Arulrajah and Affendi[2] and Bo and Choa[3] for further details on the methodology of the settlement design for stone columns.

2.1.2. Time rate of settlement. Time rate of settlement can be calculated by using the Terzaghi equation. The time factor for a degree of consolidation of 90% can be obtained from the Balaam and Booker chart,[4] shown in Figure 3, which is applicable for rigid inclusions. The equations relevant to these calculation are as follows

$$t = T_h d_e^2 / c_h \tag{1}$$

$$d_e = 1{\cdot}128 \times \text{spacing (for square grid)} \tag{2}$$

$$T_h = 0{\cdot}044 \tag{3}$$

(Balaam and Booker chart for $U = 90\%$, $d_e/d = 3$), where t is time; T_h is the time factor for consolidation by horizontal drainage; d_e is the diameter of the equivalent soil cylinder; and c_h is the coefficient of consolidation for horizontal flow.

2.1.3. Strength properties of improved ground. Stone columns deform until any overload has been transferred to the neighbouring soil. The stone columns receive an increased portion of the load, m', which depends on the area ratio, A_{col}/A, and the final improvement factor n_2. The process described below has to be repeated for each of the various soil layers

$$m' = (n_2 - 1)/n_2 \tag{4}$$

where $\boldsymbol{m'}$ is the proportional load on the column.

The cohesion of the composite system depends on the proportional area of the soil and can be calculated as follows

$$C' = (1 - m')C_u \tag{5}$$

where C' is the cohesion of the composite system and C_u is the undrained shear strength of the soil.

The shear resistance from the friction of the composite system can be determined as follows

$$\tan\phi' = m' \tan\phi_c + (1 - m')\tan\phi_s \tag{6}$$

where ϕ' is the friction angle of the composite system; ϕ_c is the friction angle of the column; and ϕ_s is the friction angle of the soil layer.

2.1.4. Stability. The improved cohesion and friction angle values of the soil–column matrix is calculated from the final improvement factor, n_2, and these values are input into a slope stability analysis program to attain the factor of safety of the improved ground.

2.1.5. Design details. Based on the analyses of the stone column areas, the following design parameters and design spacing were adopted

(*a*) diameter of column, $d = 0{\cdot}8$ to $1{\cdot}0$ m
(*b*) unit weight of column, $\gamma_c = 22$ kN/m^3
(*c*) friction angle of column, $\phi_c = 40°$
(*d*) constrained moduli of columns, $D_c = 120$ MPa
(*e*) constrained moduli of soil, $D_s = 100 \times C_u = 500 \times \text{SPT}$
(*f*) unit weight of fill, $\gamma_f = 20$ kN/m^3
(*g*) traffic load = 30 kN/m^3.

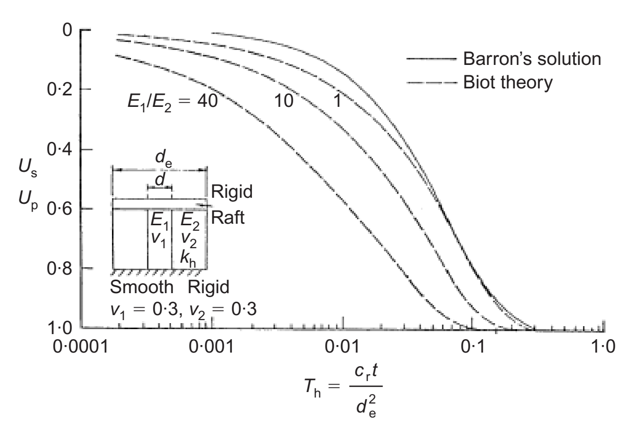

Figure 3. Time rate of settlement of stone columns for $d_e/d = 3$ (Balaam and Booker[4])

For soft soils conditions encountered in the Rawang to Bidor stretch, stone column spacings were generally in the range 1·8–2·3 m for embankment heights of 5–12 m.

Predicted total settlements were of the order of 0·3–0·5 m. Factors of safety for slope stability were greater than 1·5. Time required for 90% degree of consolidation in the predominantly sandy silts was less than two months. The treatment area ratio for the stone columns varied from 13 to 20%, depending on the design spacings for the stone columns.

2.2. Stone column installation

Arulrajah *et al.*,[5] have described the soil conditions and soil parameters relevant for stone column design in the project site. The results of site investigations revealed the presence of a wide range of soils along the track, ranging from very soft silty clay or clayey silt to loose silty clayey sand. Figure 4 shows a typical cone penetration test (CPT) plot at one such stone column treatment location.

Stone columns were used to treat soils over about 14 km length of the railway line. Approximately 1 100 000 linear metres of 0·8–1·0 m diameter stone columns were installed on the project site to depths of 6–30 m.

Figure 5 shows the schematic diagram of stone column treatment works as carried out in the project at locations of new alignment. Figure 6 shows the schematic diagram of stone column treatment works as carried out in the project at locations next to the existing railway track.

2.3. Plate load testing on stone columns

After completion of stone column installation, plate load tests were carried out on single-columns, or groups of four columns for acceptance purposes. The load was applied on the stone column and the soil surrounding the column. For the first cycle, the allowable design load was applied and maintained for a 24 h duration. In the second cycle, a maximum load of 150% of the

