

# STRENGTHENING REINFORCED EARTH WALLS – KNOX ROAD DUPLICATION CASE STUDY

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## ABSTRACT

In 1979, The Reinforced Earth Company (RECO) designed and supplied Reinforced Earth walls (REWs) at Knox Road bridge over main Western rail line at Doonside, NSW. The arrangement comprised a pair of abutments spanning over rail. In 1979, the REWs were constructed with two traffic lanes on the Eastern side (Stage 1), and with provision for a further two traffic lanes to be added on the Western side (Stage 2). Between 1979 and 2011, the Western side of the abutments were left in a relatively unfinished condition in readiness for the Stage 2 abutment to be constructed.

The original design of the REWs was carried out on the assumption that when the Stage 2 duplication was to occur, the same abutment beam seat, relative founding level and bridge loading would apply. However, the Stage 2 duplication design, developed in 2011, required a larger abutment beam seat and significantly higher bridge loads than the original design assumed. This meant that there was insufficient earth reinforcement capacity within the existing REW structure to safely support the new bridge loading.

To increase the existing capacity, additional galvanised metallic soil reinforcing strips (RE strips) were incorporated into the REWs. This increase in capacity was achieved by a combination of installing new reinforcing strip connections to existing panels and fabricating new facing panels. The work involved a staged construction process with the initial stage comprising removal and replacement of under-strength panels and retro-fitting with new reinforcing strip connections. The second stage of construction comprised conventional construction of the REW to the new finished surface level incorporating newly fabricated facing panels. Impact on existing road and rail users was paramount during the design and construction of the bridge. It was important to keep the existing road open as much as possible, and to reduce the amount of works in the Rail Corridor. This Case Study presents an innovative method of strengthening existing structures, while reducing costs and construction impacts on both road and rail users in an urban environment.

## 1 INTRODUCTION

Reinforced Earth™ (RE) technology was introduced to Australia in 1974 with the establishment of The Reinforced Earth Company in NSW. RE combines granular backfill with RE Strips and concrete facing panels to produce a coherent gravity structure, capable of retaining significant amounts of earth with a relatively thin facing. The arrangement used on the Knox Road design, both in 1979 and in 2011, was that of a ‘true abutment’, where the abutment beam seat bears directly on the RE fill, and is not supported on piles hidden by the RE mass and founded in competent strata beneath the RE. ‘True’ abutments (refer figure 1) were popular at this time but have become less so in recent years due to the requirements of codes such as AS 5100 Bridge Design (Standards Australia, 2017), which requires that bridges over rail be supported independently of the REW. Within Australasia, true abutments are still very popular in New Zealand and the design and supply of a true abutment has been completed in 2019 for the Gawler Freeway Upgrade project in South Australia.

The Knox Road Bridge over rail was a true abutment built in 1979 with provision for two future traffic lanes to be added at a later stage. These added lanes were designed in 2011, with completion occurring in 2013. Between the initial construction date and the duplication, there was a significant increase in bridge loads, with an increase in relative size and lowering the founding level of abutment beam seat.

A detailed analysis of the structure was undertaken in order to assess whether the new bridge loads could be catered for in the existing structure. Several important factors were found to mitigate the effects of the higher bridge loads. A number of these factors will be presented here:

- The method of design in 1979 involved using a lower bound estimate of the effective angle of friction. At the original design, 32.5 degrees was the design value while in 2011, during the design assessment, the value was

found to be a minimum of 39 degrees following shear box testing. The results of the laboratory testing are appended to this paper, which describes the material as a silty sand. Given that RE technology is a recent innovation in civil engineering, there were no design specifications in place in NSW at that time, such as Transport for New South Wales (TfNSW) Specification R57 (TfNSW, 2020) which is now used in most infrastructure projects in the state.

- The original abutment beam seat width was 2.4 m. In the 2011 design, the abutment beam seat was 3.2 m wide, which significantly increased the footprint of the beam seat on the RE below, thereby reducing the vertical bearing pressure below the 200 kPa threshold, as recommended by RE.
- During the design phase in 2011, a representative sample of RE strips incorporated in the 1979 works were excavated and assessed by RECO engineers. The site inspection revealed that no loss of the galvanic coating, nor loss of thickness of the structural steel section was observed. Therefore, this allowed the design life to be assessed assuming zero loss of section, effectively resetting the service life of the structure to zero years.

