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Design and construction of an anchored soil nail wall close to movement sensitive structures

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Abstract

Legacy Way is a 4.6 km twin 12.4 m diameter road tunnel with approximately 3 km of surface connections to facilitate entering and exiting the tunnels. The new roadway bypasses Brisbane CBD to the west and will connect the Western Freeway at Toowong with the Inner City Bypass (ICB) at Kelvin Grove.

As part of the Eastern surface connection works, a 12 m high × 55 m long soil nail and anchored wall retaining a soil and weathered rock slope was constructed as abutment support to the extended Inner Northern Busway (INB) bridge and Brisbane Grammar School (BGS) pedestrian bridge. The new abutment wall was an integral element of an overall solution that required addition of spans to the existing bridges and removal of the original Reinforced Soil Structure (RSS) abutments.

The near vertical new abutment wall is located approximately 1 m in front of large diameter bored piles which support the extended bridge. Resistance of lateral pile loads and restriction of movement of the new bridge abutment were key constraints in the design.

Bridge extension and construction of the new abutment wall has created sufficient space for a large buried drainage structure to carry floodwater outflow from nearby playing fields to an open channel further east. The buried drainage structure runs parallel to the ICB and supports a shared (pedestrian/bikeway) user path.

This paper provides a commentary on the design approach adopted and construction issues encountered. Validation of the design through construction support and monitoring records is also summarised.

1 Introduction

Legacy Way is a 4.6 km twin 12.4 m diameter road tunnel with approximately 3 km of surface connections to facilitate entering and exiting the tunnels. The new road bypasses Brisbane CBD to the west and will connect the Western Freeway at Toowong with the Inner City Bypass (ICB) at Kelvin Grove.

The Western Connection comprises some 2 km of new roadway formed in trough and deep cut and cover structures and connects the Western Freeway to the western tunnel portal. The 12.4 m diameter parallel twin tunnels commence at the Toowong roundabout. The tunnels pass underneath sections of Auchenflower, Milton, Paddington and Red Hill before surfacing at Kelvin Grove, where the Eastern Connection (again trough and deep cut and cover structures) connect to the existing ICB.

Transcity, a joint venture between BMD Constructions, Ghella and Acciona Infrastructures, is responsible for the design, construction and initial operation of Legacy Way (Transcity, 2011).

Design of Legacy Way has been carried out by Transcity Design Alliance, an Alliance between Cardno, GHD and URS.

2 Proposed INB Bridge extension works

The Eastern Connection involves construction of two realigned eastbound lanes of the ICB at Kelvin Grove. These lanes are located under the Inner Northern Busway (INB) bridge, which spans the ICB. The northern abutment of the bridge was previously supported by a Reinforced Soil Structure (RSS) wall. To construct the additional lanes, the following construction sequence was undertaken:

1. Extension of the INB bridge using two additional spans supported on large (1.2 m) diameter bored piles. Piles and beams were constructed on the outside of the original road barriers, with a series of precast beams supporting the in-situ cast deck.
2. Demolish existing RSS wall once new structure was fully operational.
3. Construct anchored/soil nail wall to support northern abutment.
4. Construct culvert drainage structure and ventilation tunnel structure below the original RSS location.
5. Backfill and construct new ICB lanes and shared user path above buried structures.

Figure 1 Original site layout just prior to commencement of Legacy Way works

3 Anchored soil nail abutment wall

Extension of the INB and BGS Pedestrian bridges and removal of the RSS abutment fill was required to allow construction of buried drainage and tunnel ventilation structures as part of the Legacy Way works.

The buried drainage and tunnel ventilation structures pass, at depths of 12 and 14 m respectively, below the undersides of the INB Bridge and BGS Pedestrian Bridge decks.

Excavation, steepening and support of the soil and weathered rock slope were required to facilitate the buried structure works. The overall slope height was 12 m formed at a gradient of 1H:5V (79^o). The lower 3.5 m of excavation was to be backfilled after drainage structure construction, whilst the upper 8.5 m of the excavation formed a permanent slope cutting. A clear distance of less than 1 m was available between the top of the cut slope and the new bridge piles.