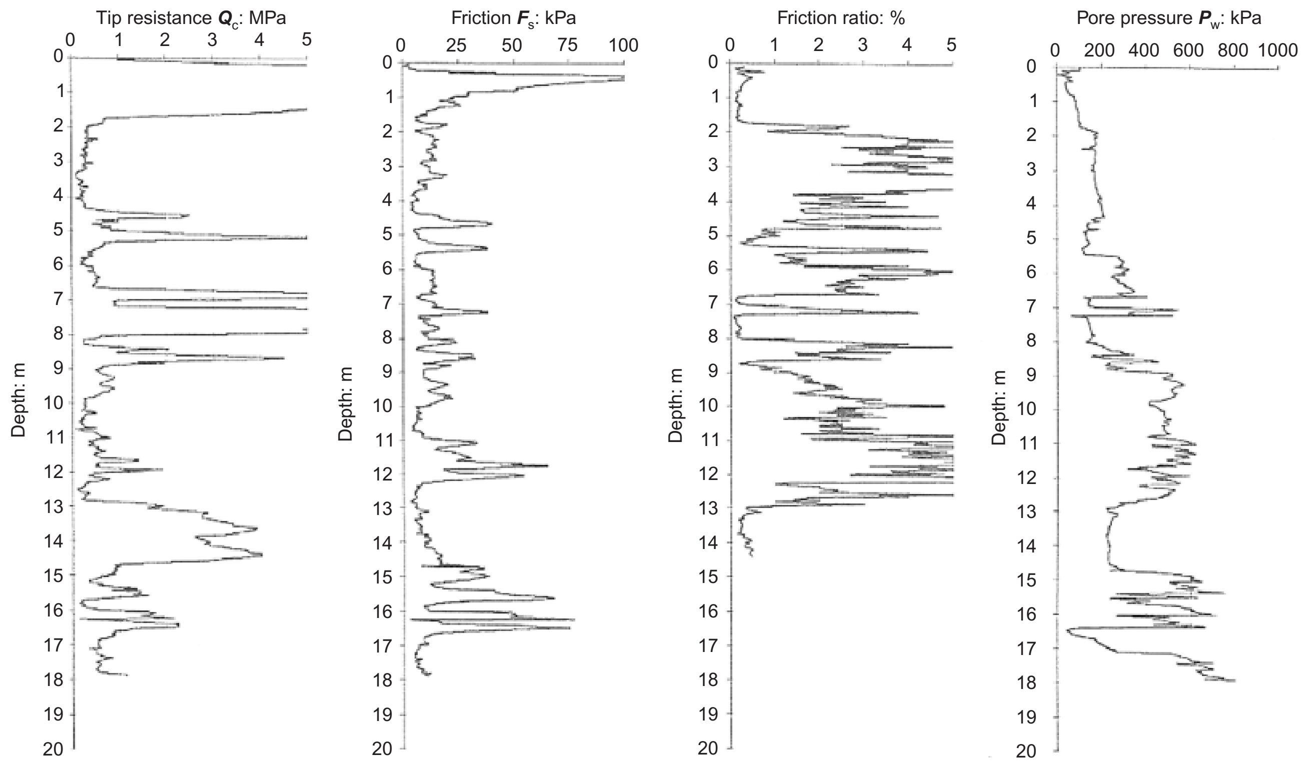

Figure 4. Plot of typical pre-treatment CPT result at stone column location (chainage 352130)

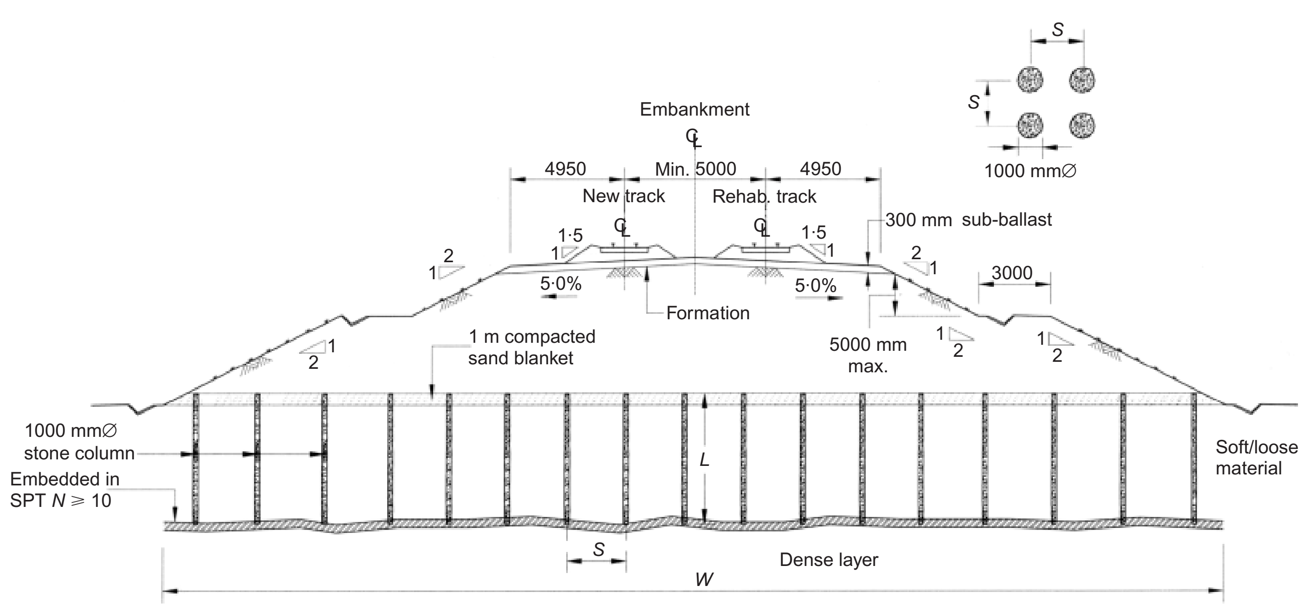

Figure 5. Schematic of stone column treatment scheme for new alignment comprising two new tracks (dimensions in mm)

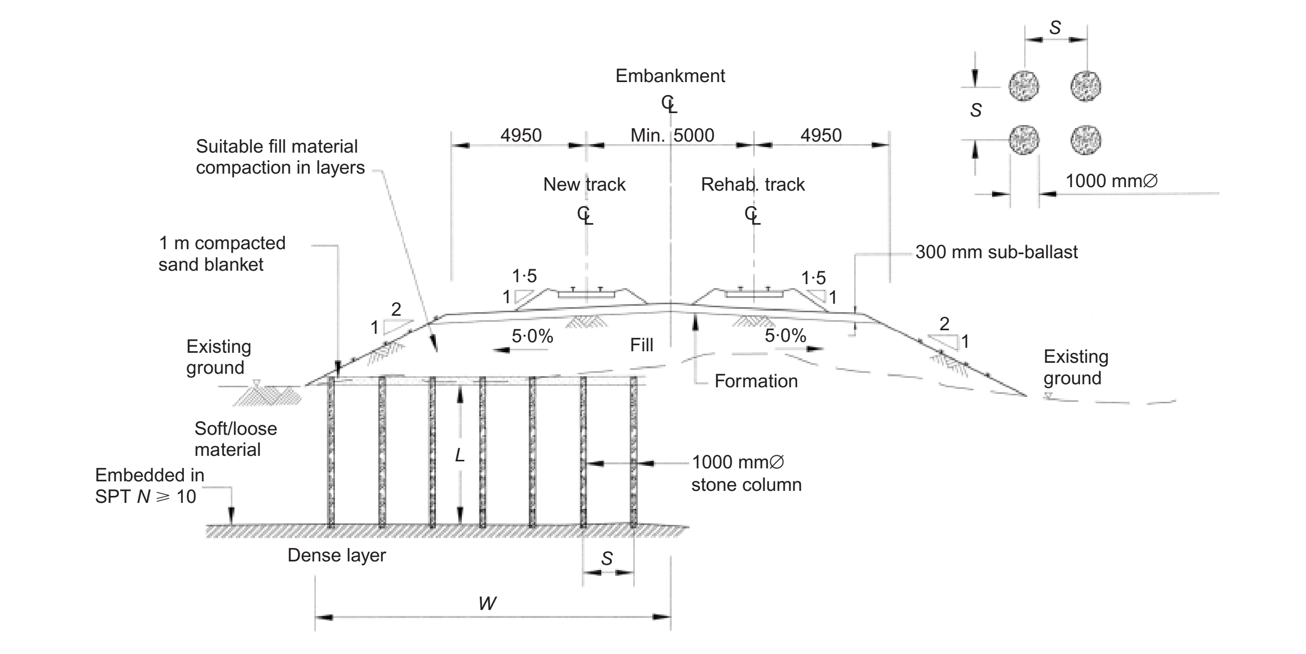

Figure 6. Schematic of stone column treatment scheme for partial width treatment next to the existing railway track (dimensions in mm)

design load was applied. The acceptance requirement of the load test was that the settlement should not exceed 50 mm under the allowable design load and not exceed 80 mm under 150% of the allowable design load. The size of plate used for the load test was 1·5 m by 1·5 m for a single column and 3 m by 3 m for a group of four columns. Figure 7 presents the schematic diagram showing the plate load test set-up for a single column load test. The results of a typical single column plate load test carried out in the project are presented in Figure 8.

