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Case Studies of Geotechnical Risk Management of Construction Near a Live Railway

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ABSTRACT

Construction near a live railway involves geotechnical risks that must be managed to ensure safe operations, within a very limited timeframe. Three successful case histories are presented. Case Study 1 presents the study of a temporary retaining wall to install a bridge pile foundation. Case Study 2 presents the study of demolishing an existing road bridge over a railway. It discusses in detail protecting the existing retaining wall. Case Study 3 presents the study of a drainage pipe installation underneath the railway. The geotechnical and construction uncertainties are discussed and highlighted in the case histories.

Keywords: case study, geotechnical risk management, live railway

1 INTRODUCTION

Construction in urban environments sometimes involves construction near a live railway. Geotechnical risks must be managed to ensure the safe operations within a very limited timeframe. The factors that are to be considered include geotechnical uncertainty, construction uncertainty, time constraints, safety, cost, and adjacent and underground structures. Three case histories are presented to study the risk mitigation approaches involved in construction next to live railway traffic.

2 CASE STUDY 1 – TEMPORARY RETAINING WALL FOR BRIDGE PILE FOUNDATION INSTALLATION

2.1 Site description

As shown in the site plan (Figure 1), there were eight bridge foundation piles to be installed at Pier 1 of Bridge BR440. Pier 1 was located 3.5 m away from the tracks of live railway. As shown in Figure 2, there was a nearly vertical existing slope at the pier location and the piling rig would be located on the top of the slope.

2.2 Geotechnical risks

The geotechnical condition near the slope was not well understood at the time of initial design. The ground condition was assumed as Class 6 rock (R6) based on adjacent investigation results. The adopted rock classification system is summarised in Table 1. The potential geotechnical risks were: (a) slope instability due to surcharge from the piling rig; and (b) rock blocks could fall onto the railway. In both cases, railway operation would be jeopardised and construction personnel could be injured or killed.

2.3 Design evolvement

2.3.1 Initial design

To mitigate the potential geotechnical risks, a temporary retaining wall comprising bored piles was proposed next to the Pier 1 piles. The design of the temporary retaining wall was based on the loading diagram, as shown in Figure 2. A Soilmec R10 drilling rig was proposed with a maximum applied outrigger pressure of 77 kPa. Steel plates were proposed to distribute the loads from the piling rig. Temporary bored piles of 600 mm diameter at 1.2 m c/c spacing were proposed to be installed to 5.5 m deep.

