Geotechnical Failure Investigation of a Reinforced Soil Wall and Remedial Work Design

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This paper presents a case study of a displaced reinforced soil (RS) wall which is located on a fill slope with a bulging wall and opening of gaps within the wall. Subsurface Investigation (S.I.) had been carried out using exploratory boreholes, mackintosh probes and laboratory tests to determine the relevant engineering properties of the subsoil for subsequent slope stability analyses and investigation. The investigation revealed that the failure is mainly attributed to inappropriate foundation design. This paper describes the site conditions, including foundation design and the details of the geotechnical investigation of the distressed wall. Following the geotechnical investigation and analyses, remedial works have been proposed and successfully constructed which include reconstructing part of the wall to be supported by slab with spun piles and the use of lightweight materials of Expanded Polystyrene (EPS).

1 INTRODUCTION

The platform of a petrol station was constructed with a reinforced soil (RS) wall of up to a height of 7.5m. The site is located in Kuala Lumpur. The RS wall is located on top of a fill slope and is supported by reinforced concrete (RC) square piles. The movements of foundation and progressive wall bulging had led to the opening of gaps within the wall and aggravated the stability of the wall. This paper presents a geotechnical investigation carried out by the Authors to find out the causes of the failure. Appropriate remedial measures were subsequently proposed based on the findings of the geotechnical investigation works.

2 TOPOGRAPHICAL AND GEOLOGICAL CONDITIONS

Generally the original ground surface at site varies from about RL 87m to RL 109m before the construction of the platform at RL 102.83m. The geological map of Selangor Darul Ehsan, compiled by the Geological Survey Department, indicates that the project site is underlain by Kuala Lumpur Granite. The texture and composition of granitic rock generally ranges from coarse to very coarse-grained. The subsoils based on grain size generally consists of clayey sandy SILT with some layers of silty sandy CLAY and silty sandy GRAVEL which is typical of granitic residual soils.

3 ORIGINAL REINFORCED SOIL WALL AND FOUNDATION DESIGN THAT FAILED

The original RS wall foundation consists of 150mm x 150mm square Reinforced Concrete (RC) piles at 1m c/c spacing driven to a penetration length of 9m. A 350mm thick RC slab is subsequently cast on the piles as the foundation of the RS wall. The piling layout of the original RS wall design is shown in Figure 1.

The 7.5m high RS wall design consists of well compacted granular backfill and galvanised steel reinforcing strips of highly adherent type (5mm thick x 45mm width) with length of 5.8m for the top half height of the wall and length of 5.0m for the lower half height of the wall. Typical cross section of RS wall design is shown in Figure 2.
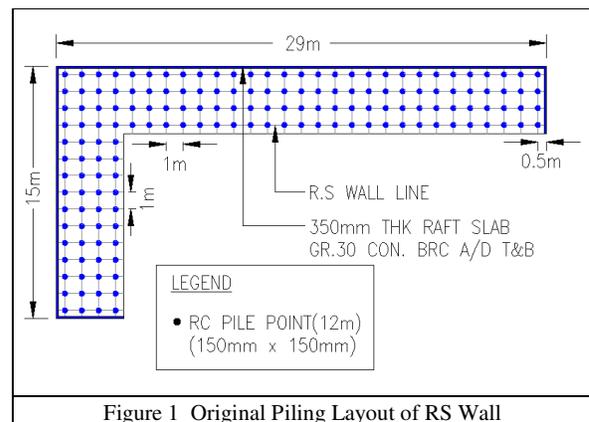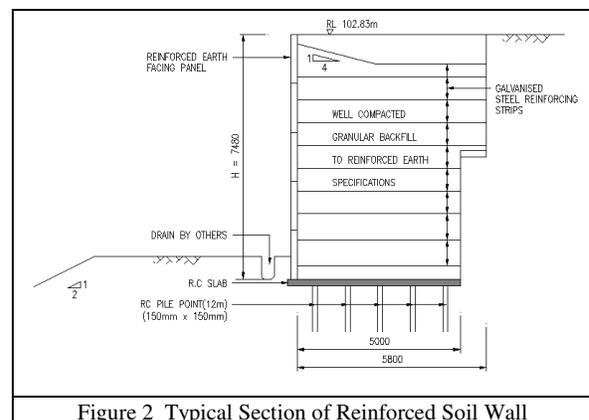

Figure 1 Original Piling Layout of RS Wall

Figure 2 Typical Section of Reinforced Soil Wall

4 SITE CONDITIONS AND OBSERVATIONS

The RS wall panel showed visible signs of displacement, bulging and opening of gaps between the wall panels at the south-western corner of the wall as shown in Figure 3. This had caused backfilled material to be washed away when rain, inducing voids behind the RS wall as shown in Figure 4 that further aggravated the internal stability of the wall.

Bulging and Gap Opening

Figure 4   Voids Behind Wall Due to Backfill Materials Being Washed Away

5 SUBSURFACE INVESTIGATION

There were two stages of Subsurface Investigation (S.I) works being carried out at the site. The first stage of S.I. works (borehole designation with ABH) were carried out prior to earthwork by the design consultant. The S.I works consist of two (2) boreholes and ten (10) Mackintosh Probes (AMP). However, the results obtained from the first stage of S.I works were limited and insufficient for detailed investigation and remedial work design. In addition, the boreholes are located at quite a distance apart from the location of the wall. Therefore, some of the important soil parameters could not be established due to insufficient data. A second stage of S.I. works (borehole designation with BH) was subsequently proposed by the Authors to assist in the investigation. The second stage S.I. works consist of one (1) borehole and eighteen (18) Mackin-

tosh Probes (MP). The locations of field tests for both stages of S.I are shown in Figure 5.
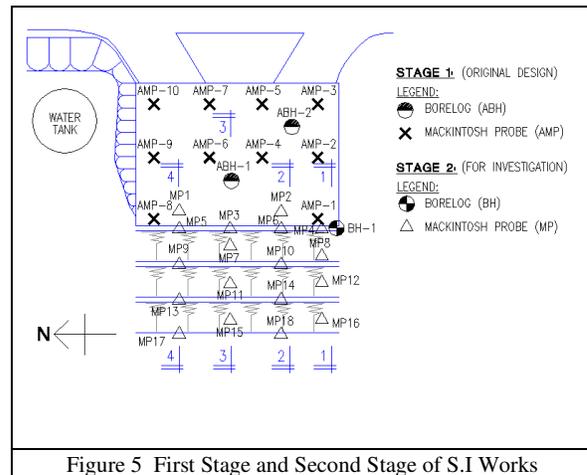

Figure 5   First Stage and Second Stage of S.I Works

Figure 6 shows the interpreted subsoil profiles for both stages of S.I. works. Generally, borelogs of ABH-1 and ABH-2 indicate that the original subsoil consists of sandy SILT and silty GRAVELS. Based on the subsurface investigation carried out, the overburden soils in borehole BH-1 generally consist of clayey SAND at the top 5 m, which are the fill materials and underlain by the original residual subsoil. The depth of fill materials is consistent with the original topography plan, which shows that the original ground level is about 6m below the existing profile. The clayey SAND and sandy CLAY are generally very loose to loose and soft to medium stiff respectively with SPT 'N' values ranging between 3 and 11 up to a depth of approximately 8 m followed by slightly weathered granitic bedrock.