2.4. Field instrumentation of stone columns

Extensive field instrumentation was carried out in the stone column treatment areas after the installation of the stone columns. The majority of the field instrumentation comprised surface settlement plates and settlement markers. The surface settlement gauges on site consistently indicated settlements occurring for each additional lift and minimal post-construction settlements. Figure 9 presents typical results from a settlement plate installed on the project site. Figure 10 presents a typical Asaoka plot for the said settlement plate, which indicates that the degree of consolidation of the improved ground at the location had achieved 94%. The long-term performance of the stone columns were predicted by means of the Asaoka method of back-analyses based on the field settlement results. Settlement markers were also placed to monitor the long-term performance of the stone columns after the railway tracks were placed. The long-term monitoring results indicated that the stone columns had performed very

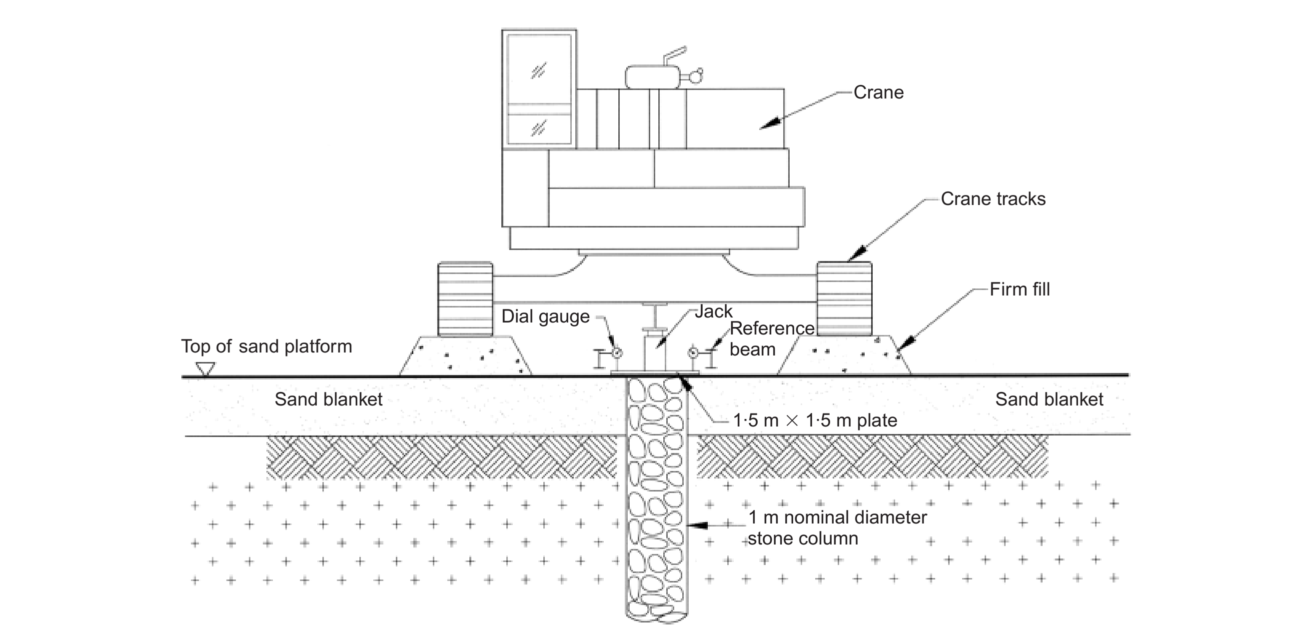

Figure 7. Schematic diagram showing single column plate load test set-up

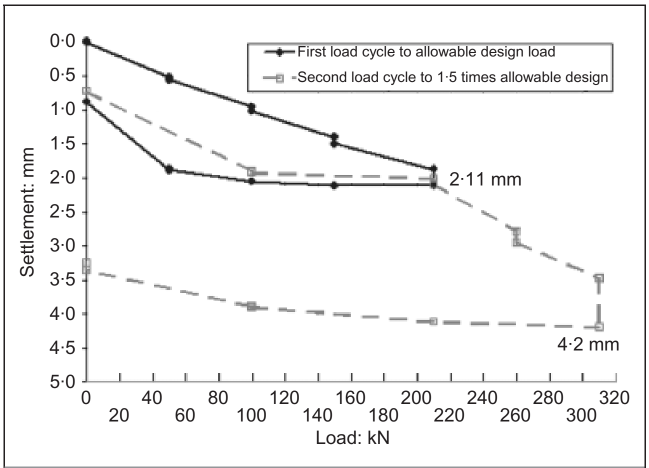

Figure 8. Load–settlement curves for a single column plate load test (chainage 352066)

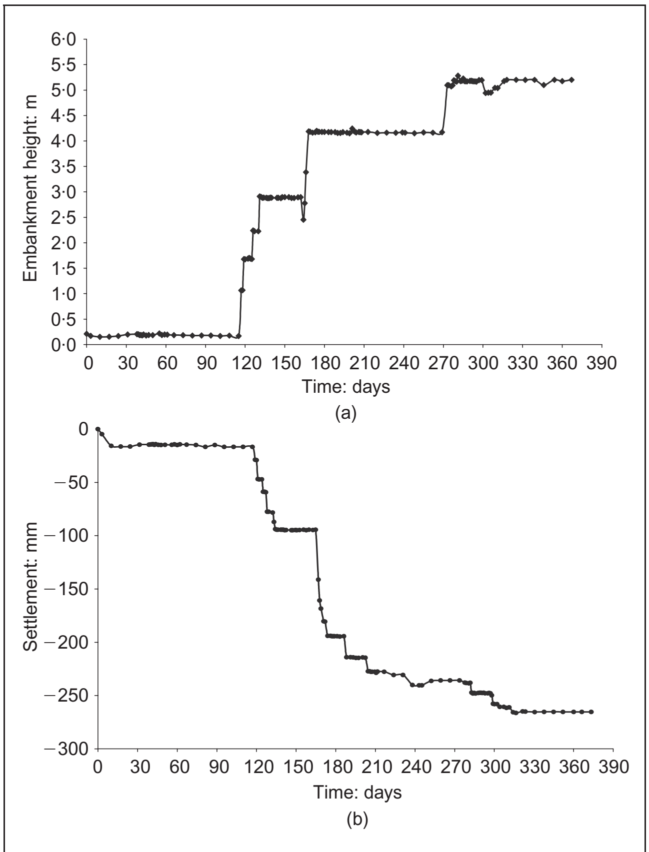

Figure 9. Plot showing the result of a settlement plate installed in a stone column treatment area (chainage 291050)

well in the area in which they were installed and within the predictions made at the design stage.

