A Vibro Stone Column Supported Test Embankment for a High-speed Rail Project in Malaysia

Y. W. Yee[1], L. H. Ooi[2] and J. Daramalinggam[3]
[1]*Keller (M) Sdn Bhd*
[2]*Gamuda Engineering*
[3]*Keller AsiaPacific Ltd*
E-mail: ywyee@keller.com.my

ABSTRACTS: The Ipoh- Padang Besar Electrified Double Track project is a multibillion dollar high-speed rail project that involved installation of double tracks, electrification work, construction of stations, bridges and tunnels. Stringent performance specifications governed all aspects of the project. Various ground improvement techniques were employed, including Vibro stone columns to support railway embankments. The primary function of Vibro stone columns was to reduce settlements of the newly constructed railway embankments. As part of the project requirements, a low test embankment supported by Vibro stone columns was built and monitored. The purpose of this test was firstly to demonstrate that Vibro stone columns would not result in "hard points" at the surface of even a low embankment. The second purpose was to validate the designed rest periods for consolidation settlements, based on the proposed calculation methods. Vibro stone column installation commenced in June 2008, embankment construction commenced in February 2009 and the test embankment was monitored up till March 2010. Throughout the monitoring period, instrumentation and visual inspection showed that no "hard points" were observed on the embankment surface. In addition, it was shown that Priebe's (1995) method adequately predicts the magnitude of settlements, and that Han & Ye's (2001) method adequately predicts the rate of settlements. The track has been operational since 2013.

KEYWORDS: Vibro stone columns, Test embankment, High-speed rail

1. INTRODUCTION

1.1 Project Background

In 2007, MMC- Gamuda JV began construction work on the Ipoh-Padang Besar Double Tracking (IPDT) project. The project, which costs over RM 12 billion involves the installation of double tracks, electrification work, construction of bridges, road-over bridges, stations and tunnels over 329 km of railway line. The alignment winds through varying ground conditions, from soft alluvial deposits to stiff residual soils. Track geometry tolerances are tight, given that passenger trains will be travelling at design speeds of 180 km/h. Among the many construction challenges, the proposed railway tracks were to be built in the existing right-of-way, adjacent to the existing tracks, without interruption to the operation of the existing trains.

In view of the construction restrictions, stringent performance requirements and varying ground conditions, different ground improvement methods were adopted in this project. Lee *et al.* (2013) describe some of the ground improvement methods that were adopted in this project. These included these driven piles at transition zones, installation of prefabricated vertical drains, the use of geotextiles for basal reinforcement and removal and replacement of soft soils. Among the ground improvement techniques used was Vibro stone columns.

Vibro stone columns has a long history of use in Malaysia (Raju & Sondermann, 2005), including the Ipoh- Rawang double tracking railway project from 2001 to 2004, and the Kuantan- Kerteh railway line from 2000 to 2001. As part of the IPDT project, an instrumented test embankment (consisting of two zones, 2 m high- 15 m x 10 m and 4 m high- 25 m x 15 m, including working platform) was constructed in Kodiang, Kedah.

1.2 Literature Review

There have been a large number of stone column load tests published over the years, for example Hughes *et al.* (1975), Greenwood (1991), Watts *et al.* (2000) and McCabe *et al* (2009). Greenwood (1991) carefully points out that the soil-stone column interaction is very different from soil-pile interaction, and therefore the value of large scale testing, which more accurately replicates the actual mechanism. McCabe *et al* (2009) presents a compilation of data from both footing-sized and embankment-sized tests, and lists settlement improvement factors from the data set. Factor such as the type of installation method, average column diameter and area replacement ratios are listed.

A recent and interesting case study by Adam *et al.* (2010) describe the application of stone columns for the foundation of the Klagenfurt football stadium in Austria. Stone columns were used in a "floating" design, to support the spectators' grandstands, and several other stadium structures. Settlements over a 4-year period (2006 to 2010) were measured and reported, and a back-analysis using 2-D finite element modelling was presented. As part of the project a six-month long large scale field test was done in 2006. A 10.5 m high earth ramp, which would be part of the final stadium was monitored. The stone columns were installed to about 14.5 m, into a clayey silt. Extensometers and piezometers were placed within the stone column treated soil, as well as below it.

1.3 Test Embankment Objectives

The present Kodiang test had the following objectives:

(a) To verify that the use of stone columns for low embankments would not result in hard-points at the embankment surface, or the so-called "mushroom effect".
(b) To determine if the design "rest periods" for the surcharge were Adequate (Tan *et al.,* 2010 describe another test embankment for the IPDT project where PVD and surcharge were used)

In some highway projects in Malaysia, piled embankment with individual pilecaps showed some "mushrooming" (Gue *et al.*, 2007). The excessive clear distance between pilecaps relative to embankment height and material properties resulted in depressions on the surface of the road. These depressions resulted in frequent maintenance of the roads, and the eventual construction of a reinforced concrete raft to resolve the issue.