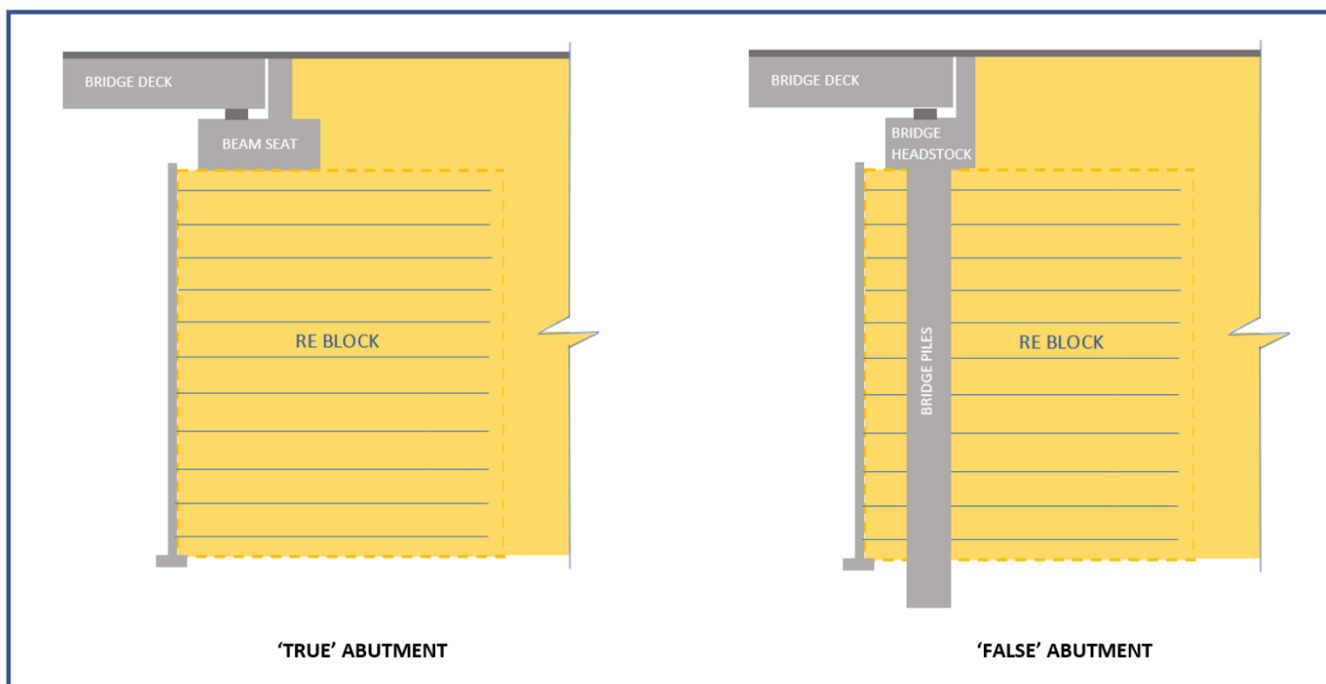


Figure 1: Diagrammatic representation of typical bridge abutment types

## 2 DESIGN METHODS

In order to assess the new bridge loads in 2011, three methods of design analyses were utilised. Firstly, a now-superseded Working Stress design approach that was originally used for the 1979 design. This method applies material strength reduction factors applied in conjunction with the unfactored applied loads. Maximum stress within the RE strips is taken as 0.6 of the nominal yield stress (equivalent to a FoS=1.67) and maximum allowable frictional resistance of RE strips is taken as 2/3 of the theoretical resistance available (equivalent to a FoS=1.5). Note the original 1979 design was carried out assuming the effective friction angle of select backfill  $\phi' = 32.5^\circ$  while the present design assessment is based on a nominal effective friction angle of select backfill  $\phi' = 39^\circ$ . In preparation for this design assessment, several samples of the existing select backfill material were removed and tested for direct shear strength following the AS1289 6.2.1 test methodology (Standards Australia, 2001), returning a favourable result which yielded excess capacity in the existing RE strips.

The second analysis method utilised is based on Limit State design principles to assess RE strips capacity (referred to as the internal stability). This analysis was included only as a comparative design analysis check using current design recommendations and guidelines. This design method would also satisfy the requirements of Australian Standard AS4678 – Earth Retaining structures (Standards Australia, 2002).

The third method was a Limit Equilibrium stability analysis using the program 'Stares'. This program was written at the University of Sydney (Balaam, 1999) for the analysis of REWs and is based on Bishop's simplified method of slices (Bishop, 1955). This method divides the soil above the assumed failure surfaces into "n" number of slices with vertical boundaries. However, the analysis method in Stares accounts for additional terms which take the action of the reinforcement strips into account. Thus, in addition to moment equilibrium between slices, it allows for both pull-out and rupture type failures of each strip. Corrosion is considered by reducing the net area of the strips to account for the predicted reduction in cross-sectional area at the end of the design life. The stabilisation provided by the earth reinforcements are

assumed to be entirely provided by the axial forces generated along their length. Thus, any contribution towards stability due to any shear forces in the reinforcements are ignored. The results of this analysis are expressed in terms of a factor of safety FoS against shear failure along a circular arc. This factor of safety is also assumed to be the factor of safety against rupture and pull-out failure of the reinforcements. This can be expressed as follows:

$$F_{REO} = F_{MAX}/FoS \quad (\text{Equation 1})$$

where,

$F_{REO}$  = Sum of the restraining forces generated in all reinforcing strips that intersect with the base of the slice being considered

$F_{MAX}$  = Sum of the maximum restraining forces that each strip can provide which intersects with the base of the slice being considered. The force for each strip is the minimum of the force required to rupture the reinforcement and the force required to pull-out the reinforcement strip.

The simplified Limit Equilibrium method satisfies the laws of statics, which is a limitation in its' usefulness. Krahn (2001) has described the fundamental shortcoming of Limit Equilibrium methods being the failure to consider strain and displacement compatibility. He further postulates that a method of overcoming this is to use Finite Element computed stresses within a Limit Equilibrium framework, which will result in more accurate analyses within the familiarity of Limit Equilibrium methods. This type of analysis has not been completed for this structure. However, the use of three methods in analysing the structure is considered to provide counterchecks to support the final design conclusions.

For a bridge structure such as this, a minimum factor safety of, approximately 1.5, is sought. The material parameters adopted in the design are presented in Table 1.

**Table 1: Reinforced earth wall design parameters**

Parameter	Value	Unit
Soil Maximum (Minimum) Unit Weight in RE Block	21.5 (20)	kN/m <sup>3</sup>
Soil Unit Weight in General Backfill	21.5	kN/m <sup>3</sup>
Soil Friction Angle in RE Block and General Backfill	39	deg
Soil Friction Angle on Base of RE Block	36	deg
Maximum vertical spacing of RE strips	750	mm
Existing Strip Width (thickness)	60/40 (5)	mm
Existing Strip Minimum Tensile (Yield) Strength	410 (275)	MPa
New Strip Width (thickness)	45 (5)	mm
New Strip Minimum Tensile (Yield) Strength	550 (400)	MPa
Strip Friction coefficient at surface, f*max (f*min)	1.6 (0.80)	-
Strip Corrosion Allowance	0.75 per face 1.50 Total	mm

Further to the above, it is important to note that REWs are typically sensitive to horizontal loading. Where true abutment configurations are used, there are typically significant horizontal loading considered in the design. These loads typically act normal to the abutment beam, outwards towards the centre of the bridge. The original design catered for 22 kN/m SLS across the abutment beam seat, but in 2011 the loading proposed was more than double, at 45 kN/m SLS. The choice of a fixed restraint on Abutment B (Southern Abutment), and expandable arrangement at Abutment A (Northern Abutment) was made by the bridge engineer, Cardno. This additional loading was not able to be transferred to the RE wall, even considering the addition of new straps. Therefore, in order to avoid a very expensive solution, and specifically to avoid a far more extensive removal of the existing REW structure, new RE strips were attached to the rear of Abutment B beam seat to provide an earth anchor restraint to resist the horizontal braking loads. Abutment A did not have any horizontal loads at the abutment beam seat. The increased loads were provided by the primary designer, Cardno. It is unknown why the bridge loads differ so much between 1979 and 2011, although the requirements of AS5100:2004 are considered in the 2011 design. These requirements include consideration of dynamic loading allowances and minimum ultimate horizontal forces acting normal to the bridge. These considerations may not have been included in the original design.

