Design and Performance of Arch Culvert and RSW Constructed on Improved Soft Ground

Bosco Poon[1] and Kim Chan[2]

[1]Associate – Ground Engineering, Aurecon, Neutral Bay, Australia
[2]Senior Principal and Service Line Leader – Geotechnical Engineering, GHD, St. Leonards, Australia
Corresponding author's E-mail: bosco.poon@aurecongroup.com ; kim.chan@ghd.com

Abstract

A concrete arch culvert with reinforced soil wing wall structures was constructed on soft soil treated with nonconforming short dynamic replacement columns. A novel remedial design solution was developed which comprised (i) installation of remedial stone columns in the vicinity of the culvert and wing wall structures, (ii) provision of a detached connection system to allow for greater tolerable movement at the culvert-wing wall interface, and (iii) the use of a dead-man anchorage system to tie-back the ground beam that underpins the wall facing. This paper focuses on the numerical analyses of the arch culvert and wing wall structures, and compares the predictions with actual performance under short term loading conditions.

Keywords: arch culvert, reinforced soil wall, stone column, dynamic replacement, stress concentration.

1. INTRODUCTION

A 16m span arch culvert was constructed to underlie an 8m high embankment. The installed culvert is flanked by two reinforced soil wing walls at the northern (Wing Wall A) and southern end (Wing Wall C) as shown in Figure 1a. The arch culvert structure is supported by a piled foundation whereas the wing walls were originally designed to be a reinforced soil wall (RSW) founding on full depth dynamic replacement (DR) columns. Prior to the construction of the arch culvert and the reinforced soil wing walls, a post-DR investigation indicated that many of the DR columns at the wing wall locations were not installed to their design depth, and the untreated soft soil thickness beneath the short DR columns was in excess of 0.5m. Re-designs of the arch and the soft ground treatment were subsequently implemented, which involved a reduction of pile spacing at the northern footing of the arch, a small increase in temporary surcharge height and the installation of stone columns (SC) between the DR columns. As for the wing wall structures, the remedial solution comprised (i) a detached connection system to allow for greater tolerable movements between the wing wall and the spandrel wall of the arch structure; (ii) installation of stone columns (SC) underneath the reinforced soil block as well as the wall facing to reduce settlement and lateral spreading; and (iii) the use of dead-man anchor to prevent excessive yielding of the front row of DR columns supporting the wall facing. This paper focuses on the numerical analysis of the remedial arch and wing wall system, and compares its prediction with actual performance under short term loading conditions.

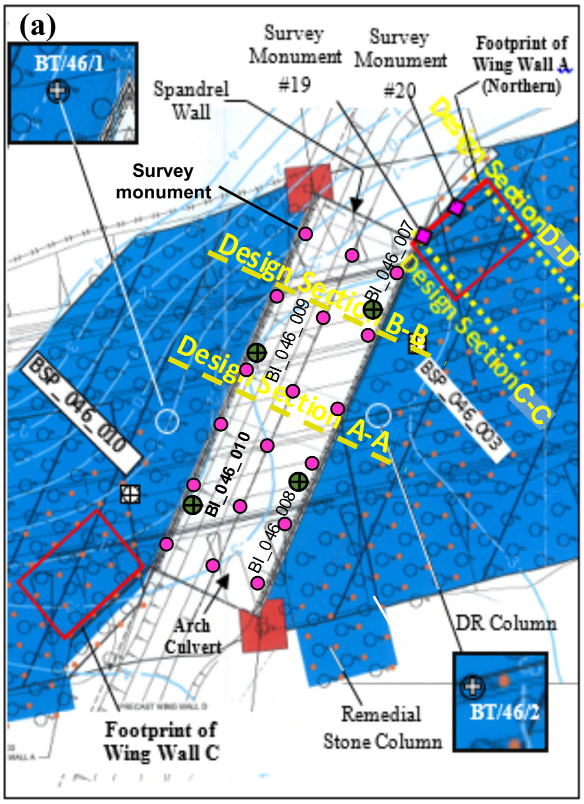
2. GROUND CONDITIONS

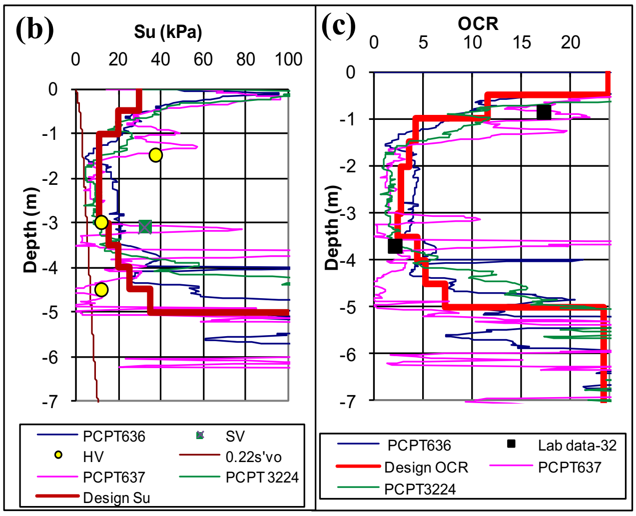
The subsurface stratigraphy was derived from field investigations comprising piezocone tests (CPTU) and boreholes. The soft ground at the arch culvert area comprises about 5m thick very soft to firm clay. The clay unit thins towards the south of the culvert. Figure 1d presents the geotechnical model which was developed at the location where soft soil deposits are the deepest. The corresponding profiles of inferred undrained shear strength (s_u) and over-consolidation ratio (OCR) are shown in Figures 1b and 1c. The salient features of the soil model are summarised as follows:

- The soft soil deposit comprises high plasticity clay with a liquid limit w_L of between 78% and 88%.

The plasticity index I_P is about 50%.

