

FIELD BEHAVIOUR OF FILL STRUCTURES ON THE SOUTH WEST RAIL LINK

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ABSTRACT

This paper provides a summary of the instrumentation and monitoring program implemented during the Glenfield Transport Interchange component of the South West Rail Link. Works included the construction of reinforced soil walls and fill embankments up to 9 m in height as part of the connection alignment between the existing East Hills Line and the Glenfield to Leppington rail line. The key challenge in the region was meeting the stringent long term settlement criteria specified by RailCorp. This was managed by preloading of the fill embankments, staged construction and a detailed instrumentation program which included inclinometers, magnetic extensometers, vibrating wire piezometers and settlement plates. Ongoing monitoring of retaining wall and embankment performance during construction allowed for confirmation of design estimates and management of preloading time periods. This paper provides a comparison of the predicted and actual ground behaviour, together with a discussion of the effectiveness of the techniques employed.

1 INTRODUCTION

The South West Rail Link is an initiative to respond to issues of rail reliability and passenger growth on the metropolitan rail network as well as issues of population growth in south-west Sydney. The South West Rail Link includes a new 11 kilometre rail line from Glenfield to Leppington with new stations at Edmondson Park and Leppington before terminating at a new stabling facility.

The first component of the South West Rail Link is the design and construction of the Glenfield Transport Interchange, comprising the following works:

- i) Upgrade of the existing Glenfield station including construction of a new platform, footbridge and overhead concourse.
- ii) Realignment and replacement of approximately 2 km of track including new up and down East Hills lines.
- iii) Construction of a flyover structure to the north of Glenfield station. The northern flyover structure will remove the need for the existing at grade crossing, taking the East Hills line over the Main South lines.
- iv) Construction of a flyover structure to the south of Glenfield station. The southern flyover structure will allow trains travelling to and from the South West Rail Link to join with either the existing Main South or East Hills lines.
- v) Construction of a series of fill embankments, viaducts and reinforced soil walls connecting the southern flyover viaducts to the South West Rail Link.

The behaviour of the fill embankments and reinforced soil walls at the southern flyover connection is the primary focus of this paper.

2 KEY CHARACTERISTICS OF THE SOUTHERN FLYOVER CONNECTION

2.1 REGIONAL GEOLOGY

The subsurface conditions in the region of the southern flyover are generally characterised by 'Modern Alluvium', 'Alluvial Terraces', 'Ashfield Shale', and 'Hawkesbury Sandstone'. Modern Alluvium is identified as Holocene period sediments comprising sand, silty sand, silt and clay. Alluvial Terraces in this region are identified as Pleistocene and Holocene age sediments including sand, silt and clay. Ashfield Shale is the basal unit of the Wianamatta group and comprises dark grey to black sideritic claystone and siltstone. Hawkesbury Sandstone is described as a quartz sandstone, generally medium to coarse grained, and may include pebbles of white vein quartz and fine elongated laminae.

2.2 SOUTHERN FLYOVER CONNECTION SITE DESCRIPTION

The southern flyover branches off from the existing Main South and East Hills lines approximately 500 m to the south of the existing Glenfield station. This then connects to the South West line via a series of fill embankments, viaducts and reinforced soil walls as shown in Figure 1.