3. DRY DEEP SOIL MIXING (CEMENT COLUMNS)

Dry deep soil mixing (DSM) technology is a development of the lime–cement column method. It is a form of soil improvement involving mechanical mixing of in situ soft and weak soils with a cementitious compound such as lime, cement or a combination of both in different proportions. The mixture is often referred to as the binder. The binder is injected into the soil in a dry form. The moisture in the soil is utilised for the binding process, resulting in an improved soil with higher shear strength and lower compressibility. The removal of the

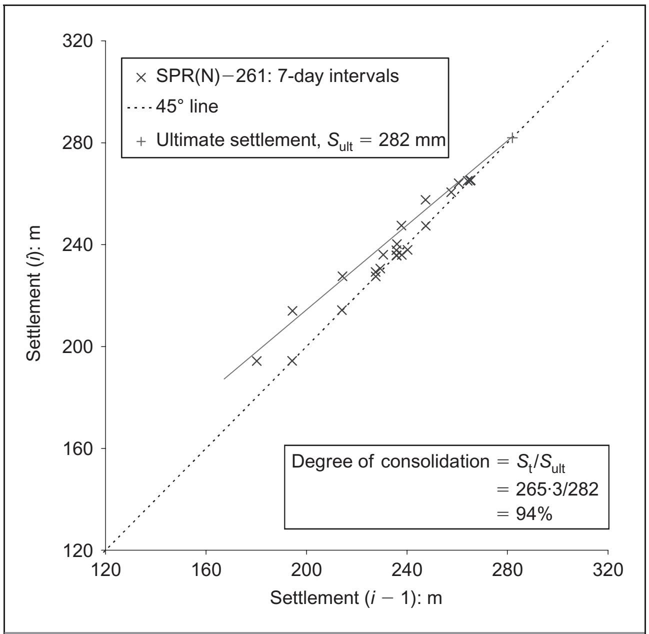

Figure 10. Asaoka plot and determination of degree of consolidation for a settlement plate (chainage 291050)

moisture from the soil also results in an improvement in the soft soil surrounding the mixed soil. Holm[6,7] provides further details on this technique.

Ground improvement by means of DSM allows for the treatment of a wide range of soils, ranging from soft clays to loose sands by forming stronger reinforcing elements of low compressibility and high shear strength. The technology is primarily used to reduce subsidence and increase the shear strength and bearing capacity of the composite soil mass. It can also be used in cases in which the reduction of vibrations is required. For example vibrations caused by high-speed trains can be reduced by the dry DSM technique in order to achieve an acceptable dynamic performance of the rail system.[6,7] Cement was used as the binding agent in the project, consisting of standard Portland type, grain sizes 0–0·01 mm, and with approximately 65% of activated CaO. The strength develops differently over time depending on the type of soil, amount of cement and ratio of proportion used. In most cases, the strength starts to increase after a few hours and then continues to increase rapidly during the first week. In normal cases, approximately 90% of the final strength is reached about three weeks following installation.

A typical dry DSM unit consists of a track-mounted installation rig fitted with a leader and a drill motor. The binder is carried in pressurised tanks, which are mounted on the rig itself or on a separate shuttle. Mixing is achieved by using an auger-mixing tool connected to the drill motor by a Kelly bar. The mixing tool is drilled down to firm ground or the intended depth. Once at the required depth, the tool is drilled out with the simultaneous injection of the binder. The binder is transported from the container to the mixing point in the ground in a dry state using compressed air. The rate of rotation, rate of withdrawal and the rate of injection of binder are adjusted such that the desired amount of binder is thoroughly mixed with the soil. The amount of binder is usually in the range 100–150 kg/m^3 of soil. The final result of the deep soil mixing process is a soil mass in the shape of a cylindrical column with improved deformation and shear resistance characteristics.

3.1. Dry deep soil mixing design methodology

The design philosophy for dry DSM is to produce a stabilised soil mass that mechanically interacts with the surrounding natural soil. The intention is not to produce rigid pile-like elements which will carry all the load. This method of semi-rigid stabilisation is often referred to as the 'soft treatment'. The 'soft treatment' can be achieved by designing with low binder contents, which can achieve improved shear strength values (typically undrained shear strengths ranging between 100 and 250 kPa depending on characteristics of the in situ soil). The applied load is partly carried by the columns and partly by the natural soil between the columns. Therefore, a too rigidly stabilised material is not necessarily the best solution since such a material will prevent an effective interaction and load distribution between the stabilised soil mass and surrounding natural soil.

The design approach and technical development for the dry DSM in order to evaluate improved deformation and shear strength parameters are derived from the work carried out by Broms[8,9] and the Swedish Geotechnical Society,[10] and are summarised below.

3.1.1. Stress and load on stabilised columns. The failure stress that the column can sustain is

$$\sigma_{\text{f col}} = 2c_{\text{u col}} + 3(\sigma_{\text{h}} + 5c_{\text{u soil}}) \tag{7}$$

where $c_{\text{u col}}$ and $c_{\text{u soil}}$ are the undrained cohesion of the column and in situ soil respectively and σ_{h} is the horizontal stress calculated at the top of soft layer using $K = 1$ and 50% of the embankment load.

The creep stress of the column is generally calculated as

$$\sigma_{\text{creep col}} = m_{\text{c}}\sigma_{\text{f col}} \tag{8}$$

The Young's modulus of the columns can be estimated as

$$E_{\text{col}} = m_{\text{E}}\sigma_{\text{creep col}} \tag{9}$$

where m_{c} and m_{E} are constants and their values depend on type of in situ soil as shown in Table 1.

To ensure that the resultant settlement on the treated ground is less than 0·5% of the treated depth, it is a general practice to limit the allowable stress on the column to 70% (higher values can be used depending on the soil condition) of the creep stress

$$\sigma_{\text{all col}} = 0{\cdot}7\sigma_{\text{creep col}} \tag{10}$$

The allowable load on the column can be calculated as

Soil description	m_{c}	m_{E}
Clayey silt	0·8–0·9	150–200
Silty clay	0·8	150–200
Clay	0·7–0·8	150
Organic clay	0·6–0·7	100
Peat	0·6	50–75
Silty, clayey sand	0·9	200–250

Table 1. Values of constants m_{c} and m_{E} applicable for deep soil mixing design

$$P_{\text{all col}} = \sigma_{\text{all col}} A_{\text{col}} \tag{11}$$

The spacing of the columns in square grid pattern can be assessed as

$$s = \left(\frac{P_{\text{all col}}}{\sigma_{\text{imp}}}\right)^{0{\cdot}5} \tag{12}$$

where σ_{imp} is the imposed stress on behalf of dead load and live load on top of the ground surface.