- The derivation of the undrained shear strength s_u profile with depth was estimated from the measured piezocone data using cone bearing factor N_{kt} value of about 15. The adopted N_{kt} value has been calibrated against corrected vane shear data.
- The adopted over consolidation ratio (OCR) profile is consistent with the design s_u profile through the relationship proposed by Ladd (1991), $s_u = S(OCR)^m \sigma'_{vo}$, where S = 0.20+0.05I_P (~0.22 for I_P=0.5); and m = 0.88(1-CRR/CR) ± 0.06 (~0.8 for CR/CRR = 7). In addition, the design OCR profiles compare reasonably well with the OCR values obtained from oedometer tests via the conventional Casagrande (1936) technique (see Figure 1c).
- The coefficients of consolidation c_h (horizontal) were derived based on pore pressure dissipation test within the CPTU. The adopted c_v (vertical) was taken to be half of the c_h values. Note that c_v and c_h derived from piezocone are significantly higher than those from the oedometer test results, which were considered to be too conservative since the testing sample may have been subject to disturbance.





(d) Layer	Thick-ness m	CR[(1)] –	CRR[(2)] –	OCR –	c_h m²/yr	γ [(3)] kN/m³	s_u kPa
1	0.5	0.2	0.03	70	20	18	30
2	0.5	0.35	0.05	11.6	5	15	20
3	0.5	0.35	0.05	4.3	5	14.5	11
4	0.5	0.35	0.05	3.6	5	14.5	11
5	1	0.35	0.05	2.8	5	14.5	11
6	0.5	0.35	0.05	2.4	5	14.5	15
7	0.5	0.35	0.05	4.5	5	14.5	20
8	0.5	0.3	0.05	5.3	5	15	25
9	0.5	0.3	0.05	7.3	5	15	35
10	4.5	0.1	0.015	23.5	50	19	150

(1) CR = compression ratio = $C_c/(1+e_o)$; (2) CRR = recompression ratio = $C_r/(1+e_o)$; (3) γ = bulk unit weight

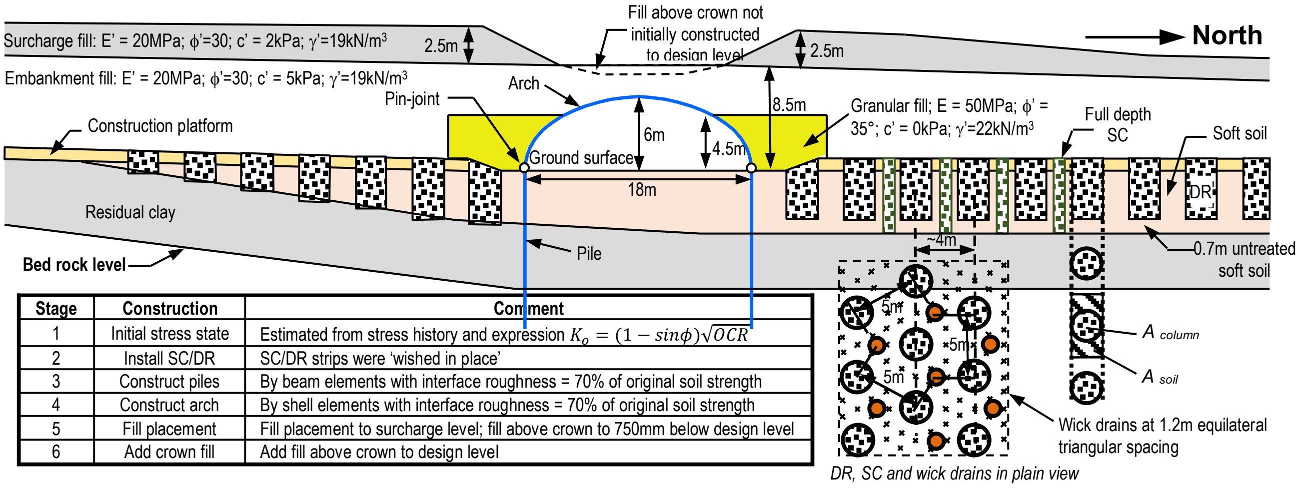
Figure 1. (a) Ground treatment and monitoring plan, (b) S_u profile inferred from CPT, (c) OCR profile inferred from CPT, (d) Adopted ground model and design parameters

3. GROUND TREATMENTS

The adopted ground treatments for the culvert approach included installation of dynamic replacement (DR) columns in conjunction with placing surcharge fill on either side of the culvert. Wick drains were installed after DR formation to increase the consolidation rate even though the DR columns would have already facilitated radial drainage in the soft clay.

DR columns were introduced into the ground by a heavy weight dropped repeatedly onto gravel layer while the craters created by the impact of the heavy weight were backfilled with gravel. The resulting DR columns were about 2.5m in diameter. The disadvantage of DR, however, was that there was a limiting depth to which the columns can be installed. For the northern side of the arch culvert where soft soil deposits are thickest, the maximum penetration depth of the DR columns through the top of the 1.2m thick working platform was about 5.5m, thereby leaving about 0.7m thickness of the soft clay (for a total soft clay thickness of 5m) untreated. It was also assessed that many of the DR columns at the wing wall areas did not fully penetrate to the soft soil base. Remedial ground treatments using stone columns (SC) and extra surcharge fill were subsequently introduced. The final treatment solutions are summarised in Table 1 and outlined in Figures 1 to 3.

Relevant instrumentations to the arch culvert are shown in Figure 1a, which include four settlement plates (BSP_046_002, 010, 003 and 011) near the arch culvert; two pressure cells (BT/46/1 and BT/46/2) installed at the top of DR columns to measure the imparted vertical stress; and four inclinometers (BI_046_007 to BI_046_010) to monitor sub-soil movement adjacent to the arch supporting piles.