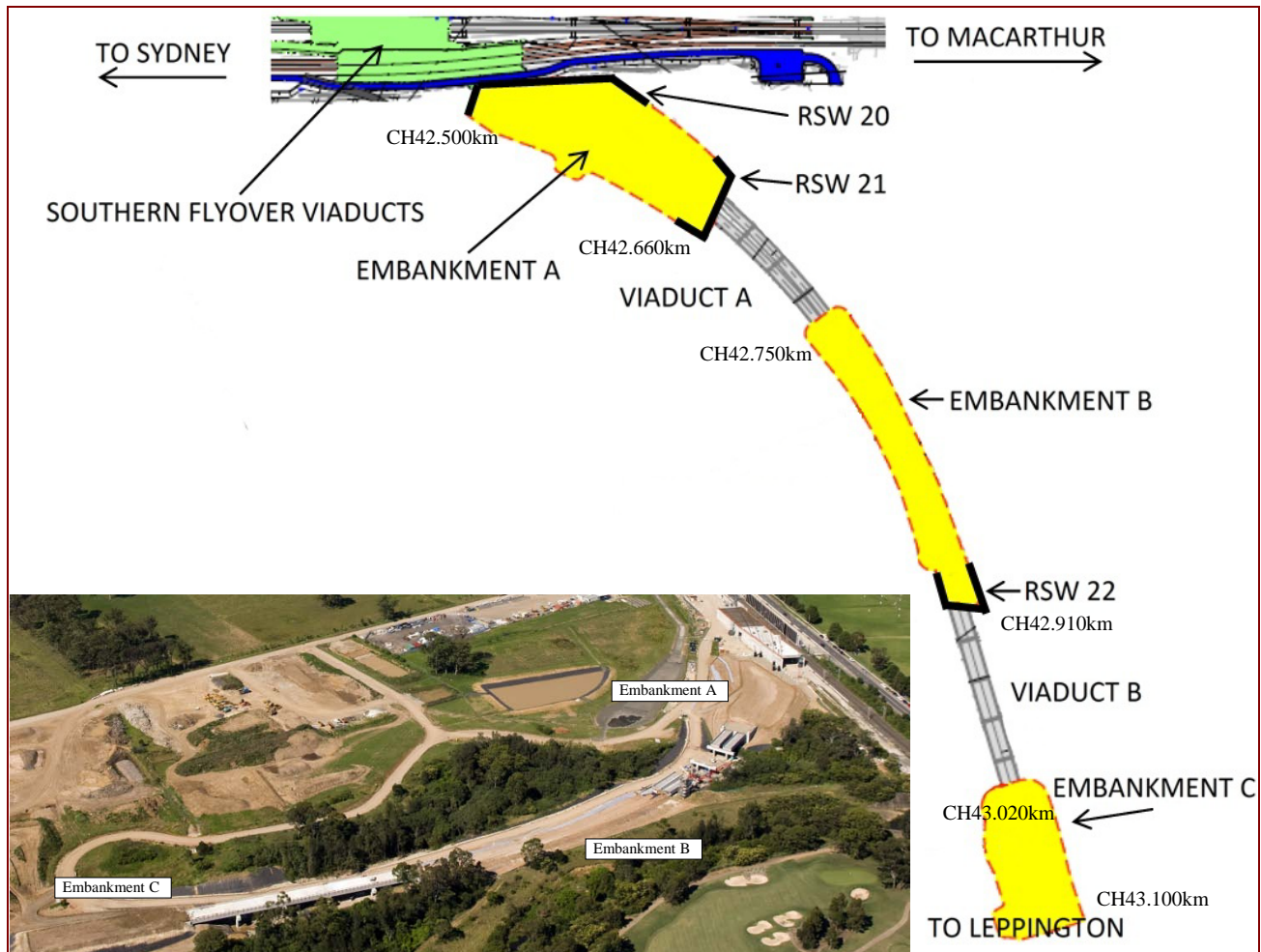


Figure 1: Embankment, viaduct and RSW locations at the southern flyover connection site.

2.3 ALLOWABLE SETTLEMENT CRITERIA

Geotechnical design settlement criteria with respect to the railway embankment were discussed and agreed with RailCorp. These design criteria were as follows:

- i) Tolerable maximum long term settlement was to be limited to 50 mm over 20 years.
- ii) Tolerable maximum change in grade in both longitudinal and transverse direction of the railway embankment was to be limited to 1%.

Although other design criteria were in place, compliance with the long term settlement requirement set down by RailCorp became the governing design criteria for this particular design package.

2.4 SUBSURFACE CONDITIONS IN THE REGION OF THE SOUTHERN FLYOVER CONNECTION

The southern flyover connection site lies approximately between chainages 42.500 km and 43.100 km on the planned Leppington line. The area is characterised by dense vegetation and steeply inclined and undulating ground surfaces. Subsurface conditions generally comprise of firm to very stiff alluvial clays and medium-dense to dense alluvial sands with some minor fill deposits. Existing creek beds with some localised soft alluvial deposits were identified in the regions of Viaduct A and Viaduct B. Groundwater levels were typically observed to lie between RL12 and RL15. Table 1 summarises the inferred geological profile throughout the region.



Photographs 1 and 2: Creek bed area at approximately 42.950 km

Table 1: Inferred geological profile at each embankment location.

Material	Embankment A	Embankment B	Embankment C
Fill	-	-	3 m at city end only
Firm clay	-	-	0 – 0.5 m
Stiff clay	1 – 9 m	-	-
Very stiff to hard clay	1.5 – 10 m	7 – 10 m	8 – 10 m
Medium dense clayey sand	1 – 2 m lenses	2 – 4 m as discontinuous layers	-
Firm clay	-	2 m	-
Hard residual soil	1 – 2 m	1 – 3 m	1 – 2 m
Class V ¹ Sandstone / Shale	1 m	1 – 2 m	1 – 1.5 m
Class III ¹ Shale	-	0 – 3 m	4 – 5 m
Class III ¹ Sandstone	>5 m	>2 m	>2.5 m