3.1.2. Shear strength of the column. The governing equation for shear strength is the Mohr–Coulomb equation

$$\tau = c + \sigma'_{\text{vert}} \tan(\phi) \tag{13}$$

The undrained shear strength of the column is assumed as the undrained cohesion of the column

$$\tau_{\text{u col}} = c_{\text{u col}} \tag{14}$$

where $c_{\text{u col}}$ is assumed as 100–250 kPa depending on the characteristics of the binder and in situ soil.

The drained shear strength of the columns is calculated as

$$\tau_{\text{d col}} = c_{\text{d col}} + \sigma'_{\text{vert}}\tan(\phi_{\text{d col}}) \tag{15}$$

where $c_{\text{d col}}$ is assumed as 30% of $c_{\text{u col}}$ and $\phi_{\text{d col}}$ is assumed as 40–45°.

3.1.3. Composite soil parameters. The soil within the stabilised block will be treated as a composite soil matrix with new soil parameters. The Young's modulus and undrained cohesion of the composite soil is estimated as follows

$$E_{\text{comp}} = aE_{\text{col}} + (1 - a)E_{\text{soil}} \tag{16}$$

$$c_{\text{u comp}} = ac_{\text{u col}} + (1 - a)c_{\text{u soil}} \tag{17}$$

$$c_{\text{d comp}} = ac_{\text{d col}} + (1 - a)c_{\text{d soil}} \tag{18}$$

19	$\phi_{\text{u comp}} = \tan^{-1}[a\tan(\phi_{\text{u col}}) + (1-a)\tan(\phi_{\text{u soil}})]$

20	$\phi_{\text{d comp}} = \tan^{-1}[a\tan(\phi_{\text{d col}}) + (1-a)\tan(\phi_{\text{d soil}})]$

where E_{col} and E_{soil} are Young's modulus; $c_{\text{u col}}$ and $c_{\text{u soil}}$ are undrained cohesion; $c_{\text{d col}}$ and $c_{\text{d soil}}$ are drained cohesion; $\phi_{\text{u col}}$ and $\phi_{\text{u soil}}$ are undrained angle of friction; $\phi_{\text{d col}}$ and $\phi_{\text{d soil}}$ are drained angle of friction of the column and in situ soil, respectively; and a is the area replacement ratio calculated as $a = (A_{\text{col}}/s^2)$ for square grid pattern of spacing, s. In the calculations it can be assumed that the settlements will be equal in the soil and in the stabilised columns to ensure compatibility.

3.2. Dry deep soil mixing installation

The results of soil investigation at the treatment area revealed the presence of a wide range of soils along the track, ranging from very soft silty clay or clayey silt to loose silty clayey sand.[11] Figure 11 shows a typical CPT plot at the DSM treatment location.

Deep soil mixing treatment was used to treat soft soils over an 800 m length of the railway line. Over 50 000 linear metres of 0·6 m diameter columns were installed at the site to depths of 6–14 m.[11] The embankment heights in the DSM treatment areas varied between 1·5–3 m. Column spacing generally ranges between 1·0–1·5 m. Typically the spacing of the column grids (square/rectangle) varies between 1·0–1·3 m centre to centre under the location of the proposed rail tracks and 1·4–1·5 m centre to centre in the remaining area underneath the embankment. The strength of the columns used directly under the proposed track was 250 kPa and that used in other areas was 150 kPa. Figure 12 shows the schematic diagram of dry deep soil mixing treatment works as carried out in the project at locations of new alignment.

3.3. Plate load testing on dry deep soil mixing columns

Figure 13 presents the schematic diagram of a group of four-column plate load test set-up for DSM. The requirements of the plate load test were that the settlement should not exceed 50 mm under the allowable design load and not exceed 80 mm under 150% of the allowable design load. Figure 14 presents the typical load settlement curve of a four-column plate load test at the treatment area which shows settlement within 7 mm for 150% of design load.

3.4. Field instrumentation of dry deep soil mixing

During the construction of the embankment over the treated ground, settlements and lateral movements of the embankment were monitored using rod settlement gauges and inclinometers. Typical results from the rod settlement gauges are shown in Fig. 15. The settlement gauges showed virtually no settlement (< 10 mm) for an embankment of height ranging from 1 to 1·5 m. Typical results of lateral displacement (perpendicular to the alignment of the embankment) are shown in Figure 16. The inclinometers showed lateral movements to be within 15 mm in the direction perpendicular to the embankment alignment. The inclinometer measurements presented in the figure were monitored for seven months. Maximum displacement was observed at the ground level.[11] Details of the field instrumentation of the dry deep soil mixing works at the project site have been described by Raju.[11]

