

DESIGN AND CONSTRUCTION OF CANTILEVER RETAINING WALL NEAR RAILWAY LINE: (CASE STUDY – 359 ILLAWARRA ROAD, MARRICKVILLE NSW)

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ABSTRACT

The proposed “Revolution Apartments” is located at the corner of Illawarra Road and Byrnes Street, Marrickville NSW and replaces the Marrickville RSL Club. The development comprises seven level building, partly underlain by a two level basement car park. The Marrickville Railway Station sits on the southern boundary. The nearest track of the railway line was set back by about 3m to the site boundary. An existing Sydney Water “Storm-water Underground Channel” was running through the proposed building. The development required the decommissioning of the existing Sydney Water culvert and the construction of a new concrete lined culvert between the proposed building and the south boundary. A permanent cantilever wall is proposed because the anchor system is not permitted into the railway Corridor. The maximum excavation depth is about 3.5m. As a result of the excavation works, ground movement is expected. The challenge of this project is that, the proposed retaining wall shall meet the track settlement limits in accordance with RailCorp SPC 207 (Track Monitoring Requirements for Undertrack Excavation, 2013). It was required to limit any movement to “Alarm Level 1” as per the Railcorp specification, which is defined by any detected movement, both vertical and horizontal, to be less than 15mm. Frankipile submitted a design and construct proposal comprising a cantilever contiguous pile wall along the boundary to the railway Corridor. In order to monitor the deflection of the wall during the excavation works, three inclinometers were installed at the most critical section of the contiguous shoring wall adjacent to the railway track boundary. The railway lines were monitored independently by surveyor engaged by Railcorp at some 200 points and the presentation will review these results in the light of the inclinometer monitoring results. Computer software WALLAP and PLAXIS were used in design to estimate the wall deflection and ground movements. The analysis results are compared with the monitoring results and soil parameter studies are also carried out.

1 INTRODUCTION

With increasing residential densities and business activities near rail corridors, there is a growing need to better integrate land use and transport. This particularly applies to major development near railway stations.

The proposed “Revolution Apartments”, a seven level building located at the corner of Illawarra Road and Byrnes Street, Marrickville NSW. The Marrickville Railway Station sits on the southern boundary. The nearest track of the railway line was set back by about 3m to the site boundary. An existing Sydney Water “Storm-water Underground Channel” was running through the proposed building shown in Figure 1.

For a cantilever retaining wall with 3.5m excavation depth, ground settlement and horizontal displacement are expected. In accordance with Railcorp specification (SPC 207, 2012), the proposed retaining wall shall meet the track settlement limits. For this particular project, it was required to limit any movement to “Alarm Level 1”, which is defined by any detected movement, both vertical and horizontal, to be less than 15mm.

Traditional design method includes the stages simplified as (1) building up geotechnical model and using conservative soil design parameters, (2) with the help of computer software (i.e. WALLAP and PLAXIS etc.), (3) estimating the wall deflection and ground movements with an assumed pile length and stiffness, (4) adjust the pile length and stiffness to limit the ground movement to the target value, (5) taking the bending moment and shear force induced in the wall and design the structural strength for the piles.

This paper shows the studies on the predicted wall deflections, compared with the inclinometer monitoring results. Discussions and comments on the design and displacement estimation for cantilever retaining wall are also presented.

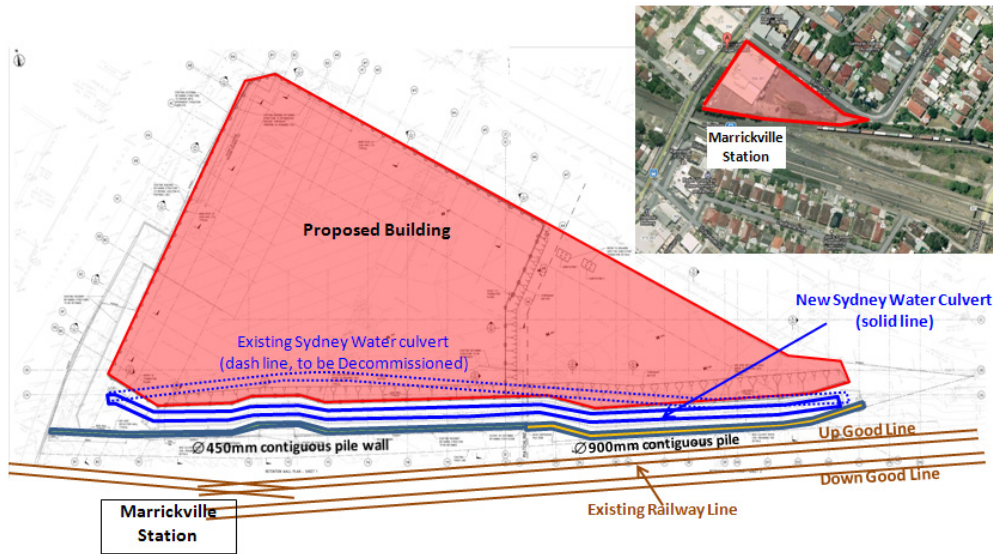


Figure 1: Site Location & Layout (Existing & Proposed Structures)

2 ASSESSMENT OF GEOTECHNICAL CONDITION

The topography of the site is gently sloping down to the east. The preliminary geotechnical investigations was carried out in 2009 for DA purpose with 2 boreholes drilled into rock and additional geotechnical investigations was carried out in 2011 for construction tender purpose. Additional 4 boreholes were drilled into rock with Point Load Strength Index Tests on cored samples.