Stage	Construction	Comment
1	Initial stress state	Estimated from stress history and expression $K_o = (1 - sin\phi)\sqrt{OCR}$
2	Install SC/DR	SC/DR strips were 'wished in place'
3	Construct piles	By beam elements with interface roughness = 70% of original soil strength
4	Construct arch	By shell elements with interface roughness = 70% of original soil strength
5	Fill placement	Fill placement to surcharge level; fill above crown to 750mm below design level
6	Add crown fill	Add fill above crown to design level

Figure 2. 2D FEA for arch culvert

Table 1. Adopted ground treatment at Upper Sand Flat

Ground treatment	Details
DR column	Nominally 2.5m in diameter; 5m equilateral triangular spacing; area replacement ratio, a_r = 23%.
Remedial SC	Remedial SCs were installed on the northern side of the culvert. A clearance of about 7m to 8m has been allowed between the arch and the 1st row of SC to avoid disturbance on the installed arch supporting piles. The remedial SC are of nominally 1m diameter, 5m equilateral triangular spacing and with a_r = 3.6%.
Surcharge Fill	2.5m thick surcharge on either side of the culvert for 6 months. No surcharge immediately above the culvert.
Wick Drain	Installed after DR formation at 1.2m equilateral triangular spacing.

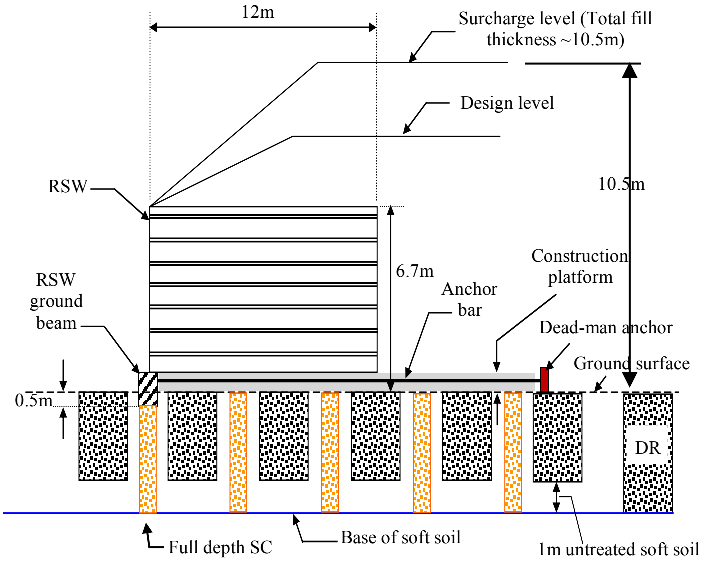


Figure 3. Reinforced soil wing wall design configuration

4. GEOMETRY AND LOADING HISTORY OF ARCH CULVERT

Figure 2 shows a cross section of the arch structure and its principal dimensions. The arch comprises a 350mm thick pre-cast reinforced concrete section, with a span of 18m and a 6m rise. The culvert was aligned at a skewed angle of 55° from the centre of the alignment. It was constructed by 36 arch units, each of which 1.8m wide. Some of the key steps in construction and monitoring are as follows:

- The arch structure was erected on ground beams that are supported by a single row of 750mm bored piles with a centre to centre spacing of 2m and 1.8 on each side of the arch. The piles are 12m long

founded in extremely weathered argillite rock with a socket length of about 2.5m.
- Each arch unit comprises two half sections joined at the crown. The joint is tied together by a capping beam of 2.25m wide by 0.35m thick. Bending moment is allowed to develop at the crown and the final arch structure can be considered as a two-pin arch (i.e. pin joints at the base only).
- Survey monuments were introduced at ground beams and top of the culvert on the inner face of the arch to measure the horizontal movements at the spring lines and the vertical movement at the crown. The monument location plan for the culvert is shown in Figure 1a.
- Back fill was placed with a maximum side differential lift of 600mm. In particular, granular fill (well graded gravelly sand) was placed at the side of the arch up to 4.5m high. It was compacted to not less than 95% of the maximum dry density at optimum moisture content for standard compaction.
- During fill placement, the embankment fill immediately above the crown was constructed to about 750mm below design level, while full surcharge thickness was placed on either side of the culvert (see Figure 2). This has resulted in large amounts of upward movement in the arch. Backfilling to design level above the arch took place subsequently.

5. REINFORCED SOIL WALL DESIGN

While the total fill behind the RSW is at a constant height of 10.5m, the design wall height varies linearly from a lowest end of about 3.6m to a top end of about 7m next to the spandrel wall of the arch culvert unit. There is no shear connection between the wing wall and the spandrel wall to allow movements of the wing wall. The gap at the detached connection is sealed with deformable material in conjunction with geotextile. The following design criteria were incorporated in the wing wall design: (i) maximum allowable differential movements (both vertically and horizontally) for the wall face of 1.0 percent change in grade to prevent cracking of wall panels; and (ii) Maximum horizontal movement of the wall face of 150mm over 100 years to avoid potential closure of the 170mm clearance at the wing wall / spandrel wall juncture.

The design configuration of the reinforced soil wing wall for the required criteria is outlined in Figure 3. In essence, the reinforced soil block is 12m wide and is built upon a 0.75m thick working platform that was constructed over the DR and remedial SC columns. To limit the total and differential settlements of the wall facing, a ground beam spanning over a row of remedial stone columns is provided to support the precast wall panels. To limit the applied horizontal force on the supporting stone columns which are geotechnical elements without significant bending stiffness, a dead-man anchorage system is adopted to tie back the ground beam into the platform fill. The dead-man anchor block is a continuous beam with dimension 1.25m (H) × 1m (W). The anchor bars that connect the ground beam and the dead-man block are 20m long 32mm diameter stress-bar at a horizontal spacing of 3m. Survey monuments were introduced at the ground beam to monitor wall-face movements. Typical monument location plan for Wing Wall A is shown in Figure 1a. The dead-man anchor also assists the overall stability against sliding of the reinforced soil block. If the reinforced soil wall was built without the dead-man anchor, the reinforced soil block would need to be embedded for the required sliding resistance. The soil excavation within the reinforced soil block footprint may be subject to construction and/or environmental constraints including (i) water inflow during excavation, (ii) damage of the DR and SC columns and (iii) disposal of excavated soft soil.