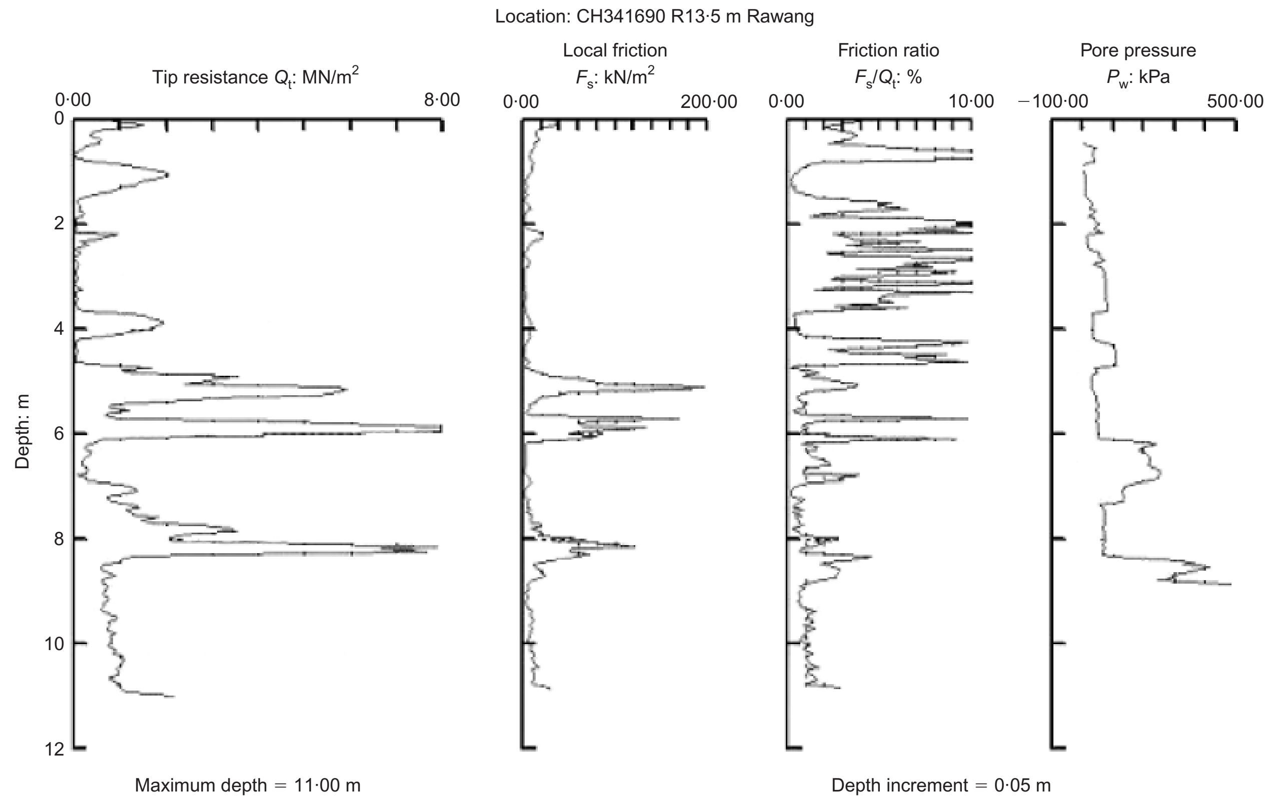

Figure 11. Plot of typical pre-treatment CPT result at deep soil mixing location (chainage 341690)

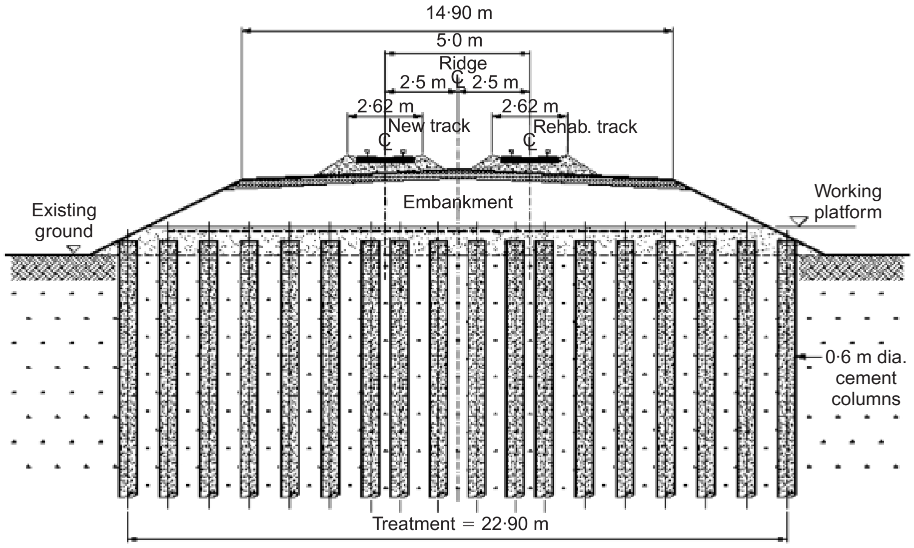

Figure 12. Schematic of dry deep soil mixing treatment scheme (Raju[11])

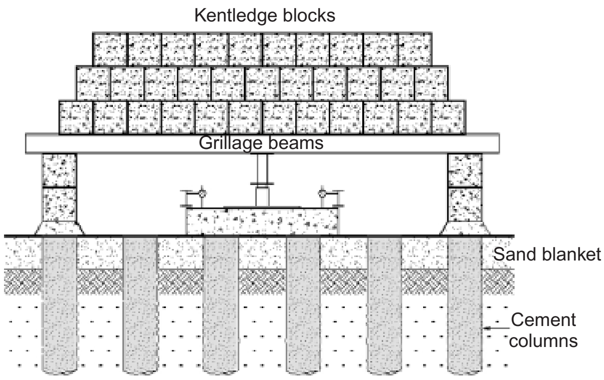

Figure 13. Schematic diagram showing four-column plate load test set-up (Raju[11])

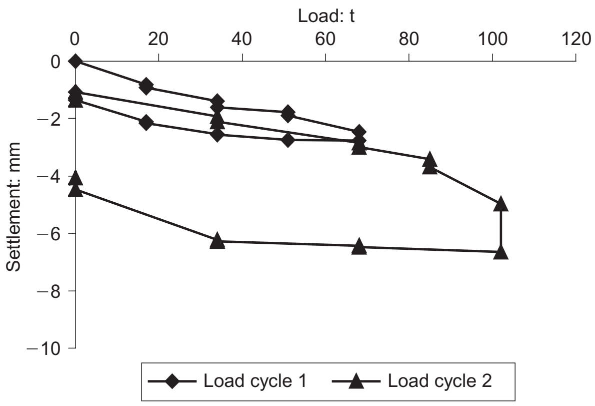

Figure 14. Typical load–settlement curve of a four-column plate load test (Raju[11])

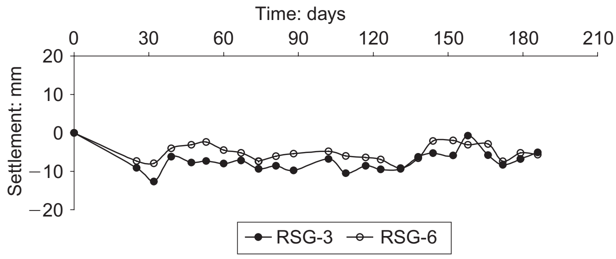

Figure 15. Time–settlement plot showing the typical results of rod settlement gauges installed in a deep soil mixing treatment area

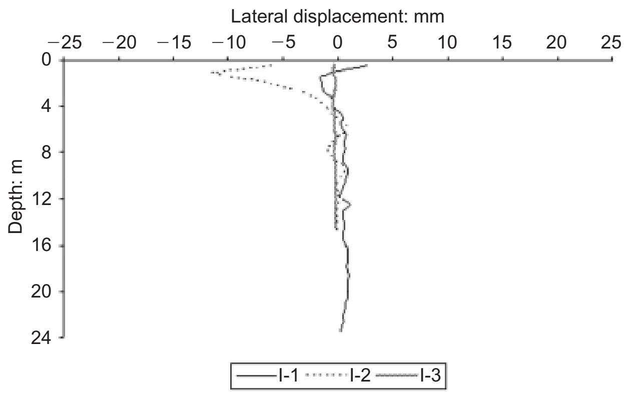

Figure 16. Depth–lateral displacement plot showing the typical results of inclinometer measurements in a deep soil mixing treatment area

4. GEOGRID-REINFORCED PILED EMBANKMENTS WITH INDIVIDUAL PILE CAPS

Piled embankments were designed for the railway bridge approach transitions. Piling allows for the embankments to be constructed rapidly without any slowdown in the construction rate or sequence. Piled embankments will also eliminate the effect of settlement and stability problems. It can be safely assumed that all the embankment loads will be transferred through the piles down to the dense underlying formation below.

