# Widening of Existing Bridge Abutments – A Geotechnical Approach

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## ABSTRACT

Widening of existing infrastructure is slowly becoming more commonplace but is not yet routine and requires a different thought process. Since the infrastructure was first constructed design codes and acceptable factors of safety are quite often different to the present day which creates a challenge when upgrading or widening the infrastructure thirty or so years later. This paper presents the challenges, options considered and final solution adopted for bridge abutment widening at a major highway interchange. The adopted approach was compliant with the project Scope of Works and Technical Criteria (SWTC) and enabled the widened abutments to wrap around the existing abutments with minimal excavation into the existing abutments, minimising the length and duration of lane closures and the need for temporary retaining walls. The adopted approach can be applied to small and large bridges alike.

## THE PROJECT

The Project involves upgrading a section of a major Australian Motorway. The current four-lane motorway is a critical transport corridor for more than 75,000 vehicles each day. The upgrade primarily includes: widening of over 10 km of the motorway from four to six lanes; modifications to road interchanges including extended on- and off-ramps; construction of off-road cycle / pedestrian facilities; widening of more than five existing bridges; deconstruction and reconstruction of three existing bridges and construction of two new structures.

This paper focuses in detail on a particular Bridge constructed in 1985 and forms part of the southbound widening of a major interchange. The approach adopted at this bridge was adopted for all bridge widenings.

## SITE GEOLOGY

Published geological mapping corroborated with borehole information indicates that this Bridge is underlain by Pleistocene age alluvial plane materials (comprising sand, silt, clay and gravel). To the east of the Bridge site Holocene age undifferentiated coastal plains sediments (comprising mud and sand) is shown to be present. These more recent deposits are underlain by residual soil and a deeply weathered rock profile from the Tertiary age Petrie Formation (comprising sandstone, siltstone, mudstone, and basalt). Boreholes indicate occasional fill overlying up to 5m of alluvium overlying 3 to 5m of residual soil overlying extremely weathered rock. There is a variable weathering profile across the bridge with several boreholes encountering residual soil layers or extremely weathered rock underlying a stronger upper basalt cap.

Groundwater monitoring wells were not installed in the vicinity of the Bridge, however, given the vicinity of the bridge site to the creek and wetlands, groundwater levels have been taken to be at ground level.

## BRIDGE SPILL THROUGH ARRANGEMENT

The widening of the southbound Bridge requires the existing Bridge approaches to be widened and the existing 1v:1h batter slope under the Bridge to be maintained. The 1v:1h batter slope transitions to a 1v:2h slope for the main approach embankment to the Bridge.

## DESIGN CHALLENGES

No one in the team had previously designed a spill through abutment for a bridge widening. The team initially designed a reinforced soil slope for the spill through for a green field site ignoring the presence of the existing bridge. This design would have generated extensive temporary works including temporary sheet pile walls, lane closures and excavation into the existing road carriageway to make space for the lengths of reinforcement required.

The as-built drawings for this Bridge indicated that the spill throughs were constructed with Select Fill immediately behind Terrafix interlocking concrete block facing at a batter slope of 1v:1h. There were no details on what this Select Fill was likely to comprise. This exercise exemplified the importance of good as built drawings for safety in design and construction but also to enable future works and ensure future works were not overly conservative.

1v:1h batter slopes in Select Fill do not as a rule stand up unaided over the long term. The as-built drawings did not show any form of ground treatment, however visual observations indicated that the current Terrafix spill through protection was not showing any signs of bulging, distress or repair. This observation was also supported by the absence of any physical repair or remediation work or records which indicated that there had been a requirement for previous repairs. Therefore, the existing embankments were assumed to be stable in their current state.

There were no Highway Authority standard details for 1v:1h slopes. Under the latest Highway Authority requirements, new spill throughs must not be steeper than 1v:1.5h for which there is a standard detail.

Due to construction sequencing the piles for the bridge widening were installed from headstock level. This meant that piling platforms would be constructed out of rockfill at the spill through locations and would be trimmed to match the spill through geometry.

The geometry of every spill through abutment was different and therefore every spill through had to be analysed separately and a series of cross sections as shown below had to be cut to determine what was happening in 3D. There was no one size fits all solution. This is illustrated in Figure 1**Error! Reference source not found.** to Figure 4.