MP tests from the second stage of S.I. works were carried out along the slope as shown in Figure 5. Figures 7 and 8 show the MP profiles, which are generally consistent with the fill thickness.
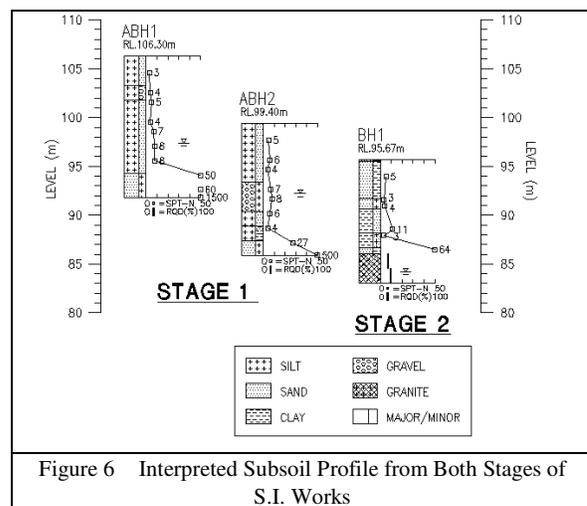

Figure 6   Interpreted Subsoil Profile from Both Stages of S.I. Works
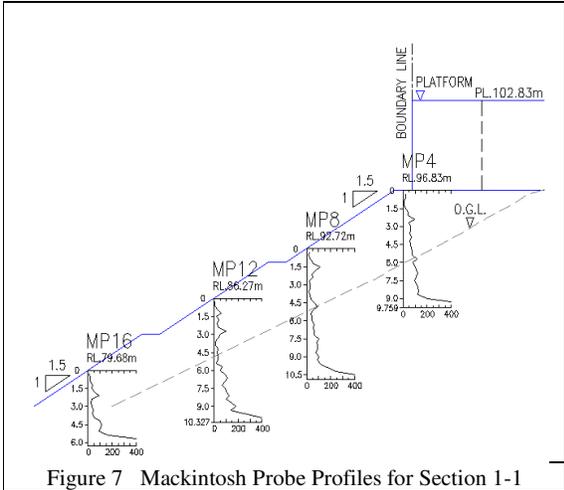
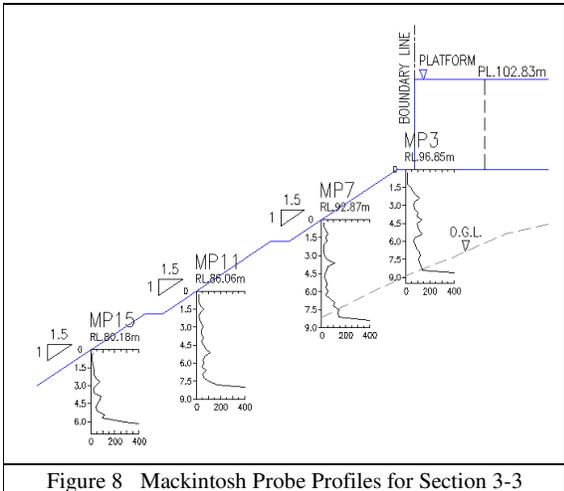Figure 7   Mackintosh Probe Profiles for Section 1-1

Figure 8   Mackintosh Probe Profiles for Section 3-3

5.1 Laboratory Test Results

A series of laboratory tests were carried out on the samples obtained from the second stage of subsurface investigation works, they are:

1. Atterberg limits – see Figure 9
2. Particle size distribution
3. Single stage direct shear box test for wall granular backfill – see Figure 10
4. Isotropically consolidated undrained triaxial test (C.I.U) for in-situ subsoil – see Figure 11

The results of the above laboratory tests are summarised in Table 1.

Table 1: Summary of Laboratory Test Results

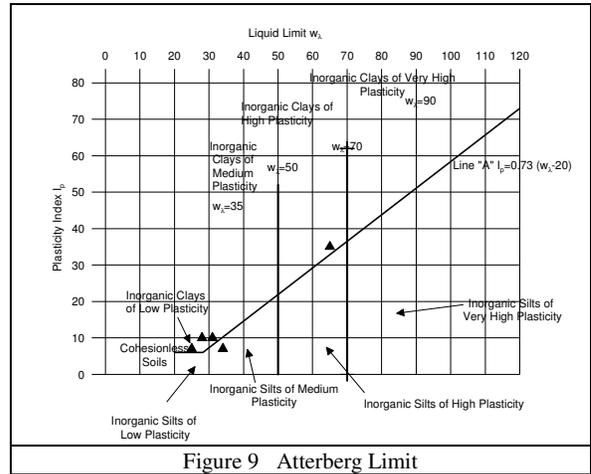| Type of Laboratory Test	Results
Atterberg Limits & Particle Size Distribution	Samples collected from residual sub-soil are mostly clayey sand of low plasticity
Single Stage Direct Shear Box Test	$c = 0$ kPa and $\phi = 36°$ for granular backfill material of the RS wall
C.I.U	$c = 3$ kPa and $\phi = 33°$ for residual subsoil

Figure 9   Atterberg Limit

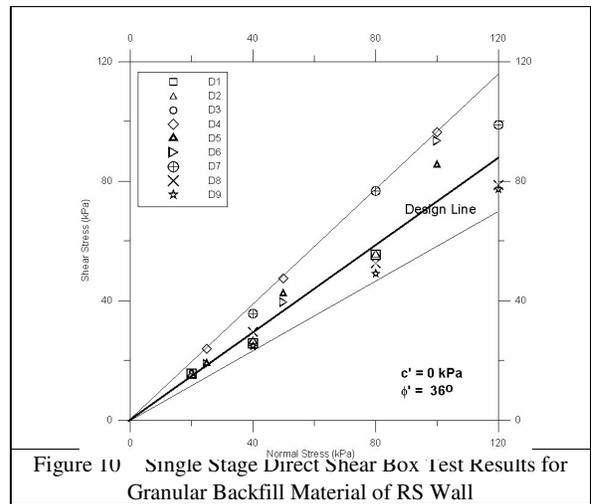Figure 10   Single Stage Direct Shear Box Test Results for Granular Backfill Material of RS Wall