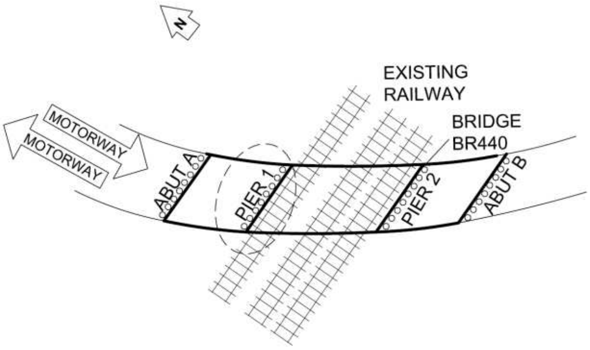

Figure 1. Site plan – Case Study 1

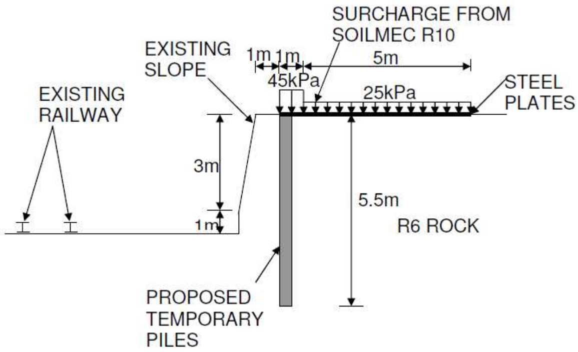

Figure 2. Slope near the railway

Table 1: Rock mass classification system adopted for Cases 1 and 2

Rock Class	Rock Strength (MPa)	Strength category	Block size
R1	UCS[a]>200, $I_{s(50)}$[b]>13.3	Extremely high (EH)	>300 mm
R2	UCS>60, $I_{s(50)}$ >4 or	Very high (VH)	<300 mm
	UCS=20-60, $I_{s(50)}$ =1.3–4	High (H)	>300 mm
R3	UCS=20-60, $I_{s(50)}$ =1.3–4 or	High (H)	<300 mm
	UCS=6-20, $I_{s(50)}$ =0.4–1.3	Medium (M)	>300 mm
R4	UCS=6-20, $I_{s(50)}$ =0.4–1.3 or	Medium (M)	<300 mm
	UCS=2-6, $I_{s(50)}$ =0.13–0.4	Low (L)	>300 mm
R5	UCS=2-6, $I_{s(50)}$ =0.13–0.4 or	Low (L)	<300 mm
	UCS<2, $I_{s(50)}$ <0.13	Very low (VL)	>300 mm
R6	UCS<0.6, $I_{s(50)}$ <0.04	Extremely low (EL)	<300 mm

[a] UCS = Unconfined compressive strength
[b] $I_{s(50)}$ = point load strength

2.3.2 Design check for changed site conditions

The temporary 600 mm diameter piles were installed with an actual installed pile 6.2 m long, slightly longer than designed. Just before the BR440 Pier 1 piles were installed, it was realised the adopted loading of the piling rig was not correct. Instead the rig on site was a 75 t Casagrande TRD100 and the maximum applied ground pressure provided by the piling subcontractor was 624 kPa, much higher than that assumed in the initial design. A thorough review was made and it was identified that the following conditions had changed since the initial design (Table 2):

Table 2: Changed site conditions since initial design

Initial Design	Changed site condition	Final design
R6 rock	Better ground condition	R6/R5 rock (based on adjacent construction)
Up to 4 m	Lower retained height	Up to 3.0 m with a 3.4 m wide bench in front of wall (Figure 3)
77 kPa (Soilmec R10)	Higher piling rig pressure	624 kPa (75 t Casagrande TRD 100)

The better ground condition and lower retained height compensated for the higher applied pressure. Steel plates sized 6 m x 2.4 m were used to reduce the applied ground pressure to about 140 kPa. Figure 3 shows the adopted geotechnical model and the calculated factor of safety (FOS) against global stability was 1.4, which satisfied the required minimum FOS of 1.3.

2.3.3 Design check for pile cap excavation

After the Pier 1 pile was installed, the rocks between the piles were to be broken to construct pile caps. To speed up the pile cap construction, the construction team proposed using the Casagrande TRD100 piling rig to break the rocks. The excavation geometry and loading diagram would be like those shown in Figure 3 (with pile cap excavation). The calculated FOS was 1.1, which was considered unacceptable. Therefore tracked excavators were used to break the rocks instead of the piling rig. Figure 4 shows the exposed pile heads at Pier 1.