Due to the access constraints under the INB bridge structure and the need to maintain a fully operational INB roadway, piling was not possible. The only feasible option was a staged, top-down construction method. A major consideration for the abutment design was the need to resist lateral loads of 400 kN at the level of the bridge deck/ pile connection. In addition, as the abutment was immediately adjacent to the at-grade INB roadway, it was critical to limit outward and downward movement of the abutment wall. As such the solution for the slope cutting was a soil nail wall with structural shotcrete facing and with a single row of anchors, as close as practical to the underside of the bridge decks.

The soil nail abutment wall extends from Ch 0 to Ch 55 along control line MD12 (refer Figure 2). The permanent wall height varies from 0 m at Ch 3, rising to approximately 8.5 m at Ch 30 (below the INB Bridge) and reducing to approximately 6.0 m at Ch 43 (BGS footbridge) and 3.5 m at Ch 55, where it changes to a soldier pile wall.

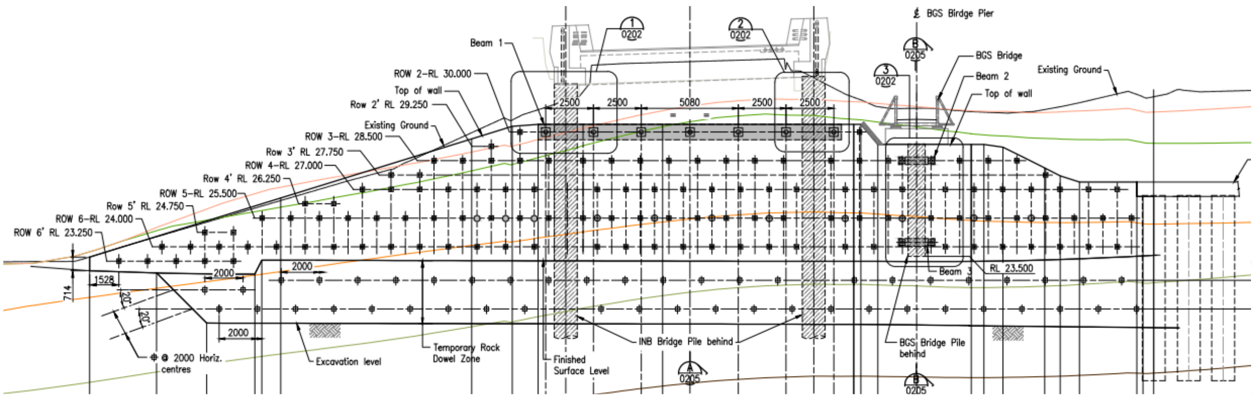

Figure 2 Soil nail wall elevation

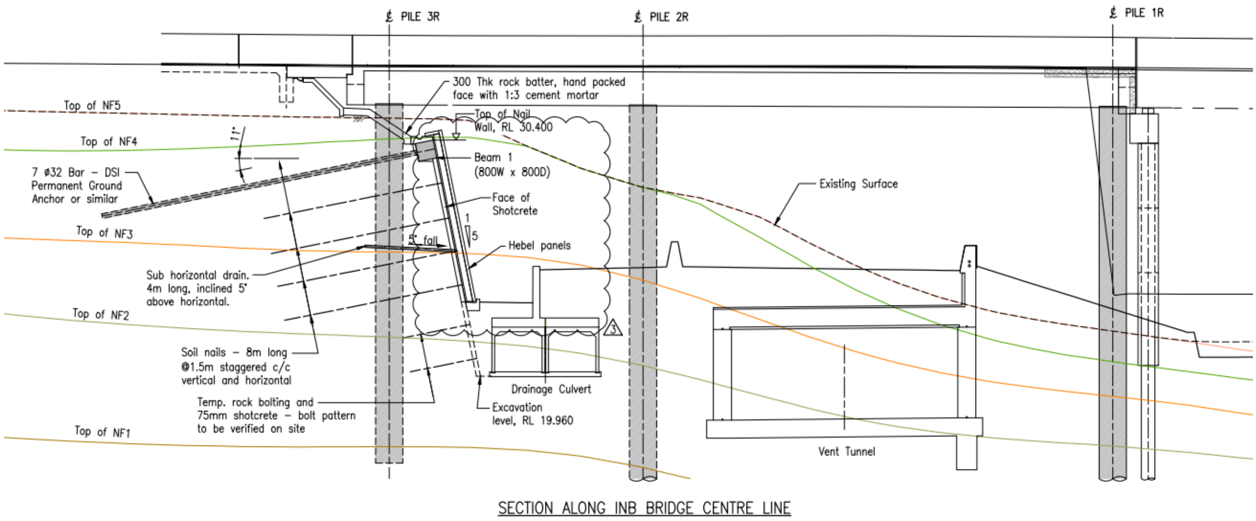

Figure 3 Design cross section at Ch30

4 Design approach

4.1 Design standards

The Performance Deed for the Legacy Way project required the anchored soil nail wall to be designed for a service life of 100 years.

The Performance Specification for the Legacy Way project generally required the works to be designed to Australian and Queensland Transport and Main Roads (TMR) standards. However, very limited design guidance was available in the various local standards, including Standards Australia (2002, 2004) and Department of Transport and Main Roads (2012).

As such, the design for the abutment wall below the INB bridge was therefore based on the US Federal Highways (2003) which provides a robust and comprehensive method of assessment of soil nail walls. Standards Australia (2002 and 2004) were adopted for the design of ground anchors.

4.2 Design ground model

The project area is underlain by the Neranleigh Fernvale (NF) beds, commonly referred to as the Brisbane Metamorphics, comprising phyllite, metagreywacke, arenite, quartz arenite, quartzite and spilite, occurring both as interbedded sequences and thick 'beds'. From various past projects, the NF beds are known for their high variability due to their often steeply dipping foliation, changing from fresh and slightly weathered to highly and completely weathered and back again, within metres of exposed surface.