3

Interpreted Soil Strength Parameters (C.I.U. Tests)

c = 3 kPa
φ = 33°

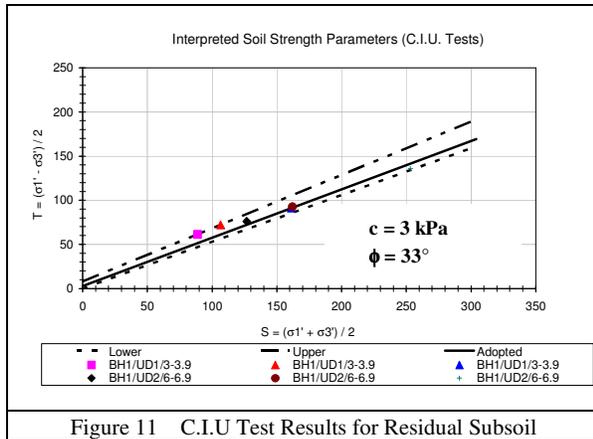|		
--- Lower	— Upper	— Adopted
■ BH1/UD1/3-3.9	▲ BH1/UD1/3-3.9	▲ BH1/UD1/3-3.9
◆ BH1/UD2/6-6.9	● BH1/UD2/6-6.9	+ BH1/UD2/6-6.9

Figure 11   C.I.U Test Results for Residual Subsoil

6 GEOTECHNICAL INVESTIGATION INTO THE FAILURE OF RS WALL

In order to determine the causes of RS wall bulging and displacement, the following analyses as shown in Table 2 were carried out based on the interpreted subsoil parameters and backfill material properties obtained from laboratory tests:-

Table 2: Type of Analyses Carried Out

Type of Analysis	Details
Global Stability of the Wall and Slope	• Slip Failure Analysis • Overturning and Bearing Capacity Check • Sliding Failure Check
Internal Stability of RS Wall	• Rupture • Adherence
Structural and Geotechnical Capacity Check of the Piled Foundation	• Pile Axial Capacity Check • Pile Lateral Resistance Check • RC Base Slab Check
Degree of Compaction of Fill Slope Materials	• Check on Adequacy of Degree of Compaction

The following subsection describes in detail the analyses carried out to investigate the causes of distress.

6.1 Global Stability Analyses

6.1.1 Slip Failure Analysis

Independent slope stability analyses were carried out to check for different modes of failures namely circular and irregular shaped failure surfaces using the Modified Bishop's and Spencer's Methods respectively. Therefore, all possible slip failure surfaces have been checked and the most critical Factor of Safety (FOS) for the slopes is computed.

In the slope stability analyses, the following conditions were analysed:

Case 1: RS wall with the presence of 150mm x 150mm reinforced concrete (RC) piles as per the original design by C&S consultant (assuming the piles were not displaced)

Case 2: RS wall without the piles to simulate the FOS in the initial construction before installation of piles and if the small piles had displaced

Case 3: Local fill slope stability in front of RS wall

The acceptability of a slope's stability is based on its ability to achieve an adequate Factor of Safety (FOS) against slope failure. An appropriate FOS against the failure of a slope is dependent upon the extent to which that failure could potentially result in the loss of life. It is also dependent upon the degree of economic loss that would result if a failure does occur. The assignment of an appropriate FOS for various types of slopes would be done in accordance to the Hong Kong Geotechnical Engineering Office's (GEO) "Geotechnical Manual for Slopes". Factor of Safety of 1.4 is recommended in the stability analyses of the slopes. In addition, the slopes should achieve a factor of safety of 1.1 for the condition of "worst predicted groundwater level".

Based on the results of the analyses as shown in Table 3, the RS wall displacement is not likely to be induced by local instability of fill slope in front of the wall. Based on the slip failure analyses results, it can be deduced that the pile displacement and lateral resistance are possible critical factors influencing the global stability. As deformation could not be analysed in the slip failure analyses using the limit equilibrium method, finite element method (FEM) analyses are required to assess the pile displacement and lateral resistance.

Table 3: Stability Analyses Results

Case	Section	Long Term Factor of Safety	
		Modified Bishop Method (Circular Failure)	Spencer's Method (Non-Circular Failure)
Case 1	With Piles	1.58 (>1.4)	1.69 (>1.4)
	With Piles (worst case of WL)	1.26 (>1.1)	1.37 (≥1.1)
Case 2	Without Piles	1.25 (<1.4) Not acceptable	1.36 (<1.4) Not acceptable
	Without Piles (worst case of WL)	1.20 (> 1.1)	1.25 (>1.1)
Case 3	Local Stability of 1V:1.5H Fill Slope	1.37 (<1.4) Not acceptable	1.44(>1.4)
	Local Stability of 1V:1.5H Fill Slope (worst case of WL)	1.37 (>1.1)	1.44 (>1.1)

As shown in Table 3, the FOS for Case 1 is adequate in both abovementioned groundwater conditions. It indicates the global stability of the slope is adequate if the RC piles were not displaced significantly and could provide adequate lateral resistance against slope movement. However, if the piles have displaced or damaged due to slope movement, the global stability would be inadequate, as demonstrated by Case 2. Although the local stability of the slopes is slightly inadequate (FOS=1.37 < 1.4) as shown in Case 3, it is only marginal. In view of this, the RS wall displacement is not likely to be induced by local instability of fill slope in front of the wall, but rather due to global slope instability with loading from RS wall when the small piles had been displaced, as shown in Case 2.

The original configuration of RS wall supported by RC piles has been analysed using a two-dimensional finite element method programme. Figure 12 shows the graphical printouts of the FEM analyses results (the deformation is plotted in exaggerated scale). As shown in Figure 12, the RC piles beneath the RS wall have bent and displaced significantly, and therefore part of the RS wall supported by piles also displaced together. The estimated pile displacement ranges from about 150mm to 170mm with induced maximum bending moment and shear force to the pile of 36 kNm and 64 kN respectively. The maximum mobilised bending moment and shear force induced onto the 350 mm thick slab is 146 kNm and 192 kN respectively. The comparison of the induced and ultimate bending moment and shear force of piles and slab are elaborated in Section 6.3.2 and 6.3.3 respectively.

Figure 12   FEM Analysis – Displaced RC Piles
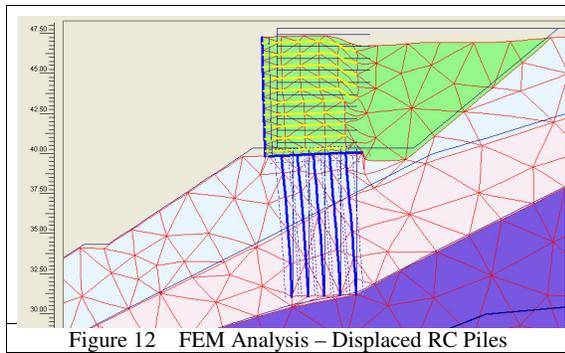
6.1.2 Overturning and Bearing Capacity Check

The long term bearing capacity of the wall is governed by the pile foundation supporting it. As for overturning, the independent analyses carried out indicate that the wall is stable against overturning with factor of safety against failure of greater than 2.0.