For Abutment B, where a horizontal braking load is applied, the Working Stress and Limit State analyses did not consider this horizontal loading. It is assumed that all horizontal forces are resisted by the earth anchor RE strips attached to rear of abutment beam seat. However, the Limit Equilibrium analysis considers the horizontal braking force and the presence of the RE strips. The capacity of the RE strips to resist pull-out failure governs the design of these beam seat RE strips. The RE Strip pull out capacity is dependent on the frictional characteristics of an inextensible material and the physical characteristics of the soil backfill and is calculated directly from the vertical stresses obtained from the soil mass above and RE strip resistant length taken as the free length behind the abutment beam seat. The number of RE Strips required to resist the applied loading was calculated using basic principles of soil mechanics. At each layer, the frictional and tensile resistance of the straps was calculated for a number of load cases. To determine the number of straps needed across the abutment beam seat, for each load case the critical resistance per strap was divided into the total force acting across the abutment beam seat.

A critical assumption in the analysis is the Soil – RE Strip Friction Coefficient,  $f^*$ . This friction coefficient is found to be above unity at low overburden pressures, being a maximum at the finished surface level and reducing to a value of  $\tan \Phi'$  at about 120 kPa overburden, or 6 m below surface level. Historically, a maximum of 1.5 was adopted by RE entities worldwide, based on equation 2 below. However, as testing data has been obtained over the past 50 years, and as specifications have become stricter in terms of the physical characteristics of select fill, the original values of  $f^*$  adopted would appear to be on the conservative side (Lawson et al, 2013; Jayawickrama et al, 2015). The value of  $f^*$  is related to the characteristics of the soil being used as backfill, but also related to the earth reinforcement being used. The ribbed strips which RE utilises contribute to the increased  $f^*$  observed in the top 6 m of a typical RE wall. Testing of this specific backfill was not carried out during the design or reconstruction stage. For the Abutment B strip design, the selected value of  $f^*$  at 2.8 m below the surface level is calculated using equation 3. The  $f^*_0$  values used in 1979 and in 2011 are consistent, while the  $f^*$  at 2.8 m is based on a  $\Phi'$  of 36 degrees. This is considered prudent given uncertainty in the choice of  $f^*$  under the instantaneous loading which would be expected in the event of a sudden braking force on the bridge.

- $f^*_{\max}$  at surface =  $f^*_0 = 2.1 \times \tan(36 \text{ degrees}) = 1.5$  (Equation 2)
- $f^*_{2.8 \text{ m}} = f^*_0 \times (6 - 2.8)/6 + \tan(36 \text{ degrees}) \times 2.8/6 = 1.14$  (Equation 3)

### 3 EFFECTS OF ABUTMENT BEAM SEAT GEOMETRY AND LOADING

The bridge abutment beam seat was designed by the primary bridge designer, Cardno (NSW) Pty Ltd. The 1979 and 2011 bridge loading and geometry is presented below. The significant increase in the vertical dead load is mitigated by the increased footprint of the abutment beam seat, which significantly reduces the pressure on the RE fill underneath to about 125 kPa for dead loading only, and 185 kPa when both dead loads and live loads are combined, considered to be within the capabilities of a typical REW to sustain.

**Table 3: 1979 vs 2011 abutment beam seat**

Parameter	1979 Value	2011 Value	Unit
Beam Seat Width	2.4	3.2	m
Vertical Dead Load	100	220	kN/m
Abutment Beam Seat Self Weight	120	120	kN/m
Superimposed Dead Load	0	17	kN/m
Vertical Live Load	80	177 (average)	kN/m
Horizontal Dead Load	0	0	kN/m
Horizontal Live Load	22	45 (Abutment B Only)	kN/m
Uniform Distributed Live Load at Road Level	24	22	kPa

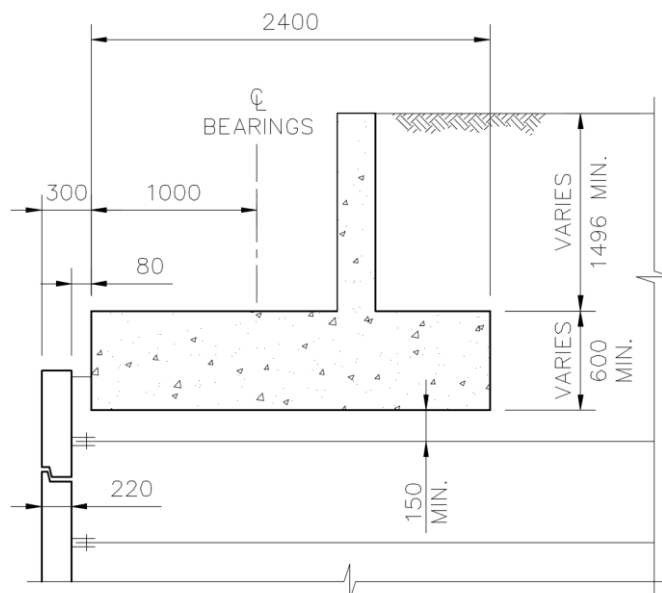


Figure 2a: Diagrammatic representation of 1979 abutment beam seat (not to scale)

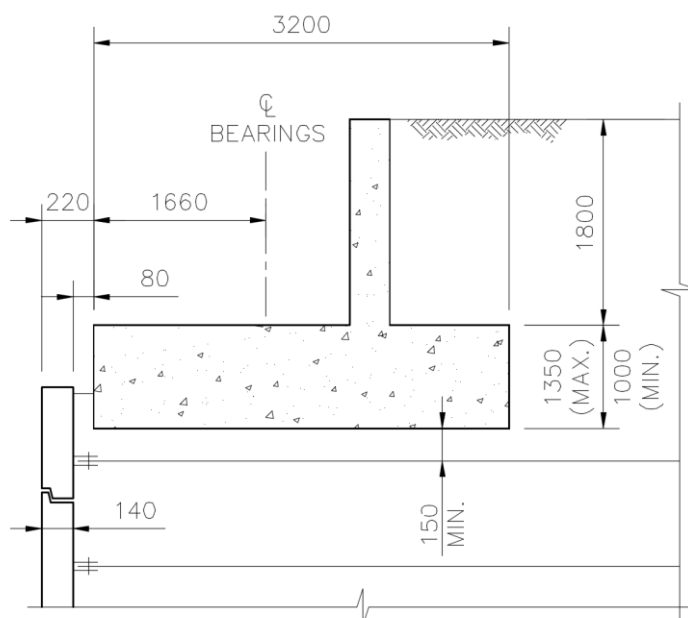


Figure 2b: Diagrammatic representation of 2011 abutment beam seat (not to scale)