6. PERFORMANCE PREDICTION APPROACH

The modelling of the arch and the wing wall was carried out based on 2D finite element analyses (FEA) using PLAXIS 2D software program. Owing to the 3D nature of the SC/DR columns, a separate 3D FEA (using the PLAXIS 3D Foundation software program) was undertaken for a group of SC/DR columns to assess the equivalent 2D column stress concentration parameters. The 3D group analysis also involved consolidation such that the time for the stabilisation of the arch movement can be assessed.

Conventionally, the design of SC/DR columns involves the prediction of their settlements using a composite material approach in which equivalent strength and deformation parameters are derived using semi-empirical correlation to represent the entire treated soil. For the current problem, however, the key design criteria are horizontal displacements. The above composite material approach, while has been accepted as a reasonable method for vertical displacement prediction, is less certain for the prediction of horizontal displacement. The adopted design approach was to explicitly model the SC/DR as strips in the 2D FE model with the appropriate diameter, spacing and smeared properties of the columns.

In the 2D FE model, the soft clays were modelled using Soft Soil Model in PLAXIS program, which resembles the Modified Cam-Clay model with a Mohr-Coulomb hexagon yield surface in the deviatoric plane. The adopted model parameters have been given in Figure 1d. To model the equivalent area replacement ratio in the 2D analyses, the widths of the DR/SC strips were the same as their diameters and the DR/SC properties were smeared based on 5m spacing within the strips, but the strip spacing was 4m because of the triangular configuration. The DR/SC strips were modeled as Mohr-Coulomb materials with adopted Poisson's ratio of 0.3. The equivalent Young's modulus, E_{eq}, of the strips can be calculated based on weighted average approach as given by Equation 1, where A_{soil} and A_{col} are the area of the soil and the column inside a unit cell within the strips.

$$E_{eq} = \frac{E_{soil} A_{soil} + E_{col} A_{col}}{A_{soil} + A_{col}} \tag{1}$$

The design Young's moduli of the SC/SR columns (E_{col}) are given in Table 3 and the adopted soil Young's modulus (E_{soil}) was 1.65MPa, which was equal to 150 times the soil undrained shear strength of 11kPa. The equivalent 2D strip friction angle, ϕ_{eq}, can be derived based on force equilibrium by:

$$\tan(\phi_{eq}) = \frac{A_{soil}\tan(\phi_{soil}) + n \cdot A_{col}\tan(\phi_{col})}{A_{soil} + n \cdot A_{col}} \tag{2}$$

where ϕ_{col} are given in Table 3; ϕ_{soil} was assumed to be 25°; and n was the stress concentration factors over the SC and DR columns (i.e. column stress / soil stress). For the parameter n, it may not be able to estimate adequately using published correlations due to the presence of SC and DR with different lengths and diameters; and the presence of in-homogenous soil layers. In the design analysis, n was assessed separately by carrying out a full 3D FEA for a group of SC/DR columns under axially symmetric condition as shown in Figure 3.

Table 2. Construction sequence for the modelling of reinforced soil wing wall

Stage	Construction Operation	Comment
1	Calculate initial stress for in-situ ground	Initial in situ effective stresses were estimated from the assumed stress history (Figure 1d) and the expression $K_0 = (1-\sin\phi)\sqrt{OCR}$
2	Install construction platform and SC/DR columns	The SC/DR strips were 'wished in place'; No installation effects have been considered. The smeared SC/DR properties are assessed from Equations 2 and 3, in conjunction with the stress concentration factors obtained from a separate 3D FEA.
3	Construct ground beam and dead-man anchor	The Ground beam and dead-man anchor were represented by linear elastic material with E' = 32GPa and ν = 0.15. The anchor bars were modelled using two-node elastic spring element without pre-stressing. The long term axial stiffness of the bar was derived based on E' = 200GPa and a reduced cross section sacrificial thickness of 0.85mm.
4	Reset Displacement to zero	
5	Construct RSW and embank. fill to top of surcharge level	Membrane elements with limiting tensile strength were used to model reinforced strips. No slippage was allowed at the membrane face since it had been designed (based on limit equilibrium) to have its pull-out resistance > the tensile capacity. As layers of soil and reinforcement were placed at the RSW, a wall facing represented by discrete beam elements were also included. Interface elements were introduced at the soil-wall contact to allow for slippage. The roughness of the interface was assumed to be 70% of the original soil strength values.
6	Strip surcharge to design level; applied traffic surcharge for long term performance assessment.	

The salient features of the 2D FEA model, as well as the adopted construction sequences are summarised in Figure 2 for the arch culvert and Table 2 for the reinforced soil wing wall. Decay of excess pore pressure with time during primary consolidation was not considered in the 2D (drained) FEA, since it is difficult to convert SC/DR columns and wick drains into equivalent drain walls. In the design analysis, the time for consolidation was assessed separately by carrying out a coupled consolidation analysis in the 3D modeling for the SC/DR group under 10.5m embankment fill as depicted in Figure 4. The radial drainage towards the SC/DR columns, which were treated as large diameter drains with high permeability, can be modelled directly. The time for the stabilisation of the wall movements can be inferred from the time-settlement curve obtained in the 3D FEA. One challenge in the consolidation analysis is the selection of soil permeability values k_v (vertical) and k_h (horizontal), which can be inferred from piezocone data. Owing to the installation method for the SC and DR columns, the soil surrounding the columns may have been remolded/smeared. This would lead to a lower permeability than that of the in-situ state. However, it is noticed from the comparison with the monitoring data that the reduced k_v and k_h of the remolded clay may have been compensated by the wick drains such that a reasonable agreement between measurement and prediction was obtained by adopting the in-situ k_v and k_h of a normally consolidated clay without smearing, and without the modelling of wick drains.