6.1.3 Sliding Failure Check

The sliding resistance is provided by the soil-to-soil contact between the reinforced soil wall fill and the slab supporting it. The analysis indicates that the sliding resistance is adequate against the lateral forces imposed on the RS wall. However, it is important to note that the lateral forces have been transferred to the piles which should have been designed to cater for it.

6.2 Internal Stability Analyses

Internal stability analyses were carried out to check the internal stability of the RS wall in terms of rupture, adherence and wedge stability in accordance to BS 8006:1995. Coherent Gravity Method is adopted as recommended in Clause 6.3 of the BS 8006:1995. As required, three load cases of load combination with different partial load factors are considered in the design.

Table 4 shows the load combinations used for the checking of the design of the reinforced soil wall. In addition, the effect of groundwater level on the internal stability is also investigated using the method proposed by Tan & Khoo (2006).

Combination A considers the maximum values of all loads and therefore normally generates the maximum reinforcement tension and foundation bearing pressure. It may also determine the reinforcement requirements to satisfy pull-out resistance although pull-out resistance is usually governed by combination B.

Combination B considers the maximum overturning loads together with minimum self mass of structure and superimposed traffic load. This combination normally dictates the reinforcement requirements for pull-out resistance and is normally the worst case for sliding along the base.

Combination C considers dead loads only without partial load factors. This combination is used to determine foundation settlements as well as generating reinforcement tensions for checking the serviceability limit state.

Table 4: Partial Load Factors for Load Combinations associated with Walls (from BS8006:1995)

	Load Combination		
Effects	A	B	C
Mass of reinforced soil body	1.5	1.0	1.0
Mass of backfill on top of RS Wall	1.5	1.0	1.0
Earth Pressure behind structure	1.5	1.5	1.0
Traffic Load:			
(a) on reinforced soil block	1.5	0.0	0.0
(b) behind reinforced soil block	1.5	1.5	0.0

6.2.1 Rupture

Rupture is the tensile capacity of the reinforcement to resist the tensile forces generated from the lateral earth pressure and the external loads. This is specified under BS 8006:1995 clause 6.6.5.2.5. Independent analyses carried out indicate that the reinforcement is adequate against rupture failure.

6.2.2 Adherence

Adherence is the resistance of the reinforcing fill against local sliding of the reinforcement from the tensile forces generated. The adherence check is specified under BS8006:1995 clause 6.6.5.2.4. Based on the design calculations provided, the original design methodology of RS wall is in accordance to BS8006:1995. However, the values for the coefficient of friction (μ) used are following the French Ministry of Transport's (FMT) recommendations for Reinforced Earth Structures (FMTRRES) instead of BS8006:1995.

The FMT's recommendations use the parameter f* which is similar to BS8006:1995's μ parameter. The value used varies with depth of the reinforced soil wall as follows (Cl. 2.3.3.1, FMTRRES):

$$f^* = f_o^* (1 - z/z_o) + \tan (\phi') \cdot z/z_o \qquad \text{for } Z \leq Z_o = 6m.$$

$$f^* = \tan (\phi') \qquad \text{for } Z > Z_o$$

Where the minimum f_o^* value is taken as 1.50 in absence of accurate measurements as recommended by FMT.

British Standard for Reinforced Soil, BS8006:1995 states that the coefficient of friction is derived from $\mu = \tan \delta'$. However, BS8006:1995 does not state the value of δ' and in consistent with the standards, the value of δ' is obtained from British Code of Practice for Earth Retaining Structures (BS8002:1994). Clause 3.2.6 of BS8002:1994 states that for design values, the values of parameter δ' can be adopted as approximately two-thirds of the peak angle of friction of fill materials in the reinforced soil ($2/3*\phi'$).

Using BS8002:1994's design values for δ' results in coefficient of friction value of 0.45 (for a soil peak angle of friction of 36°. However, using FMT's recommendations, the f^* parameter values varies from maximum 1.5 at the top of the RS wall to $\tan \phi'$ (which is 0.73) from the depth below 6 m. Therefore, the FMT's recommendations of μ values are higher than the British Standards by 62%. The design of RS wall using FMT recommendations is more optimistic than design using British Standards.

Figure 13 presents the results of the tensile force and adherence (calculated in accordance to BS8006:1995 and FMT) for all cases. As can be seen from Figure 13, the tensile force induced is higher than adherence. The non-compliance on adherence failure according to BS8006:1995, is due to insufficient reinforcement strip length provided there is no change to the strip width. The reinforcement for adherence is marginally adequate at the top 3m based on FMT's recommendations. It can be seen that the adherence reduces at the depth of approximately 3.5m for both BS8006 and FMT. This is attributed to the reduction in the length of steel reinforcing strips as explained in Section 3.

In addition, the level of compaction of the reinforced fill materials during construction (if inadequate) will also affect the adherence.

Figure 13   Tensile Force Induced and Adherence Profile With Depth
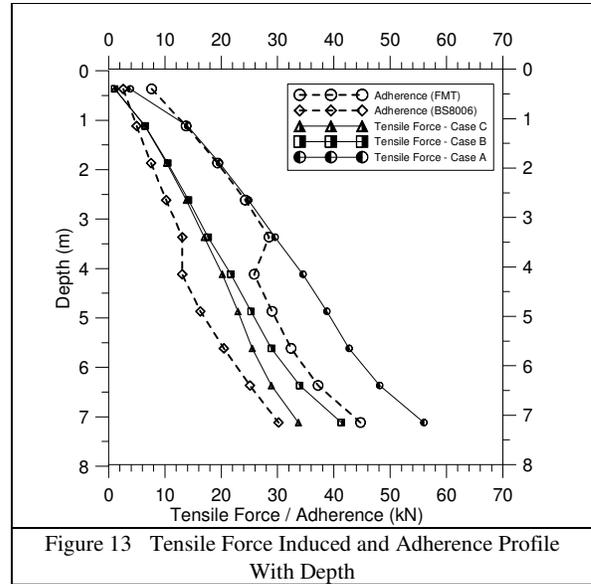
6.2.3 Effect of Groundwater Level on Internal Stability

In order to investigate the effect of groundwater tables on the internal stability of RS wall, groundwater table of 1/3 and 2/3 of RS wall height are assumed in the analyses according to BS8006:1995. The tensile force does not significantly increase with the presence of groundwater table (GWL) as can be seen from Figure 14.

The effect of groundwater table on the adherence is also investigated. The adherence (in accordance to BS8006) with the presence of the groundwater table (at 1/3H and 2/3H) is expectedly lower than that without the presence of the groundwater table as illustrated in Figure 15.