#### 4 DURABILITY ASSESSMENT OF EXISTING RE STRIPS

As part of the duplication, an inspection and assessment of the existing buried RE Strips was requested by Blacktown Council. The inspection and assessment were requested by Blacktown City Council with the aim of:

- Establishing the present condition of RE strips.
- Assess whether the existing RE strips remain were corroding in accordance with original design assumptions.
- Assess the anticipated design life of the existing REW sections which are required to be integrated with new sections of REW to support the duplication works.

Steel buried in soil will corrode slowly with time. The REW was originally designed in 1979 assuming a sacrificial allowance in the thickness of the buried RE strips. The design philosophy is to assume that a certain amount of steel corrodes and so this part of the strip section is ignored in the design at the end of the service period.

For the original structure, a total corrosion allowance of 1.50 mm (or 0.75 mm per face) was assumed in the original design. Figure 2 indicates the expected linear rate of loss of thickness over time. This allowance for corrosion is based on predictable and well-defined rates of steel corrosion in soil with chemically controlled parameters. In particular, the pH,

chloride, sulphide, resistivity and sulphate parameters of a soil backfill are controlled where RE Strips and backfill are in contact. Results of testing of three representative samples of existing backfill undertaken in March 1979 indicates the chemical and electrochemical parameters are low and well within the specification requirements of the time. Test results are reproduced below and compared to existing specification limits. The limits below are for REWs built in dry conditions and match the current TfNSW R58 (TfNSW, 2020) specification limits.

**Table 4: Soil chemical results from 1979**

Sample ID	pH	Chloride (ppm)	Sulphate (ppm)	Resistivity at Saturation ohm.cm
15607	8.3	30	25	1435
15606	7.1	18	31	3500
15605	7.5	17	Not Detected	1795
Current RECO Specification Limits	5 - 10	200	N/A – Sulphate test has been replaced by a Peroxide Sulphur test. Previous limit was 1000 ppm prior to 2014 R57 specification revision.	1000

A total of two (2), full-length, 40 mm wide x 5mm thick RE strips were excavated and removed from REW 4 (North Abutment). Each of the samples were assessed for average section thickness and widths, and thickness of galvanizing coating. The condition of these sampled RE strips was then assumed to be representative of all other RE strips within the structure in order to carry out the design analysis. It is important to note that although the initial RE strip sample size is small, the results obtained were consistent with the excellent chemical results from 1979. Furthermore, validation of the design assumptions was possible during the construction stages which included partial demolition of the REWs, and replacement of 1979 RE strips with new ones. Measurements of the galvanic coating carried out with a galvanising thickness meter confirms the average zinc coating thickness is well above the minimum 85µm thickness specified in 1979, with minimum values of 105 µm and 130 µm observed on the two samples respectively, and average values of 130 µm and 149 µm. The fact that the average measured thicknesses are so much higher than the specified minimum is not unusual based on our experience with hot-dipped galvanised products and suggests no degradation of zinc coating occurred between 1979 and 2011/2012.

## 5 RESULTS OF ANALYSIS

Four wall sections were analysed, two per abutment. For the Limit State and Working Stress analysis of the internal stability of the REW, any embedment of the REW was neglected. However, given that there is significant stabilising embedment in front of the wall in the form of a rail embankment, this embedment was considered in the Limit Equilibrium analysis. Results from both the Working Stress and Limit State analysis programs confirmed both Abutment A and B structures in their current condition, assumed to have been built in accordance with specifications, did not have adequate structural capacity to support the new proposed bridge duplication loading in 2011. In particular, the top sections of walls were found to have insufficient earth reinforcement attached to the panel to resist failure in rupture. Additional RE Strips were determined to be necessary to support the increased loading, and this additional capacity was required in the top three layers of RE Strips directly underneath the abutment beam seat. Panel modifications in the form of new physical connections and/or casting new panels were designed and detailed to provide the extra capacity.

For Limit Equilibrium analysis, the calculated Factor of Safety with the incorporation of the additional soil reinforcement capacity and new loads is presented below. A typical output of the Limit Equilibrium analysis is shown below, with the critical failure surface have an FoS of 1.54 in this case, with the new loads applied and panel/RE Strip modifications complete. Results from the 1979 design analysis are not available, and the values shown in the right hand column of Table 5 are the assumed minimum values which would have been sought in 1979.

Table 5: Summary for the limit equilibrium stability analysis to determine the factor of safety

Abutment	REW Height (m)	FoS After modifications and applying new loads	FoS Of Existing Bridge without modifications and with original loads
A	11.25	1.54	-
A	11.57	1.53	-
B	11.25	1.56	-
B	11.55	1.61	-
A/B	11.25	-	1.50
A/B	11.795	-	1.50