Piled embankment with geogrids and individual pile caps are supported by three distinct actions: the piles reinforce and stiffen the underlying subsoil, the piles give direct support to the embankment transferred through arching action between adjacent pile caps and finally where a geogrid is used and laid over the pile caps its tension will provide support and prevent lateral spreading of the embankment. The advantage of using geogrids is that geogrids absorb the stress induced during construction until arching is formed and prevents lateral movement of the soil. The design of the geogrids involves calculations for the serviceability and ultimate limit states, which incorporate the future, anticipated dead and live loads. Individual pile caps were designed for usage with the geogrids as they were found to be more economical in comparison with a continuous slab. In the project, piles of size 250 mm by 250 mm (Concrete Grade 45) were installed at 1·5 m square spacing. The geogrid design was carried out as per BS 8006,[12] and incorporated the published method of Hewlett and Randolph,[13] utilising allowable strain (ultimate) $\varepsilon = 12\%$ and allowable strain (serviceability), $\varepsilon = 5\%$. A sand blanket was also provided just below the pile cap to provide a working platform and some lateral restraint on the pile during driving. Figure 17 shows the schematic drawing of geogrid-reinforced piled embankments.

For the design of transitions to railway bridges, the pile lengths were reduced by 1 m for each pile spacing from the integrated bridge abutment slab. By this approach, the piles near the bridge are long and will settle little since they were designed to carry the full weight of the embankment. Further away the piles are shorter and will settle more. The ground conditions at the location of the piled embankment transitions are homogeneous which enables this design intent to be achieved at site. This solution thus provides a gradual transition from bridge to embankment and ensures there is no sudden change in settlement profile. This design significantly reduces lateral pressure on the bridge abutment piles and eliminates differential settlement between the adjacent ground and the bridge hard point.[5,14] Figure 18 shows a schematic diagram of geogrid-reinforced piled embankments used for the railway bridge approach transitions.

5. REMOVAL/REPLACEMENT

This method is possibly the most widely used and economical treatment option for improving the presence of shallow soft soil deposits. The removal and replacement method was used in the project at locations where there was soft cohesive material present. The unsuitable materials were removed from the site and the excavation trench and they were replaced with suitable fill materials, which were subsequently compacted.[5] Excavation to depths greater than 2 m may require temporary protection methods such as the use of temporary sheet piles. Non-woven geotextiles were provided as a separation layer at the base of the excavation works to ensure an effective separation between the in situ soils at the base of the excavation and the suitable fill. Figure 19 shows a schematic drawing of the removal and replacement works.

6. CONCLUSION

In this paper, the various ground improvement techniques used in a major high-speed railway project in Malaysia have been discussed.

Vibro-replacement with stone columns allows for the treatment of a wide range of soils, ranging from soft clays to loose sands by forming reinforcing elements of low compressibility and high shear strength. In addition to improving strength and deformation properties, stone columns densify in situ soil, rapidly drain the generated excess pore water pressures, accelerate consolidation and minimise post-construction settlement. In this paper, the design methodology, installation methodology, load testing and field instrumentation for vibro replacement with stone columns for railway embankments

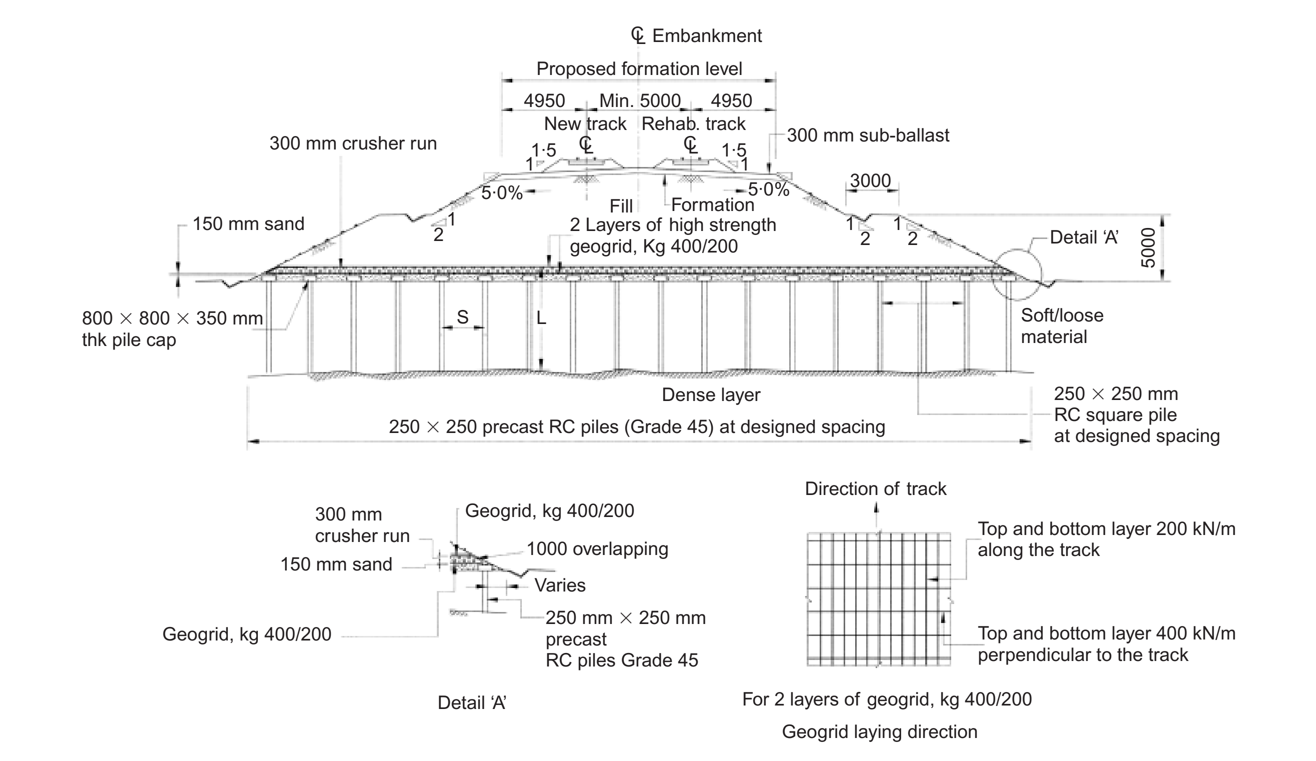

Figure 17. Schematic diagram of geogrid-reinforced piled embankments (dimensions in mm)