7. RESULTS OF 3D GROUP ANALSIS FOR SC/DR COLUMNS

Prior to the modelling of the arch culvert and the RSW in 2D, a pilot 3D group analysis for the SC/DR columns was carried out to assess the stress distribution between columns and soil. Further, the 3D analysis also involved coupled consolidation such that the time for the stabilisation of wall movement can be assessed. Figure 4 shows a slightly exaggerated deformed 3D mesh of the SC/DR columns under full embankment load. The full depth SC under full embankment load exhibited bulging in the soft soils, while the floating DR columns have undergone punching type deformation at the column base. The increases in vertical effective stress for the SC, DR and the surrounding soil are shown in Figure 5. For the DR with a_r = 23%, the effective vertical stress peaked at about 1m below ground level. Below this level, the imparted DR stress reduced as the load was transferred from the floating DR to the surrounding soil and SC (a_r = 3.6%), which exhibit increased vertical effective stress with depth. The predicted column stress of about 400kPa at the top of the short DR agrees reasonably well with pressure cell BT_046_002 (Figure 7). BT_046_001 at the southern side of the arch has measured slightly higher column stress due to the presence of full depth DR columns. Figure 6 shows the predicted stress concentration factors n_{SC} (for SC) and n_{DR} (for DR) versus depth. In particular, n_{SC} = 4.5 and n_{DR} = 3.5 were adopted for the derivation of equivalent friction angle, ϕ_{eq}, for the SC/DR strips in the 2D modelling. Table 3 summarises the adopted 2D column parameters, including the ϕ_{eq} for the full depth DR at the southern side calculated based on axi-symmetric analysis.

Figure 8 shows a comparison of the predicted time-settlement from the 3D analysis and the measurement from settlement plate BSP_036_003. The agreement between the two results is considered satisfactory. Note that wick drains were not included in the 3D model, and the results given in Figure 8 might have suggested that such an omission in the analysis was compensated for also not modelling smearing of the remolded soil surrounding the SC/DR columns. Due to the fact that the construction platform was built much earlier than the embankment fill, some settlement has already occurred prior to the fill placement. In the design analysis, however, the platform fill and the embankment fill were constructed at the same rate, potentially contributing to the discrepancy between the prediction and measurement at the onset of the fill loading. The assessed time for the wing wall to settle, as inferred from the time to achieve 90 percent degree of consolidation in Figure 8, is about 5.5 months from the start of fill placement.

Table 3. Design parameters for SC and DR columns

DR/SC	n [(1)]	Adopted E' for column	Smeared E' in 2D FEA	Adopted ϕ' for column	Smeared ϕ' in 2D FEA
	(–)	(MPa)	(MPa)	(degrees)	(degrees)
Northern side of arch with short DR and remedial SC					
DR	3.5	30	12.8	35	31
SC	4.5	50	9.3	40	32
Southern side of arch with full depth DR only					
DR	4.0	30	12.8	35	33.5

(1) n = stress concentration factor = column stress / soil stress at the same level

