PERFORMANCE OF ANCHORED PILE WALLS FOR A DEEP CUT

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ABSTRACT

The Banora Point Upgrade Project in NSW Australia comprised key features including two interchanges, a 300m long viaduct, a large cutting through Sexton Hill with an associated 100m wide Land Bridge and retaining walls reaching 490m in length and up to 22m high. The geology at Sexton Hill comprises a volcanic succession of variably weathered basalt flows and agglomerate deposits overlying Mesozoic sedimentary rocks at a depth of approximately 48m beneath the crest of the hill. Two types of retention system were selected to support the Sexton Hill cutting and were required to suit the ground conditions, satisfy the performance criteria and consider the narrow project corridor. These included cantilever/anchored piled retaining walls and soil nail walls. The paper focuses on the design approach and numerical modelling techniques and compares the predicted and actual performance of the anchored pile wall system based on the monitoring data collected during and post construction. Numerical analyses using computer programs PLAXIS, Phase^2 and WALLAP have been undertaken to model the behaviour of the anchored pile wall including wall deflections and anchor loads. In addition, the analyses sought to determine whether neighbouring residential properties were adversely affected by the construction of the cutting. The predicted values of anchor loads and wall deflections have been compared with actual performance and are presented. A back analysis has been performed focusing on a portion of the retaining wall section where the predicted and actual performance differed significantly. The results from the back analysis revealed that the deflections displayed by the piles are particularly sensitive to small changes in stiffness of high strength rock just above the final excavation level.

1 INTRODUCTION

The Banora Point Upgrade, a part of the Pacific Highway upgrade along the east coast of Australia, provides a high standard 2.5km length of dual carriageway motorway from the Tweed River in the south to the Tweed Heads Bypass in the north, improving safety, reducing highway congestion and providing a safer local road network. The project has been constructed for the Roads and Maritime Services (RMS) by an Alliance comprising Abigroup Contractors, Seymour Whyte, SMEC Australia and RMS. The main structures that required civil design input included two interchanges, a 300m long viaduct, a large cutting through Sexton Hill with an associated 100m wide Land Bridge and retaining structures up to 22m high, 4 cuts ranging in height from 2m to 6m, embankment fills, 21 retaining walls, noise walls, and pavements. See Figure 1 for the route plan and key structures.

One of the main features of the project was the realignment of the Pacific Highway through Sexton Hill (Ch 84500 to Ch 84900) which required the formation and retention of a 22m deep cutting with vertical cut faces. The cut is situated in a narrow corridor between the existing Pacific Highway to the west and a well-established residential area to the east. Since the existing Pacific Highway is a major road connecting New South Wales to Queensland it was a requirement of the project that it had to remain in operation during the entire construction period. The main retention system comprised anchored pile walls which are the main focus of this paper.

This paper presents an overview of design and construction of Sexton Hill Cutting, the geology and geotechnical characteristics of the ground, design considerations including potential risks identified, design philosophy and methodology, numerical modelling techniques, issues and solutions associated with adverse ground conditions encountered during excavation of the cut, and actual performance of critical areas of the cutting.

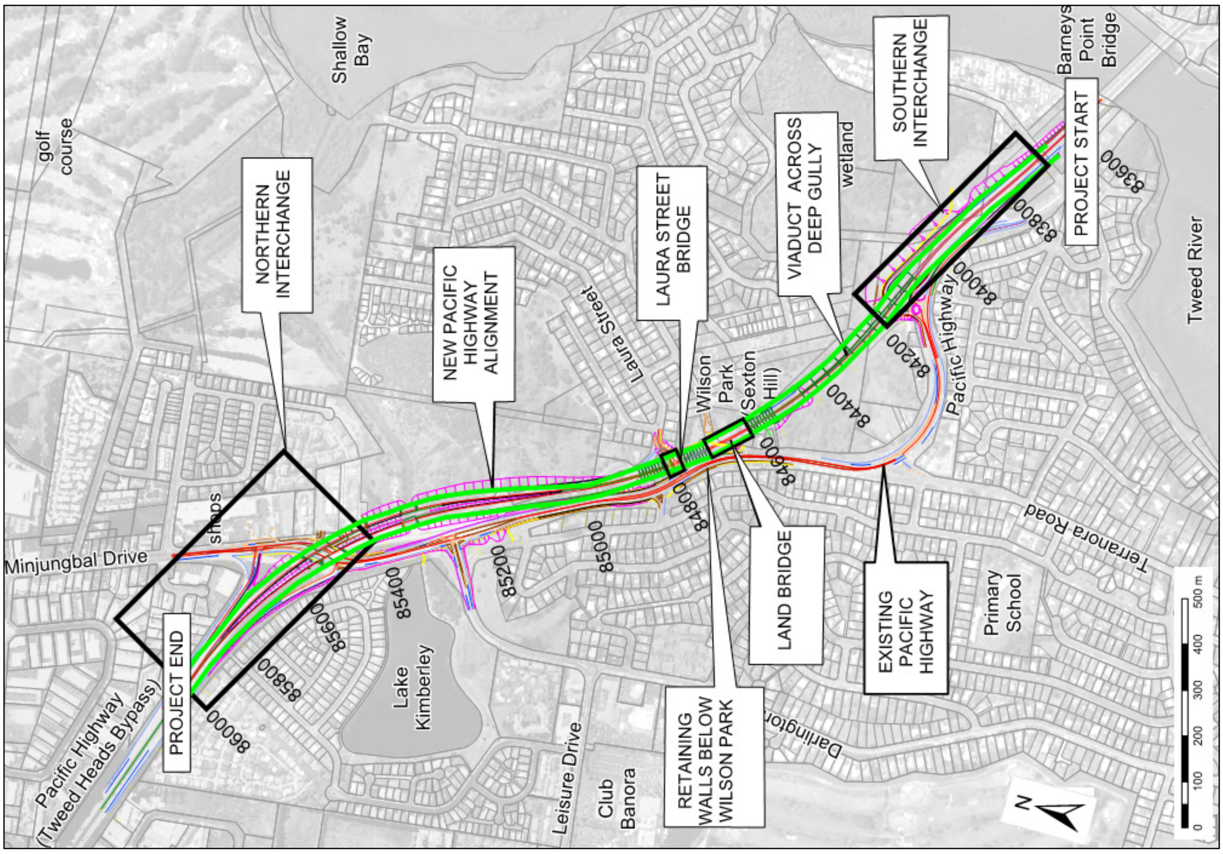

Figure 1: Project location plan

2 GEOLOGICAL CHARACTERISATION

2.1 GEOLOGY

The Banora Point region is located on the eastern edge of the Clarence-Moreton Basin which is a large intra-cratonic sedimentary basin extending from north of Brisbane in Queensland to south of Grafton in New South Wales. The sediments are continental in origin and the basin is cut by major structural features, being intruded by extensive plutonic suites (Branagan and Packham, 2000).

Early Tertiary rifting and sea floor spreading off the eastern Australian continental margin resulted in heating and uplift of the eastern extent of the basin. Late Tertiary intrusives and volcanics resulted in regional heating and deformation of the eastern parts. Extensive basaltic volcanism occurred across large areas including the Banora Point region. These eruptions involved basaltic lava flows and explosive pyroclastic ejection of material comprising agglomerate and tuff. Due to long periods between eruptions, weathering of the surface of the basalt lava flows over time resulted in the development of palaeosol soil horizons between successive flows.

The elevated Sexton Hill area is underlain by Tertiary volcanics comprising layers of variably weathered basalt lava flow and weathered agglomerate, with Palaeozoic metasediments occurring at depth.

2.2 SUBSURFACE SITE CONDITIONS

The subsoil and bedrock encountered within the depth of the proposed excavations at Sexton Hill are described below:

- **Residual Soil** – Comprising very stiff to hard clays and silty clays of variable thickness (up to 6m).