Ground investigation records of the existing INB bridge and RSS wall could not be sourced, due to relocation of archive materials with Government Agencies. Further ground investigation at the proposed INB abutment wall was significantly constrained due to the elevated nature of the existing RSS wall and the inability to obtain temporary INB road closure.

A preliminary design ground model of the abutment area was therefore based on extrapolation of one borehole in the vicinity of the slope crest, along with mapping of adjacent weathered outcrops and assessment of original ground surface topography. The model was refined using geophysics (reflection and refraction surveys) around the original RSS wall. The refined ground model for the new INB abutment was part of overall three-dimensional ground models developed for the Legacy Way Eastern and Western Connections.

Based on the ground model, the cut batter slope at the new INB abutment was expected to expose thin engineered fill and residual soil, over mostly extremely weathered material to highly weathered material, with moderately then slightly weathered, medium to high strength rock over the lower one-third of excavated slope.

Based on collation of boreholes, geophysics and surface mapping for the Eastern Connection generally, the NF beds were divided into 5 units (NF1 being fresh to maintain consistency with tunnel notation). Table 1 summarises the design engineering units.

Table 1 Working description of NF Units around the INB abutment area

Unit	Typical Weathering Grade	Typical Strength (to Standards Australia, 1993)	Typical Mean Defect Spacing	Approximate Thickness
NF5	Residual to ext. weathered	–	–	<2 m
NF4	Highly weathered	Low to Medium	Very close to medium (<0.6 m)	5.0 m
NF3	Moderately weathered	Medium	Medium (0.2 to 0.6 m)	3.5 m
NF2	Slightly weathered	High	Medium to wide (0.6 to 2.0 m)	4.5 m
NF1	Fresh	High to very high	Wide to very wide (0.6 to >2.0 m)	–

4.3 Geotechnical design parameters

Geotechnical design parameters for soils were derived using in-situ and laboratory testing, together with published empirical and theoretical relationships (Hoek and Diederichs, 2006; Mesri and Shahien, 2003). Design parameters for rock units made use of geological mapping of a number of exposures along with laboratory testing and geophysics. The geotechnical parameters adopted in the abutment wall design are summarised in Tables 2 and 3.