At the post tender stage, an extra five cored boreholes and three EFCP tests were carried out after the existing RSL building was demolished. The subsurface soil profile is summarised below:

- FILL, extended to depths varying from 0.5m to 2m, comprised silty clays and silty sands with varying content of gravel, in a highly variable state of compaction.
- SANDY/SILTY CLAYS, low to high plasticity, highly variable in strength or relative densities. Some soft, loose bands encountered of variable thickness.
- SANDSTONE, encountered at depths ranging from 3.0m in the west of the site to 13.4m in the east. The rock is initially distinctly weathered of very low to low strength, improving with depth to fresh of medium to high strength.
- GROUNDWATER, seepage was encountered during drilling in all boreholes at depths ranging from RL4.5m to RL-0.1m. Measured groundwater in the borehole standpipe was at RL3.9m to RL3.5m.

The geotechnical investigations disclosed highly variable subsurface conditions of fill and natural soils grading into sandstone bedrock at depths. The sandstone was assessed as being initially very low to low strength, improving with depth to medium to high strength. A contour plan of medium strength rock has been interpreted based on the current borehole logs in Figure 2.

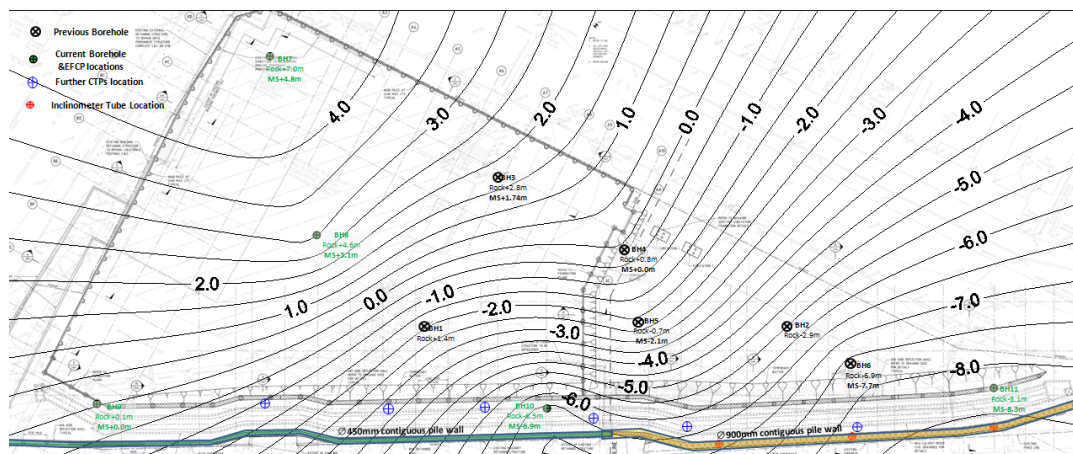


Figure 2: Shoring Plan & MS Rock Contour (included borehole location)

3 RETAINING WALL DESIGN

In response to the excavation work for the new culvert, the potential impact induced below the adjoining railway line must be considered in retaining wall design. The retaining wall analysis shall meet the track settlement limits of 15mm (“Alarm Level 1” as per the Railcorp specification). Ø900mm contiguous pile wall was proposed in this section. At the preliminary design stage, the worst case scenario was analysed with the site boundary located 3.6m to the centre line of the closest rail track.

During the post tender stage, Frankipile confirmed with the Geotechnical Consultant, the construction stages for the worst case scenario shown below and the simplified design model and soil parameters of Mohr-Coulomb Model are summarised in Table 1. A typical section of the Ø900mm contiguous Wall is shown in Figure 3.

- Top of Wall at RL7.0m
- The closest track is at RL8.0m and set back 3.5m from the centre line of the contiguous pile wall.
- The design surcharge of 60kPa applied over two 3.0m widths located immediately below the two closest tracks applied at the base of the ballast of RL7.4m (assumed the ballast thickness = 0.6m)
- Install Ø900mm contiguous piles to toe level at RL-5.0m
- Soil/Wall interface reduction factor $R_{inter} (\tan\delta/\tan\phi) = 0.5$
- The culvert excavation to RL3.5m with a 45° batter left in front of the wall

Table 1: Design Model for Ø900mm Contiguous Wall (Mohr-Coulomb Model)

Material	Top RLm	γ_{dry} kN/m ³	γ_{sat} kN/m ³	E MPa	c_{ref} kPa	ϕ (°)	ν
Ballast	8.0	16	19	21	0.5	42	0.3
Fill	8.0	18	21	13.5	2.0	28	0.3
Very Stiff Clay	3.5	18	21	38	2.0	28	0.3
MD Sand	-2.5	18	21	35	0.5	30	0.3
Stiff Clay	-5.0	18	21	15	2.0	28	0.3
Rock	-7.0	21	21	100	100	30	0.25