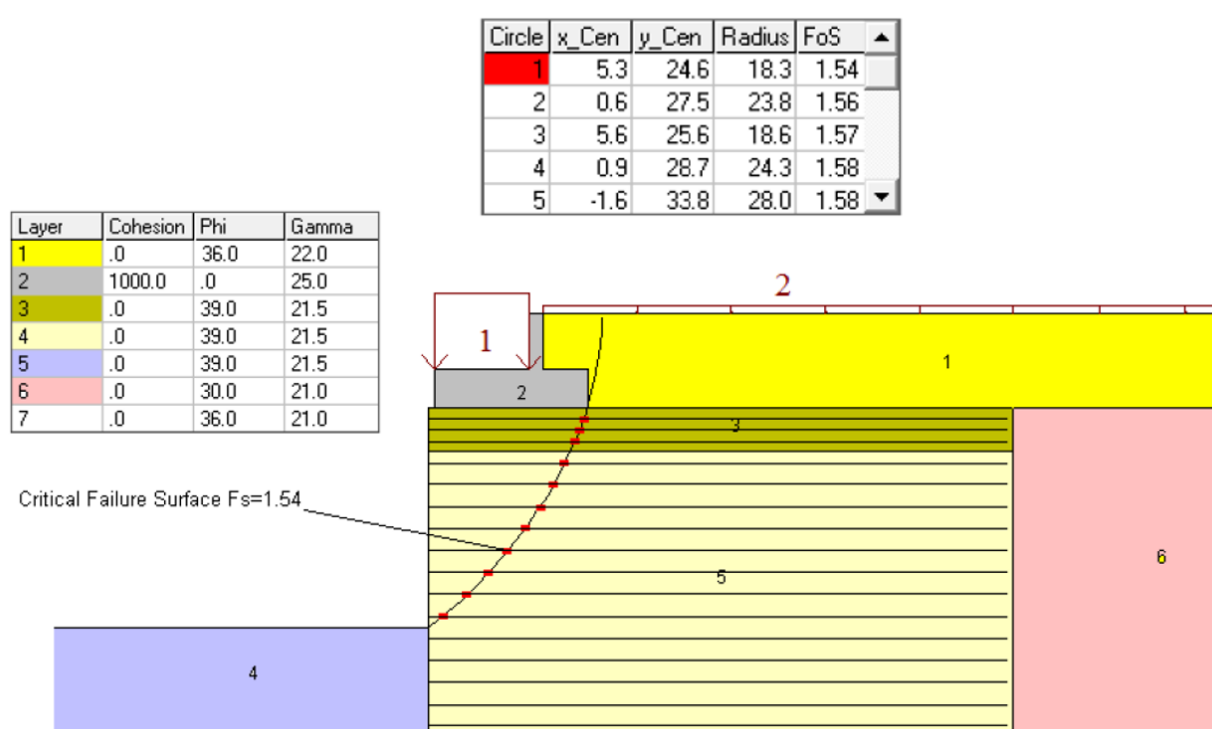


Figure 3 Typical limit equilibrium analysis output

## 6 CONSTRUCTION WORKS

The construction sequence after the 2011 design of the bridge extension can be briefly depicted as follows.

- Step 1 - Excavation and dismantling of redundant top RE strips from the original provision

After carrying out the design of the new section of the bridge and RE walls, the majority of the existing panels (including the strips attached to them) that do not comply with the new design have to be removed. Dismantling of facing panels and RE strips is to be carried out in a top-bottom sequence down to the level where the strips are found satisfactory with the new design. Removal of panels and RE strips implies removal of the backfill as well.

- Step 2 - Strengthening on some of the existing panels to remain in the wall (additional RE strips and post-fixed connections to panels to be added). Refer to Figure 9 for post-fixed connection details.

Due to the staggered arrangement of the facing panels, there are panels that cannot be dismantled and will remain in the wall. However, there will not be sufficient cast in tie strip connections for all the additional strips required by the new design. Therefore, new post-fixed connections need to be added to the existing panels to remain in the wall. Both the remaining tie strips and the additional post-fixed ones will provide the facing panels with the required amount of connections for the RE strips numbers required by the new wall design.

- Step 3 - Installation of the new facing panels & RE strips and backfilling to the underside of the new beam seat

This stage of the re-construction does not differ from a standard RE wall construction sequence. New facing panels are installed and RE connected to them in a sequence of backfill layers from bottom to top (underside of the new Beam Seat).

- Step 4 - Construction of the new bridge beam seat

Forming of the new bridge beam seat as per new bridge design details. Installation of the new bridge beams and construction of the new bridge deck can be carried out right after the construction of the new beam or at a later stage.

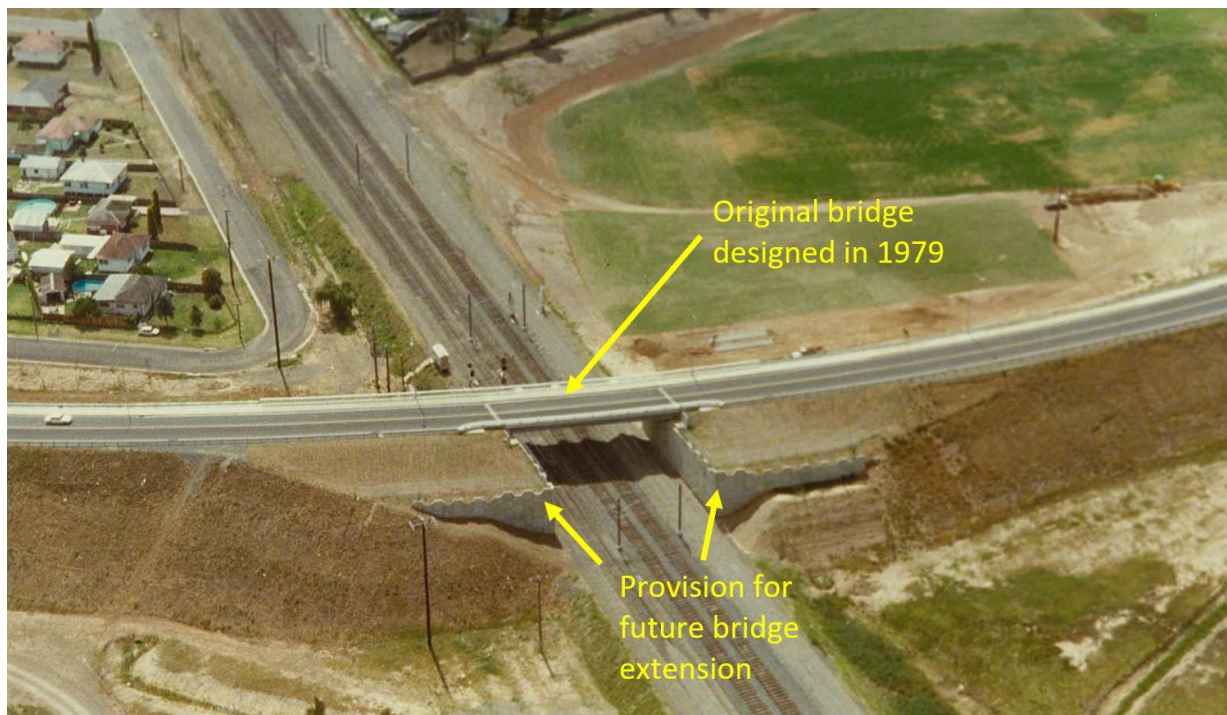
- Step 5- Installation of new panels above the underside of the new beam seat.