Table 2 Soil geotechnical design parameters (adopted for INB abutment soil nail design)

Soil Unit	Unit Weight (kN/m^3)	Effective Cohesion, c' (kPa)	Effective Friction Angle, ϕ (°)	Undrained Shear Strength, S_u (kPa)	Undrained Modulus, Eu (MPa)	At-rest Pressure Coefficient, K_o	Drained Modulus, E' (MPa)
Eng. Fill	21	5	30	50	20	0.50	15
NF5	22	2	28	100	70	1.00	45

Table 3 NF rock geotechnical design parameters (adopted for INB abutment soil nail design)

Rock Unit	Unit Weight (kN/m^3)	Adopted Intact UCS (MPa)	GSI	Material Constant (m_i)	Disturbance Factor (D)	Modulus Ratio, MR	Rock Mass Deformation Modulus, E_{rm} (MPa)
NF4	25	10	35	7	0.5	450	257
NF3	26	20	45	8	0.5	500	1,059
NF2	26	40	55	9	0.5	550	4,449
NF1	27	50	65	10	0.5	600	10,577

4.4 Loadings considered

In accordance with the performance specification for the project, various loading conditions were considered for the abutment wall design. The loads are summarised in Table 4 along with the adopted intensities.

Table 4 Description and intensities of loadings considered

Load Type	Description	Intensity
Dead load	Concrete self-weight	25 kN/m^3
	Soil weight	As per Tables 2 and 3
Imposed load	Traffic load (SM 1600 design loads and HLP 400 vehicles in accordance with Standards Australia, 2004)	25 kPa
Surcharge	During construction	20 kN/m^2
	Long-term (future development)	20 kN/m^2
Earth pressures	At-rest conditions from soil structure interaction	As per Tables 2 and 3
Flood loading	ULS (10,000 year ARI)	RL varies 25.16 – 24.20
	SLS (100 year ARI)	RL varies 23.90 – 23.44
Anchor-waler beam load	INB abutment	400 kN per Pile (ULS)
	BGS footbridge pier	70 kN (ULS)
Water pressure	Hydrostatic (24.2 mRL for Q100 and 25.5 mRL for Q10,000)	10 kN/m^3
Earthquake	Lateral acceleration coefficient (in accordance with Standards Australia, 2007)	0.06 g

4.5 Lateral loading and allowable abutment movements

A major consideration for the abutment wall design was the need to resist lateral loads of 400 kN at the level of the bridge deck/pile connection. The loads were determined from structural analysis of the extended bridge spans and soil-structure interaction of the two supporting abutment piles.

In addition, as the abutment was immediately adjacent to and below the at-grade INB roadway, it was critical to limit outward and downward movement of the abutment wall to avoid unacceptable movement at pavement level.

From assessment of the effects of outward wall movement on surface movement behind the wall, a limit of 5 mm was adopted as the allowable abutment wall movement to stay within allowable road pavement settlement limits. The movement criteria was a governing factor in the wall design, requiring a significantly higher level of slope support than what would be required simply to achieve a Factor of Safety of 1.5 while disregarding movement.

5 Geotechnical design

5.1 Analysed sections

The abutment wall extends from Ch 0 to Ch 55 along control line MD12 (refer Figure 2). The permanent wall height varies from 0m (at CH3) increasing to a maximum of 8.5 m (between CH24 and CH40), then reducing to 6.0 m at CH43 and 3.5 m at CH55, where it changes to a soldier pile wall.

The design ground model showed slightly varying geological units along the wall, with NF3 just below the wall toe between CH0 and CH20, and then rising above the wall toe over the remainder of the wall (CH20 to CH55). Figures 2 and 3 show the soil nail wall elevation and a cross section at CH30 respectively. Three critical cross sections were selected for analysis as follows:

Table 5 Analysed cross sections

Chainage	Reason for Selection
20	Moderate slope height (6 m) and NF3 geology below wall toe
30	Maximum slope height (8.5 m permanent; 12 m temporary) and considers load from INB piles
43	Moderate slope height (6 m) and considers load from BGS footbridge piles

5.2 Design approach

Design parameters were derived for each rock unit using generalised Hoek–Brown criterion. Related stiffness derivations were implemented in RocLab (Rocscience, 2011) for use in design modelling.

5.2.1 Finite element modelling (FEM)

Finite element stress-deformation modelling was undertaken using Phase^2 (Rocscience, 2011) to estimate movements of the proposed abutment wall design. The level of slope support was iterated until outward movement at the slope crest (and elsewhere within the slope face) was within the adopted 5 mm allowable abutment wall movement. Iterations included number of anchors, anchor prestress, soil nail spacing/length/etc. and shotcrete thickness. Table 6 shows the estimated deflections from the FEM analysis for the adopted slope support.