1. Rock classification in accordance with the 1998 rock classification system developed by Pells *et al.*

3 MANAGEMENT OF GROUND BEHAVIOUR

3.1 STAGED CONSTRUCTION PROGRAM

Settlement analyses indicated that long term settlement throughout the southern flyover connection region would exceed the 50 mm in 20 years limit specified by RailCorp. It therefore became necessary to accelerate consolidation during construction in order to limit the long term deformation of the live rail line. To support this requirement a staged construction program was developed, with mandatory preloading periods to allow for settlement. The following general methodology was followed during construction:

- i) Installation of abutment piles.
- ii) Construction of viaduct abutments.
- iii) Construction of embankment and reinforced soil walls.
- iv) Completion of filling works. Fill material would be placed and compacted to finished level, and a further layer of general fill material would be placed to final track level. This resulted in an average preload layer thickness of approximately 2 m.
- v) With the additional general fill material in place, a preloading period of 4 months was applied. In the case of RSW 20, a staged filling schedule with two separate 3 month preload periods was implemented. This approach was adopted because the short term bearing capacity of the subsurface strata was insufficient and the intermediate waiting period allowed for sufficient strength gain in the founding material.
- vi) Installation of bridge decks.
- vii) Removal of preload material to the top of the general fill layer, placement of structural fill and capping material followed by completion of track works.

3.2 INSTRUMENTATION AND MONITORING PROGRAM

In support of the staged construction program, a detailed instrumentation and monitoring program was developed. The principal aims of the instrumentation program were as follows:

- i) Confirmation that the degree of primary consolidation of foundations beneath constructed embankments and reinforced soil walls met the settlement performance criteria agreed with RailCorp.
- ii) Comparison of actual ground movements against design predictions.
- iii) Allow re-evaluation of the ground strength and design parameters and provide opportunity for correction should the actual performance be underestimated.

Table 2 summarises the types and quantities of instruments installed at various locations throughout the southern flyover connection site.

Table 2: Types and quantities of instrumentation installed through the southern flyover connection site.

Location	Inclinometer	Vibrating Wire Piezometer	Magnetic Extensometer	Settlement Plate
Embankment A	5	1	0	5
Embankment B	4	1	0	6
Embankment C	1	1	0	5
RSW 20	0	2	1	2
RSW 21	0	1	1	1
RSW 22	0	0	0	1

Readings were generally taken twice per week until the end of the preload period, at which point the frequency was decreased to one reading per month. Instrumentation data was reviewed on a weekly basis to facilitate the release of preloading hold points and make adjustments to the construction program where necessary.

4 COMPARISON OF PREDICTED VS ACTUAL GROUND MOVEMENTS

4.1 PREDICTED GROUND SETTLEMENT

Settlement analyses were carried out using one dimensional consolidation analysis and the finite element modelling software PLAXIS 2D. Table 3 summarises the geotechnical design parameters adopted in these analyses.

Table 3: Geotechnical design parameters.

Material	γ (kN/m ³)	c' (kPa)	ϕ' (deg)	ν	E' (MPa)	m_v (m ² /MN)	$C_{\alpha c}$	c_v (m ² /year)
Fill	19	0-5	26-28	0.30	8	0.20	0.002	40
Firm clay	18	2	26	0.30	8	0.18	0.003	15
Stiff clay	19	5	28	0.30	15	0.12	0.002	30
Very stiff clay	20	10	28	0.30	30	0.08	0.001	70
Hard clay	20	10	28	0.30	50	0.05	0	150
Medium dense clayey sand	19	0	31	0.30	35	-	-	-
Class V ¹ Shale	22	10	28	0.30	80	-	-	-
Class V ¹ Sandstone	22	30	35	0.30	100	-	-	-
Class III ¹ Shale	24	100	30	0.20	500	-	-	-
Class III ¹ Sandstone	24	200	35	0.20	700	-	-	-

1. Rock classification in accordance with the 1998 rock classification system developed by Pells *et al.*

Table 4 summarises the ground deformations that were predicted at each element of the southern flyover connection site during the design phase.

Table 4: Calculated settlement values during and after construction.