- **Extremely Weathered Basalt** – Basalt corestones in an extremely weathered basalt matrix which has soil strength properties and can be remoulded to silty clay. The corestones range from cobble size to subrounded boulders up to 2m in diameter.
- **Extremely Weathered Agglomerate** – A very low strength rock to very stiff soil which is typically friable or can be remoulded to silty clay. The unit is often weathered to the extent that original rock textures are difficult to distinguish.
- **Upper Basalt** – A relatively thin, but laterally extensive layer of slightly weathered to fresh basalt that varies from about 1m to 4m in thickness and the rock strength ranges from high to extremely high.
- **Weathered Agglomerate** – An agglomerate sequence up to about 9m thick that displays a highly variable weathering pattern ranging from extremely weathered through to fresh rock over very short distances.
- **Lower Basalt** – An extensive basalt flow comprising slightly weathered to fresh basalt of unconfirmed thickness. It is inferred that the flow thickness could be of the order of 20m to 25m based on ground investigations to the north and south of Sexton Hill.

A geological long-section and cross-section are reproduced in Figures 2 and 3 to depict the complex geological sequence at Sexton Hill. The figures, particularly the cross-section, highlight the rapid changes in geology over relatively short distances.

2.3 GROUNDWATER

Groundwater at Sexton Hill occurs in under-unconfined to semi-confined conditions. The water table is a subdued version of topography. Local recharge is via rainfall with discharge at springs along the break of slope and to lower lying surrounds towards the Tweed River, Kimberley Canal, and Alexandra Lagoon. Monitoring of groundwater levels prior to construction confirmed local recharge. The topographic high of Sexton Hill forms a surface and groundwater divide, with water moving away from the high point (recharge) to the discharge points.

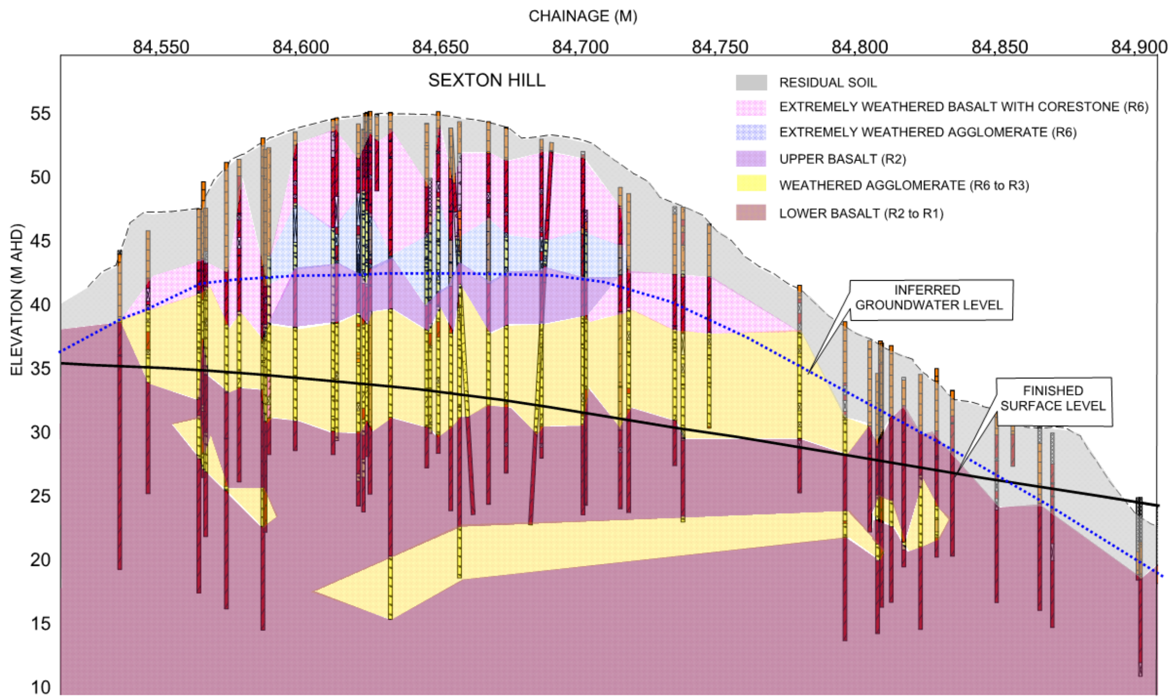

Figure 2: Geological long section

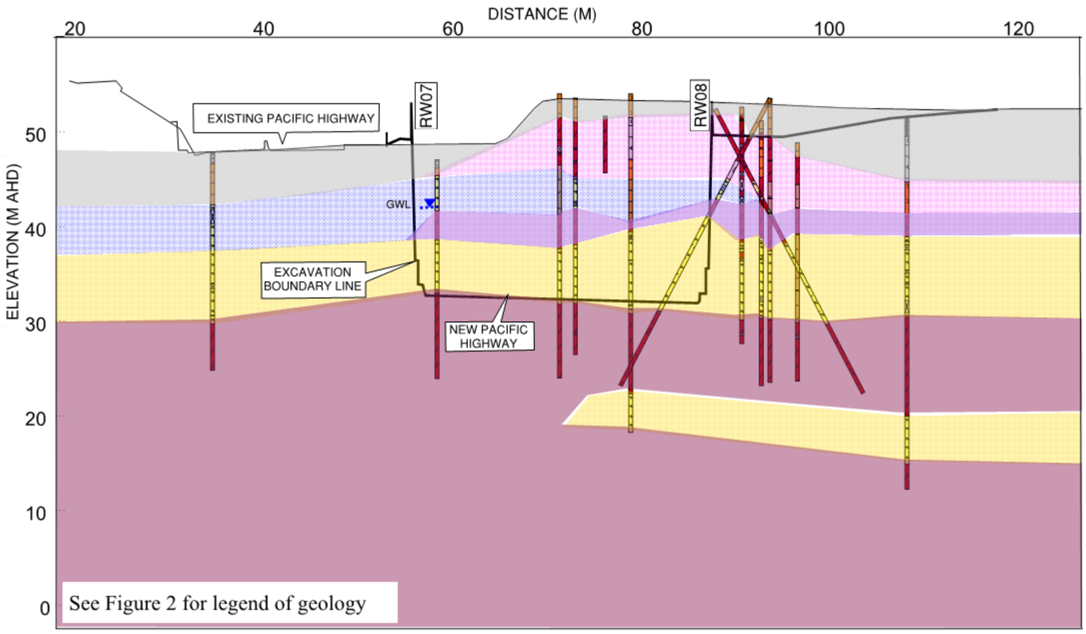

Figure 3: Typical cross-section geological profile (Ch84680)

2.4 SOIL CLASSIFICATION SYSTEM

The engineering soils at the site were classified using the Unified Soil Classification (USC) nomenclature. The residual soils and extremely weathered basalt horizons present in the cutting classified as CL or CH and of stiff (St) to very stiff (VSt) consistency.

2.5 ROCK CLASSIFICATION SYSTEM

The engineering behaviour of a rock mass depends on in-situ stresses, rock mass strength and rock mass modulus. The modulus of a rock mass depends on the properties of the defects (i.e. defect spacing, orientation, roughness, aperture etc.) and infill materials. Due to the heterogeneous, anisotropic, and discontinuous micro to macro scale nature of rocks there is no unified system to classify rocks into discrete engineering materials, as is available for soils.

In the cutting, the various rock units have different geological origin and strength characteristics at various scales. For geotechnical design purposes, all the rock units were classified based on the engineering properties governing the rock mass behaviour i.e. intact rock strength and defect characteristics (spacing, aperture, infilling, wall roughness and weathering) regardless of the geological origin.