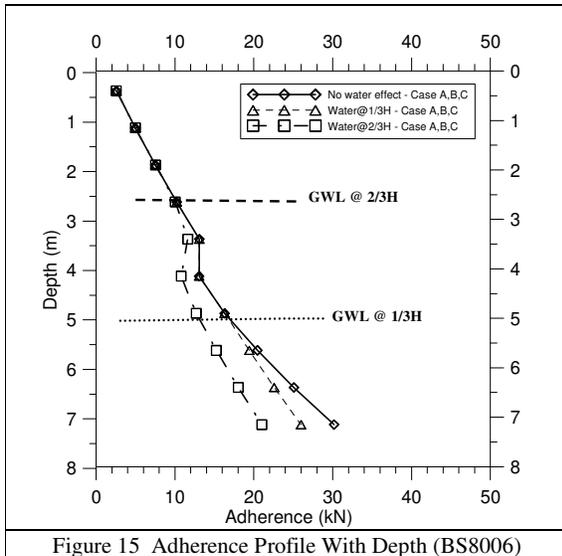Figure 14   Acting Tensile Force Profile with Depth (Case A, B, C)
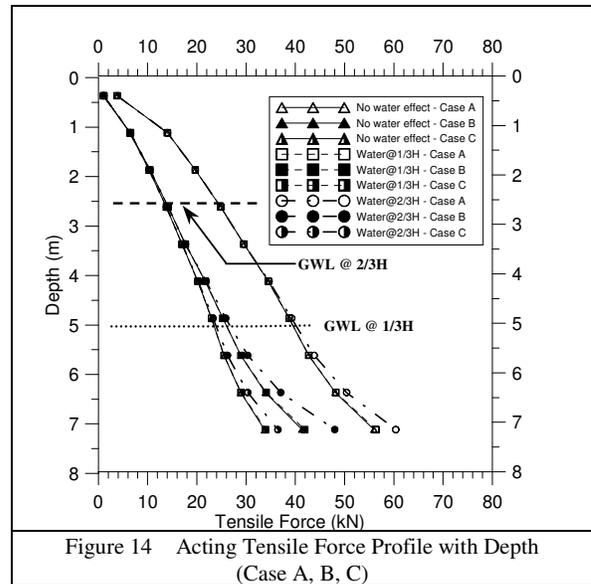
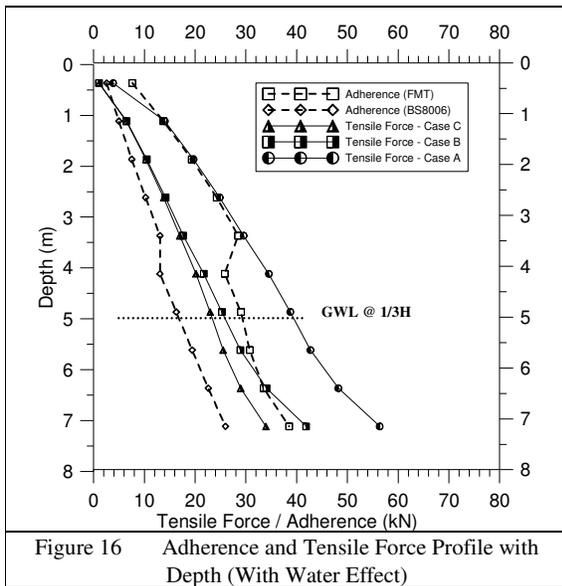Figure 15 Adherence Profile With Depth (BS8006)

Figure 16   Adherence and Tensile Force Profile with Depth (With Water Effect)

When comparing the adherence and tensile force induced with the presence of groundwater table (Figure 16), it was found that the adherence is still inadequate. The adherence calculated using FMT is also included for comparison. In both methods of calculation of adherence, the tensile force exceeds the adherence with and without the presence of groundwater table indicating inadequacy of design length.

6.3 Structural and Capacity Check of Piled Foundation

6.3.1 Pile Axial Capacity Check

The geotechnical capacity of the piles is estimated by using the borehole information. The calculations show that the geo-technical axial capacity of the piles is adequate for the proposed pile working load of 200kN. High strain dynamic load tests were carried out on two piles and the results indicate that the tested piles had achieved an axial static resistance of more than twice the working load at the time of testing before plac-

ing of the wall. Therefore, the geotechnical axial compression capacity of the piles is adequate.

6.3.2 Pile Lateral and Bending Resistance Check

Shear force and bending moment mobilised in the RC piles are extracted from the FEM analyses. As mentioned in Section 5.1.1, the maximum bending moment and shear force induced on the RC piles are 36 kNm and 64 kN respectively. The steel reinforcement details of the 150mm x 150mm RC pile that are commonly available in the local market using 4Y10 with link of ϕ5.5@69mm are adopted for computation of moment and shear resistances. The ultimate bending moment and shear force capacity of 150mm x 150mm pile are 10 kNm and 31 kN respectively and are found to be grossly inadequate. As a result of inadequate lateral resistance, the RC piles have structurally failed under bending and shear.

Since the piles are embedded within the 4m thick soil layer, these piles maybe subjected to axial and lateral loads which are induced by these soil movements. Tschebotarioff (1973) presented design method to predict maximum induced bending moment for such scenario. As such, additional analysis using Tschebotarioff's method was also carried out to check the adequacy of the piles to resist such loads. The estimation of the lateral force and bending moment is based on the assumption of fixed pile head condition. The results show that the estimated lateral force and maximum bending moment are 28 kN and 40 kNm respectively which are relatively consistent with the findings from FEM analyses. However, it shall be noted that this method only provides an estimate of the pile bending moment and does not consider either the lateral deflection or the axial response.

6.3.3 RC Base Slab Check

Independent analysis of the adequacy of the RC base slab was carried out based on the information from drawings prepared by the original consultant. Based on the drawings, BRC mesh A10 was provided at the top and bottom of the base slab of 350mm thick. As mentioned in section 6.1.1, from the FEM analyses, the maximum bending moment and the shear force induced onto the RC slab are about 146 kNm/m and 192 kN/m respectively. Based on BS8110:1985, the ultimate moment and shear resistances of the slab are 55 kNm/m and 102 kN/m respectively. Therefore, the moment and shear resistances of the slab are grossly inadequate.