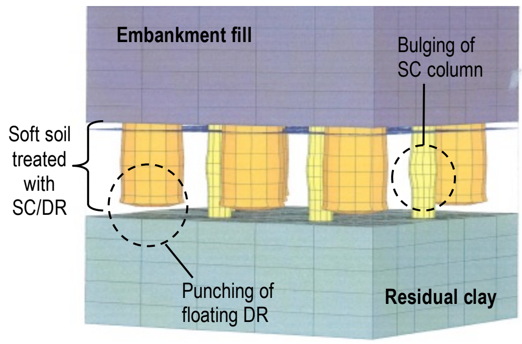


Figure 4. 3D FEA for SC/DR group

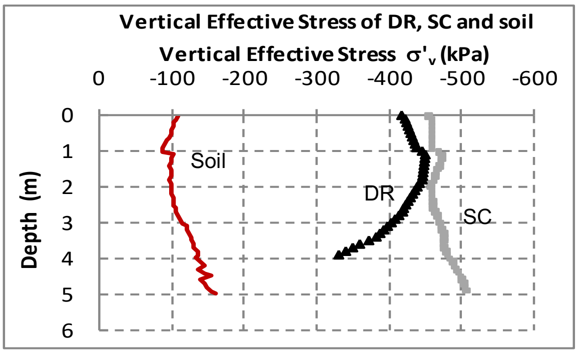


Figure 5. Vertical effective stress

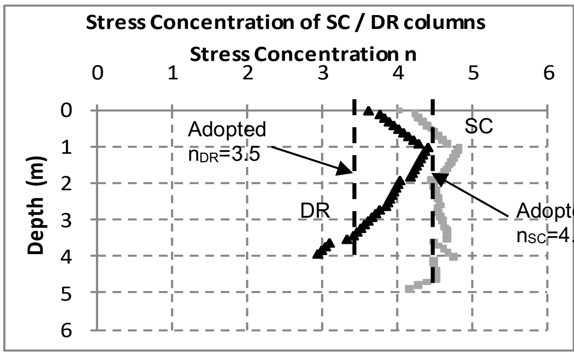


Figure 6. Column stress concentration

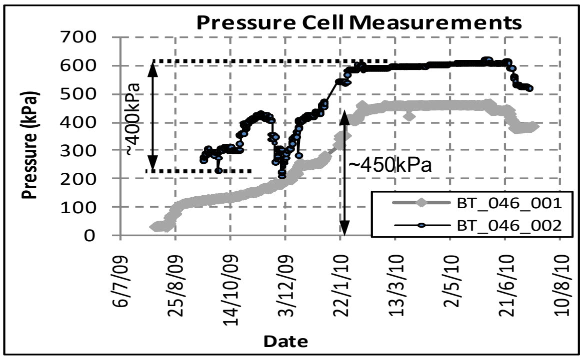


Figure 7. Pressure cell measurement

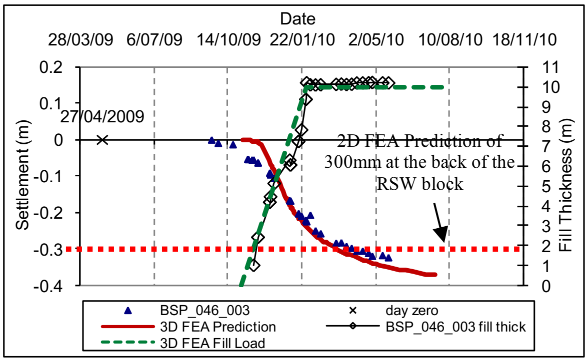


Figure 8. Time-settlement prediction from 3D FEA

8. RESULTS OF 2D ANALYSIS FOR ARCH CULVERT

Figure 9 shows the 2D FEA predictions and measurements for Section A-A of the arch. Lateral squeezing of the arch is evident when surcharge fill was placed on either side of the arch. This has resulted in upward movement of the arch crown and inward movement of the ground beams (Figure 9e).

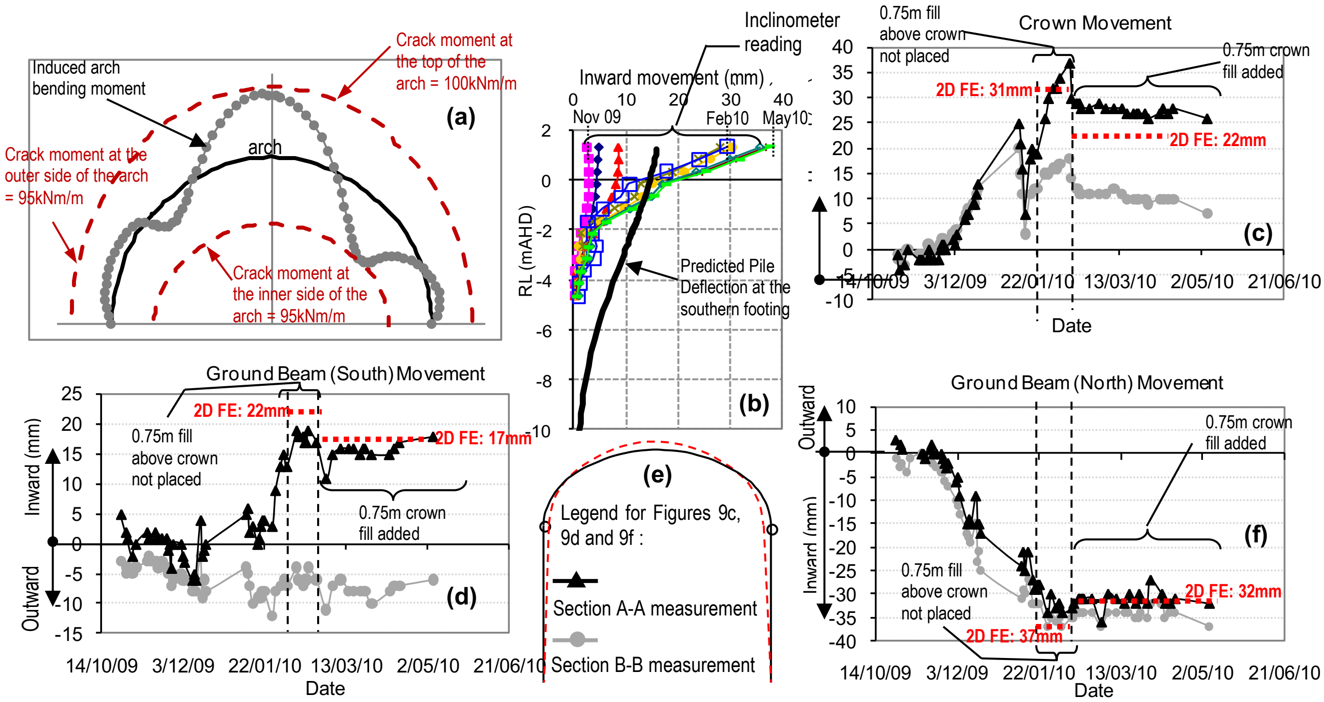
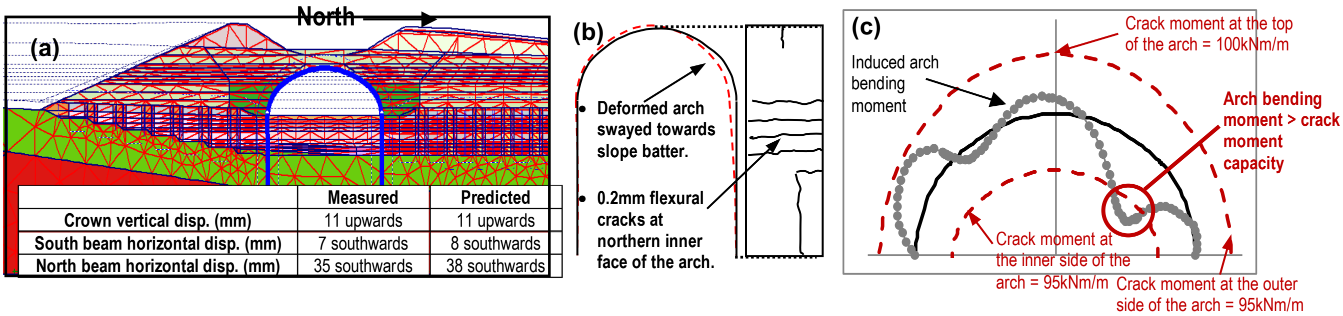