Table 1: Rock classification system

Rock Mass Class	Sub-Class	UCS (MPa)	Defect Spacing (mm)	Rock Strength
R1	a	>70	>300	Extremely High (EH)
R2	a	>70	60-300	Very High (VH) – High (H)
	b	20-70	>300	
R3	a	20-70	60-300	High (H) – Medium (M)
	b	6-20	>300	

Rock Mass Class	Sub-Class	UCS (MPa)	Defect Spacing (mm)	Rock Strength
R4	a	>20	<60	Medium (M) – Low (L)
	b	6-20	60-300	
	c	2-6	>300	
R5	a	6-20	<60	Very low (VL) – Low (L)
	b	2-6	60-300	
	c	<2	>300	
R6	a	2-6	<60	Very Low (VL) – Extremely Low (EL)
	b	<2	<300	
	c	<0.3	N/A	
UCS=Unconfined Compressive Strength Sub-class c represents Residual Soil.				

2.6 GEOTECHNICAL DESIGN PARAMETERS

Soil parameters have been selected based on all available data obtained from laboratory and insitu testing. The data has been statistically analysed in order to establish representative design values and were checked against published data and relevant past experience.

Rock parameters include both intact rock and rock mass parameters. Intact rock parameters comprise UCS, elastic modulus and unit weights available from field and laboratory testing. The data was statistically analysed in order to obtain most representative values for each rock class. The Geological Strength Index (GSI) for each rock mass unit has been estimated using the procedures outlined in Cai et. al. (2004). RocLab 1.0 software was used to determine rock mass strength parameters, based on the generalized Hoek-Brown failure criterion (Hoek et al. 2002). The derived rock mass parameters for each rock class were further analysed on the basis of past experience and literature in order to develop representative parameters.

The soil hardening approach (Vermeer & Brinkgreve 1998) was used to model the residual soil and two different strength failure criteria, i.e. generalized Hoek Brown and equivalent Mohr-Coulomb, were used to simulate the rock. The key geotechnical parameters for soil and rock are summarized in Table 2.

Table 2: Geotechnical design parameters

Soil/Rock Type	γ (KN/m^3)	E' (MPa)	υ	c' (kPa)	ϕ' (deg.)	UCS (MPa)	GSI	Mi	D	K_0
RS (St Clay)	18	2N	0.3	10	26	-	-	-	-	1.0
RS (VSt Clay)	18	2N	0.3	10	28	-	-	-	-	1.0
R6	20	100	0.3	50	30	-	-	-	-	1.0
R5	20	180	0.3	50	32	2	40	19	0.0	1.0
R4	21	250	0.2	100	33	5	45	19	0.7	1.0
R3	22	400	0.2	150	35	11	50	19	0.7	1.0
R2	25	1000	0.2	200	42	45	55	25	0.7	1.0
R1	26	6000	0.2	500	50	80	65	25	0.7	1.0
RS=Residual Soil, γ = Bulk Unit Weight, N= SPT 'N' value, E'=Elastic Modulus, St= Stiff, VSt= Very Stiff, υ=Poisson Ratio, c'= Drained Cohesion ϕ'= Drained Friction Angle, GSI=Geotechnical Strength Index, Mi=Material Constant, D=Disturbance Factor, K_0=Coefficient of Lateral Earth/Rock Pressure.										

3 DESIGN CONSIDERATIONS

3.1 GENERAL

The Sexton Hill cut comprises two retaining walls (RW07 and RW08) and two bridge structures (Land Bridge and Laura Street Bridge). The Land Bridge is a single 34.2m span bridge over the main alignment at the Sexton Hill cut and has an overall width of 75m, providing a vertical clearance of 18m. The bridge forms part of the Wilson Park precinct and supports a grassed field for pedestrian use. Laura Street Bridge is a single span structure with an overall span length of approximately 36m. The minimum vertical clearance of 7m has been provided between the main alignment and the underside of the bridge. The width of the bridge is approximately 19m. Retaining walls RW07 and RW08 are approximately 490m in length and retain a maximum height of 22m. A plan of the Sexton Hill Cutting is presented in Figure 4 and shows the locations of the various structures along the project alignment.

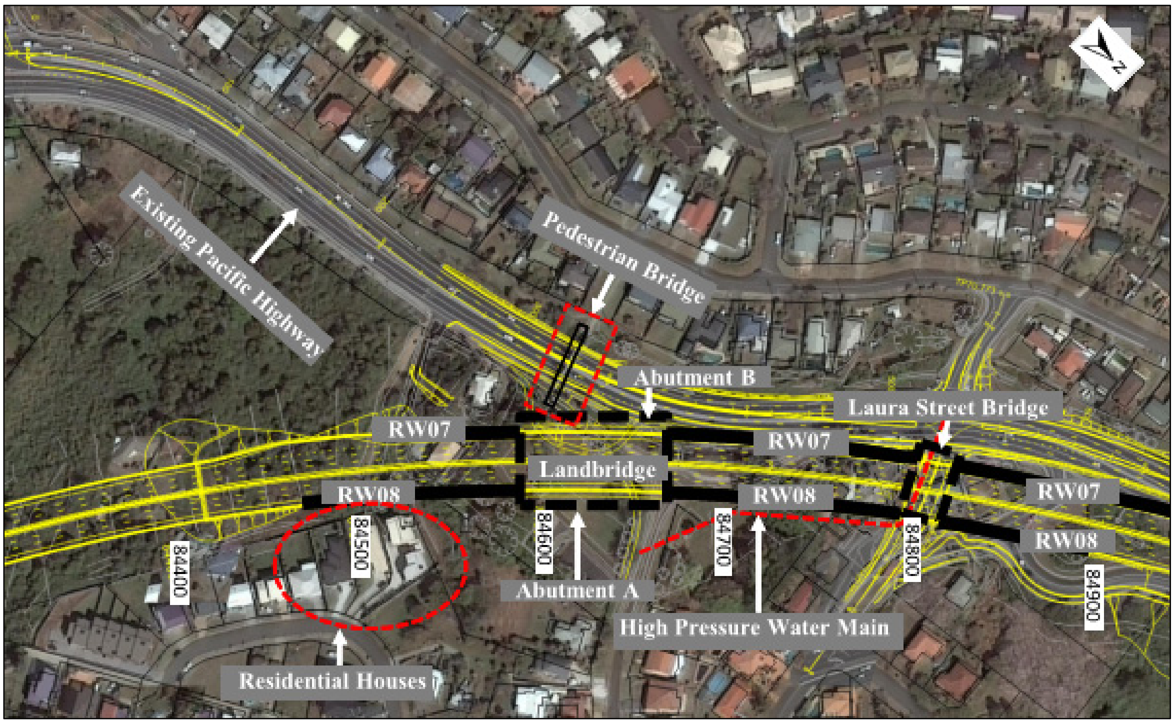

Figure 4: Plan of Sexton Hill Cutting

3.2 SELECTION OF RETENTION SYSTEMS

At the design stage, consideration was given to various retention options and involved identification of advantages and disadvantages of each one including constructability and associated risks, cost effectiveness and relative impacts on neighbouring structures. The key infrastructure that was affected by the proposed retention systems included:

- **Existing Pacific Highway** running parallel to retaining wall RW07. On the western side of the Land Bridge, the plan position of RW07 falls inside the road shoulder of the existing highway;
- **Live high pressure water main** that supplies water to the surrounding residences, located 8m behind and parallel to the rear of RW08;
- **Two story residences** located as close as 1m to the rear of retaining wall RW08; and
- **Existing pedestrian bridge** located near the Land Bridge.