The FEM analysis formed the basis of both the slope support as well as the monitoring plan, with proposed trigger levels for alarm and action.

Table 6 Results of FEM settlement analysis

Chainage	Estimated Displacement at Slope Crest (mm)
20	4 mm
30	4 mm
43	1 mm

5.2.2 Slope stability

In accordance with the FHWA, the three cross sections were analysed taking into consideration Ultimate Limit State (external, internal and facing failures) as well as Serviceability Limit State (deflection). Global stability analysis was carried out in Slide version 3.0 (Rocscience, 2011) for undrained (during construction) and drained (permanent) conditions and under seismic loading. As an independent check, the slope stability Factor of Safety was externally verified using the SlopeW (Geostudio, 2011) software package, adopting the Morgenstern and Price method.

Pore pressure coefficient R_u was incorporated in the ultimate limit state stability analysis of the permanent works to take account of Q10,000 flood levels, which were up to one third the height of the permanent soil nail wall.

As expected for governing deflection design, all Factors of Safety were well above required global stability limits required by the performance deed (1.5 for permanent stability, 1.3 for temporary stability and 1.1 under seismic loading).

5.3 Design slope support

The developed permanent cut slope design support comprised an upper row of post-tensioned anchors (at 2.5 m horizontal centres) underlain by four rows of soil nails installed at 1.5 m centres horizontally and vertically.

The anchors comprised high yield steel bar rather than conventional strands to facilitate ease of construction in the confined working area below the bridge deck. The anchors were post tensioned to mitigate slope movement and to provide high level lateral support to the existing bridge piles.

Soil nails were designed at 11° horizontal declination, i.e. perpendicular to the face, and up to 8 m in length below the INB Bridge. Either side of the INB bridge, nail lengths were reduced to 6 and 4 m as the height of wall decreased. Soil nails comprised double corrosion protected (galvanised and sheathed) N24 bars at 1.5 m spacing horizontally and vertically.

The wall facing comprised 200 mm thick shotcrete facing with SL81 reinforcement. Drainage was achieved via a combination of proprietary drainage geosynthetic behind the shotcrete and sub-horizontal drains. The shotcrete facing was overlain by a precast panel architectural finish.

The FEM analysis showed that temporary excavation in the lower part of the profile would have little effect on slope movements or overall stability. Support for the lower slope was therefore based on mapping and kinematic assessment based on exposed conditions.

5.4 Construction sequencing

Construction sequencing was instrumental to the design of the abutment wall, particularly below bridge locations where the design was reliant on anchors and soil nails to limit movement. The construction sequence for the anchored soil nail wall was:

1. Excavate to 1 m below 1st row of anchors/soil nails.
2. Place 1st layer of shotcrete.
3. Drill anchor/soil nail hole, place bar and grout.
4. Install mesh reinforcement.
5. Place anchor/soil nail face plate and lock off nut. Post-tension anchor prior to lock off of nut.
6. Place 2nd/final layer of shotcrete.
7. Repeat until final excavation. Excavation not permitted until nail grout achieves 80% design strength.

5.5 Specification and verification testing

Roads and Maritime Services (2009 and 2011) were the proposed construction standards for soil nails and ground anchors for this discrete element of the works. These provided a comprehensive assessment of suitability and acceptance testing, as well as a well-defined materials specification to ensure the 100 year design life.

6 Construction

Construction commenced in early 2012 and was undertaken top-down, enabling excavation mapping and movement monitoring in each staged cut prior to installation of support or further excavation.

Figure 4 Soil nail wall during construction

6.1 Verification of the ground model

As part of the construction oversight process, verification of the ground model was provided by a geotechnical engineer from the Construction Design Support Team (CDST) who mapped the exposed slope face after each staged excavation.

As shown in Figure 5, weathering, particularly at the western end of the wall was deeper than the design ground model. Re-analysis was undertaken and the design soil nail levels adjusted to achieve slope support at a slightly higher level, without the need to increase the overall design support.