One of the objectives of the Kodiang test embankment was to demonstrate unambiguously that such a phenomenon would not occur with stone columns. This is because stone column heads are fairly ductile, and the column itself picks up and sheds load in a manner very different from a piled embankment. Nevertheless, confirming that the "mushroom effect" is absent was important. This is because if it were present, it would require the use of geosynthetics or a thicker load transfer layer, which would lead to additional costs and time.

As Vibro stone columns function as drainage elements, in addition to providing reinforcement, determining the correct rest period is important for planning the construction schedule, and also the amount of earth to be used as a surcharge. Both of factors these are critical in a large railway project that stringent performance requirements.

The site location is shown in Figure 1, and a picture of the site is shown in Figure 2.

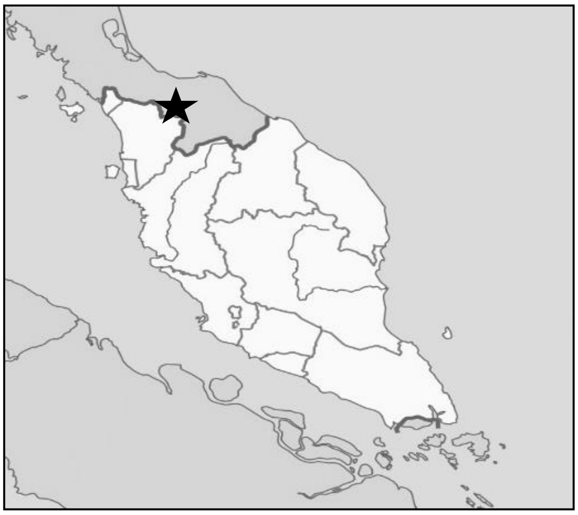

Figure 1 Map of West Malaysia and location of test embankment in Kodiang

Figure 2 Photograph of test location

2. EMBANKMENT DESIGN REQUIREMENTS

The stone columns spacing and diameters were designed to take the following design loads (Table 1), for stability and settlement analyses. The unit weight of the compacted embankment fill was taken as 20 kN/m^3.

The track was required to have a maximum total settlement of 25 mm over six months from start of service. Differential settlement was to be limited to 10 mm over a length of 10 m. During construction, the required factor of safety against slope failure was 1.2. During service life, the required factor of safety is 1.4.

3. SOIL CONDITIONS

Prior to stone column installation and embankment construction, one dynamic penetration test and one cone penetration test (CPT) was performed. The CPT plot is shown in Figure 3, and is consistent with nearby boreholes.

Based on the CPT and taking into account nearby boreholes, the soil was idealised as shown in Table 2. The correlations between undrained shear strength and constrained modulus were based on past experience in Malaysia, consistent with other published data such as Duncan & Buchignani (1976).

Table 1 Design Loads

Loading Condition	Stability Analysis	Settlement Analysis
During Construction:		
Dead load	H_{gross} x 20 kN/m^3	H_{gross} x 20 kN/m^3
Live load	10 kPa (over entire embankment)	
During Service		
Dead load	$H_{service}$ x 20 kN/m^3 + $H_{sub\text{-}ballast}$ x 22 kN/m^3	$H_{service}$ x 20 kN/m^3 + $H_{sub\text{-}ballast}$ x 22 kN/m^3 + 12.5 kPa (over ballast width)
Live load	38.3 kPa (over ballast width)	

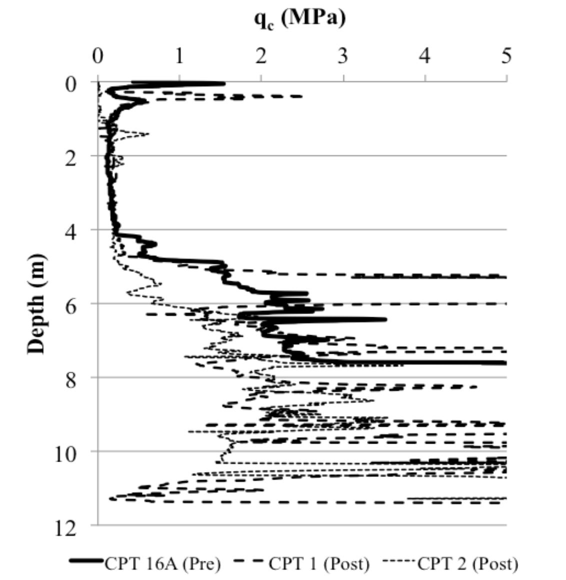

Figure 3 Cone penetration test results

4. GROUND IMPROVEMENT DESIGN

Priebe's (1995) method was used to estimate total settlements. Priebe's method uses a unit-cell concept for settlement estimation, and has the advantage of simplicity and a large database of soils and structure types behind it. By itself however, it cannot be used to estimate the rate of settlement of a structure built on stone columns. Usually, the Han & Ye (2001) method is used.

For Zone 1, based on an embankment height of 4 m (1 m working platform, 2 m permanent fill, 1 m surcharge), the total settlements were estimated at 250 mm, with a 2.2 m x 2.25 m square grid. The design length of the columns was 6 m. The stone column grid for Zone 2 is identical with Zone 1 (2.2 m x 2.25 m). Spacings on the slope of the embankment were wider at 2.5 m x 2.25 m. Figure 4 shows the working platform and embankment underlain by the stone columns.