Two types of retaining wall systems were selected to support the cutting as follows:

- **Rigid support system** – cantilever and anchored pile walls, in areas where minimal deformation of the retaining system was permissible to limit the effects on adjacent structures, or where a considerable depth of excavation was required and in areas associated with bridge structures.
- **Flexible support system** – soil nails with reinforced shotcrete facing in areas where there was adequate clearance from adjacent structures and large deformation was permissible.

4 RETENTION SYSTEM

4.1 ADOPTED RETENTION SYSTEM

Working with the Alliance Contractors, three different retention systems were adopted to support the Sexton Hill Cutting. These were:

- Cantilever/anchored pile walls;
- Laura Street Bridge (propped pile wall);
- Soil nail walls.

The selected wall systems along the cutting are presented in Figure 5 and Table 3.

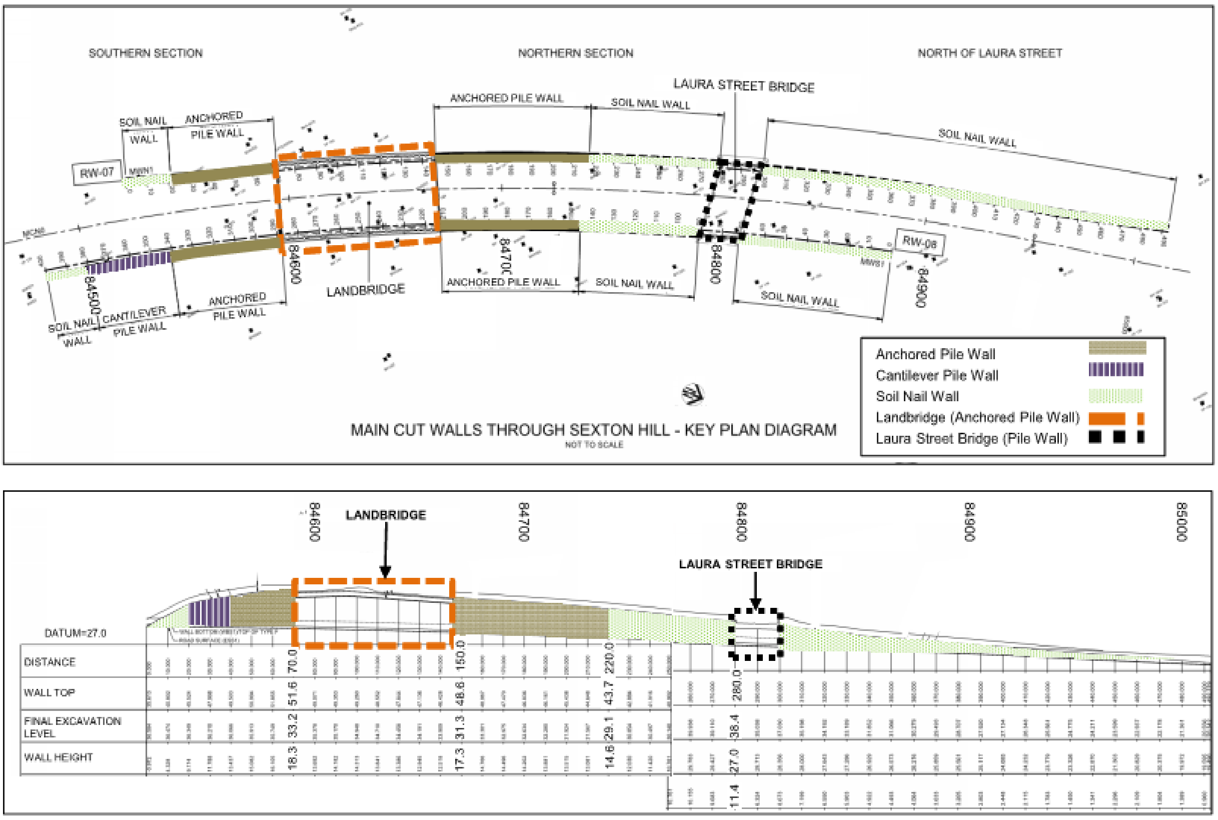

Figure 5: Plan and typical section of retention system

Table 3: Summary of wall types and locations

Wall	Chainages	Wall Height (m)	Wall Type	Remarks
RW07	84525 – 84545	~13	Soil Nail Wall	9m to 14m long soil nails
	84545 – 84730	~15 to ~21	Anchored Pile Wall	1.2m dia @ 2.5m to 3m c/c, 2 to 3 rows of anchors
	84730 – 84790	~15	Soil Nail Wall	12m to 14m long soil nails
	84790 – 84810	~12	Propped Wall	1.2m dia @1.3m c/c
	84810 – 84900	~12	Soil Nail Wall	5.5m to 15m long soil nails
RW08	84480 – 84500	~7	Soil Nail Wall	5m long soil nails
	84500 - 84540	~13	Cantilever Pile Wall	1.2m dia @ 1.5m to 3m c/c
	84540 - 84730	~18 to ~22	Anchored Pile Wall	1.2m dia @ 3m c/c, 2 to 3 rows of anchors
	84730 – 84790	~18	Soil Nail Wall	7m to 14m long soil nails
	84790 – 84810	~11	Propped Wall	1.2m dia @1.3m c/c
	84810 – 84880	~11	Soil Nail Wall	5.5m to 12m long soil nails

4.2 CANTILEVER / ANCHORED PILE WALLS

Cantilever/anchored pile walls, comprising free cantilever cast in-situ piles (1.2m diameter) with centre spacings of 1.5m or 3.0m and cast in-situ piles (1.2m diameter) with centre spacings of 2.5m or 3.0m supported by two or three layers of ground anchors, were adopted as retaining walls on either side of the Land Bridge. To the south of the Land Bridge the retained heights were typically greater than 13m, and to the north they were typically greater than 18 m. The Land Bridge itself was also supported by anchored pile walls. Piles were socketed a minimum of 2.0m into the R2 Basalt. Between adjacent piles, shotcrete was applied to provide support to the rock and soil.

4.3 GROUND ANCHORS

The basic components of a ground anchor include 1) anchorage; 2) free length (non-bonded); and 3) fixed length (bonded). The fixed length has to be located behind the critical potential failure surface. The location of potential failure surface behind the pile wall was evaluated to determine the suitable free length and fixed length of individual ground anchors. The failure surface was assumed at 45 degrees from the base of the excavation to the top of existing ground on the retained side, based on the kinematic assessment of the RAAX imaging data. The free lengths were typically extended a minimum distance of 1.5m beyond the critical potential failure surface. This assumption is made in accordance with BS 8081.

Ground anchors and stress bars were designed in such manner that the design geotechnical capacity of anchor was greater than the structural capacity of the anchors. Ground anchors and stress bars were generally anchored into R2 Basalt or stiffer material as determined in the geotechnical design. Pre-stressing is a design requirement for safety to ensure that end anchorage has been successfully developed and that the anchors are ready to accept load. The ground anchors were preloaded to 50kN per strand, a total of 350kN to 600kN per ground anchor. The stress bars were preloaded to 300kN.

The adopted the ground anchors comprised the following:

- **Upper anchors** – 12/15.2mm multi-strand tendon within a 175mm grouted diameter hole located within the headstock and positioned at the centreline of piles. The anchors were inclined at 30 degrees to 45 degrees to horizontal, the shallower inclination used where the upper basalt layer was found to be persistent and therefore reliable to support the anchor.
- **Middle anchor** – The middle anchors were similar in type (12/15.2mm multi-strand tendon) to the top anchors and were installed at the intermediate level through the pile. A guided sleeve was provided so that anchor could be installed through the pile avoiding the pile reinforcement.

- **Lower anchors** –7/15.2mm multi-strand tendon within a 150mm grouted diameter hole, or alternatively two 36mm diameter high strength stress bars within a 150mm grouted diameter hole. The stress bars were located between the piles through a waler beam, the top of which was located 3.65m above the finished road surface level. Two stress bars were provided for each pile, inclined at 45 degrees to the horizontal.