Figure 9. 2D FEA design predictions and measurements for Section A-A

The FEA provided reasonable predictions for the ground beam (horizontal) and crown (vertical) movements as shown in Figures 9c, 9d, 9f. In particular, the FEA has captured satisfactorily the movements before and after the placement of the 750mm crown fill. The inward movement of the southern arch beam (~15mm) is less than that of the northern arch beam (~32mm), which may be attributed to the stiffer treated soft ground in the southern side (with full depth DR) than in the northern side (with short DR). The arch movements stabilised soon after the 0.75m fill above crown was placed. This soil weight above crown gave the arch supporting piles some outward tendency to counter act the inward subsoil movement, leading to an early stabilisation of arch movement at about 4 months, as compared to the inferred 5-month prediction from settlement analysis. Figure 9b shows the measured soil movement (from inclinometer) and the predicted long-term deflection of the southern piles. The reinforced concrete arch was designed to be free of cracks. Figure 9a shows that the predicted arch bending moments is within the cracking moment capacity of the arch.

Although the 2D FEA predictions are satisfactory for Section A-A in the middle of the culvert, they differ greatly from the measurements for sections near the ends of the culvert. Flexural cracking was observed at the inner face of the culvert towards the embankment batter, as indicated by the evenly spaced crack pattern. Figures 9c, d and f show the measured arch movements for Section B-B at the western end of the culvert (see Figure 1a for location), which deviate significantly from the predictions for Section A-A. Figure 10c shows the mapping of the cracks on an arch unit near Section B-B. The cracks were formed at about mid-height of the northern inner side the arch, whereas no cracks were seen on the southern side. The observed behavior at Section B-B is believed to be due to uneven loading as the culvert approaches the embankment batter. A retrospective 2D FEA was carried out as shown in Figure 10a, which accounted for the slope batter on the southern side of the arch at Section B-B. The deformed arch swayed towards the batter with the predicted movements agreeing closely with the measurements (Figures 10a, 10b). The predicted arch bending moment, although still within the ultimate capacity, has exceeded the design crack moment at the northern inner side of the arch. This is consistent with the location of the mapped cracks.



	Measured	Predicted
Crown vertical disp. (mm)	11 upwards	11 upwards
South beam horizontal disp. (mm)	7 southwards	8 southwards
North beam horizontal disp. (mm)	35 southwards	38 southwards

Figure 10. 2D FEA design predictions and measurements for Section B -B

9. RESULTS OF 2D ANALYSIS FOR REINFORCED SOIL WING WALL

Before considering the numerical results of the wing wall movements, it is illuminating to discuss the deformation mechanism of the RSW and the function of the dead-man anchor. Finite element analyses have shown that lateral soil movements of the SC and adjacent soil beneath the wall facing have a direct influence on the overall performance of the wing wall design. For example, if the first row of SC supporting the wing wall ground beam is omitted, both horizontal and differential vertical movements may exceed the design criteria. However, even with the SC in place is not a complete solution since the SC, which is stiffer than the surrounding soft clay, have attracted concentrated shear stress within the column. This has resulted in lateral yielding of the column, thus undermining its effectiveness in reducing the lateral spreading under embankment load. The dead-man anchor is therefore considered to be an essential component in the wing wall design for its role to limit the applied shear force on the SC, thus greatly reducing the outward movement of the wall facing. Figure 11 shows an exaggerated deformed mesh of the Wing Wall A under short term fill load with surcharge at cross-section C-C in Figure 1a. The RSW block deforms in a backward tilting mode, with the predicted settlements at the wall facing and behind the RSW block being equal to 210mm and 300mm, respectively. These results are in good agreement with the field measurements; refer Figure 13 for settlement at wall facing and Figure 8 for settlement behind RSW block. Also note that the time for the stabilisation of the wing wall facing is about 5.5 month according to the ground beam measurement (i.e. based on Survey Monument #19, see Figure 13). This is consistent with the measured data at settlement plate BSP_046_003 behind the RSW block, as well as with the 3D analysis result (Figure 8). Figure 14 shows a comparison of the predicted ground beam horizontal movement from the 2D analysis and measurement. The prediction is about 2.2 times greater than the measured value. Several reasons may be postulated for the difference between the measured and predicted horizontal movements:

- Installation effect on undrained soil strength – The SC/DR columns in the 2D and 3D analyses were ‘wished in place’, and the installation effects have not been considered.
- Installation effect on anisotropic column stiffness – For the SC/SR columns that are formed by granular materials, the increase in horizontal to vertical effective stress ratio ($\sigma'_h/\ \sigma'_v$) due to column installation may increase the degree of anisotropy of the column stiffness (E_h'/E_v').
- The use of reduced anchor bar stiffness – As indicated in Table 2, the anchor bar was modelled using a long-term axial stiffness (EA/L) that account for a sacrificial thickness of 0.85mm. This may however underestimate the axial stiffness for the short term.
- It is well established in literature (e.g. Poulos 1972) that it is difficult to achieve reliable predictions of lateral movements under an embankment, especially after the end of construction.