Table 2 Idealized Soil Profile

Layer	Depth	Description	Undrained Shear Strength (kPa)	Constrained Modulus (kPa)	Consolidation Parameters
1	0.0 to 6.0	Very soft silty clay	10	1,000	c_v = 1 m^2/ year c_h = 2 m^2/ year
2	6.0 to 9.0	Stiff silty clay	60	18,000	c_v = 4 m^2/ year
3	9.0 to 13.0	Stiff silty clay	Settlements assumed negligible. Borehole data indicates SPT N values from 11 to 14, CPT indicates q_c values greater than 1.5 MPa		
4	> 13.0	Limestone	Settlements assumed negligible. SPT hammer rebound. RQD values between 50 % to 100 %.		

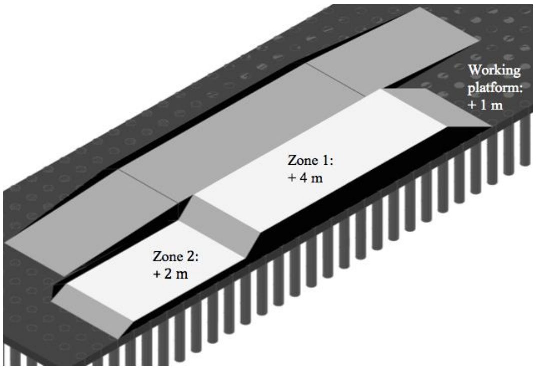

Figure 4 3-dimensional view of stone column layout under test embankments

5. SQUENCE OF ACTIVITIES

First, a 1 m thick working platform was constructed using sand. The working platform was constructed in mid-May 2008. Then Vibro stone columns were constructed using the dry bottom-feed method of construction. The columns were installed in a grid below and beyond the embankment. For the 72 columns installed directly under the embankment, the average depth of the columns was 6.0 m. The maximum column depth was 7.4 m and the minimum column depth was 5.3 m. 54 of the 72 columns were between 5.5 m and 6.5 m in depth. The range of as-built stone column depths was due to variations in thickness of the compressible soils and overall, the column installation depths were close to the design value of 6.0 m. One column just outside the test embankment was loaded to 15 tons, as a routine quality control test.

The sequence of key events is shown in Table 3. After the stone columns were installed, some additional soil testing was done. Two CPTs and some vane shear tests were performed in November 2008. (Soil samples were also retrieved, indicating that the soil is indeed a silty clay or clayey silt, generally of high plasticity.) Figure 3 shows the post-installation CPT plots. Comparing the CPT results with the test done prior to stone column installation we can draw the conclusion that there is no noticeable change of tip resistance, and hence undrained shear strength, after column installation. This is consistent with past experience in similar soil conditions. Installation of columns in clayey soils does not increase the shear strength of the in-situ soils, as clayey soils do not densify under vibrations, unlike sandy soils. (Unfortunately, no CPT data was available after completion of embankment construction and the rest period.)

Table 4 lists the instruments that were installed and monitored and Figure 5 shows the layout. However, some of the instruments were disturbed or inadvertently damaged during embankment construction or during monitoring.

Table 3 Dates of Key Events

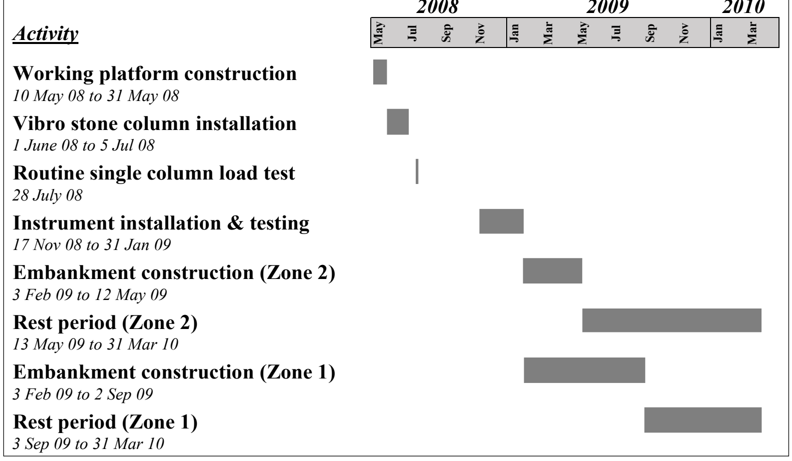

Table 4 Instruments used in Test Embankments

Location	Instruments
Zone 1	6 Nos. Rod Settlement Gauges
	6 Nos. Surface Settlement Markers
	2 Nos. Total Stress Cells
	2 Nos. Piezometers
	2 Nos. Extensometers
Zone 2	4 Nos. Rod Settlement Gauges
	6 Nos. Surface Settlement Markers
Outside Zone 1	6 Nos. Surface Settlement Markers
	4 Nos. Ground Heave Markers
	1 No. Piezometer