5 DESIGN METHODOLOGY

5.1 DESIGN CRITERIA AND CODES

The geotechnical designs of the anchored and cantilever pile walls were carried out using the Working Stress (WS) approach to satisfy the required minimum factor of safety. Serviceability of individual structural elements has been assessed based on the Finite Element Analysis. The design of the ground anchors was undertaken in accordance with BS8081 (FOS method). A summary of the design methodology and criteria is presented in Table 4.

Table 4: Summary of design methodology for cantilever/anchored pile walls

Retention System	Design Life	Element	Design Specification/Requirement	Construction Specification	Software	Design Approach	Serviceability Criteria	Targeted FOS (WS)				Ground Model	
								Bond stress	Global			Soil + R6	Rock (R1 to R5)
									Temporary	Permanent	Seismic		
Cantilever / Anchored Pile Wall	100 years	Ground Anchor	BS8081	BS8081 RMS R56	PLAXIS, Phase^2, WALLAP	WS	100mm (Lateral Deflection)	3.0	1.3	1.5	1.5	Hardening Soil or Mohr-Coulomb	Generalized Hoek Brown or Mohr-Coulomb
		Bored Pile	SWTC	AS2159									
WS=Working Stress ; ULS=Ultimate Limit State; SWTC=Scope of Works and Technical Criteria (Client's Brief)													

5.2 DESIGN APPROACH

A series of finite element analyses were performed using geotechnical numerical analysis software PLAXIS (Version 9) and Phase^2 (Version 7) from Rocscience. The PLAXIS software was used for the geotechnical design for most of representative sections with the exception of three design sections at the northern side of the Land Bridge, where the designs were developed using Phase^2. The results of the finite element analyses were compared with the limit state method using the program WALLAP (Version 5.1). Two dimensional plane strain models were developed with staged construction. Anchored piles were analysed for short term conditions (i.e. during construction using undrained parameters for residual soil and drained parameters for rock), long term condition (i.e. during service using drained parameters for both residual soil and rock) and under seismic conditions (i.e. during seismic event using undrained parameters for residual soil and drained parameters for rock).

The maximum shear forces, bending moments and deflections in the cantilever and anchored pile walls were obtained from the finite element analyses and the design action effects were used in the design of the piles and ground anchors. Pile reinforcement designs were carried out in accordance with Australian Standard AS 5100. The required ground anchorage lengths were determined using the results of ultimate pull out tests prior to the main phase of excavation. The sections below provide a brief description of design approach adopted using different numerical software and WALLAP.

PLAXIS

The computer program, PLAXIS was used to analyse the excavation of the cut supported by cantilever/anchored pile wall. PLAXIS is based on the Finite Element Method (FEM) used in the analysis of deformation and stability of soil structures. The software comprises extensive 2D FEM meshes, analysing static elastoplastic deformation, advanced soil models, stability analysis, consolidation, safety analysis and steady-state groundwater flow. The residual soil was modelled using the "Hardening Soil Model" (Vermeer & Brinkgreve 1998) and rock was modelled using Mohr-Coulomb strength parameters.

Phi-c reduction analyses in PLAXIS was undertaken to assess the global factor of safety of the anchored pile walls under three different load cases (i.e. short term, long term and seismic). Interface properties between soil/rock and piles used for the PLAXIS analyses are summarised in Table 5.

Table 5: Interface properties used in PLAXIS and Phase^2

Description		PLAXIS	Phase^2			
			c'	ϕ' (deg.)	Normal Stiffness kn (MPa/m)	Shear Stiffness ks (MPa/m)
Interface properties	RS	0.67	0	17	5000	500
	R5/R6	0.67	33	20	5000	500
	R3/R4	0.67	100	33	10000	1000
	R1/R2	0.67	200	42	10000	1000

Phase^2

Phase^2 is a 2D elasto-plastic finite element stress analysis program that was used for the study of the excavation in soil and rock. The soil and rock mass units were modelled using continuum elements and were assumed to be homogeneous, isotropic and elasto-plastic. The Generalized Hoek-Brown failure criteria were used for the rock units and the Mohr-Coulomb criteria were used for soil units.

The global factor of safety of the anchored pile supported excavation was also computed for three different load cases (i.e. short term, long term and seismic) using the strength reduction method. Similar to PLAXIS, strength parameters of the soil and rock units were reduced progressively until a global minimum reduction factor was reached. Interface properties used for the Phase^2 analyses are summarised in Table 5.

WALLAP

WALLAP software allows limit equilibrium analysis and bending moment and displacement analysis of cantilever or propped / anchored retaining walls. As an additional check on the PLAXIS and Phase^2 results, WALLAP was used to confirm broad agreement between the two methods. Similar to the finite element approach, staged construction was simulated in the WALLAP analyses.

SENSITIVITY ANALYSIS

Sensitivity analysis is part of the design development for the retention system to ensure that the design has considered impacts associated with variability of ground conditions. Sensitivity checks included:

- **Variations of elevation of top of lower basalt**. These analyses were developed to ensure adequate pile reinforcement length and pile capacity.
- **Stiffnesses of the lower basalt layer**. This analysis was carried out by varying rock stiffness of R2 to R1 of the lower basalt in the numerical analyses to verify the effect of rock stiffness on the structural loads on the piles.
- **Mohr-Coulomb and Hoek-Brown parameters**. This analysis was undertaken to verify the sensitivity of the use of Mohr-Coulomb and Hoek-Brown Parameters under the adopted in-situ stress condition. As the Hardening Soil model was not feasible in Phase2, for comparison purposes residual soils and all rock classes were modelled using the Mohr-Coulomb criteria in PLAXIS whilst all residual soils including R6

and all other rock classes were modelled using the Mohr-Coulomb and Hoek- Brown failure criteria respectively in Phase^2.

5.3 CRITICAL ISSUES

Deep excavations in complicated geological conditions, located in close proximity to residences and an operational highway were the key considerations of the various design development stages. The stages included feasibility, primary and detailed design stages. Additional detailed ground investigations were specified to address gaps in the available data identified during the initial design stage including areas where anchors were to be installed. The following critical aspects of the design were identified;

- The concept design of the anchored pile wall comprised cast in-situ piles supported by anchors at four different elevations. Some of the anchors were to be installed in the highly weathered agglomerate layer. The design was optimised by reducing the number of anchors to three which assisted in maximizing the rate of excavation. The three rows of anchors therefore had to carry higher loadings; however the agglomerate was considered to be an unreliable anchorage horizon. Significant variations in rock strength (highly to moderately weathered) over short distances between adjacent boreholes provided insufficient confidence in the actual quality of rock in the anchorage zone.
- The required fixed anchor lengths in the agglomerate horizon exceeded those permissible in design codes such as BS8081 and AS4678. Therefore the anchorage zone was targeted at higher strength rock found in the upper basalt and lower basalt horizons.
- Finite element analyses pointed to significant contrasts in stiffness at the interfaces between the basalt (both upper and lower) and agglomerate layers. This resulted in significant structural forces developing in the retaining piles. Sensitivity analyses revealed that the stiffness of the lower basalt significantly influenced loads imposed on the pile. Higher structural loads were observed when modelling the lower basalt as “R1” quality rock than “R2” quality rock (see Figure 6). Therefore, the lower basalt layer was generally modelled as rock class “R1”.
- The high structural loads required the piles to have significant reinforcement. The ground investigation revealed that the top elevation of the lower basalt was irregular. This variation had to be considered during the detailing of the arrangement of reinforcement cages in the piles so that adjustments could be made easily on site during pile installation.