As step 3, this stage of the re-construction does not differ from a standard RE wall construction sequence and so, another sequence of new facing panels are installed and RE strips connected to them completing the backfilling of the wall up to the new road level.

One can refer to Figures 6, 7 and 8 below for a graphic description of the construction steps.



**Figure 4: Bridge abutment in 2011 – view 1**



**Figure 5: Bridge abutment shortly after completion in 1979/80 – view 2**

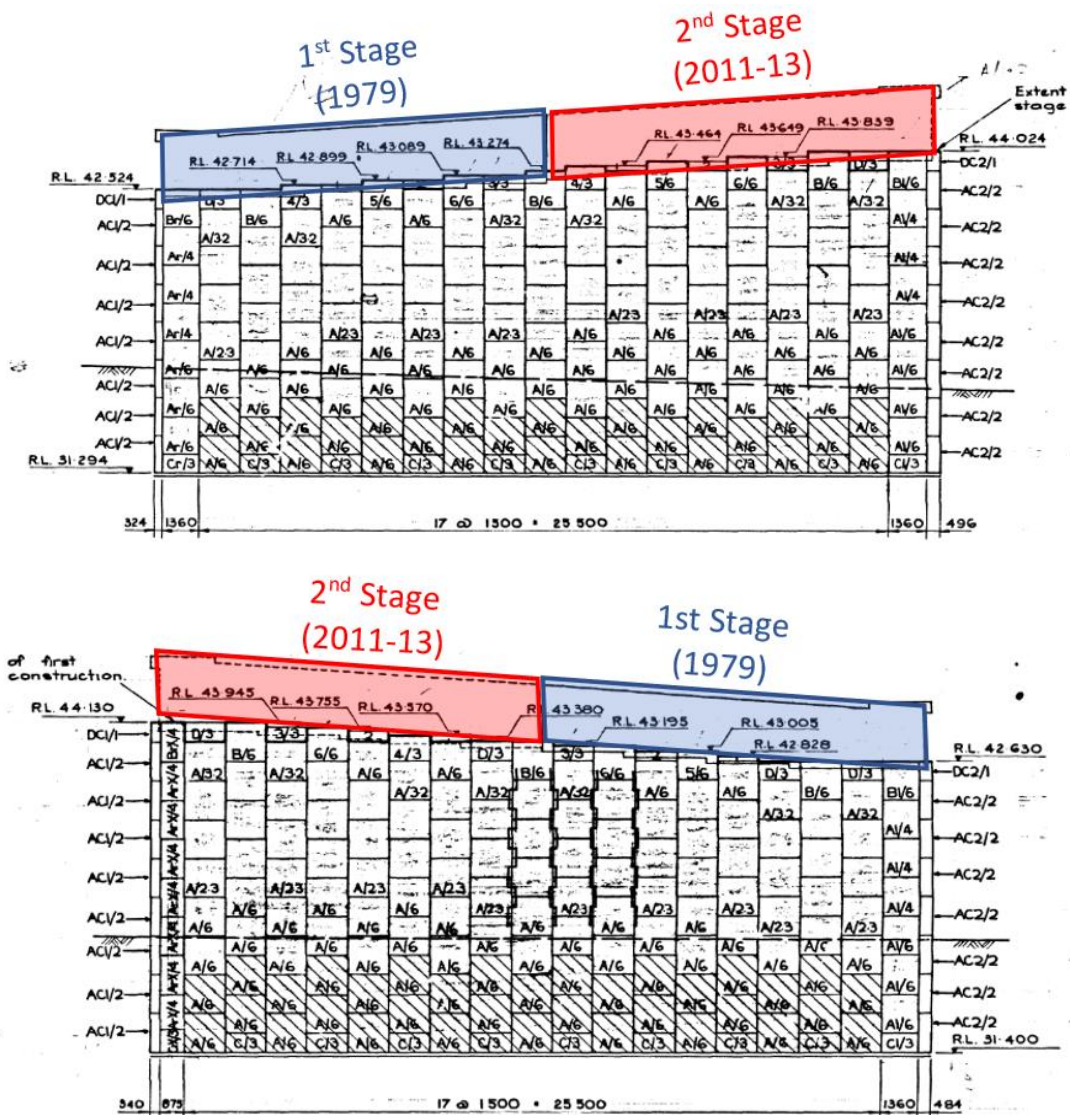


Figure 6: Abutment walls (rear view) - Original 1979 Design drawings

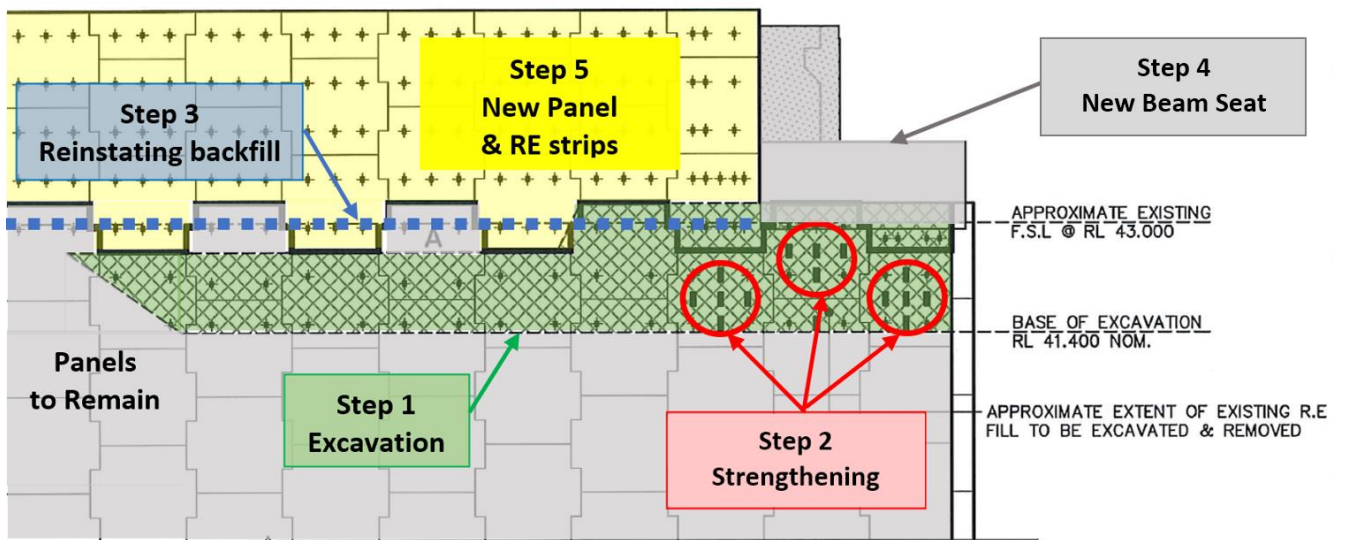


Figure 7: South West Wing-wall (extract from 2011 Design drawing) - Construction Steps