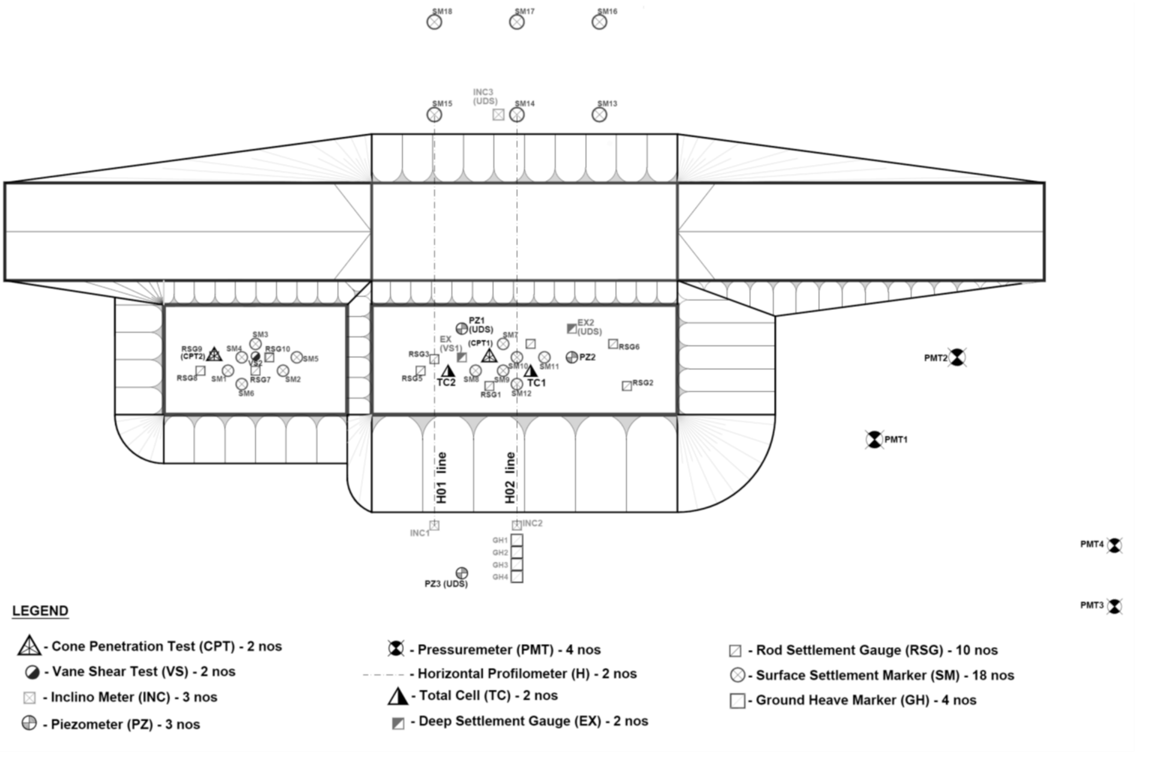

Figure 5 Layout of instruments

6. RESULTS & DISCUSSION

6.1 Results from Zone 1

Rod Settlement Gauge Readings

Six rod settlement gauges (RSG) were installed in Zone 1, allowing settlements to be monitored as the embankment is built up. Careful compaction using a light hand-director roller compactors (Figure 6) were used when filling around the RSGs. Over the monitoring period (till March 2010) the average settlement measured by the RSGs was 163 mm (Figure 7) As a 1 m thick working platform was constructed over the existing soil, prior to stone column installation, it is expected that the weight of the working platform will cause the soil to settle. However, as Table 3 indicates, there was a lapse of 7 months (July 2008 to February 2009) between the installation of the stone columns and the start of embankment construction. The theoretical period for 90 % degree of consolidation from the working platform load is 3 months, calculated by Balaam & Booker's (1981) method. As the elapsed period was 7 months, we may be confident that little or no settlements coming from the 1 m working platform remained, when the embankment construction and monitoring started in February 2009. Therefore the average settlement of 163 mm may be reasonably attributed to the 3.2 m of fill, placed from Feb 2009.

In the back analysis, the magnitude of the settlements was estimated using Priebe's (1995) method, based on the soil parameters in Table 2, computed from CPT 16A. It is worth noting that the soil parameters were fixed prior to the start of the trial. The rate of settlement was then estimated based on Han & Ye's (2001) method. Key input parameters include an assumed stress ratio (stress concentration on column) n = 3, consolidation parameters indicated in Table 2 and the actual rate of filling used in embankment construction. The stress ratio of n = 3 was selected, based on past experience, and also consistent with Han & Ye's (2001) recommendations of "steady state" n = 3 to 4. (Note that the rate of filling was much slower than originally planned, due to unanticipated challenges on site) Based on the actual loading magnitude (3.2 m of fill measured from the top of the working platform) and filling rate, the total long- term settlement was estimated at 197 mm (Figure 7).