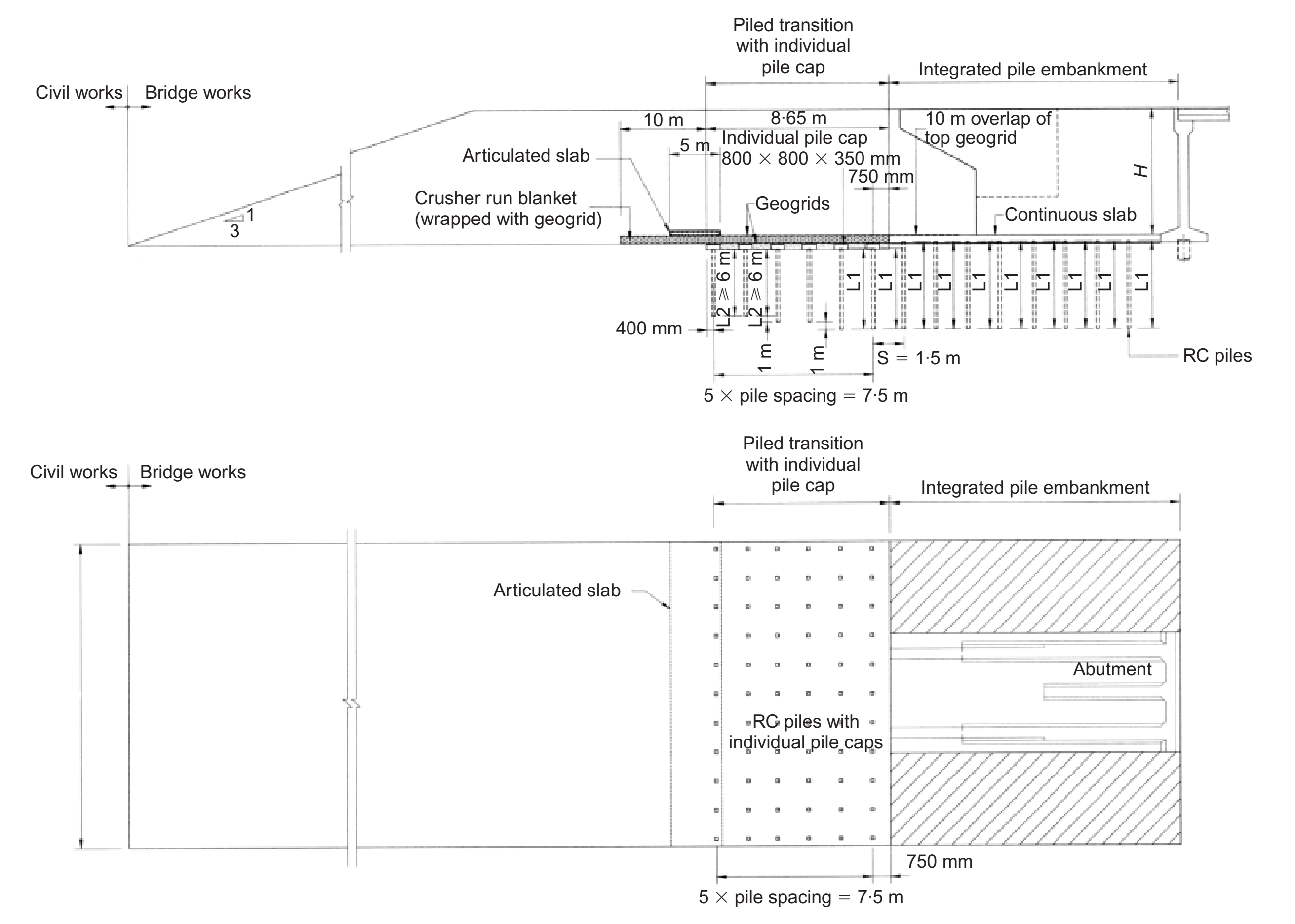

Figure 18. Schematic diagram showing bridge approach transition using geogrid-reinforced piled embankments

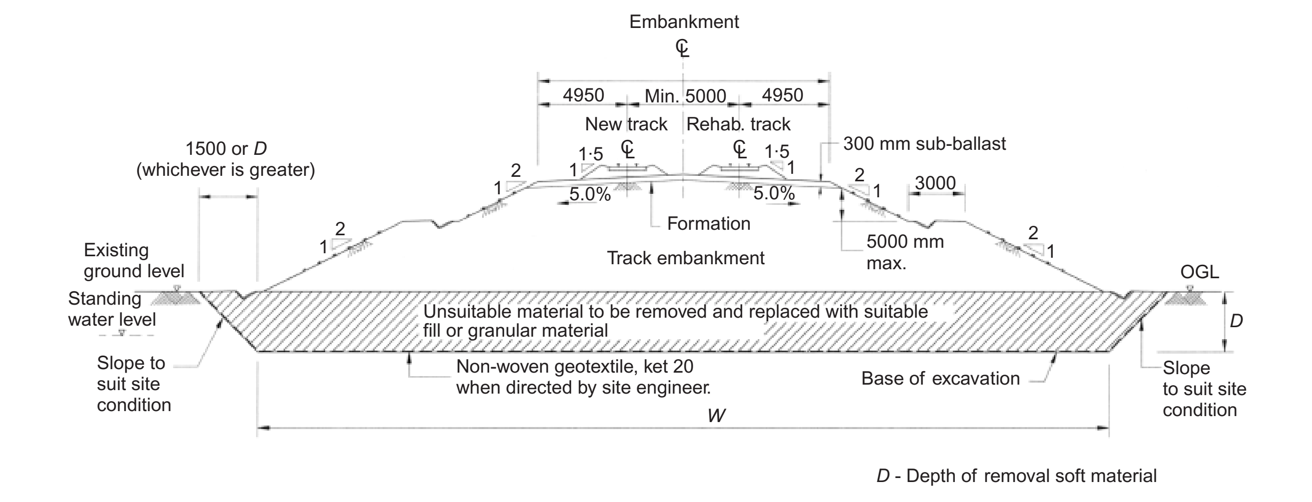

Figure 19. Schematic diagram showing typical cross-section of removal/replacement works (dimensions in mm)

have been discussed. The results from numerous load tests and settlement plates indicate that the stringent performance requirements of the new railway project were met.

Ground improvement by means of dry deep soil mixing allows for the treatment of a wide range of soils, ranging from soft clays to loose sands by forming stronger reinforcing elements of low compressibility and high shear strength. In this paper, the design methodology, installation methodology, load testing and field instrumentation for dry deep soil mixing for railway embankments have been discussed. Results from numerous load tests, settlement plates and inclinometers indicate that the stringent performance requirements of the new railway project were met.

Piled embankments were designed for use for the bridge approach transitions and allow for the embankments to be constructed rapidly without any slowdown in the construction rate or sequence. Piled embankments also eliminate the effect of settlement and stability problems.

The removal and replacement method was widely used in the project at locations where soft cohesive material was present. Removal and replacement was an economical treatment option for improving the presence of shallow soft soil deposits in the project.

The type of ground improvement method adopted in the project dependant on various factors such as type of soil, height of embankment and thickness of soft or loose deposits. The various ground improvement techniques were employed successfully in the construction of embankments in the high-speed railway project.

7. ACKNOWLEDGEMENT

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