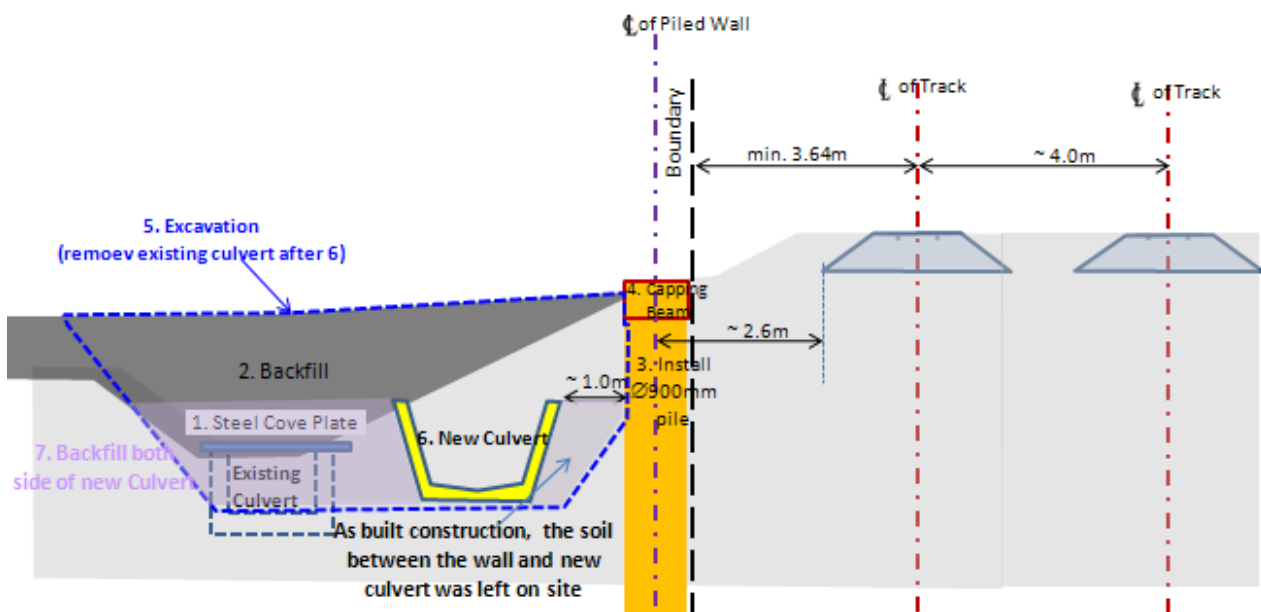


Figure 3: Proposed Construction Sequence (Typical Section of Ø900mm Contiguous Pile Wall)

Table 2: Summary of Typical Analysis Results

Analysis Software	Max. Wall Deflection	Displacement under Track			Moment kNm/m	Shear kN/m
		Vector	Vertical	Horizontal		
PLAXIS	22.3mm	23.mm	16.2mm	16.3mm	201	67
WALLAP-2D ⁽¹⁾	12mm	Factor of Safety = 2.8			85	34

NOTE: ⁽¹⁾ WALLAP does not give movement of soil behind the wall

Both PLAXIS and WALLAP analysis were carried out and the results are summarised in Table 2. Analysis results indicate that, even with a similar geotechnical model and using the same soil parameters, PLAXIS and WALLAP concluded with significantly different wall deflection. Further detail studies will be discussed hereafter.

4 RETAINING WALL ANALYSIS AND SOIL DISPLACEMENT ESTIMATION

For conservative retaining wall design, it is widely recommended to use the lower limit design value of soil strength parameters and interface reduction factor between soil and wall. In routine practice, engineers also use the design (factored) soil strength parameters in retaining wall analysis software such as WALLAP and PLAXIS and then use the analysis results of wall deflections and soil displacements for engineering estimation.

It is understood that the design (factored) soil strength parameters are used for Ultimate Limit State (ULS) Analysis. Thus the prediction of wall deflection and soil displacement are generally overestimated (WALLAP 2006).

The parametric study on the influence of interface reduction factor between soil and wall (R_{inter} or $\tan\delta/\tan\phi$) was carried out during the design stage. For identical comparison, Mohr-Coulomb Model was used (built-in soil Model in both PLAXIS and WALLAP software) and the design parameters are shown in Table 1 above.

The analysis results for the effect of R_{inter} (from 0.5 to 1.0) are given in Table 3,

- PLAXIS analysis shows significantly decrease of the wall deflection (~50%) and soil displacement (~60%); significantly increase the bending moment (44%) and Shear Force (20%) in wall element, for increasing R_{inter} ,
- WALLAP-2D analysis shown minor changes in wall deflection, moment and shear in wall element; with an increase the Factor of Safety (from 2.8 to 3.5), for increasing R_{inter} ,
- PLAXIS and WALLAP show similar wall deflection when $R_{inter}=1.0$.
- PLAXIS analysis shows the vertical settlement of pile decreased with increasing R_{inter} .

Table 3: Influence of R_{inter} on soil displacement and wall deflection

Analysis Program	Soil/wall adhesion	Wall Deflection,		Displ. under Track, mm		NOTES
		Top, mm	Toe, mm	Vertical	Horizontal	
PLAXIS (R_{inter})	1.0	11.3	0.75	5.8	6.9	toe -2.0mm (up)
	0.6	18.9	0.36	13.2	13.5	toe 3.9mm (down)
	0.5	22.3	0.19	16.2	16.3	toe 6.6mm (down)
WALLAP ($\tan\delta/\tan\phi$)	1.0	11	5	FOS = 3.5		Soil displacement & wall settlement not available
	0.6	12	5	FOS = 3.0		
	0.5	12	5	FOS = 2.8		

Further studies have been carried out after the construction works finished and will be discussed hereafter.

5 CONSTRUCTION OF CONTIGUOUS PILE WALL

For practical and safety reason, the new Sydney water culvert was built by cast in-situ method and the soil between the culvert and contiguous wall was left in-situ. For Ø900mm contiguous Wall adjacent to the railway track, the as-built construction stages are as follows:

- Backfill over the existing culvert to a level of RL6.5m to RL7.0m
- Install Ø900mm contiguous pile wall (pile toe at RL-5.0m)
- Battered excavation to the invert levels of new culvert (RL3.4m to 4.0m)
- construct the cast in-situ culvert along its new alignment
- Remove the existing culvert and backfill to top of culvert at RL5.0m to 5.5m

A typical section with construction stages is shown on Figure 3 above.

6 MONITORING SYSTEM

Throughout the excavation and shoring works a ground monitoring system was established and operated. The monitoring points along the northernmost railway track (both tracks) are located on the web of the tracks and just to the side of the track in the ballast. These monitoring points are spaced at approximately 2m along the length of the northernmost railway track. The Base Reading of Monitoring Results was first taken on 06/09/2012 and daily monitoring results were provided for review.

Although the survey monitoring showed some scatter results and system errors, the statistic analysis from large amount of data indicated the horizontal displacement and vertical settlement under the nearest tracks due to the excavation works were less than 5.0mm.