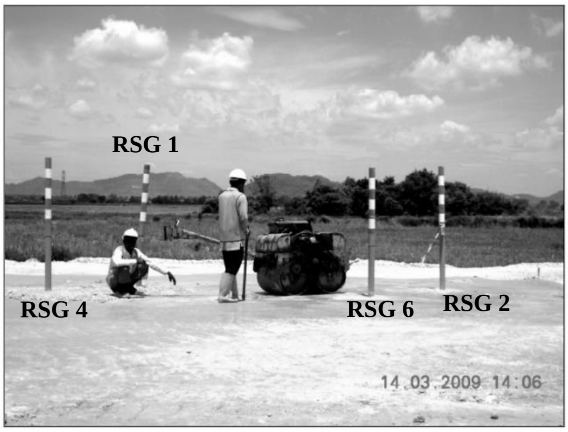

Figure 6 Compaction using a hand-directed roller compactor

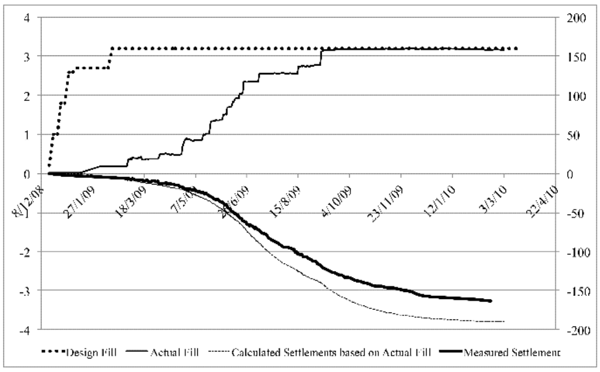

Figure 7 Settlement results from rod settlement gauges, measured and calculated (Zone 1)

Theoretically, 90 % degree of consolidation was to be reached 2 months after completion of filling. Observed settlements at the end of the trial was 163 mm (March 2010). 90% of these settlements (i.e. 147 mm) were observed at about 2.5 months after completion of filling. Both magnitude of settlements and rate of consolidation are reasonably well predicted by the simple analytical methods employed, with the magnitude of settlements predicted being slightly conservative.

Six surface settlement markers were planned for the Zone 1. However, these were unfortunately disturbed, and hence readings were unreliable and therefore not presented.

Deep Settlement Gauges

Two deep settlement gauges were installed in Zone 1, prior to embankment construction. EX 1 was placed 6 m below working platform level, while EX 3 was placed 3 m below working platform level. EX 1, which was placed approximately at the level where the soft clay ends, shows small settlements as embankment construction proceeds, with 11 mm total settlement recorded at the end of the monitoring period (18 Feb 2010). EX 2, placed about 2 m into the soft clay shows much higher settlements as expected, with a total of 98 mm recorded at the end of the monitoring period. Figure 8 presents the readings from these deep settlement gauges.

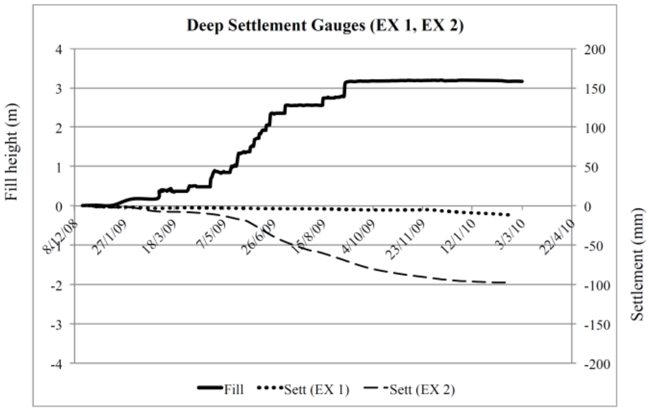

Figure 8 Deep Settlement Gauge Settlement, Fill height vs. Time

Horizontal Inclinometers

Two horizontal inclinometers were placed in Zone 1, buried just below the working platform, in between, and not over the stone column rows. They were installed just prior to the commencement of filling, and initialized on 31 Dec 2008.

As the embankment was built up, the horizontal inclinometers showed progressive settlements. The test embankment was completed on 2 Sept 2009, and the first set of readings after completion were taken on 29 Oct 2009. HO 1 (Figure 9) showed an average settlement in the middle of Zone 1 of 7 mm, while HO 2 (Figure 10) also showed an average of 7 mm. The last set of readings was taken on 10 Feb 2010, about 1.5 months before the test embankment was taken down. HO 1 showed an average settlement in the middle of Zone 1 of 97 mm, while HO 2 showed an average of 68 mm. The differences between the settlements measured by HO 1 and HO 2 are probably due to localised differences where they were installed.

It is interesting that the average total settlements measured by HO 1 and HO 2 up till Feb 2010 from Dec 2008 was approximately 80 mm, which is about 80 mm less than then 160 mm average settlements recorded by the rod settlement gauges in Zone 1.