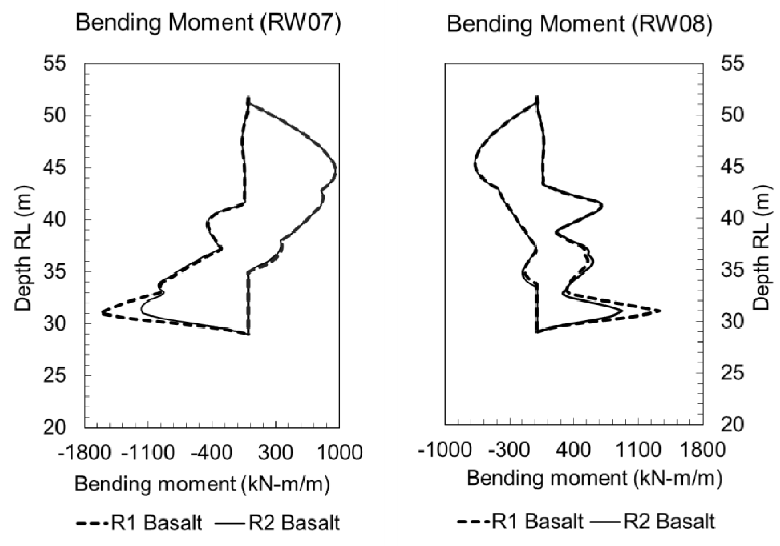

Figure 6: Comparison of bending moment envelops between R1 and R2

5.4 BOND STRESS

Prior to installation of anchors, a series of trial bond tests were conducted to verify the design bond stress of rock assumed in the initial design. The trial bond tests included a total of 47 pull-out tests in various soil and weathered agglomerate sequences and a total of 7 trial bond tests in R1/ R2 quality rock. The test results and adopted bond stresses for the design of the soil nails and ground anchors are summarized in Table 6.

Table 6: Bond stresses

Soil/Rock Type	Range of Pull-Out Test Results (KPa)	Adopted Ultimate Bond Stress (KPa)
RS	69-280	60
R6	180-400	180
R5	>400	300
R4	570-540	450
R3	>400	750
R2	>4000	2000
R1		4000
RS= Residual Soil		

Ultimate load tests in the R1/R2 basalt layer have not achieved pull-out failuare when the test load has been increased to a maximum allowable tensile strength of 80% of the strands which is equivalent to an ultimate bond stress of approximately 5500KPa. However, the ultimate bond stress was limited to 4000KPa in accordance with BS8081.

6 CONSTRUCTION PERFORMANCE

A comprehensive instrumentation and monitoring system was specified and implemented to monitor the performance of the retaining systems, particularly during construction. The monitoring system comprised the following:

- **Load cells** installed on all anchors at critical sections to monitor ground anchor loads;
- **Inclinometers** installed within selected piles to monitor horizontal movements of cantilever/anchored pile walls.
- **Optical prisms** installed on the vertical faces of the retaining walls to monitor wall movements.
- **Settlement markers** installed along the road, utilities and next to the residences behind the retaining walls to monitor ground settlements.

The plan of instrumentation along the cantilever/anchored pile walls is depicted in Figure 7.

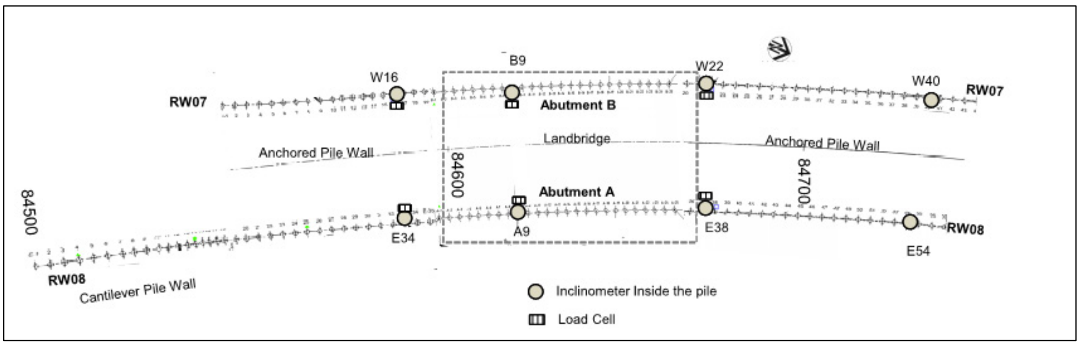

Figure 7: Instrumentation and monitoring layout

The calculated wall deflections and anchor loads from the finite element analyses have been compared with the monitoring data. The monitoring data from the construction phase is considered in this paper and therefore, design values used in the comparison are based on the short term case.

Comparisons of the results indicate that actual wall deflections were generally less than those predicted in the design (see Figure 8). The ground anchor loads (between prediction and measurement) were in reasonable agreement with the exception of the retaining wall (pile nos. A9 and B9) in the area of the Land Bridge (see Table 7). As a result of this difference, a back analysis was undertaken for this section of the cutting.