Three inclinometers were installed in the Ø900mm contiguous pile wall located in Pile Nos. P185, P209 and P231, spaced at approximately 20m. The piles were constructed on 30/10/2012. The locations and typical readings are shown in Figure 4.

The initial reading (base reading) of inclinometer was taken on 09/11/2012 which was about 11 days after pile concreting. A typical report of inclinometer reading is shown in Table 4 and cumulative displacements are shown in Figure 4 (including the maximum deflection of each location). The monitoring results of ground survey and inclinometer show the maximum wall deflection and the ground displacement are less than 5mm which is well within the target “Alarm Level 1”.

Table 4: Summary of inclinometer readings

Inclinometer Pile No.	Top of Capping Beam	BEL (passive side)	Cumulative displacement	
			29/11/2012	27/02/2013
P185	RL7.10m	RL5.4m	2.0mm	4.0mm
P209	RL7.10m	RL5.3m	1.3mm	4.4mm
P231	RL6.75m	RL4.8m	2.3mm	4.0mm

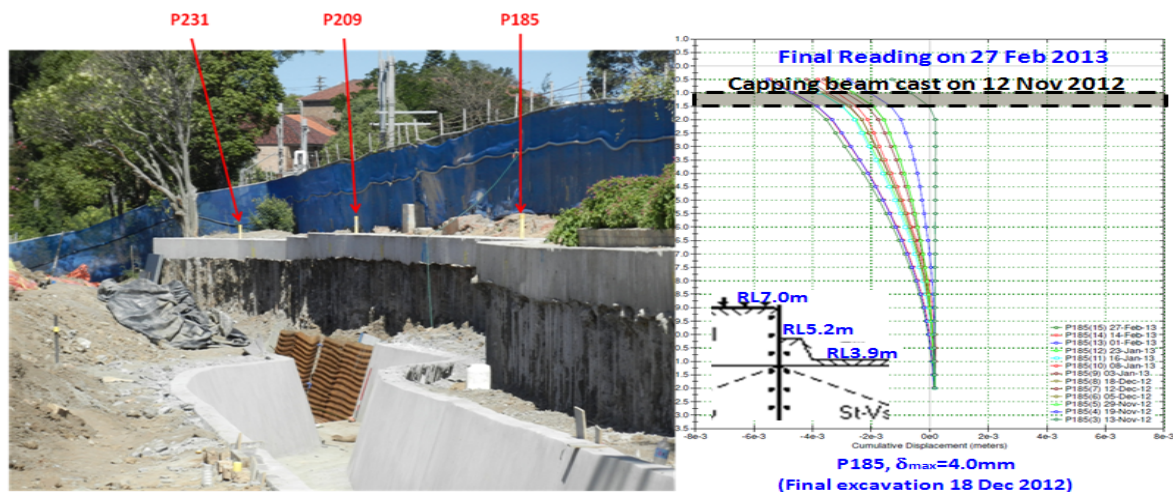


Figure 4: Location of Inclinometers and Typical Cumulative Displacements Readings

7 RETAINING WALL ANALYSIS VS. MONITORING RESULTS

A typical as-built analysis case of PLAXIS is shown in Figure 5. In order to compare with the tender design, it is assumed that the $R_{inter}=0.5$ and the tracks surcharge of 60kPa was initialled and re-set the ground displacement to zero before excavation (the same method used at tender stage).

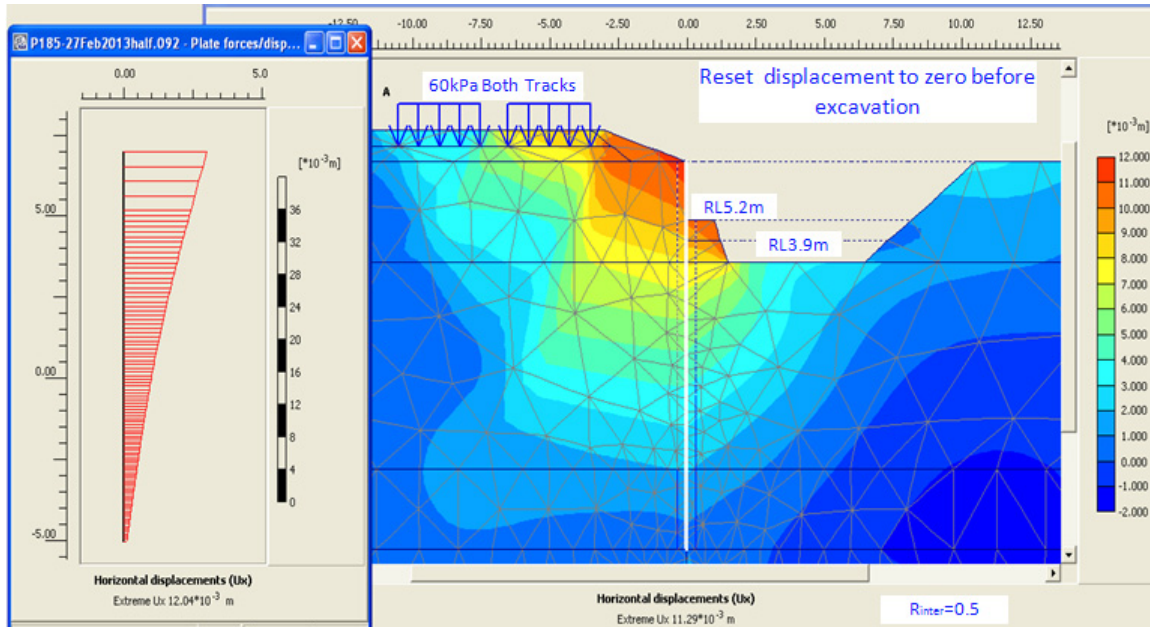


Figure 5: PLAXIS analysis Case – Horizontal displacement after excavation to RL3.9m

The PLAXIS analysis results versus the measured data of inclinometers (Figure 6) show that for an interface reduction factor of 1.0, the maximum wall deflection is very close to the measured values. However, it is noted that the PLAXIS analysis shows the toe of the wall has a movement of around 1.5mm toward the excavation, which is about 30% of the maximum wall deflection (4.5mm).