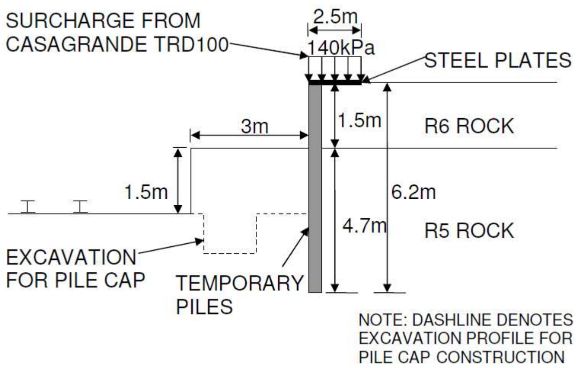

Figure 3. Cross section – Casagrande TRD100


Figure 4. Exposed pile heads at Pier 1

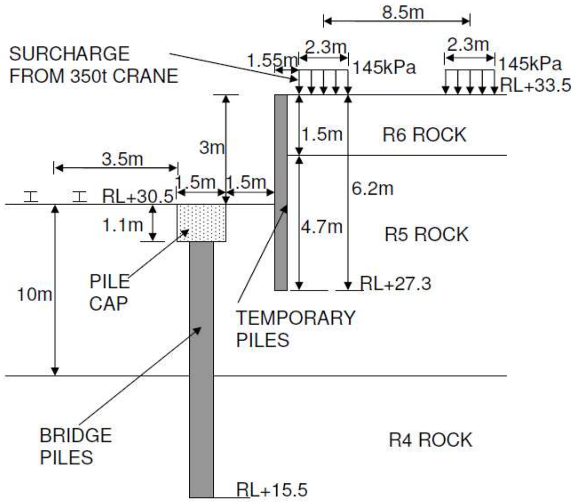

Figure 5. Loading diagram – crane loading

2.3.4 Design check for crane loading

Later on the construction team proposed setting up a 350 t crane near the temporary piles for the erection of BR440 Span 2 girders. The maximum crane outrigger loads were provided as 116 t and the maximum applied ground pressure was 145 kPa. The loading diagram was shown in Figure 5. The calculated FOS against global stability was 1.4 and considered acceptable.

2.4 Construction

The construction sequence involved the following, as shown in Table 3.

Table 3: Construction sequence – Case Study 1

Stage	Description
1	Installation of temporary 600 mm diameter piles
2	Excavation to 1.5 m deep in front of temporary retaining wall
3	Installation of 1050 mm diameter piles for Pier 1, BR440.
4	Excavation and exposure of pile heads
5	Lifting Span 2, BR440

Span 2 of BR440 (Stage 5) was successfully lifted on a night shift on 4 June 2011 (Saturday) during a railway closure.

3 CASE STUDY 2 – DEMOLITION OF AN EXISTING BRIDGE OVER A LIVE RAILWAY

3.1 Site description

As shown in Figure 6, a new bridge BR420 has been built and the old westbound carriageway bridge was to be demolished. Both the new and old bridges were over the live railway. The 3-span old bridge deck comprised a 200 mm thick reinforced concrete slab overlain by a deck wearing course up to 130 mm thick. The bridge also comprised cast in situ abutment, blade wall columns, and spread footings, which were demolished in separate operations. This discussion focuses on the demolition of the deck slabs. The demolition work required possession of railway corridor and had to be finished within a weekend.

3.2 Existing retaining wall

To lift Spans 1 and 2, one of the crane pads was located near to the existing wing wall between the abutments of the old eastbound and westbound carriageway bridges. Based on the 1957 design drawings, the wing wall was a mass concrete wall mainly of a trapezoidal shape, about 5.5 m high and 2.3 m wide at the base (Figure 7). The wing wall did not have a shear key as compared to the adjacent abutment walls, and its stability was critical. It was understood there were no connections between the wing wall and adjacent abutment walls.

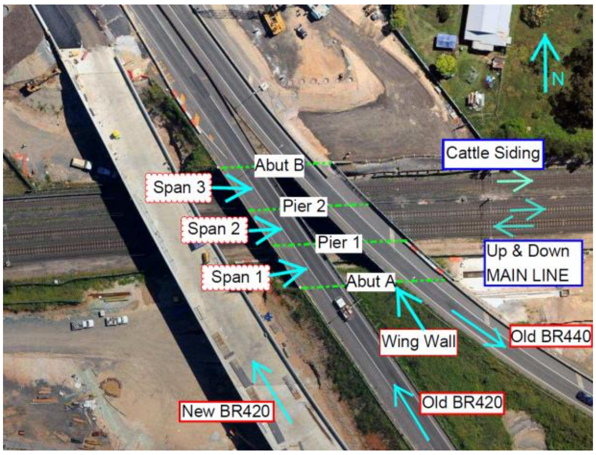

Figure 6. Plan – Case Study 2

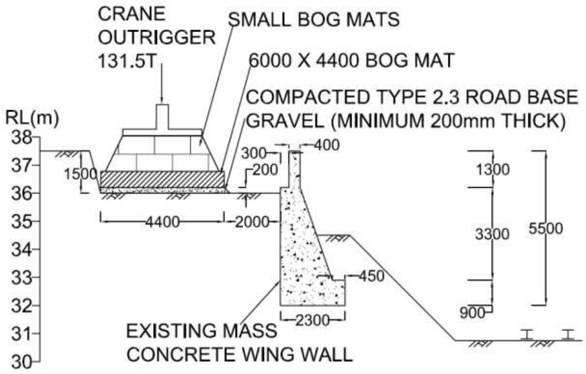

Figure 7. Existing wing wall between abutments

3.3 Geotechnical risks

Before demolition, the wing wall was propped by the existing deck slabs. The main geotechnical risk associated with the demolition would be crane or retaining wall collapse across the railway corridor after deck removal. No direct site investigation information was available for the wing wall. Based on the information from the adjacent new bridges BR420 and BR440, the wing wall was most likely sitting on R6 sandstone. However, the backfill material behind the wall was not known. It was conservatively assumed the fill material had a unit weight of 19 kN/m^3 and a friction angle of 30° and zero cohesion.