Location	Preload Period (months)	Height (m)	Settlement at ground level to end of preloading period (mm)	Post construction settlement at rail level over 20 years (including creep)(mm)
Embankment A	4	6.3 – 9.1	49 – 91	20 – 35
Embankment B	4	2.7 – 3.6	29 – 42	10 – 20
Embankment C	4	6.3	18 - 64	15 – 25
RSW 20 – Fill from subgrade to 5 m level	3	9.1	49	25 – 35
RSW 20 – Fill from 5 m level to top	3	9.1	90 ¹	
RSW 21	4	6.3	49	20 – 30
RSW 22	4	3.6	42	15 - 20

1. Cumulative settlement, i.e. 90 mm = 49 mm for fill to 5 m + 41 mm for fill to top of wall.

4.2 RECORDED GROUND SETTLEMENT

Figure 2 shows the predicted and recorded vertical settlement plotted against the height of fill placed at a settlement plate at the approximate centre of Embankment A during construction. Similar behaviour was recorded at other settlement plate locations around Embankment A. Consolidation is ongoing at the time of reporting.

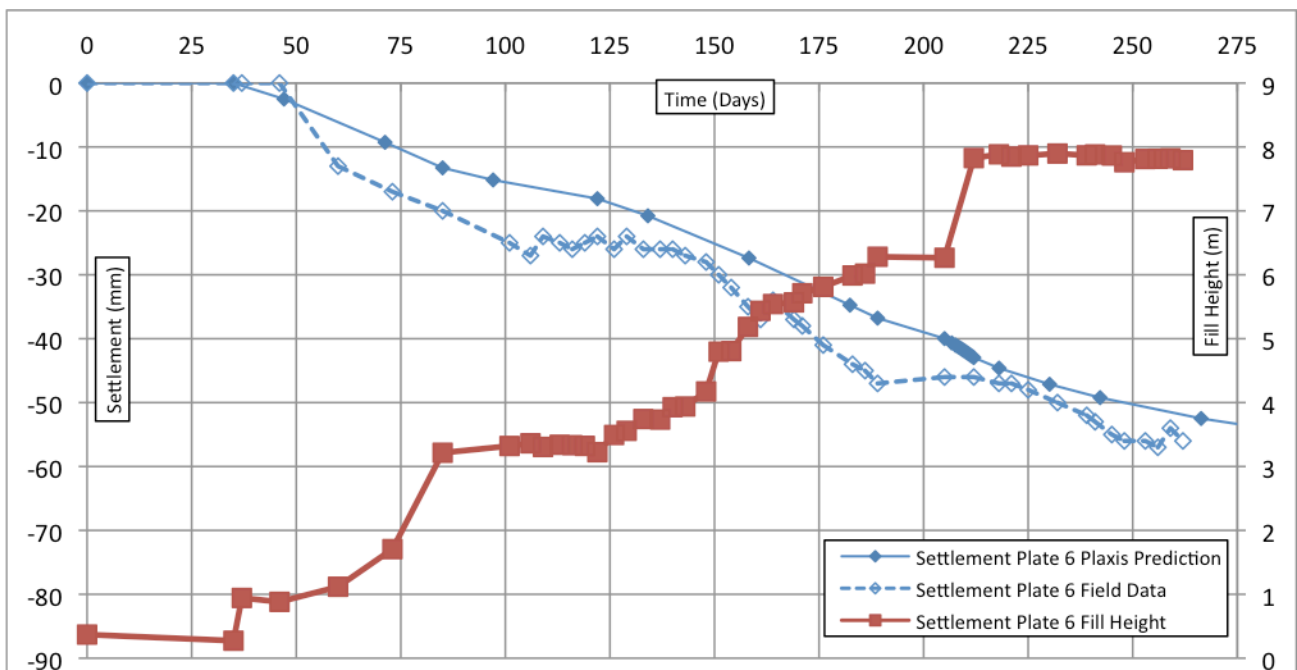


Figure 2: Predicted and observed settlement vs. fill height at Embankment A.

Figure 3 shows the predicted and recorded vertical settlement plotted against the height of fill placed at a settlement plate at the approximate centre of Embankment B during construction. Similar behaviour was recorded at other settlement plate locations around Embankment B. Consolidation is ongoing at the time of reporting.