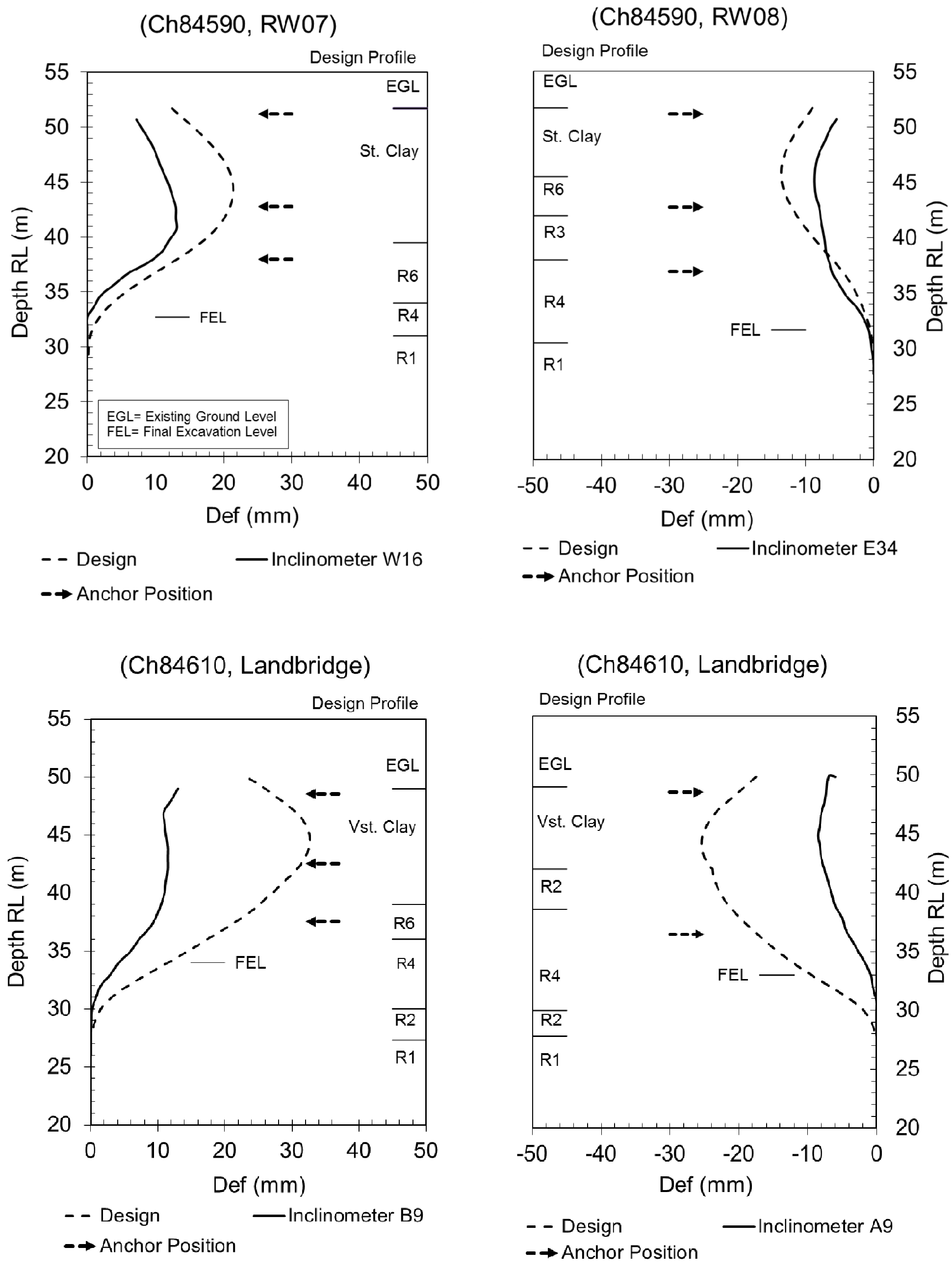

Figure 8a: Comparison of wall deflections between design and monitoring Data

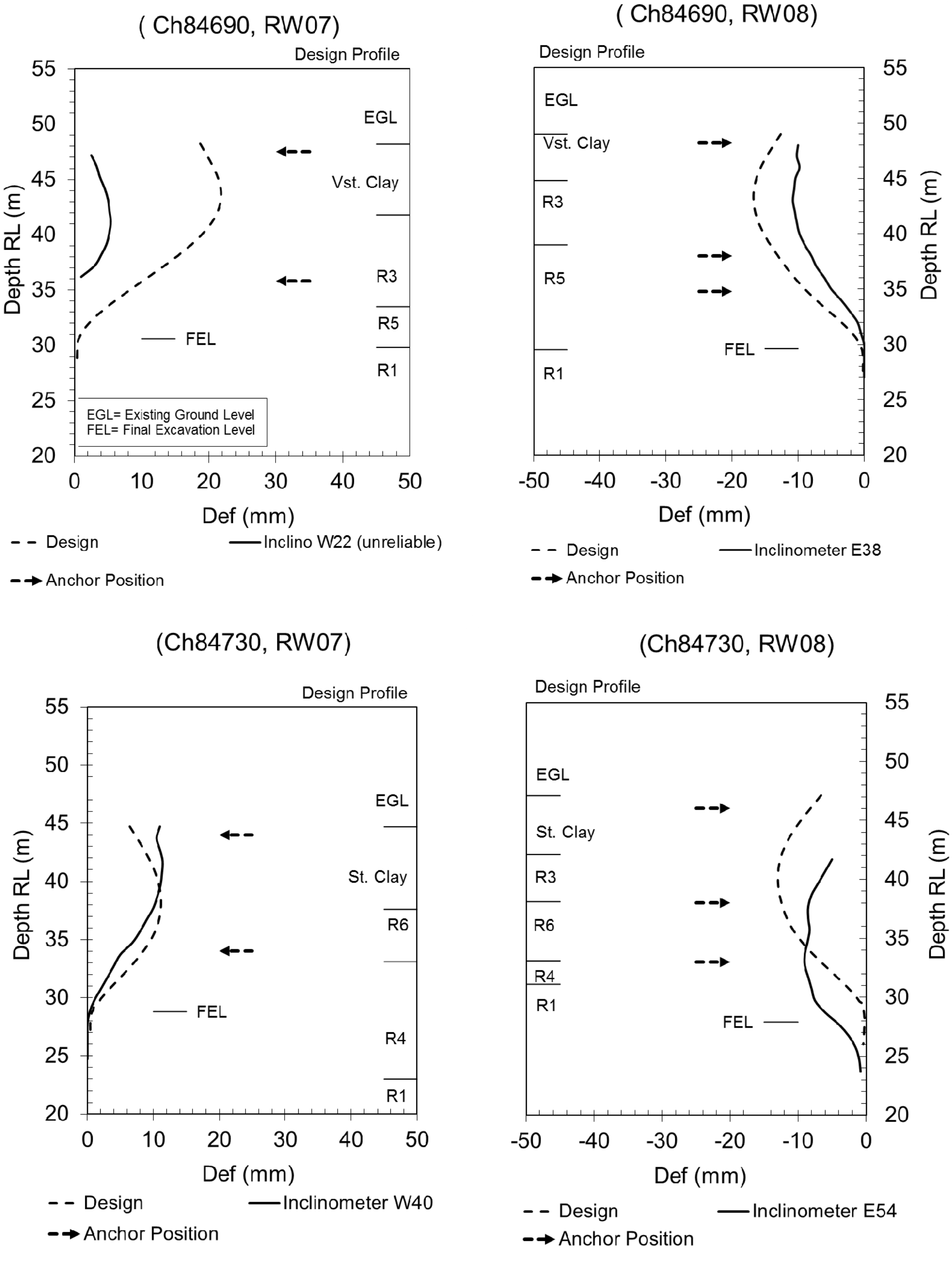

Figure 8b: Comparison of wall deflections between design and monitoring Data

Table 7: Comparison of ground anchor loads during construction

RW07					RW08				
Chainage	Pile No	Anchor Level	Monitoring Data (KN)	Design Load (KN)	Chainage	Pile No	Anchor Level	Monitoring Data (KN)	Design Load (KN)
84590	W16	Upper	710	760	84590	E34	Upper	660	750
		Middle	800	880			Middle	720	830
		Lower	760	780			Lower	780	820
84610	B9	Upper	720	850	84610	A9	Upper	690	810
		Middle	740	900			Lower	No load cell installed	
		Lower	450	510					
84690	W22	Upper	760	860	84690	E38	Upper	720	790
							Middle	780	770
		Lower	No load cell installed				Lower	550	500

7 BACK ANALYSIS

The aim of the back analysis was to review and collate the calculated values from the finite element analyses and to compare these with actual measurements of displacement and anchor loads from the monitoring data. A back analysis has focused on the retaining walls section of Piles B9 and A9 (located at Abutments A and B respectively of the Land Bridge), where anticipated values and actual performance values differed significantly. The ground profiles identified from the piling logs recorded during the pile excavation were used to amend the ground models and to conduct a back analysis. It was observed that the majority of the ground profile used to undertake the detailed design was reasonably consistent with the ground profile encountered during construction. However, it was observed that a layer of agglomerate "R2/R3" occurred immediately above the lower basalt horizon. This had been modelled as lower strength "R4" agglomerate during the design. The key data used to undertake the back analysis included the ground profile from the piling records (see Figure 9) the geotechnical model (see Table 8), wall details (see Table 9), and construction sequence (see Table 10).