3.4 Design options

The maximum outrigger load provided by the crane subcontractor was 131.5 t and the maximum applied ground pressure was around 165 kPa. The bearing capacity of the sandstone foundation was unlikely to be an issue for the wing wall. The wing wall's stability was checked against sliding and overturning. The various cases analysed are summarised in Table 4.

Table 4: Analysed cases – Case Study 2

Case	Surcharge (kPa)	Excavation depth behind wall (m)	Calculated FOS against sliding	Calculated FOS against overturning
1	None	None	1.4	1.5
2	165	None	0.6	0.4
3	50	None	1.0	0.9
4	50	1.5	1.8	2.0

The calculated factors of safety (FOS) against sliding and overturning were 1.4, and 1.5 respectively for the current condition (Case 1, no crane surcharge applied). With the applied crane surcharge of 165 kPa (Case 2), the calculated factors of safety reduced to 0.6 and 0.4 for sliding and overturning respectively, which was considered unacceptable. Therefore two bog mats sized 6 m x 2.4 m were proposed under the outrigger next to the wing wall. With the reduced surcharge of 50 kPa (Case 3), the calculated factors of safety were 1.0 and 0.9 for sliding and overturning respectively, which were still not satisfactory.

In Option 1, it was initially considered that the wing wall should be tied to the adjacent abutments using steel straps. However, this would require access to the railway corridor and it was not practical to install the straps given the limited space and time. This option was therefore discarded. Option 2 was to excavate behind the wing wall to reduce the active pressure. As shown in Figure 7, a 1.5 m deep excavation was proposed behind the wall with the bog mats located at least 2 m away from the back of the wall. As shown in Table 4, the calculated factors of safety for the excavation case (Case 4) were 1.8 and 2.0 for sliding and overturning respectively, which were considered satisfactory.

3.5 Construction

Option 2 was adopted and the fill behind the wing wall was excavated. The exposed material behind the wall was mainly composed of stiff to very stiff clay. Unbound Type 2.3 gravels were placed behind the wing wall. A gap of about 0.5 m was left between the gravel and the wing wall to reduce the earth pressure induced by compaction and the bog mat. The demolition works started on a Friday night, proceeded as scheduled despite drizzling rains, and finished on Monday morning before normal service recommenced.

4 CASE STUDY 3 – DRAINAGE PIPE INSTALLATION UNDERNEATH RAILWAY

4.1 Site description

Figure 8 shows a 900 mm diameter drainage pipe that was to be installed across the existing live railway. The invert of the pipes is located at about 4 m below the railway level. Figure 9 shows a section across the three rail tracks. The pipe installation works had to be finished within a weekend.