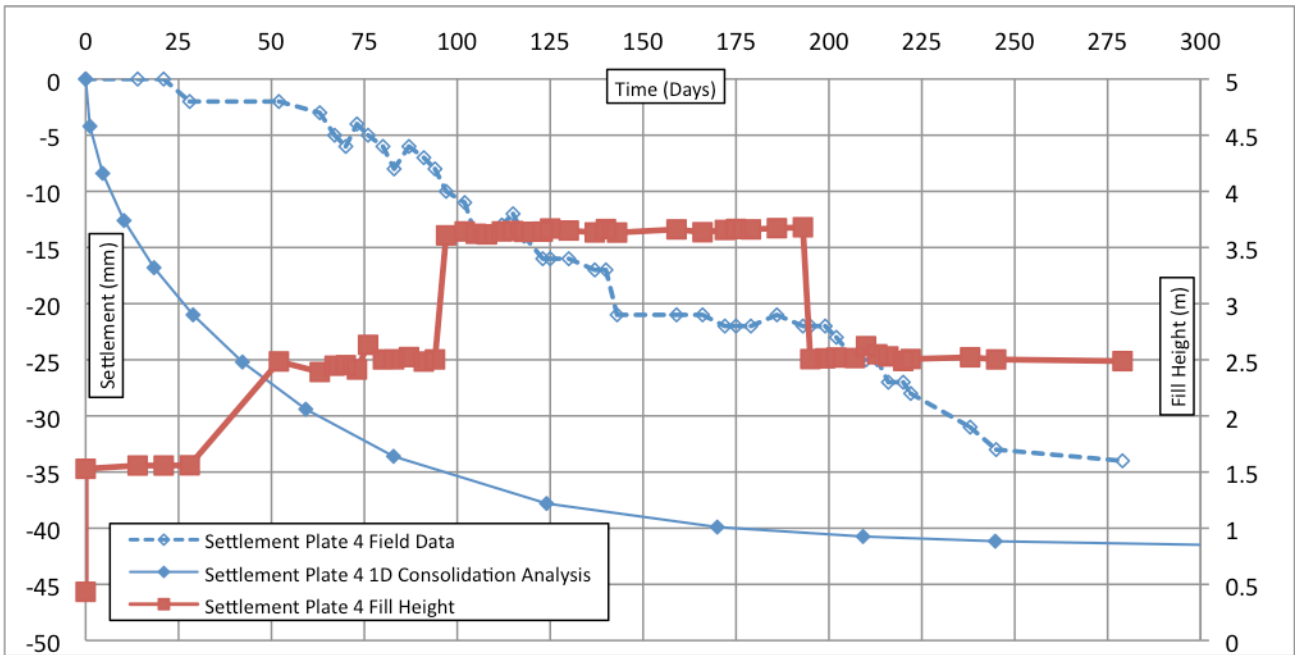


Figure 3: Predicted and observed settlement vs. fill height at Embankment B.

Figure 4 shows the predicted and recorded vertical settlement plotted against the height of fill placed at a settlement plate at the approximate centre of Embankment C during construction. Similar behaviour was recorded at other settlement plate locations around Embankment C. Consolidation is ongoing at the time of reporting.