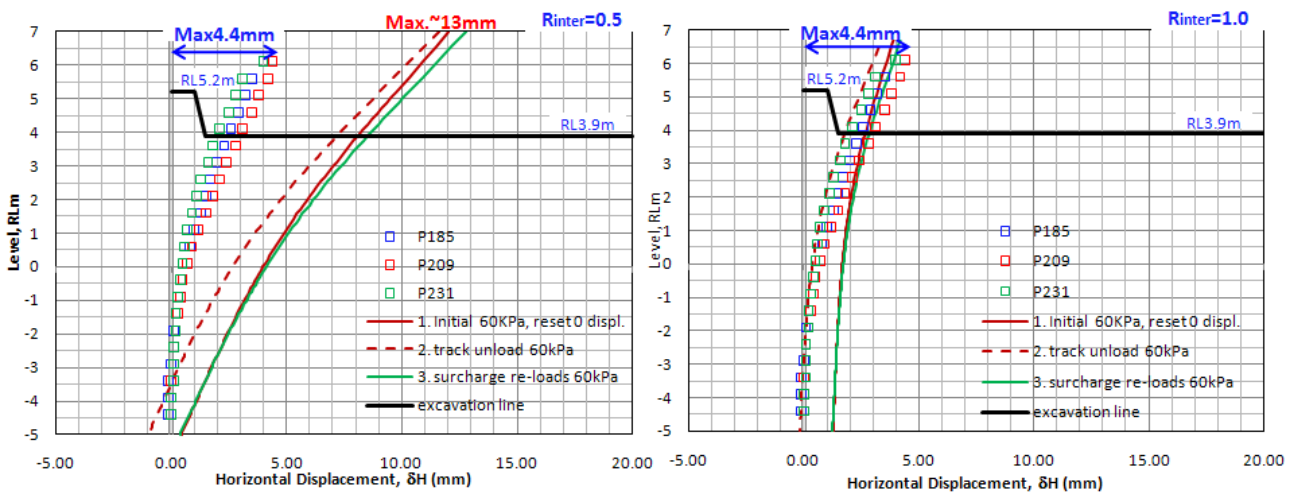


Figure 6: Comparison of Wall Deflection – Monitoring vs. Analysis

The deflection curves indicated that the strength of soil is greater than the assumed values especially at depth below excavation level. Studies also show that the numerical model will result in lateral displacement of the pile toe because the fixed lower boundary was assumed at the RL-10m.

Further studies indicate that it is questionable to assume an initial surcharge of 60kPa and to re-set the ground displacement to zero before excavation. Figure 7 shows an analysis case without track surcharge before excavation, resulting in a maximum wall deflection of approximately 5mm. However, as shown in Figure 6, with re-loading of 60kPa track surcharge, the maximum wall deflection is 4.5mm, which is less than that without track surcharge. This is unlikely true.

The Figure 7 shows a plot of inclinometer readings. We have also plotted 2 cases, with case 1 analysing nil track surcharge and case 2 analysing track surcharge, and surcharge remove as the train passes the contiguous wall.

Comparison of the curves in Figure 7, shows the predicated maximum wall deflection is much higher than the monitoring results especially in the portion below excavation level. This indicates that assumed deformation properties of the soil layers appear too low.

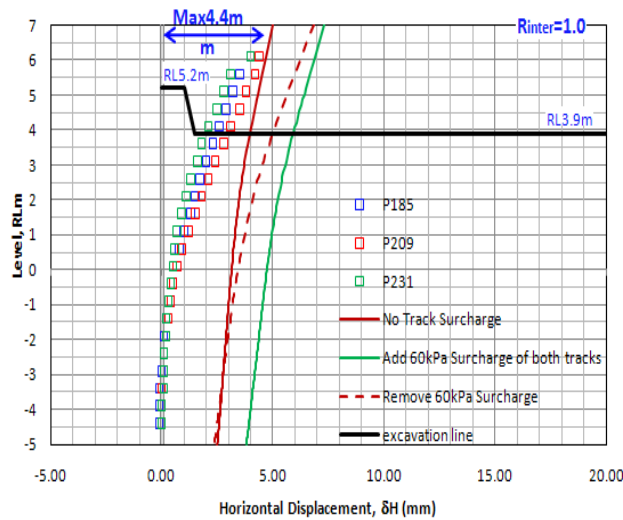


Figure 7: Monitoring vs. Analysis (surcharge load after excavation)

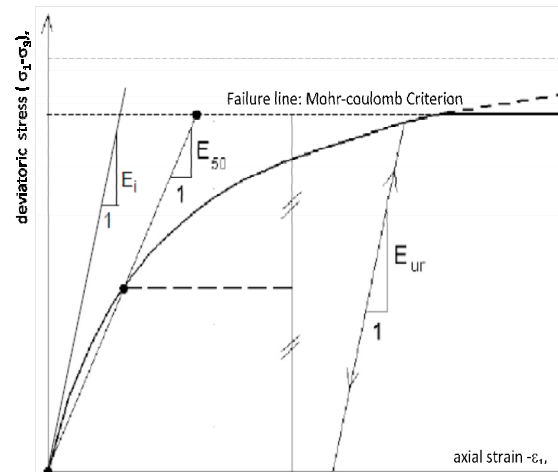


Figure 8: Definition of E₅₀ (Andrew, 2008)

8 BACK ANALYSIS AND SOIL MODEL PARAMETERS STUDY

8.1 MOHR-COULOMB AND HARDENING SOIL MODEL WITH SMALL-STRAIN STIFFNESS

The Mohr-Coulomb model, an elastic perfectly plastic model, is widely used in soil mechanics and foundation engineering. In general, the stability analysis deals with relative large strain problems where the stress states are at failure condition. The Mohr-Coulomb failure criterion well describes the stress states at failure in drained conditions with effective strength parameters ϕ' and c' (Brinkgreve, R.B.J. 2008). With Mohr-Coulomb model for soil materials, the Young's modulus at 50% strength (the secant modulus E_{50}) is usually used in engineering practice.