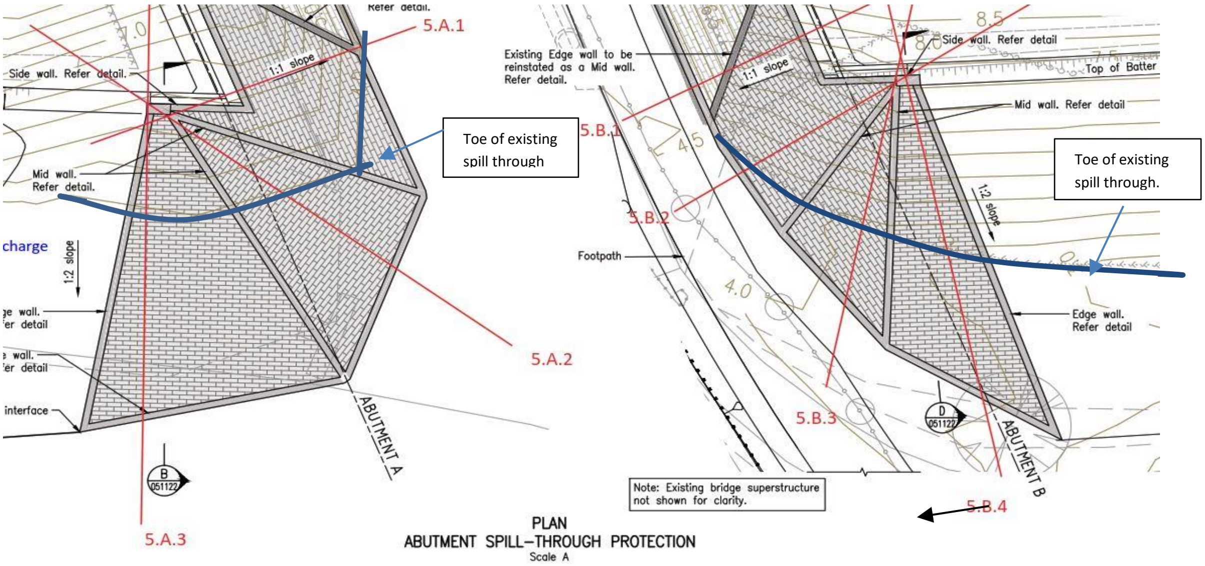



*Figure 1 Widening of the Bridge, Abutment A and B Spill Throughs*

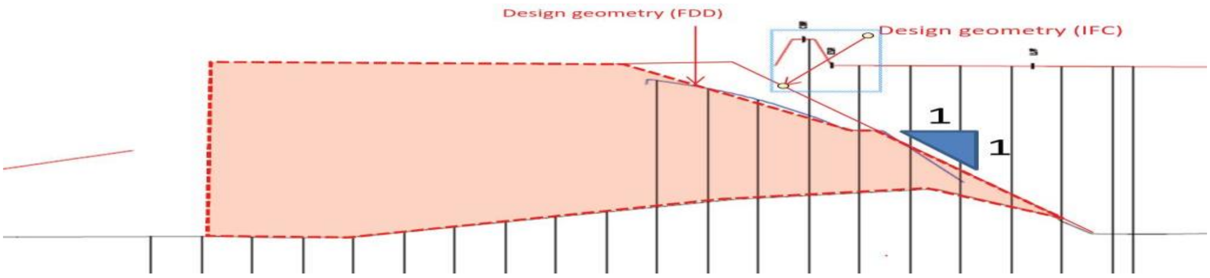



*Figure 2 Section 5.A.1*

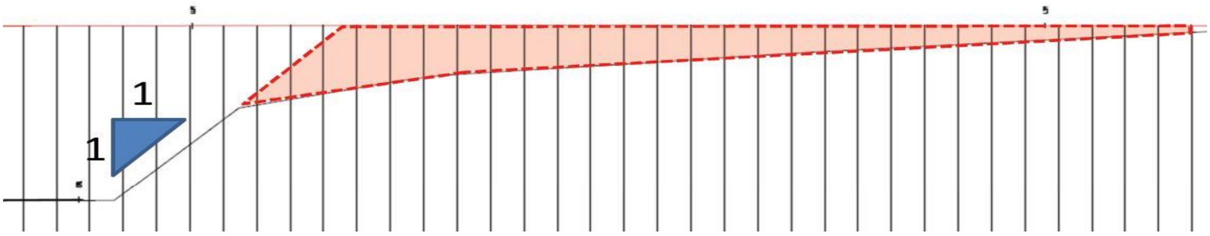



*Figure 3 Section 5.B.1*

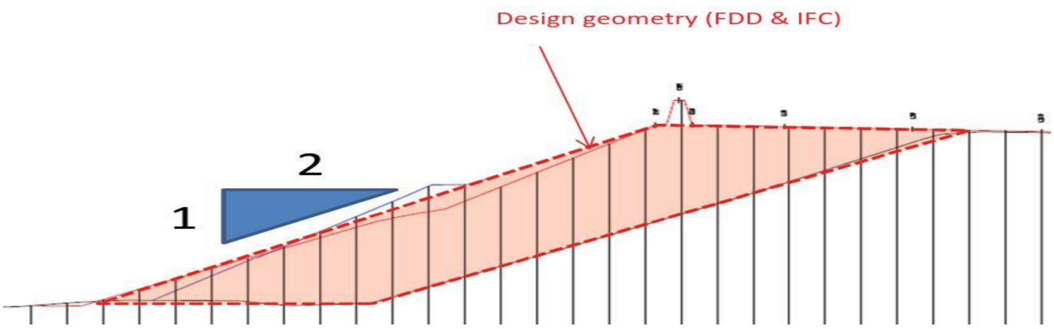



*Figure 4 Section 5.B.4*

## OPTIONS

Preliminary design was undertaken to produce four options for discussion with the Contractor. The original intent of these options was to meet the Scope of Works and Technical Criteria (SWTC) and required factors of safety for both the existing and new widened abutments. These options were only developed for the 1v:1h steepest section of slope. All options considered reusing the rock from the temporary works piling platforms built at the widened spills throughs.

Option 1: Reinforced concrete facing and shear key: Option 1 considered the reinforced concrete facing, acting as a slope stability measure. This option would wrap over and around the existing abutment and require a shear key connection with the concrete facing embedded 1.5m below ground level with 1m depth of remove and replace in front of the toe, to ensure sufficient restraint to the concrete toe/key.

Option 2: Reinforced concrete facing with fixing (rock fill with mechanical anchor). As per Option 1 but with a mechanical anchor to increase the factor of safety for slope stability. A ground anchor was modelled approximately 1m below the underside of the headstock level. A shear key would not be required with this option.

Option 3: Reinforced soil slope with reinforced concrete facing: As per Option 1, except that geogrid is placed between layers of rockfill and a shear key isn't required.

Option 4: Rockfill with soil nails and a shotcrete facing. This option was considered but discounted due to constructability issues with installing soil nails through rockfill.

Options 1 and 3 were worked up into sketches. Option 1 was selected by the Contractor based on ease of construction and this was put forward to the Highway Authority based on the Adopted Design Approach discussed below.