Because HO 1 and HO 2 were installed just below the working platform, the values should be very close to the rod settlement gauge readings. Upon investigation, it was discovered that the reference levels at the ends of the inclinometer pipe were incorrectly surveyed, and tens of millimeters of settlement were missed out. Unfortunately it was not possible to reconstruct or correct the data.

In any case, the shape of the deformation profile indicates that as the ground settles, some natural undulations are expected.

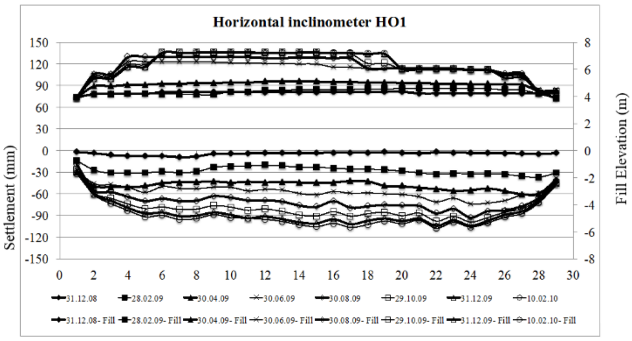

Figure 9 Horizontal inclinometer HO1 vs. Time

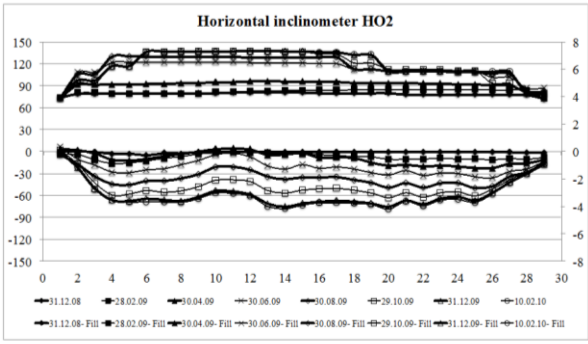

Figure 10 Horizontal inclinometer HO2 vs. Time

6.2 Results from Zone 2

Rod Settlement Gauge Readings

Similar to Zone 1, six rod settlement gauges were installed in Zone 2 to monitor the settlements as the embankment was built, and over the rest period. Just like in Zone 1, it is reasonable to assume that in the 7 month period that elapsed between stone column installation and start of embankment construction, all significant settlements from the load imposed by the working platform have occurred.

Based on the actual fill height of 1.1 m, the predicted final settlement is about 65 mm. The settlements measured by the six RSGs at the end of the monitoring period range from 50 mm to 75 mm, with the average being very close 65 mm (see Figure 11). This set of results, together with the results from Zone 1 confirm the reasonableness of the assumed parameters, as well as the Priebe (1995) and Han & Ye (2001) methods in stone column design.

Surface Settlement Markers

In Zone 2, six surface settlement markers were placed after completion of embankment construction. At the start of the monitoring period for the settlement markers, the RSGs had registered an average settlement of approximately 30 mm. At the end of the monitoring period (18 Feb 2010) the RSGs had registered an average settlement of 65 mm. This difference of about 35 mm corresponds well with the average settlement measure by the surface settlement markers over the same period of 30 mm. Figure 12 presents this data.

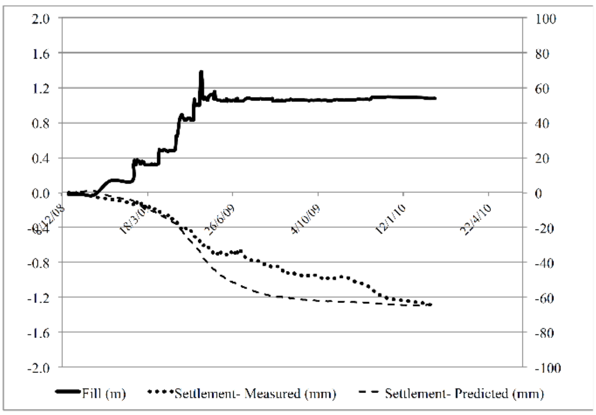

Figure 11 Rod settlement gauge settlements vs. Time (Zone 2)

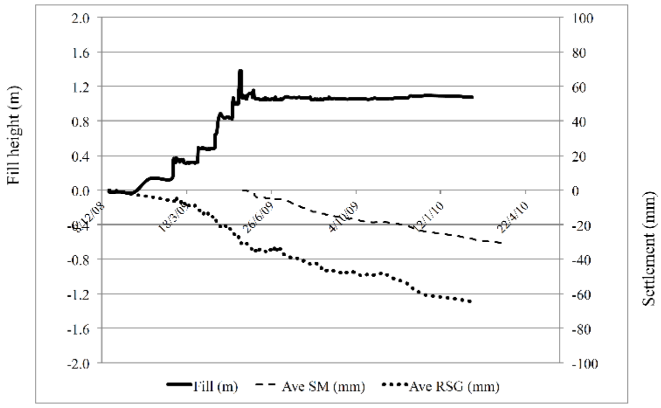

Figure 12 Comparison of Rod settlement gauge settlements and Settlement markers (Zone 2)

Figure 13 shows the plot of the individual settlement markers SM 1 to 6. SM 1, SM 2 and SM 3 were placed at the top of the embankment directly over the stone columns, while SM 4, SM 5 and SM 6 were placed at the top of the embankment in between columns. While the usual minor variations exist, it is interesting to note that no appreciable difference in settlement appears in the monitoring data. Visual inspection of Zone 2 also confirms that no "mushrooming" is seen (Figure 14).