6.4 Compaction of Fill Slope Material

Compaction tests were carried out on the fill materials during filling of the platform and slopes and prior to the construction of the RS wall. The information on the locations of the tested samples was not made available at the time of investigation. Figure 17 shows the compaction test results which indicate that the degree of compaction of tested samples ranges from 82% to 99%. In other words, 56% of the tested samples have a degree of compaction less than 95%, indicating some local-ised areas with inadequate compaction. This can be observed from some of the Mackintosh Probes results (Figures 7 to 8), in which soil consistency increases and decreases alternately with depth at some MP locations.
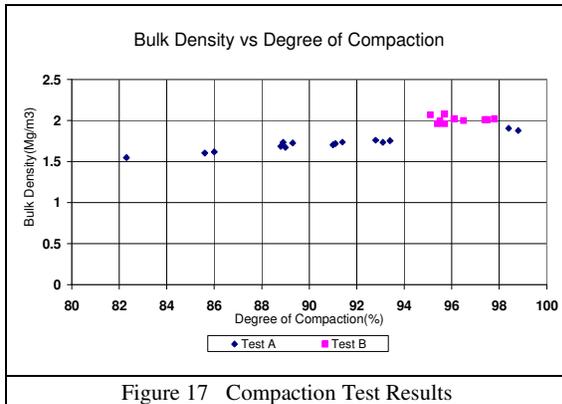
Figure 17   Compaction Test Results

6.5 Findings

Based on the above findings, the causes of RS wall displacement and bulging are as follows:

a) Main factors: Foundation instability mainly due to inadequate pile lateral resistance against lateral movement and inadequate shear and moment resistances of the RC piles and slab

b) Inadequate FOS for RS wall reinforcement strip adherence and inadequate fill compaction also contributed to some bulging of the wall

The inadequacy of the abovementioned main factors is confirmed via visible cracks observed on the slab upon exposing the base of the RS wall (as shown in Figure 18) for the remedial work.
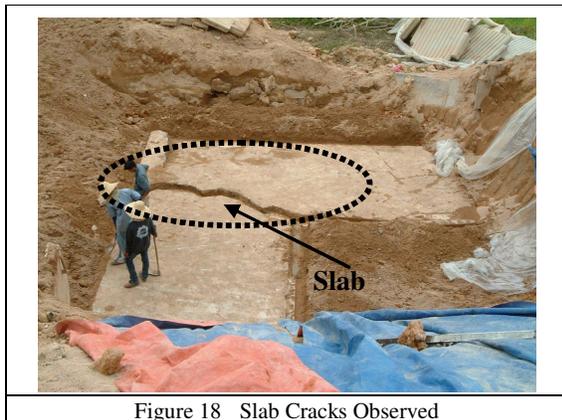
Figure 18   Slab Cracks Observed

In addition, due to the opening of the gap and bulging of the RS wall caused by the factors described above, the fill materials (granular materials) in the RS wall have dropped and washed out through the openings. The condition deteriorated more rapidly during rainy season. Due to the loss of materials, the internal stability of the wall was badly affected, inducing further widening of gap and bulging of the wall. The processes repeated until a large void was formed in the wall (as shown in Figure 4) and depression at the top of the wall.

7   PROPOSED REMEDIAL WORK

The proposed remedial work includes the following construction sequences:-
a)   Installation of sheet pile
b)   Excavation of backfill material to the base of the reinforced soil wall
c)   Hacking the base slab and drive ϕ400mm spun piles (Grade 80, Class B) filled with reinforced concrete plug
d) Reinstatement of Reinforced Soil (RS) wall and partial replacement of backfill material with Expanded Polystyrene (EPS)

Since the convenient store is located near the slope, temporary sheet piles were necessary to minimise ground movement at the convenient store during the adjacent excavation.

The spun piles were closely spaced and filled with reinforced concrete plug to provide adequate lateral resistance and the piles were driven into the competent hard layer. As such, it is not necessary to adopt raking piles to provide the lateral resistance.

Longer wall reinforcement length was also used to achieve the required factor of safety of the wall against adherence failure while Expanded Polystyrene (EPS) was adopted to reduce the vertical overburden pressure at the wall section where replacement of wall reinforcements could not be done due to site constraints. Figures 19 to 24 illustrate the details and sequence of the proposed remedial works.
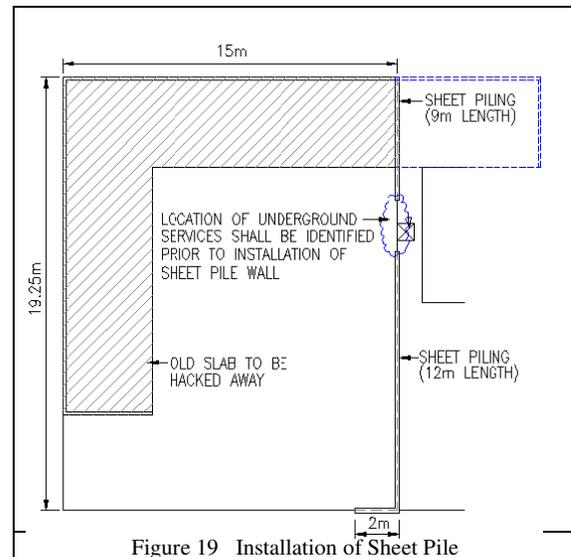

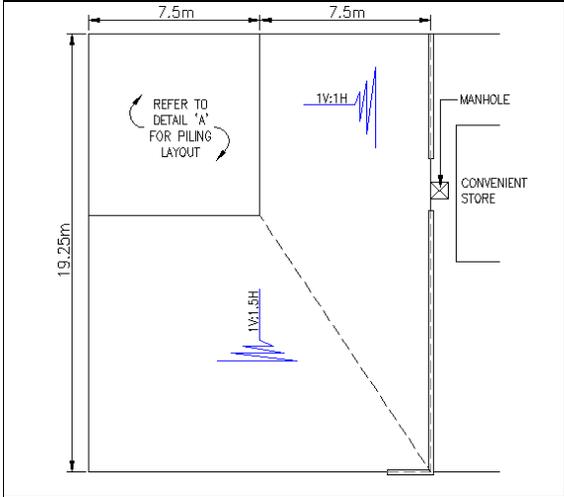Figure 19   Installation of Sheet Pile

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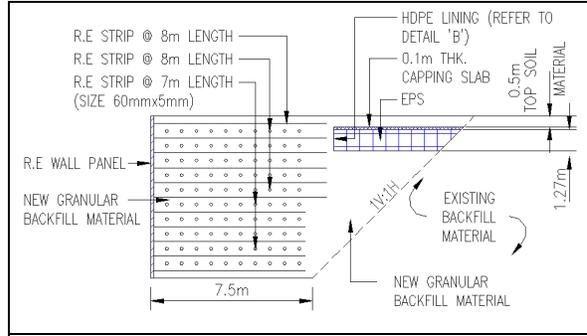Figure 20   Excavation of Backfill Material, Hack Base Slab and Install Spun Piles
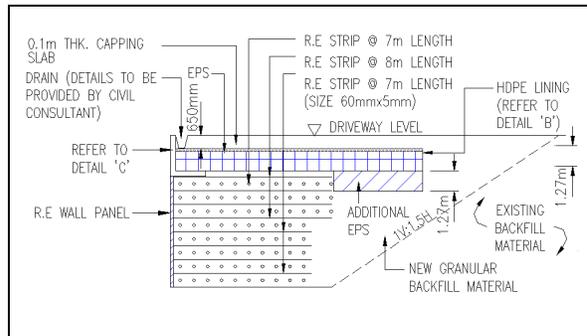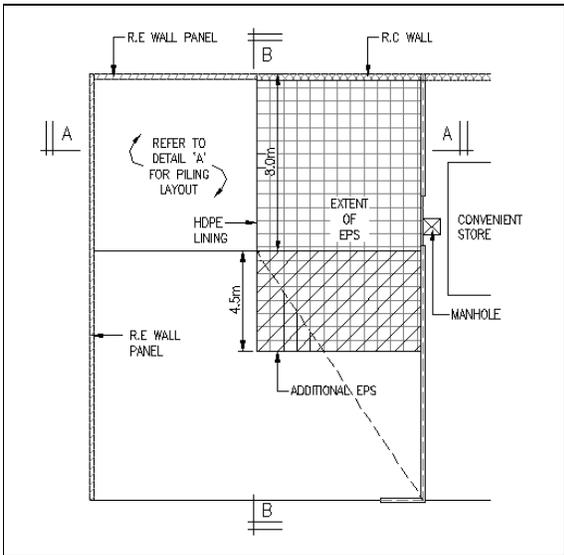