The definition of deformation modulus Young's modulus E at different stress states are shown in Figure 8. It is clear that, at the large strain near the failure state (ultimate limit state), the secant modulus E_{50} is well defined. However, the excavation involves unloading problem, the unloading modulus (E_{ur}) is required (Andrew, 2008). Observed stiffness is much higher for unloading and reloading than primary loading.

Moreover, for soil, the shear strength increases with confining pressure (Terzaghi et al., 1996). Hence, deep soil tends to have greater stiffness than the shallow layers. In retaining wall analysis, the stress-strain behavior of soil is non-linear and the Modulus (Young's modulus or shear modulus) is related the magnitude of the strains which occur during the excavation as indicated in Figure 9 (Brinkgreve, 2008).

Literature reviews indicate that to adopt the E_{50} to design the structural strength of retaining wall is appropriate, but will over-predict the displacements of soil and structures.

The Hardening Soil Model with Small-strain Stiffness (HSsmall) is one of the PLAXIS built-in models, which accounts for the increased stiffness of soil at small strains apparent in working stress condition and appropriate for estimating the stress-strain behavior of soil under un-loading and re-loading problems (Figure 9). However, the "HSsmall model" cannot simulate the soil stiffness increasing with depth.

On the other hand, because the Mohr-Coulomb model is usually in all FEM programs like PLAXIS and WALLAP, it is possible to use "MC increasing model" with the "un-loading and re-loading" Young's modulus as the reference modulus and increasing with depth.

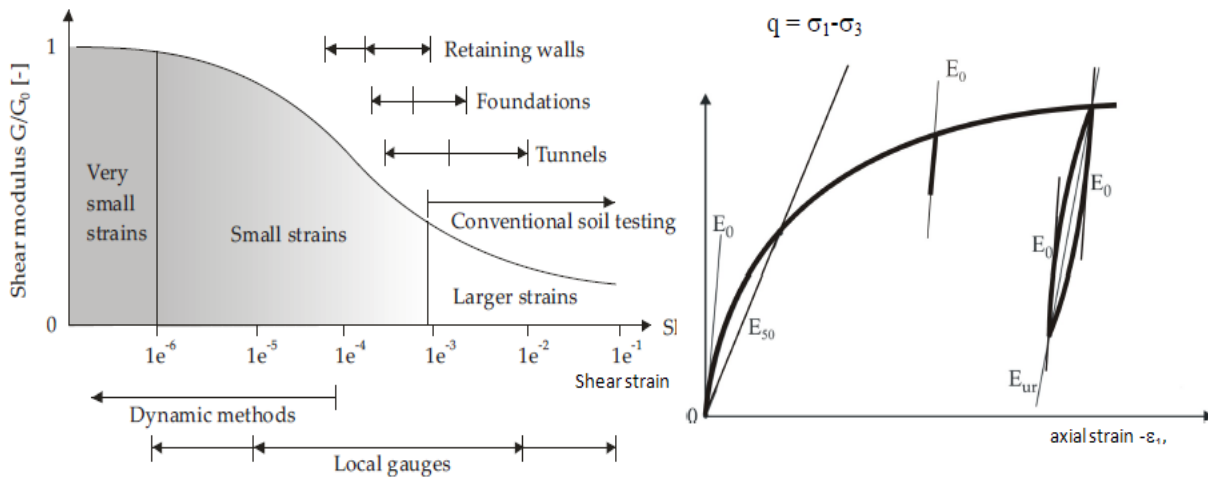


Figure 9: Stiffness–strain and stress dependent behaviour of soil (Brinkgreve, 2008)

8.2 As-Built Analysis with MC-increasing and HSsmall Models

Parametric studies have been carried out based on the inclinometer monitoring results. By adjusting the Young’s modulus, close agreement of deflection curves are obtained with PLAXIS and WALLAP. The simplified soil model and parameters are summarised in Table 5 and soil profile shown in Figure 10.

Table 5: Simplified Back Analysis Model (MC and HSsmall Models)

Soil Type	Top RLM	$\gamma_{dry}(\gamma_{sat})$ kN/m ³	E_{50}	E_{ur}	E_{ref}	ΔE MPa/m	c'_{ref} kPa	$\Delta c'$ kPa/m	ϕ (°)	ν
			(HSsmall)	(MC-increasing)						
Layer 1	7.0	18(21)	13.5	27	1	0.4	0.5	0	28	0.3
Layer 2	3.5	18(21)	38	152	40	5	2.0	1	28	0.2
Layer 3	-2.5	18(21)	38	152	140	10	1.0	1	35	0.15