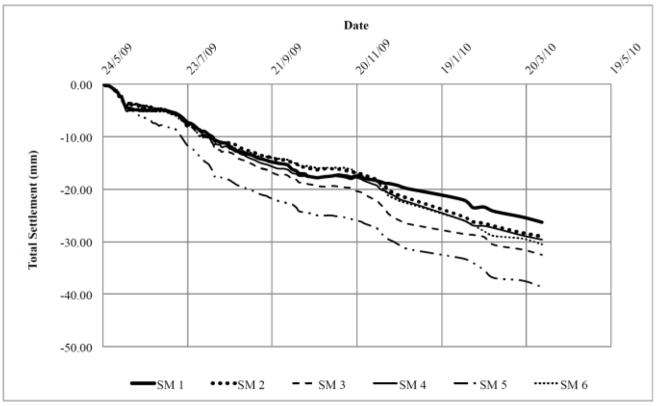

Figure 13 Settlement markers vs. Time (Zone 2) showing only 5 to 15 mm difference in measured settlements

6.3 Other results

Inclinometers outside Zone 1

Two inclinometers (INC 1 and INC 2, Figures 15 and 16) were drilled close to Zone 1, and were monitored from 22 December 2008, prior to the start of embankment construction. As embankment construction progresses, lateral movements occur, as expected. Most movements occur in "Direction A", away from the embankment. Nominal movements are recorded in "Direction B", parallel to the embankment. Near Zone 1, in Direction A, the maximum lateral movements measured are 22 mm (INC 1) and 20 mm (INC 2). This is about 12-13 % of the average recorded settlements by the rod settlement gauges (RSG 1-6). Practical experience with embankments and reinforced soil walls in Malaysia indicate that generally if lateral movements are less than 20% of vertical settlements, the rate of loading is acceptable, and unlikely to result in failure of the embankment or wall.

Figure 14 Photograph of Zone 2. No “mushrooming” seen

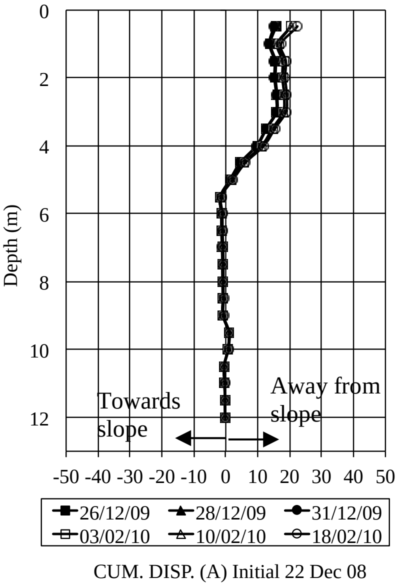

Figure 15 Inclinometer 1- Direction A

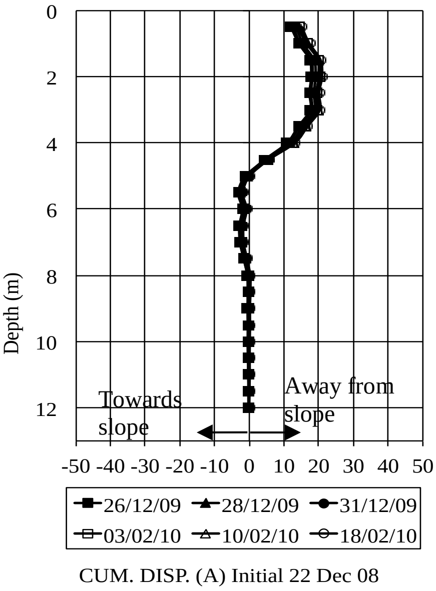

Figure 16 Inclinometer 2- Direction A

A third inclinometer (INC 3, Figure 17) was placed outside the passageway opposite Zone 1. (The passageway level was +2 m from original ground level, or identical in height to Zone 2.) Over the entire monitoring period, INC 3 recorded a maximum lateral deflection of 9 mm, reflecting the lower vertical loads placed on the passageway.

For all three inclinometers, we may observe that the lateral movements stop at about 6m from the top, of the working platform, reflecting the thickness of the soft clay, and the design length of the stone columns.