## ADOPTED DESIGN APPROACH FOR 1V:1H TO 1V:1.5H WRAP ROUND

A step change to the design approach was required. As no details of the Select Fill was provided on the as-built drawings, standard Highway Authority parameters (c'= 5kPa, Ø' = 30°) for embankment fill was assumed for back analysis of the existing spill through abutments. This analysis indicated global factor of safety in the order of 1.1 to 1.4 for 1v:1h (steepest section) of slope for the existing spill throughs on the Project and between 1.2 to 1.4 for the Bridge in the long term (drained) condition.

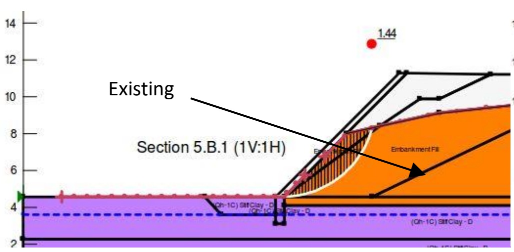
The existing abutment was visually stable and therefore the design team only needed to ensure the new widened section was stable and met the SWTC (FoS of 1.3 and 1.5 for short and long term global stability respectively). The existing spill through would have been globally stable and compliant with the codes around thirty years ago when the bridge was originally built. Therefore, the design team only needed to design the new widened section which would wrap around and extend the existing spill through abutment. This meant there was no need for temporary works or lane closures.

Using the Adopted Design Approach, the factor of safety for the existing spill-through was determined and where the new spill-through was in the zone of influence of the existing spill-through, it was designed to match the existing factor of safety. As discussed above the factors of safety for the existing spill throughs ranged from 1.1 to 1.4, however a minimum factor of safety (FoS)of 1.3 in the long term and short term condition (drained and undrained case respectively) was selected. This provided improvement in some cases to stability of the existing spill through structure. The FoS of 1.3 was selected on the basis that this was an upper bound FoS for the existing infrastructure and provided a consistent approach for all the widened spill throughs. In addition, an FoS of 1.3 is the minimum FoS that the Highway Authority will accept, albeit for the short term rather than long term case. Outside the influence zone of the existing spill-through, the design factor of safety is 1.5 for the long term condition. This meets the minimum Highway Authority criteria and SWTC requirements.