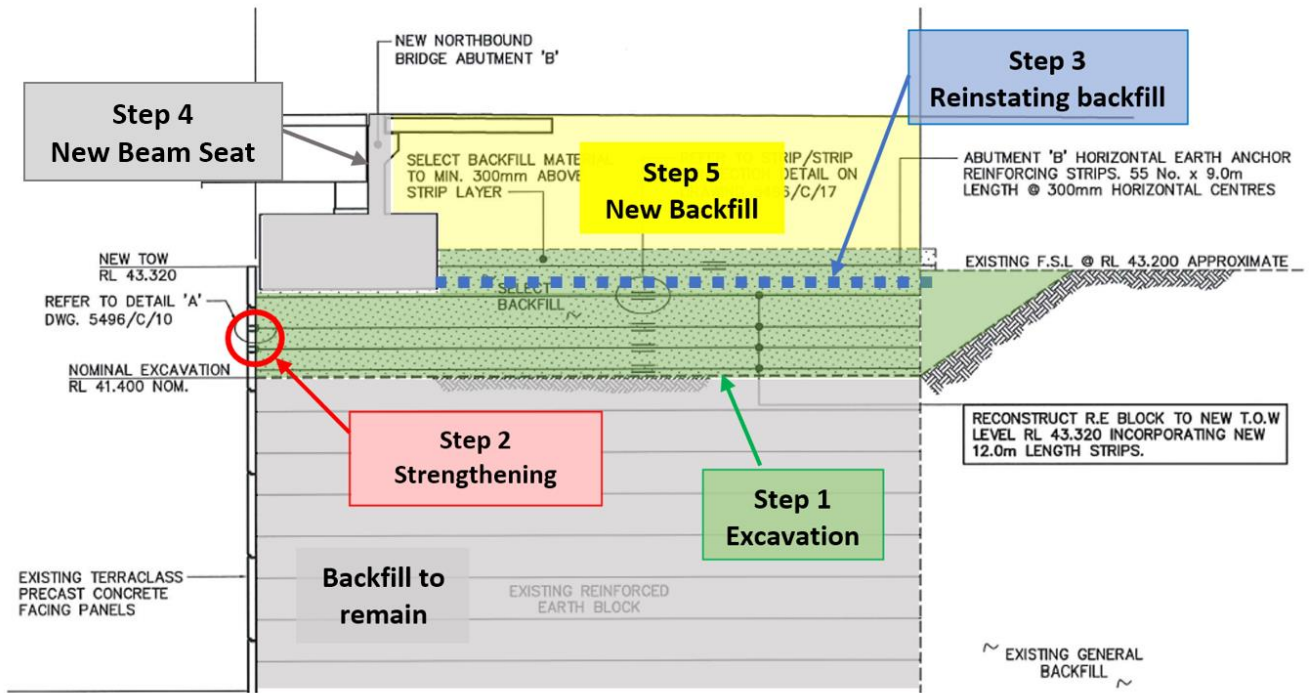


Figure 8: North Abutment (extract from 2011 Design drawing) - Construction Steps

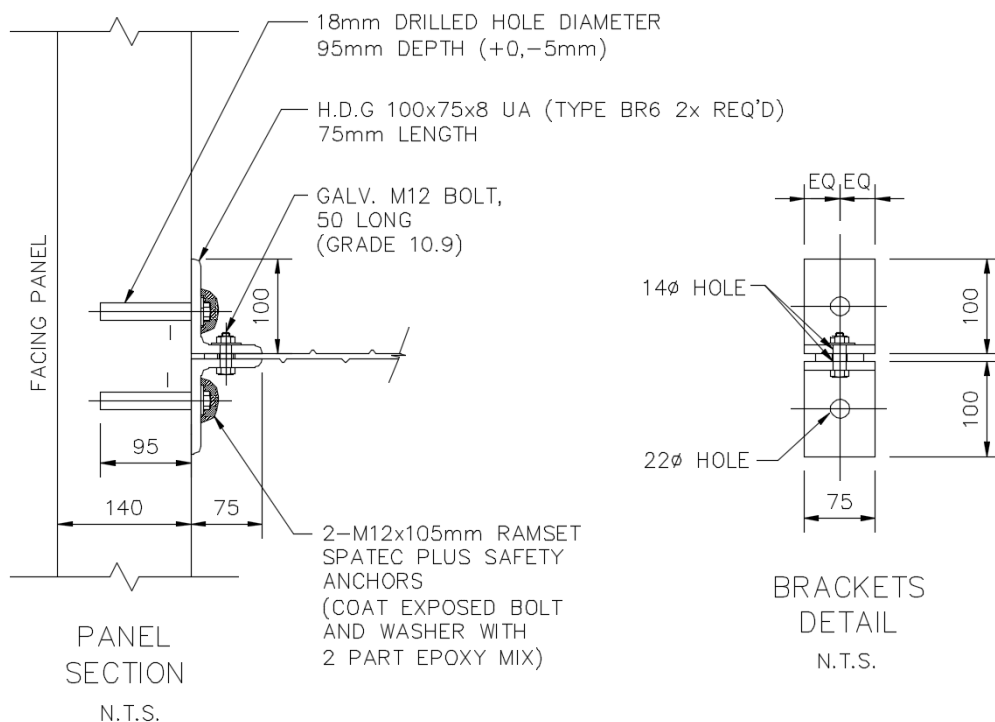


Figure 9: Panel Strengthening Detail – Additional Panel/RE strip connections



**Figure 10: New reinforced earth walls and bridge, completed in 2013**

## 7 CONCLUSIONS

The REWs at Knox Road bridge over main Western rail line at Doonside, NSW, were originally designed and built in 1979 with provision for future duplication. In 2011, the duplication was formalised but a significantly higher loading and beam seat arrangement was put forward by the bridge designers. RECO completed a detailed design and durability analysis in order to facilitate the duplication. This approach combined Working Stress, Limit State and Limit Equilibrium analyses in order to justify the new arrangement. Further optimisation work involved the utilisation of RE Strips on the new bridge beam seat in order to provide horizontal restraint to the significant bridge loads. The result yielded significant time and cost savings for Blacktown Council, and all work on the REWs was fully completed within the footprint of the walls in a populated urban environment, and outside of the adjacent rail corridor. Further to this, a critical contributor to the success of the project was the high-quality backfill which was used at initial construction. This material not only provided excellent shear strength, but also provided superb resistance to chemical attack. The use of this material allowed for the design life of the structure to be fully reset in terms of corrosion, but also contributed to the relatively minor adjustments required to justify the new design loads.

## 8 REFERENCES

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## APPENDIX A

**SMEC TESTING SERVICES Pty Ltd**

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 Wetherill Park, NSW 2164



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**DIRECT SHEAR BOX TEST REPORT**