Figure 21   Reinstatement of RS Wall and Partial Replacement of Backfill Material using EPS

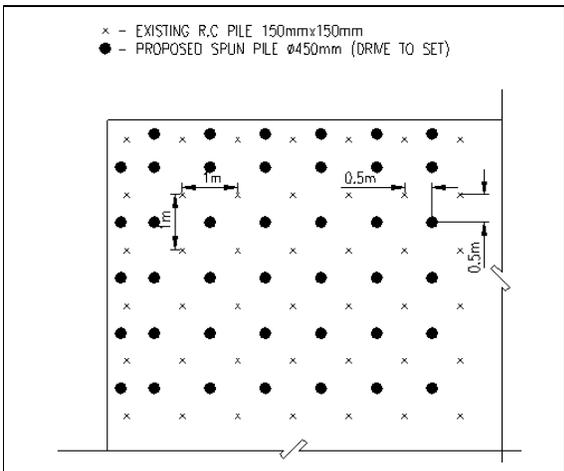Figure 22   Detail 'A'

Figure 23   Section A-A

Figure 24   Section B-B

7.1 Finite Element Method (FEM) Analyses

Prior to the actual construction of the proposed remedial works, finite element method (FEM) analyses on the proposed remedial solution were carried out to analyse the overall configuration in order to determine the performance of the proposed remedial solution. Figure 25 shows the model of RS Wall used in the FEM analyses. The Mohr Coulomb soil model was adopted in the analyses. In FEM modelling, both the spun pile and RC Raft were modelled as linear elastic materials with properties of Young's Modulus and Poisson's ratio. The spun piles and the RC raft were modelled as beam elements in order to determine the shear force and bending moment for detailed design thereafter. A summary of the material properties is presented in Table 5.
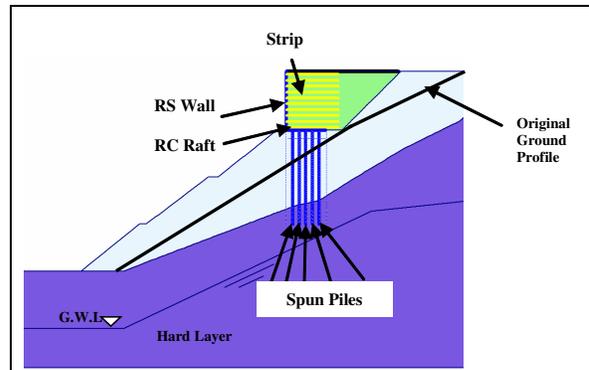

Figure 25   Reinforced Soil Wall Model

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Table 5: Material Properties for FEM Analyses

Name	Type	γ_{sat} (kN/m^3)	ν	E_{ref} (kN/m^2)
Granular Backfill	Drained	18	0.3	50,000
Existing Soil	Drained	20	0.3	12,500
Hard Layer	Drained	20	0.3	125,000
RS Wall Panel	Elastic	-	0.2	-
RC Raft	Elastic	-	0.2	-
Pile	Elastic	-	0.2	-
Strip	Elastic	-	-	-

Name	c_{ref} (kN/m^2)	ϕ (°)	EA (kN/m)	EI (kNm^2/m)
Granular Backfill	1	36	-	-
Existing Soil	3	33	-	-
Hard Layer	3	33	-	-
RS Wall Panel	-	-	3.64 E6	5,945
RC Raft	-	-	6.5 E6	33,900
Pile	-	-	1.65 E6	34,976
Strip	-	-	4.69 E4	-

The checking for punching shear, normal shear, maximum shear and bending moment on the RC raft indicates that a minimum thickness of 300mm slab with T16@125mm reinforcing steel bars are required. The structural elements were designed according to BS8110: 1985.

7.2 Reinforced Soil Wall Design

In the remedial design, the RS wall has been designed with consideration of external and internal stability according to BS8006:1995. The coherent gravity method was adopted for internal stability check.

8   PROPOSED MONITORING PROGRAMME

A monitoring programme was recommended to monitor the long-term performance of the proposed remedial work. The proposed monitoring programme consisted of three (3) inclinometers. However, one inclinometer was damaged during installation. Therefore, only two (2) inclinometers were installed. The locations and typical details of inclinometers are shown in Figures 26 and 27.   Readings of the instruments were recommended to be taken fortnightly for the first two (2) months and once a month thereafter for eight (8) months. Subsequently, the monitoring was carried out once every four (4) months for a year.

Figures 28 to 31 show that the inclinometer monitoring results for Inclinometers 1 and 2 respectively in comparison with the FEM analyses. Upon reviewing the inclinometer monitoring results, it was found that the lateral displacement of the inclinometer showed a trend of stabilising readings and are within the acceptable limit as interpreted from the FEM analyses. Therefore, it can be concluded that the proposed remedial works are effective.
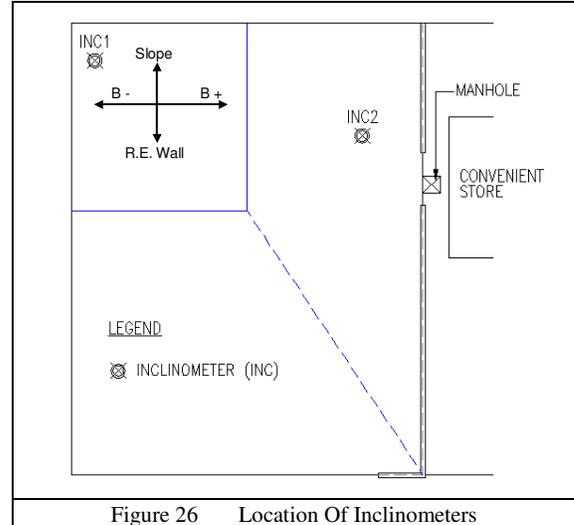