Note that the borehole information for the Bridge indicated stiff alluvium underlying the spill through abutments. This formed the basis of the ground model and was modelled in the slope stability analysis. These ground conditions had limited impact on the requirement for a shear key. The adopted ground model was a fundamental design assumption which required verification on site during construction.

> This approach was accepted by the Highway Authority as compliant with the SWTC and was adopted for all the spill through abutments on the project where the existing spill through slope is at 1v:1h. )

Table 1 to Table 4 below show how the sections were analysed and the resulting factors of safety. The cross sections associated with these section numbers are shown in Figures 1 to 4 above. Figure 5 and 6 give an example of existing and “widened” slope stability analysis for Section 5.B.1 for the drained (long term) case.



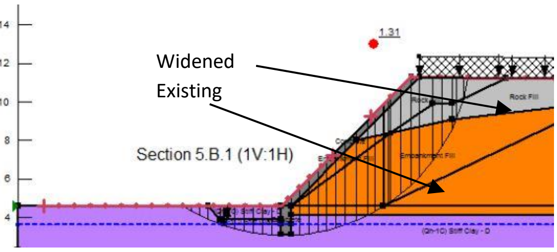




***Figure 5 Section 5.B.1: Slope stability analysis through existing section (refer Tables 1 to 4)***
***Figure 6 Section 5.B.1: Slope stability analysis through widened section (refer Tables 1 to 4)***
***(****Note the widened section is within the zone of influence of the existing spill through. Slip circle goes below shear key*)**

***Table 1 Bridge Abutment A- Representative Sections***

| Section | Description | Influence Zone | Analysed? |
|---|---|---|---|
| 5.A.1 | Raising to full height at 1V:1H | Inside influence zone | Yes |
| 5.A.2 | Widening and transition from 1V:1H to 1V:1.5H | Outside influence zone | Yes |
| 5.A.3 | Widening and transition from 1V:1.5H to 1V:2H | Outside influence zone | No |

*Table 2 Bridge Abutment A – Slope Stability Results*

| Section | Slope Angle | Undrained | | | Drained | | |
|---|---|---|---|---|---|---|---|
| | | Existing Circular | Widened[1] Circular | Widened[1] Non-Circular | Existing Circular | Widened[1] Circular | Widened[1] Non-Circular |
| 5.A.1 | 1V:1H | 1.20 | 1.83 (1.70) | 2.10 (2.03) | 1.17 | 1.33 | 1.64 |
| 5.A.2 | 1V:1.2H | N/A | 1.83 (1.73)2 | 2.21 (1.96) | N/A | 1.50 | 1.79 |

***[1]: Slope stability analysis through widened spill through abutment, refer Fig 6 A and 6A.***
(): FoS for unplanned excavation in front of shear key.

*Table 3 Bridge Abutment B – Representative Section*

| Section | Description | Influence Zone | Analysed? |
|---|---|---|---|
| 5.B.1 | Raising to full height at 1V:1H | Inside influence zone | Yes |
| 5.B.2 | Widening and transition from 1V:1H to 1V:1.5H | Inside influence zone | No |
| 5.B.3 | Widening and transition from 1V:1H to 1V:1.5H | Inside influence zone | No |

***[1]: Slope stability analysis through widened spill through abutment, refer Fig 6 A and 6A.***
(): FoS for unplanned excavation in front of shear key.

*Table 4 Bridge Abutment B – Slope Stability Results*

| Section | Slope Angle | Undrained | | | Drained | | |
|---|---|---|---|---|---|---|---|
| | | Existing Circular | Widened Circular | Widened Non-Circular | Existing Circular | Widened Circular | Widened Non-Circular |
| 5.B.1 | 1V:1H | 1.47 | 1.40 (1.31) | 1.63 (1.57) | 1.44 | 1.31 | 1.70 |
| 5.B.4 | 1V:2H | 1.44 | 1.40 (1.29) | 1.74 (1.50) | 1.61 | 1.61 | 1.72 |

This approach was applied as the spill through graded from 1v:1h at the interface with the bridge abutment to 1v:1.5h as the spill through approached the interface with the adjoining embankment.  For the section 1v:1.5h to 1v:2h the standard Highway Authority spill through detail was applied.