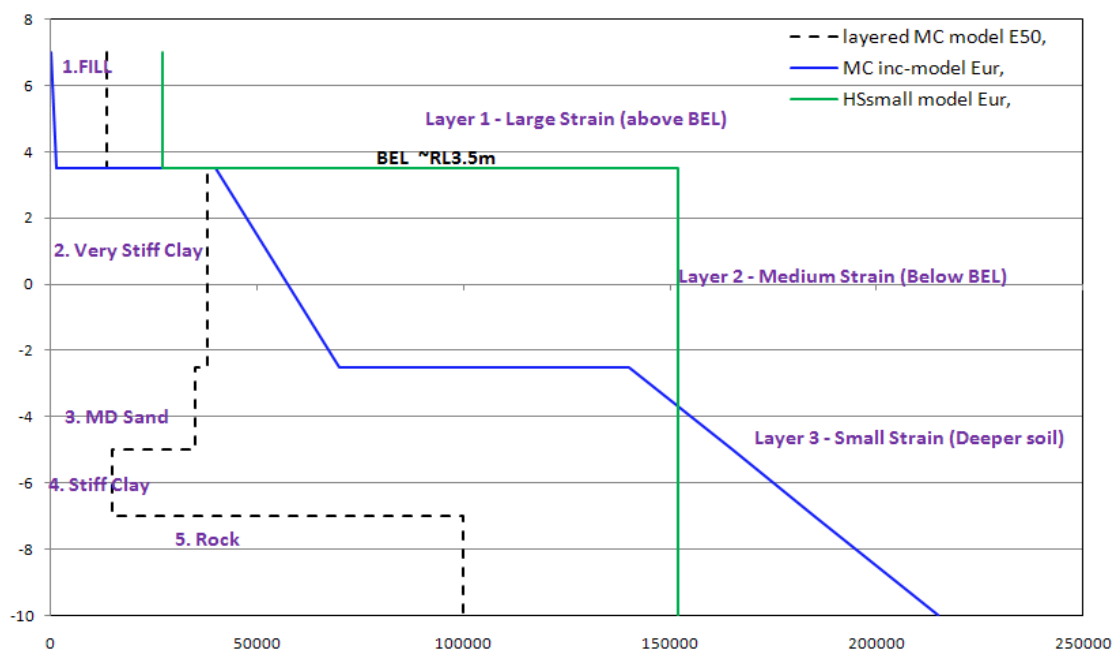


Figure 10: Simplified Analysis Model of Young’s modulus profile (MC-increasing and HSsmall Models)

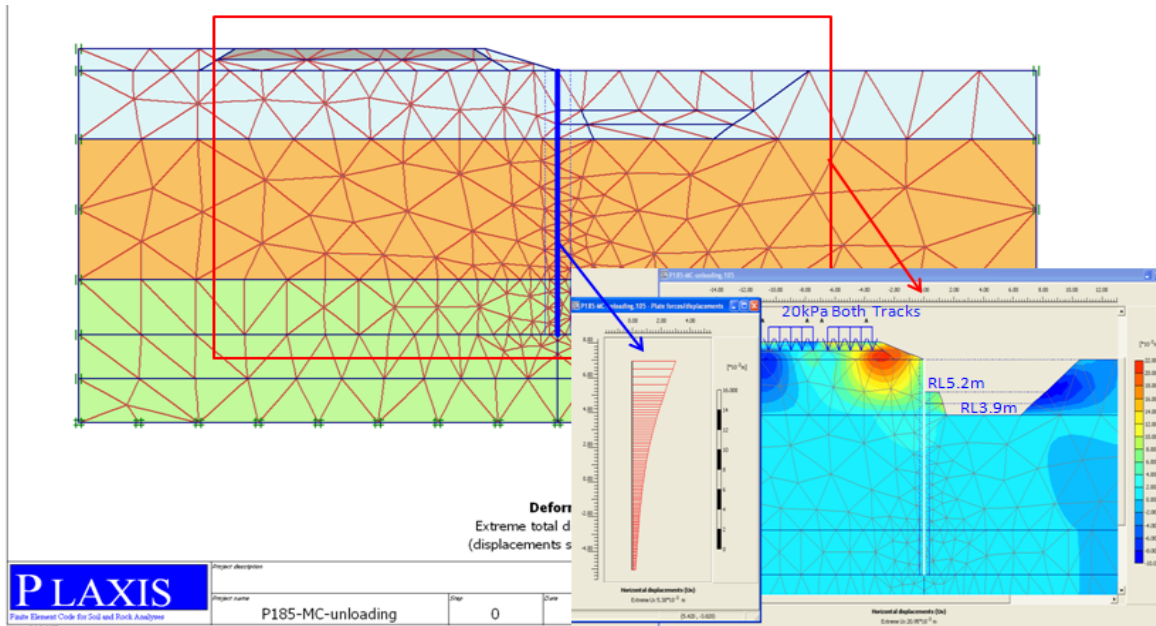


Figure 11: Simplified PLAXIS model mesh, and displacements of soil and wall (MC-increasing)

The Figure 11 shows the typical PLAXIS results of wall deflection and soil displacement around the wall with “MC-increasing” models. The Figure 12 shows a comparison of the PLAXIS result with “HSsmall” and “MC-increasing” models.

Considering the numerical approximation of Finite Element Analysis, both “HSsmall” and “MC-increasing” models show reasonable simulations of the real behaviours of soil and retaining wall.