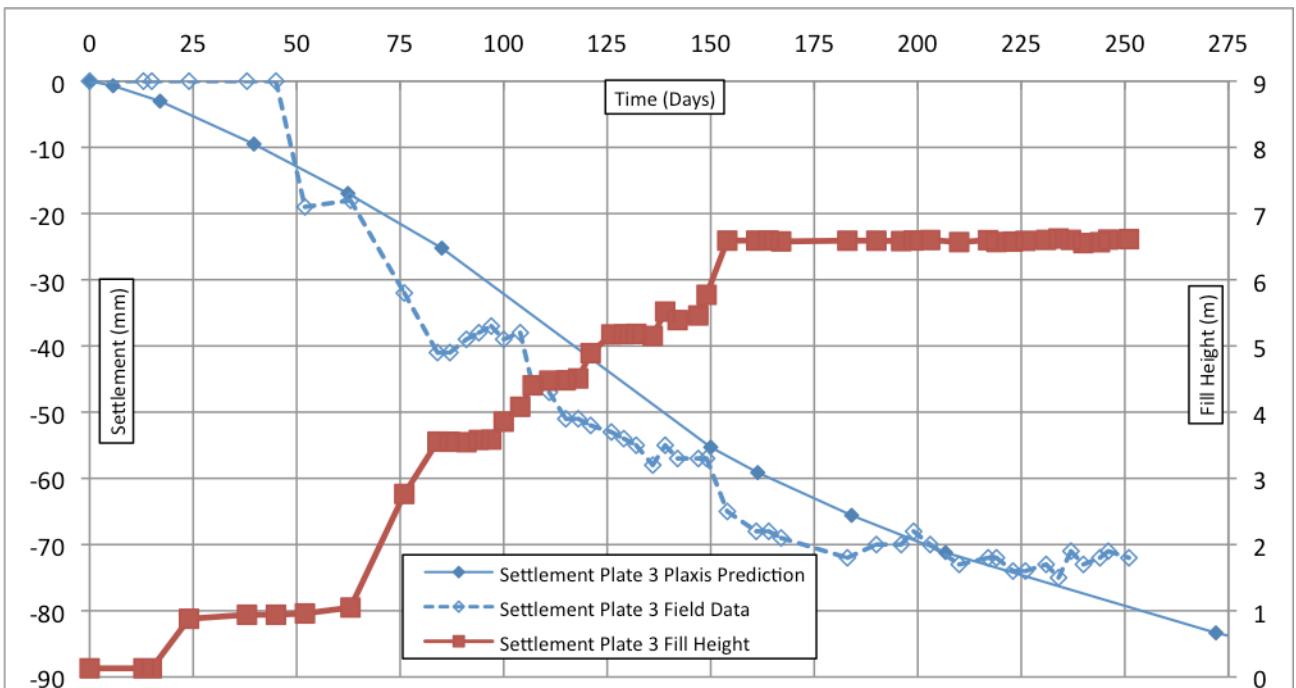


Figure 4: Predicted and observed settlement vs. fill height at Embankment C.

Figure 5 shows the predicted and recorded vertical settlement plotted against the height of fill placed at a settlement plate at the approximate centre of RSW 20 during construction. Consolidation is ongoing at the time of reporting.

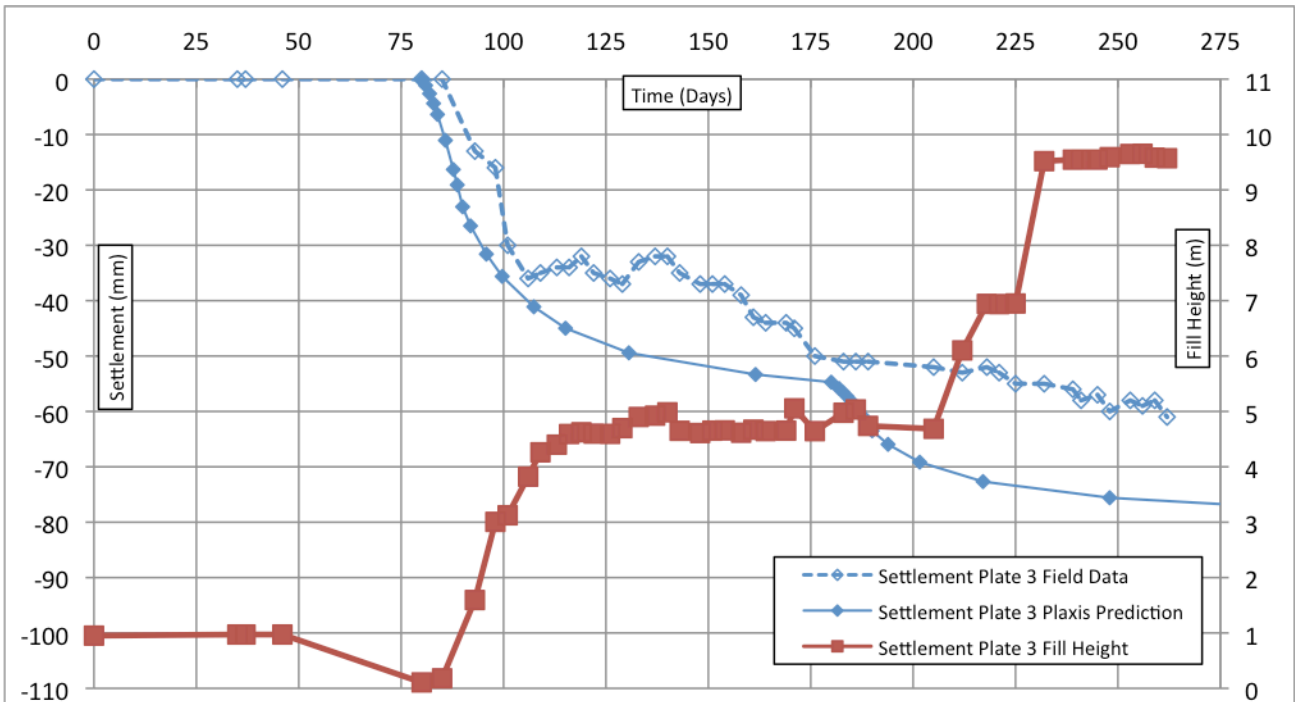


Figure 5: Predicted and observed settlement vs. Fill height at RSW 20

Figure 6 shows the predicted and recorded vertical settlement plotted against the height of fill placed at a settlement plate at the approximate centre of at RSW 21 during construction. Consolidation is ongoing at the time of reporting.