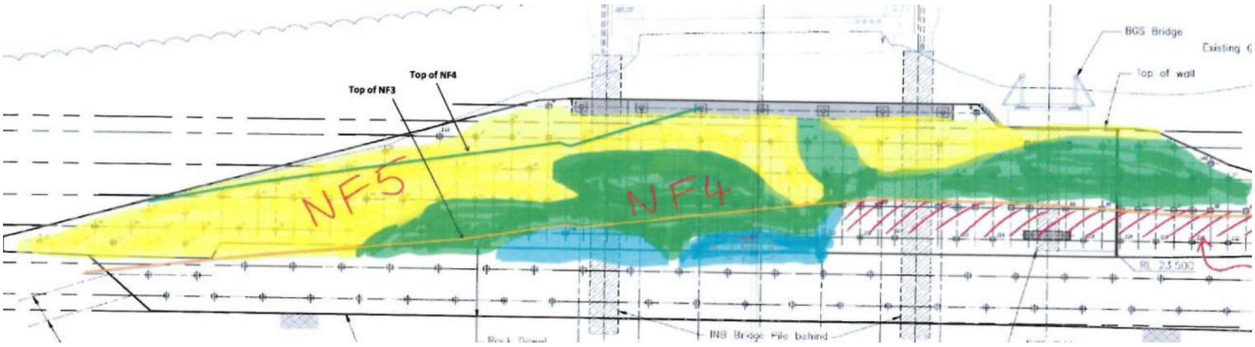

Figure 5 Marked up elevation of soil nail wall showing actual mapped NF unit boundaries (courtesy of Mr Darren Allen)

Temporary excavation for the drainage structure (lowermost 3.5 m) was mostly in moderately and slightly weathered rock. Geological mapping, in conjunction with kinematic analysis using proprietary software DIPs and SWEDGE (Rocscience, 2011), were used to confirm the need, or otherwise for rock bolting of potential rock block failures. A total of two spot bolts were used for the temporary excavation.

Suitability and acceptance testing in accordance with Roads and Maritime Services (2009 and 2011) showed that the long term soil nail and anchor design bond strengths were achieved and that working nails satisfied design load requirements.

Figure 6 Completed anchored soil nail abutment wall (background) with the ventilation tunnel and drainage structure in the foreground

7 Monitoring

7.1 Design monitoring plan

A monitoring plan, incorporating survey prisms and vibration gauges was developed as part of the design to verify the responses predicted by geotechnical analyses. Predictions of the FEM analysis formed the basis of the proposed trigger levels for alarm and action. The geotechnical monitoring specified is shown in Table 7.

Table 7 Proposed movement monitoring to verify design

Phase	Proposed Monitoring	Frequency
Construction	Survey prisms at approx. 4 m c/c horizontally and vertically	Daily during excavation and until readings have stabilised. Weekly thereafter
Operation	Survey marks as for construction	Quarterly

The locations of the prisms are shown in Figure 7.

The monitoring plan included the following for ground movement:

- Design value: 5 mm
- Alarm value: 60% of the design value.
- Yellow trigger warning: 85% of the design value and requiring action to limit further movement.
- Red action trigger value: 110% of the design value, requiring a cessation of works.

7.2 Construction monitoring

Significant variation was recorded both in and out of the slope as a result of survey tolerances (typically ± 2 mm, but up to ± 5 mm). Rolling averages over several weeks were therefore assessed as well as day–to–day variations. Table 8 presents an extract of monitoring results for point ECM18 where movement approached yellow trigger level on 24 July 2012. However, no action was taken at the time because the rolling average and daily variation values were within allowable movement limits. Subsequent readings proved that the movement was attributable to survey tolerances.

Table 8 Extract of monitoring point ECM18 (top row) between 10/07/2012 and 16/08/2012

Date	Movement – North (mm)	Daily Variation (mm)	Rolling Average (mm)
10/07/2012	-1	0	-1
17/07/2012	-3	-2	-2
24/07/2012	-5	-2	-3
26/07/2012	0	5	-3
31/07/2012	-2	-2	-3
2/08/2012	-3	-1	-3
6/08/2012	-4	-1	-2
8/08/2012	-2	2	-3
16/08/2012	-2	0	-3

Maximum movements recorded as of 15 April 2013 are summarised in Table 9. Average readings to date are within the adopted design value.