## OVERVIEW OF ADOPTED DESIGN METHODOLOGY

A general overview of the design methodology adopted is outlined below:

A geotechnical model was developed based on the latest exploratory borehole information.  The existing abutments were modelled using the embankment shear strength parameters for earth-fill (c’ = 5kPa and φ' = 30°) as specified in the Highway Authorities geotechnical design standards. The new widened sections were assumed to be formed with rockfill with shear strength parameters of c’ = 0kPa and φ' = 40.

Slope stability analysis for existing and widened sections in short-term and long-term conditions with the aid of commercial software SLOPE/W based on limit equilibrium method (using the Morgenstern-Price analysis) was then undertaken.  This was followed by short term (for unplanned excavation case) and long term stability checks against sliding and overturning using commercial software GEO5 based on limit equilibrium for the critical section. Once this was complete numerical analysis using commercial finite element software 'PLAXIS' was undertaken to calculate bending moment and shear for input into the structural design of the reinforced concrete face (as per AS5100). An additional case allowing for unplanned excavation, to a depth of 10% of the height was also modelled in the short term as per AS5100.3 Clause 13.3.1.

The design team initially used a Mohr Colomb model in Plaxis but found the Soft Soil model to be more accurate as it allowed pore water pressure to dissipate during construction. In addition, the reinforced concrete facing was able to be decoupled from the soil behind it. Bending moments were generated at ground level at the connection with the shear key and about half way down the slope due to the weight of the facing. The soft soil model showed that bending moments were minimal and the final reinforced concrete facing was only on average 10mm thicker (160mm as oppose to 150mm) for the 1v:1h slope than that required for the standard Highway Authority detail for a 1v:1.5h slope.

To achieve a Factor of Safety of 1.3 (long term drained condition) for the 1v:1h section a 1.5m deep shear key was required below the reinforced concrete facing. In addition, material in front of and behind the shear key was required to be stiff clay as a minimum, therefore any soft clay had to be removed and replaced.

## CONSTRUCTION METHODOLOGY, INTERFACE CHALLENGES AND LESSONS LEARNT

The design considered the construction methodology which was critical to the safe construction of a 1.5m deep excavation immediately below an existing bridge structure with live traffic. The original intention was that the shear key would be built ahead of construction of the piling platforms. The piling platform would then be trimmed back to form the widened spill through abutments. As a safety measure, where the abutment is at 1v:1h for safety in design the design team recommended that the Contractor build the shear key slot in no more than 2m lengths with a time limit on the length of time the 1.5m deep excavation could be open for. It was discussed with the Contractor during the design phase that the 2m sections ideally should be precast and dropped into the shear key excavation to minimise the length of time the excavation was open unsupported. There was a possibility that soft marine clay may be encountered in some locations below the spill through abutments and there was concern about instability of the shear key excavation, in particular immediately below the existing bridge abutment.

The first stage of the construction sequence was to construct the piling platforms over the footprint of the widened spill through abutment to enable piles for the widened bridge to be installed. The piling platforms were designed and constructed by the temporary works team. The temporary works teams both on the design and construction side did not consider construction of the shear key as it was part of the permanent works. This lead to the piling platforms being constructed before the shear key which meant that the shear key had to be constructed once the spill through abutments were in place. This was exacerbated, as the construction staff who had been part of the design phase of the project and with whom the construction methodology had been developed were no longer working on the project. The construction challenges around excavating a shear key immediately below a rockfill temporary works piling platform and reshaped to form the spill through abutment had not been considered. The shear key could not be precast due to the lead time required which had not been allowed for in the programme and which would now be uneconomic. Soft marine clay was encountered but due to the limited trench size and a proactive observation process with site spotters, excavation collapse was prevented from occurring. When soft marine clay was encountered, temporary shoring was used to prevent trench collapse and the trench slot was backfilled with cement stabilised sand and allowed to cure for 24 hours prior to re-excavation of the trench. This procedure worked well and proved to be a useful technique.

A further interface challenge was that the shear key was being constructed by the site structural engineers but the majority of the details for the shear key were shown in the earthworks drawings. Despite the cross referencing on both sets of drawings, details shown on the earthworks drawings were missed until clarified by the Construction Phase Services team. On future projects in a similar situation the lessons learnt here would be to put the note regarding the shear key in a separate box and highlight in bold to ensure that important details concerning construction sequencing and location of certain details are not inadvertently missed out.

This design and construction process highlighted the need to have an integrated design which included both the temporary and permanent works. The design and subsequent construction of the temporary works should not be carried out in isolation of the permanent works. Unless the temporary works are deconstructed, similar to the permanent works, good quality construction records should be kept for temporary works for future reference.