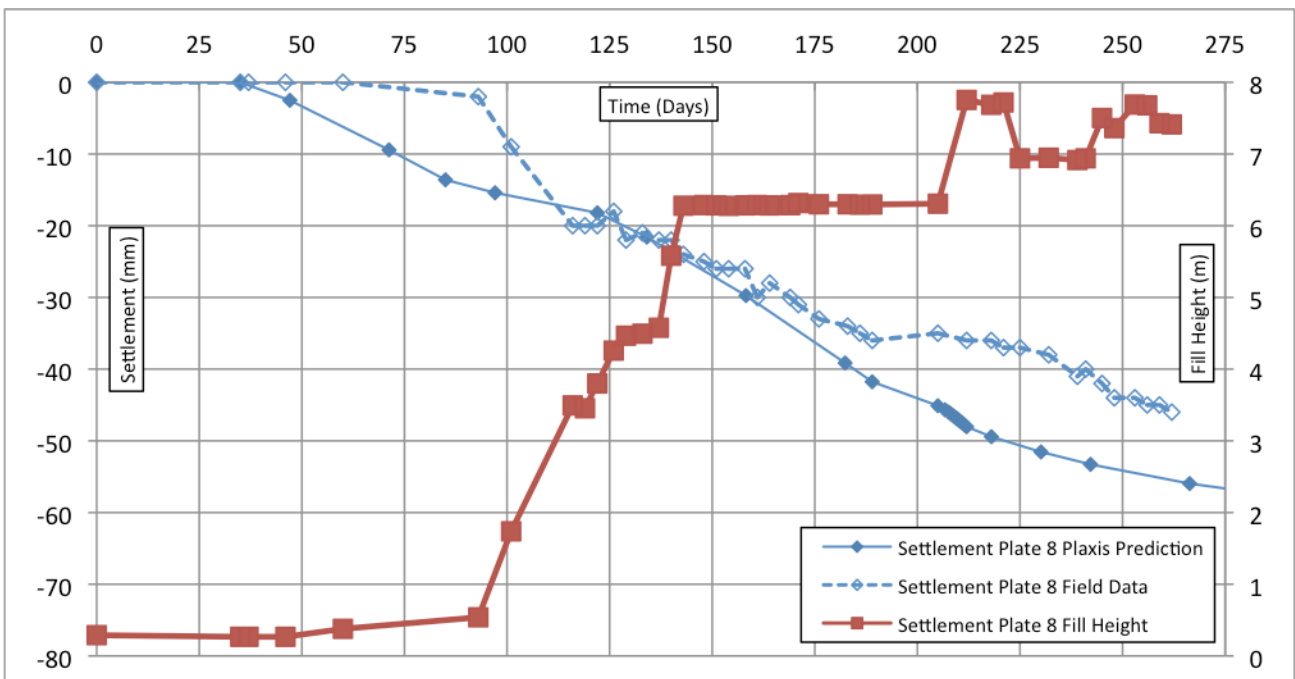


Figure 6: Predicted and observed settlement vs. fill height at RSW 21.

Figure 7 shows the predicted and recorded vertical settlement plotted against the height of fill placed at a settlement plate at the approximate centre of at RSW 22 during construction. Consolidation is ongoing at the time of reporting.