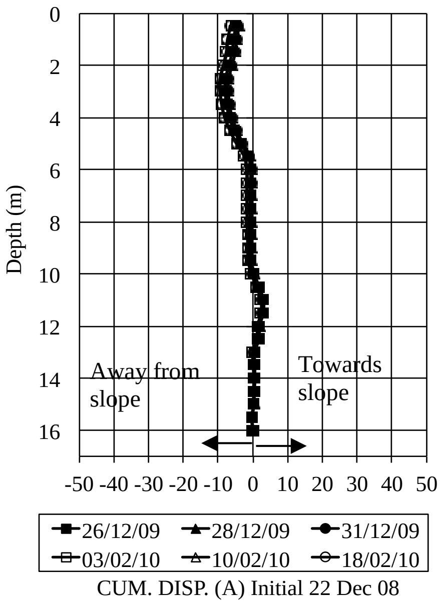

Figure 17 Inclinometer 3- Direction A

7. SUGGESTIONS FOR FURTHER WORK

While the present study validates existing design methods and knowledge about the behaviour of stone column improved ground, further work can be done.

- 2D and 3D finite element models can be calibrated and validated against test embankments such as this
- In the present study, the stone columns were founded on a stiff clay layer. In some cases, the soft clay layer extends much deeper, beyond what can be economically treated with stone columns. Although some numerical work (Ng, 2013) and field work (Adam *et al*, 2010) has been done in the past, further field research is needed on the long-term behaviour of columns that a “floating” in soft clay.
- In addition, there is a need to conduct long-term studies of embankments or walls on improved ground, at risk of creep. Madhav *et al* (2009) and others have proposed frameworks with which to analyse such problems, but field verification is scarce.

8. CONCLUSIONS

The test embankment was taken down in April 2010, and since then, the actual railways embankments and have been completed and handed over to the railway authorities. Since June 2013, the new tracks have been used for commercial traffic, with no performance issues raised.

In summary, the test embankment, and the smooth opening phase of the track allow us to conclude the following:

- In spite of its simplicity, Priebe’s (1995) method can be used to accurately predict the magnitude of settlements under an embankment. Parameters need to be appropriately selected, taking into account local experience.
- Han & Ye’s (2001) method also adequately predicts the rate of

these settlements.

- Particularly from the low test section (Zone 2), no "mushroom" effect was observed, in spite of there being only 1 m of fill over the working platform

More broadly however, this test embankment demonstrates the value of relatively simple geotechnical models, which are sufficient to reproduce and model the salient features of a problem.

9. REFERENCES

Adam, D., Schweiger, H., Markiewicz, R., Knabe, T. (2010) "Euro 2008 stadium Klagenfurt- prediction, monitoring and back calculation of settlement behavior", Proceedings: 14th Danube-European Conference on Geotechnical Engineering. From Research to Design in European Practice, Bratislava, Slovak Republic, June 2 – 4, 2010

Balaam, N. P. & Booker, J. R. (1981), "Analysis of rigid raft supported by granular piles", International Journal for Numerical and Analytical Methods in Geomechanics 5, pp. 379-403.

Duncan, J.M. & Buchignani, A.L. (1976), "An Engineering Manual for Settlement Studies", University of California, Berkeley.

Greenwood, D. A. (1991) "Load tests on stone columns", Deep Foundation Improvements: Design Construction and Testing, ASTM STP 1089, Esrig, M. I., Bachus,R. C. (eds), American Society for Testing and Materials, pp. 148-171

Han, J. & Ye, S. L. (2001), "A simplified method for computing consolidation rate of stone column reinforced foundations", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 127(7), pp. 597-603.

Hughes, J. M. O., Withers, N. J. & Greenwood, D. A. (1975) "A field trial of the reinforcing effect of a stone column in soil", Geotechnique, Volume 25, Issue 1, 01 March 1975, pages 31 –44

Lee, P. T., Tan, Y. C. & Gue, S. S. (2013) "Ground treatment for electrified double track railway in the northern part of Peninsular Malaysia", Proceedings: Advances in Geotechnical Infrastructure, 18th SEAGC cum Inaugural AGSSEA Conference, Leung, C. F., Goh, S. H., Shen, R. F. (eds) Research Publishing, Singapore, pp. 107 – 113.

Madhav, M. R., Suresh, K., Nimala Peter, E. C. (2009), "Creep effect on response of granular pile reinforced ground", Ground Improvement Technologies and Case Histories, Leung, C. F., Chu, J., Shen, R. F. (eds.), Research Publishing, Singapore pp. 275 – 284

McCabe, B. A., Nimmons, G. J. & Egan, D. (2009), "A review of field performance of stone columns in soft soils", Geotechnical Engineering 162, Issue GE6, pp. 323 – 334.

Ng, K. S. (2013), "Numerical study of floating stone columns", Doctor of Philosophy Thesis, Department of Civil and Environmental Engineering, National University of Singapore.

Priebe, H.J. (1995), "The Design of Vibro Replacement," Ground Engineering, December 1995, pp. 31-37.

Raju, V.R. & Sondermann, W., (2005) „Ground Improvement using Deep Vibro Techniques", Ground Improvement Case Histories, Indraratna, B & Chu., J. (eds.), 601-638.

Watts, K. S., Johnson, D., Wood, L. A. & Saadi, A. (2000). "An instrumented trial of vibro ground treatment supporting strip foundations in a variable fill." Geotechnique 50, No. 6, 699–708