PROJECT: Material Testing

PROJECT No: 18383

CLIENT: The Reinforced Earth Pty Ltd

REPORT No:11/1240

CLIENT ADDRESS:Level 2, 20 Georg Street Hornsby NSW 2077

DATE REPORTED:22/10/11

SHEET 1 OF 2

SAMPLING METHOD:Sample Supplied By Client

TEST METHOD:AS1289.6.2.2, 5.1.1, 2.1.1

DATE OF TEST:14/10/2011

TESTED BY:M.B.

STS No:98783

SAMPLE No:2

LOT No:Not Known

REQUEST No:PO12270

MATERIAL DESCRIPTION: Silty Sand, brown

DEPTH:Not Known

LOCATION:Knox Road

TEST TYPE: Peak / Remoulded

STRAIN RATE:0.218mm/min

SHEAR BOX SIZE: 113mm Dia

TYPE OF SPECIMEN: Disturbed

STAGE No:

STAGE 1

STAGE 2

STAGE 3

MAXIMUM DRY DENSITY( $t/m^3$ ): 1.87

OMC (%): 16.7

NORMAL STRESS(kPa):

104.1

202.0

300.9

TARGET COMPACTION (%): 100.0

INITIAL MOISTURE CONTENT(%)

16.1

16.1

16.1

MAXIMUM PARTICLE SIZE(mm): 4.75

INITIAL DRY DENSITY( $t/m^3$ ):

1.88

1.88

1.88

AVERAGE EFFECTIVE ANGLE OF FRICTION(deg.)= 39.0

INITIAL DENSITY RATIO(%)

100.0

99.9

99.8

CORRELATION COEFFICIENT( $r^2$ )=0.95

INITIAL HEIGHT(mm):

30.00

30.02

30.03

REMARKS:SHEAR STRESS IS CORRECTED FOR AREA CHANGE

CONSOLIDATED DRY DENSITY( $t/m^3$ ):

1.92

1.94

1.92

AVERAGE EFFECTIVE ANGLE OF FRICTION AND CORRELATION

CONSOLIDATED HEIGHT(mm)

29.41

29.03

29.34

COEFFICIENT CALCULATED BY LINEAR REGRESSION ANALYSIS

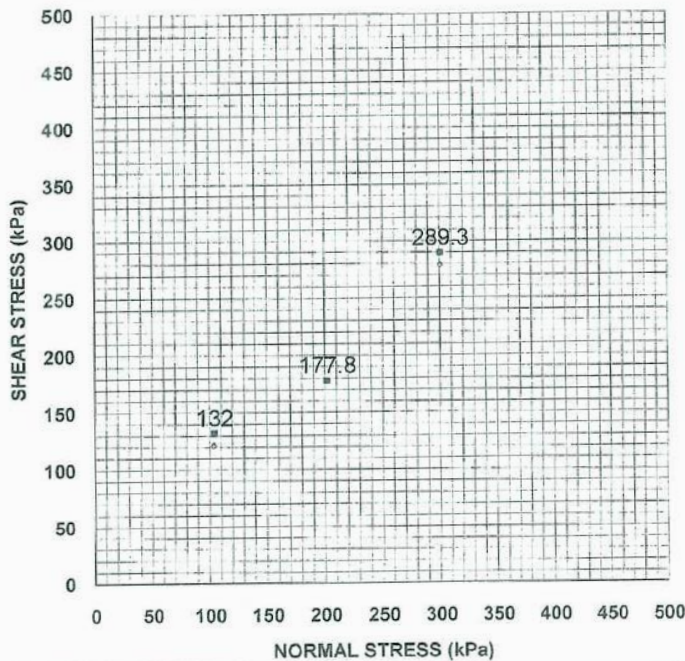
FINAL MOISTURE CONTENT(%)

19.9

19.4

19.1

UNCORRECTED NORMAL STRESS VERSUS CORRECTED SHEAR STRESS AT PEAK SHEAR STRENGTH



Approved Signatory: *[Signature]*  
 James Hughes - Senior Geotechnician

**SMEC TESTING SERVICES Pty Ltd**

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**DIRECT SHEAR BOX TEST REPORT**

**PROJECT:** Material Testing

**PROJECT No:** 18383

**STS No:**9878B

**REPORT No:**11/1240

**SAMPLE No:**2

**CLIENT:** The Reinforced Earth Pty Ltd

**DATE REPORTED:**22/10/11

**LOT No:**Not Known

**CLIENT ADDRESS:**Level 2, 20 Georg Street Hornsby NSW 2077

**SHEET 2 OF 2**

**REQUEST No:**PO12270