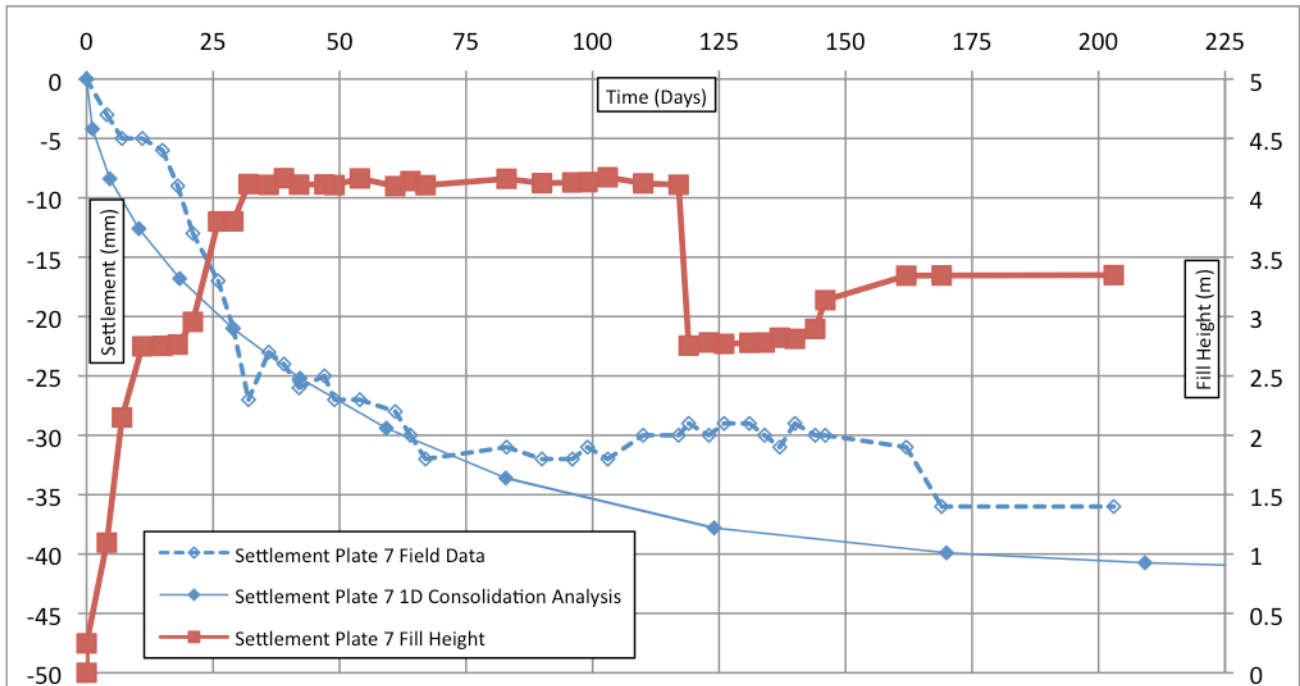


Figure 7: Predicted and observed settlement vs. fill height at RSW 22.

4.3 DISCUSSION OF SETTLEMENT RESULTS

Figure 3 to Figure 7 show a strong correlation between the predicted and observed settlement at each location of the southern flyover connection site. This correlation is generally consistent throughout the foundation settlement history. Table 5 compares the predicted and observed settlement at each location after 250 days.

Table 5: Comparison of predicted and observed settlement values after 250 days.

Location	Analysis Method	Predicted Settlement (mm)	Actual Settlement (mm)	Variation (%)
Embankment A	Plaxis 2D	50	56	+12
Embankment B	1D Consolidation Analysis	42	33	-22
Embankment C	Plaxis 2D	79	72	-9
RSW 20	Plaxis 2D	76	60	-21
RSW 21	Plaxis 2D	54	44	-19
RSW 22 ¹	1D Consolidation Analysis	41	36	-12

1. Reported settlement data for RSW 22 is after 200 days.

It can be seen that both the Plaxis 2D and 1D consolidation analyses provide consistent results for total settlement, typically overestimating the settlement at each location by 10-20%. The reasons for variations between predicted and recorded settlement include the following:

- i) To model the staged construction of fill embankments and reinforced soil walls in Plaxis 2D it is necessary to assume a rate of fill placement. In practice the rate of filling varies over time for a multitude of reasons that cannot be predicted during the design phase.
- ii) Settlement analyses are conducted on the basis of a ground profile that is inferred from site investigation data. In practice there is some divergence from the inferred ground stiffness due to localised variations in subsurface strata, imported fill material properties, and fill placement and compaction.

5 DISCUSSION OF INSTRUMENTATION TECHNIQUES

Several different types of instrumentation were installed throughout the southern flyover connection site. Settlement plate data ultimately proved the most useful in terms of monitoring foundation performance during construction for the following reasons:

- i) A significant amount of the inclinometer and magnetic extensometer data was compromised by construction traffic damage. This could be reduced in future projects through the use of high visibility secure fencing or

exclusion zones around each instrument and through promoting a greater level of awareness of instrumentation and its purpose amongst site based staff.

- ii) Vibrating wire piezometers reported little to no pore water pressure response. This is considered to be a function of the stiff to very stiff ground conditions.

Despite these challenges, sufficient data was gained from the instrumentation program to confirm the degree of primary consolidation and release all construction hold points on time.

6 CONCLUSIONS

Based on this case study, the following general conclusions can be drawn:

- i) When there is sufficient time in the construction schedule, preloading of fill structures is an effective way to accelerate the consolidation process and mitigate issues relating to excessive long term settlement.
- ii) When backed by quality site investigation data, both one dimensional consolidation and two dimensional Plaxis analyses provide robust estimations of long term settlement, typically within $\pm 20\%$ of recorded field values.
- iii) A sufficiently detailed instrumentation program can provide a wealth of data regarding ground deformations during and beyond construction. In order to reduce the likelihood of construction traffic damage, exclusion zones should be established and training should be conducted to ensure that site based staff are aware of the location and purpose of instrumentation. These practices aid in preserving the integrity of the recorded data and protecting the investment of the client.

7 ACKNOWLEDGEMENTS

The authors acknowledge Transport for NSW for permission to publish this paper and colleagues from Parsons Brinckerhoff, Macmahon Contractors, Bouygues Travaux Publics and MVM Rail for their collaboration during construction monitoring.

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