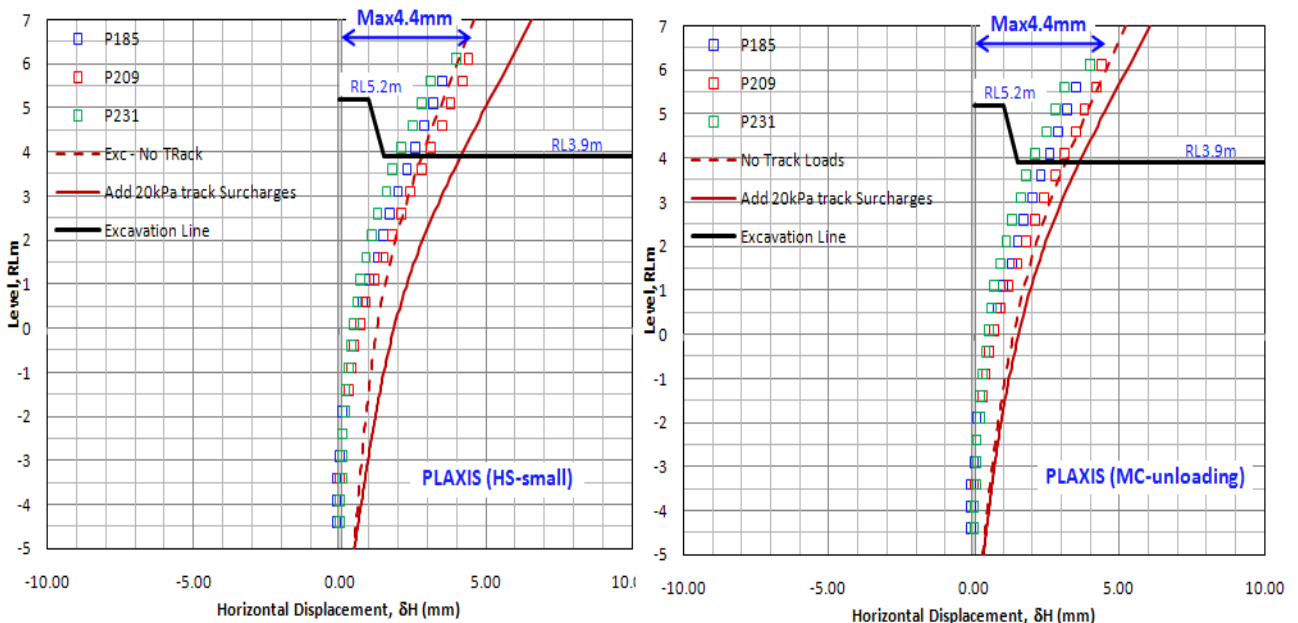


Figure 12: Predicted wall deflections with back analysis models (PLAXIS)

Similar analysis has been also carried out using WALLAP-2D with “MC-increasing” models. A very close curve has been obtained and the result is shown in Figure 13.

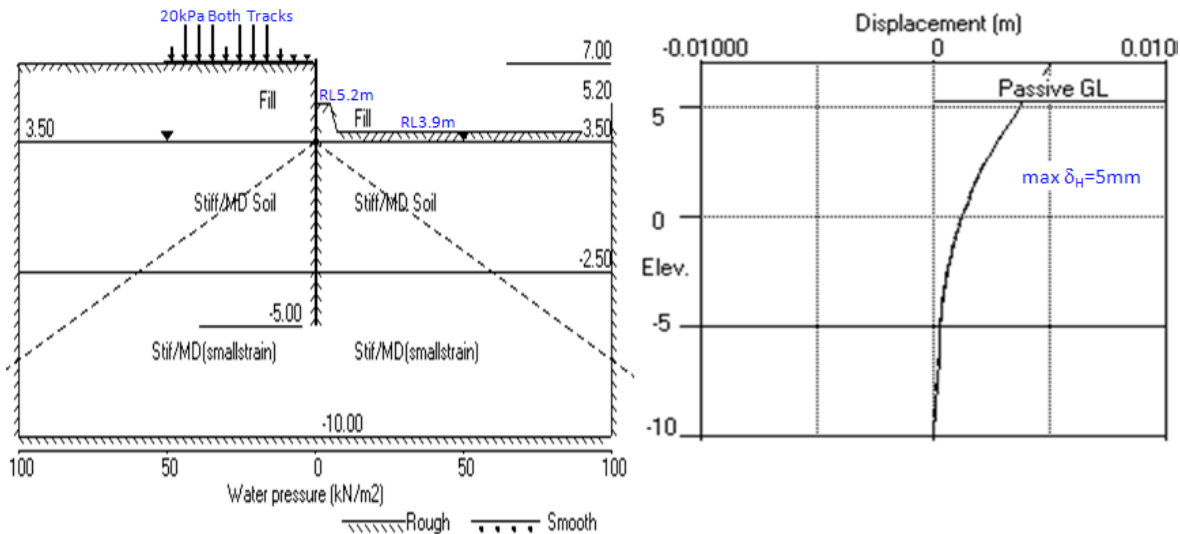


Figure 13: Simplified WALLAP-2D model, and deflection of the retaining wall (MC-increasing)

9 CONCLUSIONS

For cantilever retaining wall design, the use of the interface reduction factor between soil and wall R_{inter} ($\tan\delta/\tan\phi$) of 0.5 will produce a higher bending moment and shear force (PLAXIS) or lower factor of safety (WALLAP), which will need a stiffer or longer pile. However, to estimate the soil and retaining wall displacement, R_{inter} ($\tan\delta/\tan\phi$) of 1.0 is appropriate.

In WALLAP or PLAXIS, to monitoring the traffic loads of road or railway tracks, unlike the buildings standing behind the retaining wall, the surcharge load should be added after the piles are installed without re-setting the displacement to zero.

In engineering practice, uniform layered model are normally adopted. However, as the effective stress increases with depth, the strength parameters and deformation modulus should be stress and strain dependent especially when the soil considered at great depth.

For ground movement sensitive retaining wall, it is important to carry out the monitoring during the excavation works. The monitoring method provides a continuous review and refinement of a design based on observations of field behavior.

Mohr–Coulomb Model is widely used in soil mechanics (both WALLAP and PLAXIS with built-in model). The study shows that the Mohr-Coulomb model with un-loading/re-loading Young's modulus and increasing with depth is reasonable for retaining wall analysis. With accurate observations and appropriate parameter analyses, they are well in retaining wall design and displacement estimation.

The back analysis above shows a reasonable prediction on the behaviour of the cantilever retaining wall. We should understand that there are many factors and various parameters controlling the soil and structural behaviours. Analysis shown in this paper is only one of the possible approximations of reality.

10 ACKNOWLEDGEMENTS

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