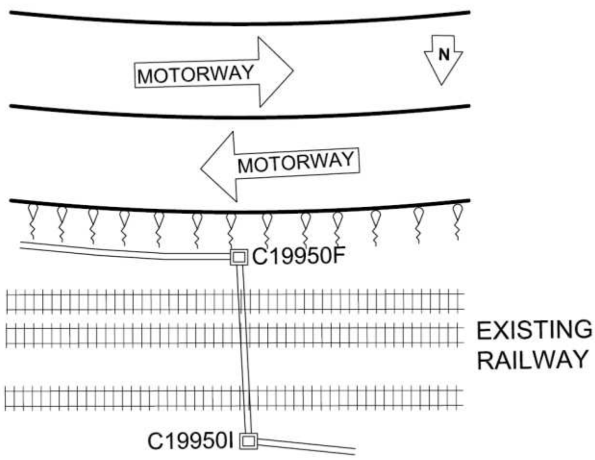

Figure 8. Site plan – Case Study 3

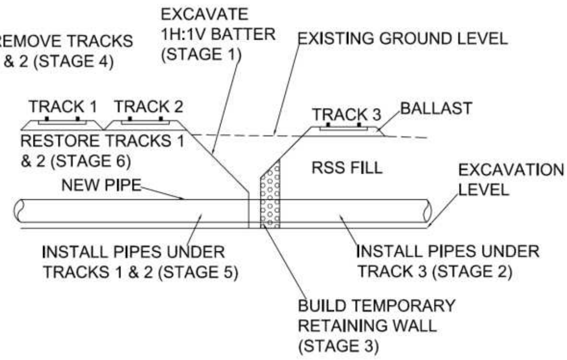

Figure 9. Cross-section – Case Study 3

4.2 Geotechnical risks

There was no geotechnical investigation information available for the site. A test pit to 4.5 m was excavated next to the railway near the proposed drainage pit. The strata at the test pit comprised 1.5 m of firm sandy clay fill overlying 3 m of stiff to very stiff residual clay. To minimise the impact on the railway, one of the tracks was to be kept operational during the pipe installation and the proposed construction sequence was as follows:

Table 5: Proposed construction sequence – Case Study 3

Stage	Description
1	Temporarily remove Track 3. Excavate a 1H:1V batter at Track 2. Trains still running on Track 2.
2	Install pipes underneath Track 3
3	Build a temporary retaining wall using stabilised sand and sand bags; Place reinforced soil fill (RSS fill) and restore Track 3 on a 1H:1V batter
4	Temporarily remove Tracks 1 and 2. Trains operating on Track 3.
5	Install pipes underneath Tracks 1 and 2.
6	Place RSS fill to track level and restore Tracks 1 and 2.

A surcharge of 90 kPa has been applied to the top of the tracks to simulate a train loading. Slope stability analysis was carried out for Stage 1, the calculated FOS was 1.1. Hand-driven soil nails were proposed, the calculated FOS only marginally increased. It was concluded the risk of the proposed operation was too high. To remove this risk it was decided to go ahead with the full railway closure and restore the three tracks after the pipes were installed.

4.3 Construction

Excavation started on a Friday. The material underneath the railway was variable with mainly loose sandy fill, which was inferior to the material found in the test pit. Water was found at the base of the excavation. The railway crossing was finished and the railway was restored by the Monday morning. The section outside the rail corridor was finished two months later.

5 DISCUSSION

All three cases discussed confirm that detailed construction planning is most critical in reducing construction risk next to a live railway. In Case Study 1, the design of the temporary wall had to consider pile installation, pile cap excavation and the crane loading for the lifting the span. The design check and assessments were reviewed during construction. In Case Studies 2 and 3, the constructing works had to be finalised over the weekend. Detailed hour-by-hour planning was required with allowance for a contingency plan in the safe working method statement.

Geotechnical information is usually either not available or difficult to obtain. As construction proceeds, additional geotechnical information is obtained. Although the geotechnical model is constantly updated to reflect the latest understanding of the site, this still cannot eliminate geotechnical uncertainties. For example, the material under the railway in Case Study 3 was inferior to that exposed in the test pit. This is quite opposite to assuming 'the materials must be good because they have been there for decades'. So a more conservative design approach is required to account for the geotechnical uncertainties.

The construction team must realise the level of geotechnical risk involved.

- In Case Study 1, after constructing the adjacent retaining wall, the site condition was better than that assumed in the original design. The construction team proposed omitting the temporary piles to speed up the construction process. The designer assessed the site condition, especially the variability of the weathered sedimentary rocks, and insisted the temporary piles be installed. In fact, weaker extremely weathered claystone was later encountered at the eastern side of the site. The risk of instability would have been unacceptable if the piles had not been installed.
- In Case Study 3, based on the risk assessment results, the geotechnical designer concluded that the proposal by the construction team was risky and a full railway closure was used. If rail track closures were not possible then other alternatives, such as pipe jacking, would have been considered.

6 CONCLUSIONS FOR RISK MANAGEMENT

The following conclusions can be drawn from the three case studies:

(1) Good planning is the most critical part to eliminate or mitigate potential risks. Contingency plans must be included in the safe working method statement.
(2) Additional information will be obtained for design from adjacent site or additional investigation.
(3) To reduce or eliminate risk, adopt a more conservative design approach when there is lack of geotechnical information or there are construction uncertainties.
(4) The construction team must consider geotechnical risks and the designer must evaluate the risks and provide advice on alternative solutions.

7 ACKNOWLEDGEMENTS

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