Table 9 Wall movement monitoring results as of 15 April 2013

Location	Horizontal Movement Out of Wall	Vertical Settlement at Top of Wall
Top of wall	2 mm (9 mm)*	2 mm (7 mm)*
Bottom of wall	2 mm (7 mm)*	1 mm (4 mm)*

*Values reported are average values with maximum values in brackets

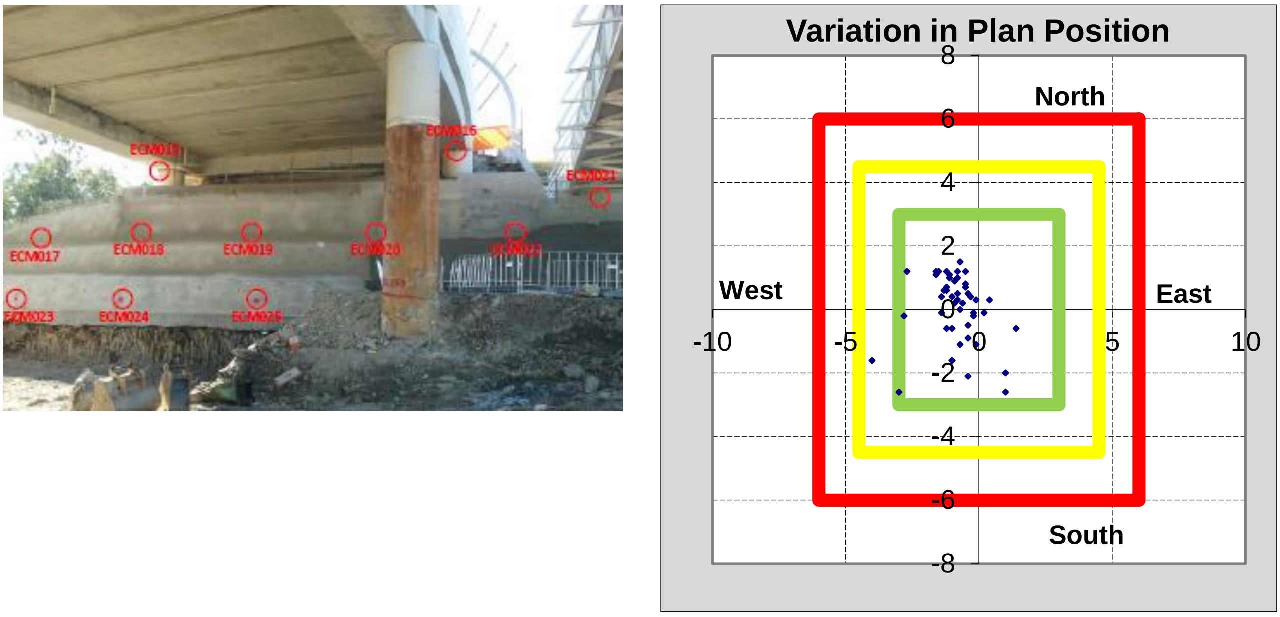

Figure 7 Movement monitoring during construction (results for ECM18, top row)

8 Conclusions

The paper has outlined the key design aspects of an innovative anchored soil nail abutment wall immediately adjacent to the movement sensitive INB bridge structure.

The new abutment wall is an integral element of an overall innovative solution that required addition of spans to existing bridges and removal of the original Reinforced Soil Structure (RSS) abutments.

Abutment wall design was governed by allowable movements, rather than limit state stability of the reinforced wall. Finite element analysis formed the basis for design slope support. The design ground model was verified during construction and the interaction between designer and ground mapping teams allowed for construction issues to be dealt with promptly.

The design analysis was verified by a comprehensive movement monitoring programme. Recorded wall movements to date are within the range estimated with no damage to the INB/BGS bridge structured recorded.

With expected completion of the Legacy Way project in mid-2015, the BGS anchored soil nail wall is an example of a well-planned design and construct process that has led to the successful completion of major infrastructure works close to movement sensitive structures.

Figure 8 Anchored soil nail wall – before and after

Acknowledgement

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