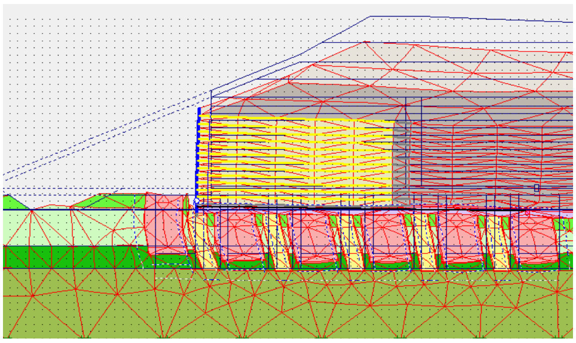


Figure 11. 2D FEA for wing wall A, section C-C

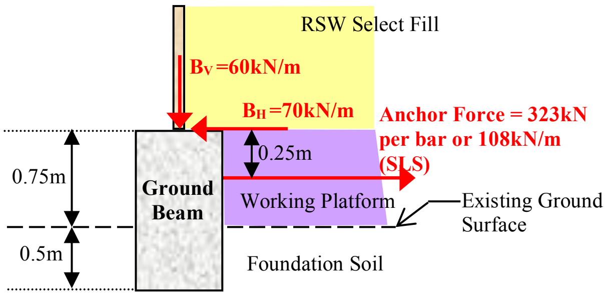


Figure 12. Reaction of ground beam and anchor

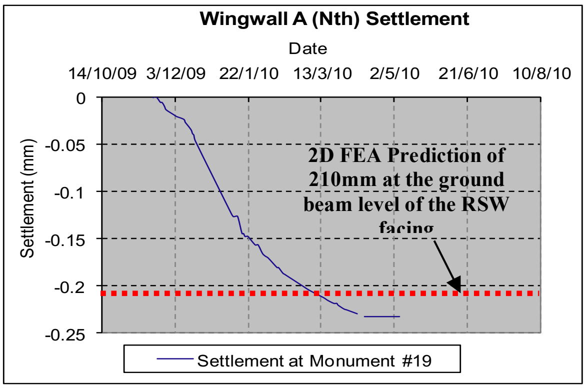


Figure 13. Measured and predicted settlement at Wing wall facing

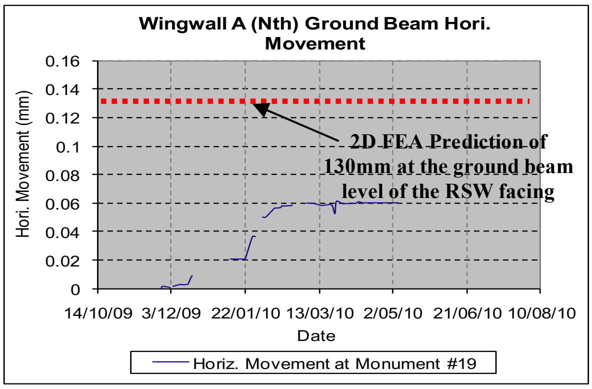


Figure 14. Measured and predicted horizontal movement at ground beam level of Wing Wall A

Table 4 summarises the performance predictions under short term surcharge loading, as well as presenting some of the measured data from monitoring results. Figure 12 summarises the predictions for the maximum reaction forces of the ground beam and anchor bar, which are within their structural capacities. A free body diagram showing the reaction forces is given in Figure 12. It is noted that while design cross-section C-C can be compared with measurements from survey monument #19 because of their similar locations, the design cross-section D-D is about 7.5m south of survey monument #20 and therefore they cannot be compared directly due to the different wall heights at the two locations. In terms of differential wall movements, the predicted results are in good agreement with the measurements both vertically and horizontally. Overall, it can be seen that the design perditions are reasonable with regard to settlement and differential movements. The prediction for the total horizontal movement is however erring on the conservative side, although it remains within the design limit of 150mm.

Table 4. Summary of Wing Wall A movements under short term construction loading

	Prediction		Measurement	
Cross section /Monument	C-C	D-D	#19	#20
Wall height (m)	6.7	4.0	6.7	5.2
Total Fill height behind batter (m)	10.5	10.5	10.5	10.5
Settlement (mm)	210	160	230	210
Hori. movement at wall base (mm)	130	100	60	45
Differential settlement	50mm settlement over 15m between Sections C-C and D-D; or 0.33% change in grade		20mm settlement over 7.5m between Monuments #19 and #20; or 0.27% change in grade	
Differential horizontal movement	30mm lateral movement over 15m between Sections C-C and D-D; 0.2% change in grade		20mm lateral movement over 7.5m between Monuments #19 and #20; 0.2% change in grade	

10. CONCLUSION

This paper presents the design and performance of a concrete arch culvert and wing walls founded on treated soft ground. A 2D FEA has been carried out to model the SC/DR columns modeled as equivalent strips. A separate 3D coupled FEA for a group of SC/DR columns was carried out to supplement the 2D FEA. The objectives of the 3D FEA were to assess (i) the equivalent 2D column stress concentration parameters, and (ii) the time for the stabilisation of arch movement. The analysis predictions have been compared with field measurements and the following conclusions can be drawn:

- The stress distribution between SC/DR columns and soil has been obtained from the 3D group analysis and was shown to be in reasonably good agreement with field measurements.
- Although wick drains and smearing were not included in the 3D FEA, the predicted time-settlement response has compared favourably with settlement plate measurements. This may suggest that the wick drains serve to compensate for the slowdown due to remolded soil. The arch movement however appeared to be stabilised before the settlement plate, which may be due to the arch supporting piles being insensitive to any small increment of soil flow at the end of the primary consolidation.
- Attention should be drawn to the effect of embankment batter when designing arch structure with a skewed alignment. For the arch culvert with a skewed angle of 55° from the main alignment, it has been shown that although the 2D FEA has given satisfactory arch movement predictions for a middle section of the culvert, the results differ greatly from the measurements towards the culvert ends. The induced arch bending moment near the batter, although within the ultimate capacity, has exceeded the crack moment of the reinforced concrete designed based on the 2D model of the middle section.
- In general the 2D FEA has given satisfactory wall movement predictions with regard to settlement and differential wall movements (both vertical and horizontal). The prediction for the total horizontal movements is however erring on the conservative side, and is within the design limit of 150mm.

REFERENCES

Casagrande, A. (1936). "The determination of the pre-consolidation load and its practical significance. " Proc. 1st ICSMFE, Cambridge, 3, 60-64.

Ladd, C.C. (1991). "Stability evaluation during staged construction: 22nd Terzaghi Lecture." Jnl of Geot Eng, ASCE, 117(4), 537-615.

Poulos, H.G. (1972). "Difficulties in prediction of horizontal deformations of foundations." J. Soil Mech. Found. Div., Proc. ASCE, 98 (SM8), 843-848.