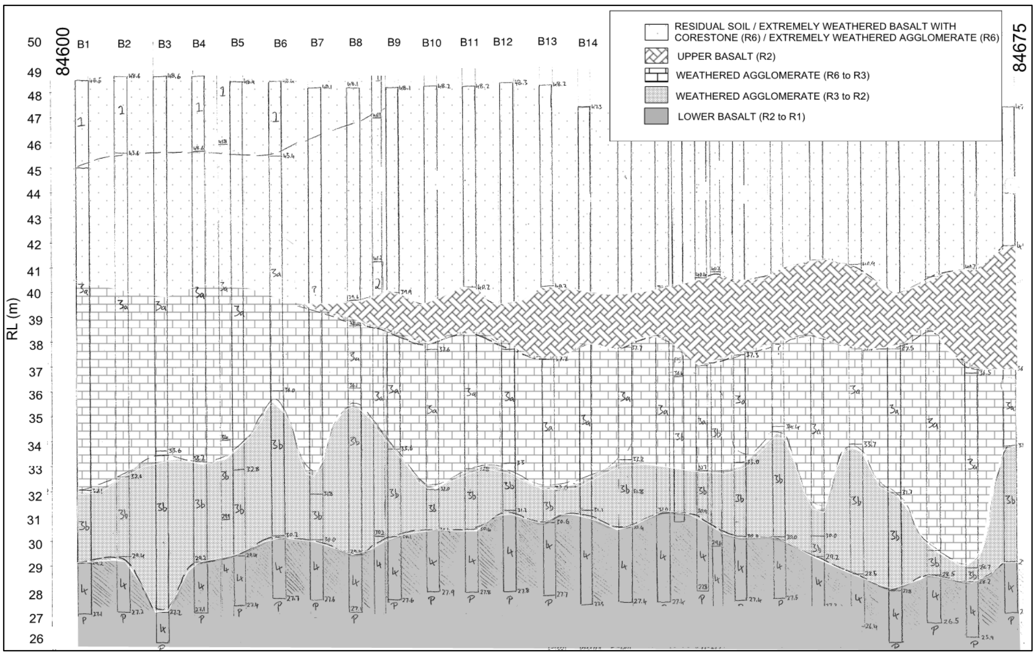

Figure 9: Ground profile from piling records

Table 8: Geotechnical model for selected back analysis section

Pile No. B9 (Abutment A)				Pile No. A9 (Abutment B)			
Soil/Rock Type	Classification		Top RL-m	Soil/Rock Type	Classification		Top RL-m
	Design	Back Analysis			Design	Back Analysis	
Residual Soil	VSt	VSt	54.5	Residual Soil	VSt	VSt	54.5
Agglomerate	R6	R6	39.0	Upper Basalt	R2	R2	42.0
Agglomerate	R4	R4	36.0	Agglomerate	R4	R4	38.5
Agglomerate	R4	R2/R3	33.0	Agglomerate	R4	R2/R3	33.0
Lower Basalt	R2	R2	30.5	Lower Basalt	R2	R2	29.8
Lower Basalt	R1	R1	27.3	Lower Basalt	R1	R1	27.5

Table 9: Wall and ground anchor details

Type		Pile No. B9 (Abutment A)	Pile No. A9 (Abutment B)
Wall	Pile wall	1.2m dia. Pile @ 3.0m c/c	1.2m dia. Pile @ 3.0m c/c
	Pile Top Level	RL 49.6m	RL 49.9m
	Pile Toe Level	RL 27.6m	RL 27.5m
	Final Excavation Level	RL 32.7m	RL 31.7m
	Wall Height	17.6m	19.4m
Anchor	Upper Anchor Level	RL 48.5m with 45° inclination	RL 48.5m with 45° inclination
	Middle Anchor Level	RL 42.5m with 45° inclination	NA
	Lower Anchor Level	RL 37.5m with 45° inclination	RL 36.5m with 45° inclination

Table 10: Summary of construction sequence

Stage	Pile No. B9 (Abutment A)	Pile No. A9 (Abutment B)
Stage 1	Initialise in-situ stresses	Initialise in-situ stresses
Stage 2	Install 1.2m dia. piles at 3.0m c/c	Install 1.2m dia. piles at 3.0m c/c
Stage 3	Excavate to RL 48m	Excavate to RL 48m
Stage 4	Install upper ground anchor @ RL48.5 and pre-stress to 600kN	Install upper ground anchor @ RL48.5m and pre-stress to 600kN
Stage 5	Excavate to RL 42m	Excavate to RL 42m
Stage 6	Install middle anchor @ RL 42.5m and pre-stress to 600kN	-
Stage 7	Excavate to RL 37m	Excavate to RL 37m
Stage 8	Install lower ground anchor @ RL 37.5m and pre-stress to 350kN	-
Stage 9	Excavate to RL 36m	Excavate to RL 36m
Stage 10	-	Install lower ground anchor @ RL 36.5m and pre-stress to 350kN
Stage 11	Excavate to final excavation level	Excavate to final excavation level

By adjusting the ground model to accommodate the layer of higher strength agglomerate in the back analysis, the resulting wall deflections and anchor loads were found to be closer to those actually monitored on site (see Figure 10 and Table 11). The back analysis revealed that the models were particularly sensitive to minor variations in rock stiffness at critical elevations along the pile length.

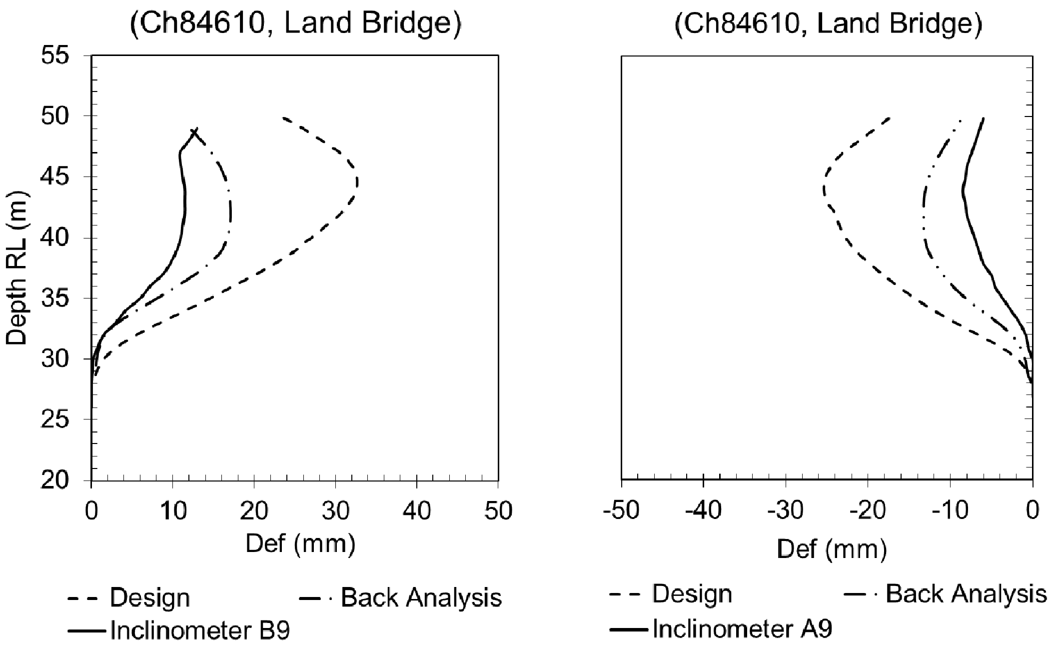

Figure 10: Wall Deflections between design, monitoring data and back analysis

Table 11: Comparison of anchor loads

Pile No. B9 (Abutment A)			Pile No. A9 (Abutment B)		
Anchor Level	Monitoring data (KN)	Back Analysis Load (KN)	Anchor Level	Monitoring data (KN)	Back Analysis Load (KN)
Upper	720	730	Upper	690	710
Middle	740	780			
Lower	450	440	Lower	No load cell installed	420

8 CONCLUSIONS

The Sexton Hill Cutting of the Banora Point Upgrade presented particular challenges to the Designer and the Constructor. These included extremely high retaining walls, considerable variability in geological and geotechnical conditions and excavations carried out adjacent to a major operational road, occupied residences and critical utilities.

The pile wall designs had been developed largely based on numerical modelling techniques and the field performance confirmed design predictions.

The monitoring data indicated that at one location the predicted movements were greater than the actual measurements of the retention system. Subsequent back analysis revealed that the deflections displayed by the piles were particularly sensitive to small changes in presence of high strength agglomerate above the Lower Basalt. Adjusting the model to reflect the actual ground conditions demonstrated reasonable agreement between the observed and predicted structural responses.

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