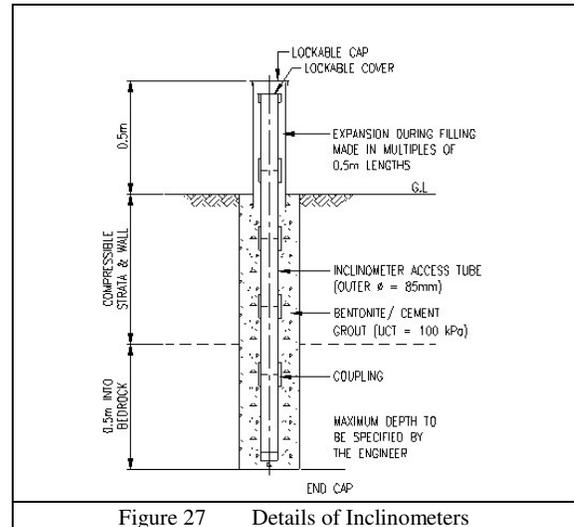Figure 26   Location Of Inclinometers
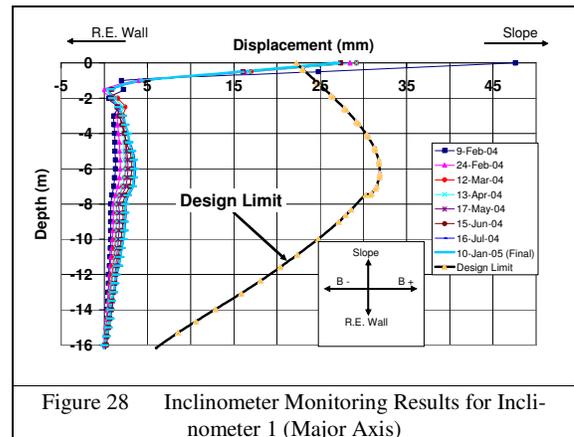

Figure 27   Details of Inclinometers

Figure 28   Inclinometer Monitoring Results for Inclinometer 1 (Major Axis)
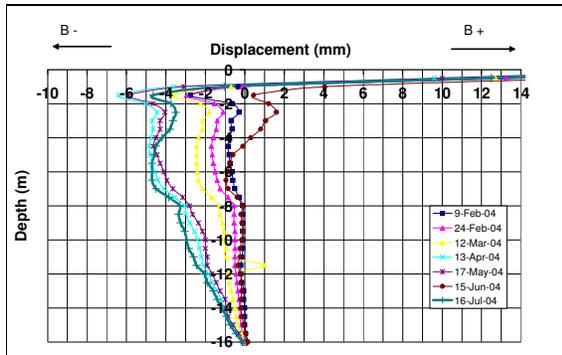
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Figure 29   Inclinometer Monitoring Results for Inclinometer 1 (Minor Axis)

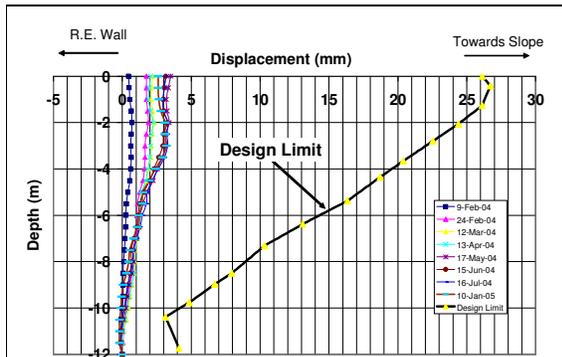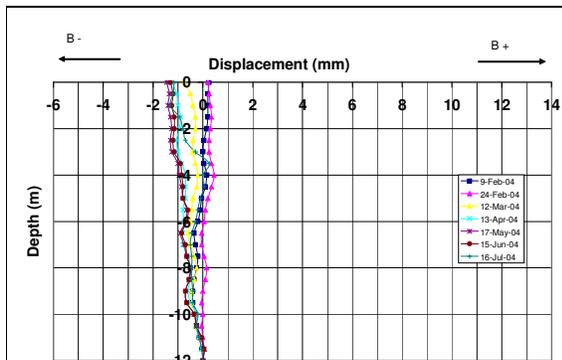Figure 30   Inclinometer Monitoring Results for Inclinometer 2 (Major Axis)

Figure 31   Inclinometer Monitoring Results for Inclinometer 2 (Minor Axis)

9   SUMMARY AND CONCLUSIONS

The investigation results of RS wall failure at the petrol station were presented. The wall bulging occurred progressively had caused the backfilled material to be washed away during rain and induced voids behind the RS wall. In an attempt to identify possible causes of failure, a comprehensive investigation was carried out.

The conclusions and recommendations can be summarised as follows:

Slope Stability Analyses

- Local stability of the fill slope in front of the RS wall was not the causes of the bulging problems.
- Global stability of overturning and sliding of the RS wall were adequate, except for the global slip failure through the slope under the loading from the wall. Slope stability analyses indicate that the global slip failure under the loading from the RS wall was inadequate if the RC piles had been displaced.

RS Wall Internal Stability Analysis

- According to BS8006:1995, all aspects of RS wall internal stability were adequate except for adherence. The wall strip reinforcements had inadequate FOS against adherence failure.
- There was insignificant increase in the acting tensile force in the presence of the groundwater table for 7.5m RS wall. However, the adherence was comparatively lower with the presence of the groundwater table than that in the absence of the groundwater table. As such, it is recommended to consider the effects of the groundwater table (hydrostatic pressure) on the reinforced soil wall internal stability design.

Structural and Geotechnical Capacity Check of Piled Foundation

- Finite element method (FEM) analyses show significant movement of the 150mm x 150mm RC piles supporting the wall.
- The shear and bending moment resistances of 150mm x 150mm RC piles and the base slab are inadequate.

Remedial Work Design

- The remedial works consist of reconstructed part of RS wall to be supported by 450mm diameter spun piles (filled with reinforced concrete plug) and partial replacement of existing backfill materials with lightweight Expanded Polystyrene (EPS).
- The monitoring results of the inclinometer show a trend of stabilising readings and are within the acceptable limit as interpreted from the FEM analyses. The proposed remedial works are effective.

REFERENCES

British Standard Institution. 1995. *BS8006:1995 British Standard Code of Practice for Strengthened / Reinforced Soils and Other Fills.*
British Standard Institution. 1994. *BS8002: 1994 British Code of Practice for Earth Retaining Structures*
The French Ministry of Transport. *Reinforced Earth Structures: Recommendations and Rules of the Art.*
Tschebotarioff G.P. 1973. *Foundations, Retaining and Earth Structures,* 2nd Edition. McGraw-Hill, New York.
Tan, Yean-Chin & Khoo, Chee-Min. 2007. *Effects of Hydrostatic Pressure on Limit State Design of Reinforced Soil Wall*

Internal Stability, Proc. of 16th SEAGC, Kuala Lumpur (submitted).