DESIGN AND CONSTRUCTION OF A CEMENT STABILISED-SHORED REINFORCED SOIL WALL

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ABSTRACT

This paper presents the design approach, methods of analysis, material testing and construction of a Cement Stabilised-Shored reinforced soil wall (RSW) for Hills M2 Upgrade project in Sydney, NSW. Particular attention was given to the deformation modulus of the backfill material and stress conditions within the RSW that could promote cracking.

1 INTRODUCTION

The NSW Government announced the approval of the Hills M2 Upgrade on Tuesday, 26 October 2010. The Hills M2 Upgrade widens the existing motorway generally between Windsor Road, Baulkham Hills and Lane Cove Road, North Ryde including delivery of four new ramps to improve access to the motorway.

The Hills M2 Motorway plays a key role in Sydney's Orbital network linking the north west region to the lower north shore and Sydney's CBD. It is a key road freight and commuter route and connects the major employment hubs of Macquarie Park and Norwest Business Park. Construction began in January 2011 and is completion is estimated for early August 2013.

Due to site constraints (e.g. existing sedimentation basins, driveways, boundary restrictions etc.), there were a number of locations throughout the Hills M2 Upgrade project alignment where limited space was available for the extension of the existing relatively high retaining walls which, in most cases, were reinforced soil walls (RSW).

Construction of RSWs is often the preferred retention solution in road works as it involves a fill strengthening process that is considered very cost effective. The current industry practice typically adopts a minimum RSW reinforcement length (L) equivalent to approximately seventy percent of the design height (H) of the wall, i.e. $L = 0.7H$. However, at some locations along the Hills M2 Motorway, the use of conventional RSWs was not feasible as the available space was limited to only 0.3H to 0.5H. In addition, the transfer or application of new loads to the existing Hills M2 RSWs was considered to be of high risk as movement of these RSWs had been observed under current loading.

Constructability issues were also identified in relation to the other solutions. For instance, one of the concept designs considered a hybrid retaining wall where the upper section of the wall consisted of a RSW limited to 8 m in height and a lower section comprising anchored precast panels. The total height of this hybrid wall was limited to 17 m. The limited available width resulted in anchors inclined at 45° or steeper in order to avoid cutting the geosynthetic reinforcement within the existing RSWs which had web type layout (Paraweb). As a result of the steep anchor inclination a structural facing would be required to accommodate the large vertical loads applied by the anchors, comprising precast concrete columns with plan dimensions of 1.2 m x 1.0 m and spaced at 3 m centres. It was also initially anticipated that the lower layers of steel reinforcement within the proposed upper RSW would be connected to the facing panels of the existing RSWs. However, during Detailed Design phase (DD), the design team raised concerns about the integrity of the existing RSW as significant movement of these RSWs had been observed. In addition, the construction team also identified difficulties in relation to the installation of the proposed steeply inclined anchors.

As a result, an alternative solution was required and a design procedure was developed that could consider the stabilising effect of the existing RSW with regard to the reduction of lateral loads acting on the new RSW. Under such conditions, Berg *et al*. (2009) presented two design approaches for RSW:

i) Shored Mechanically Stabilised Earth (SMSE) walls when excavation and shoring in steep terrains would be required to establish a flat bench to accommodate the soil reinforcements with a minimum length greater than 2.5 m or 70% of the height of the wall. In this case, shorter reinforcements are possible if the shoring system is accounted for (Figure 1).

ii) Stable Feature Mechanically Stabilised Earth (SFMSE) walls for new walls built in front of apparently stable features such as a rock face.

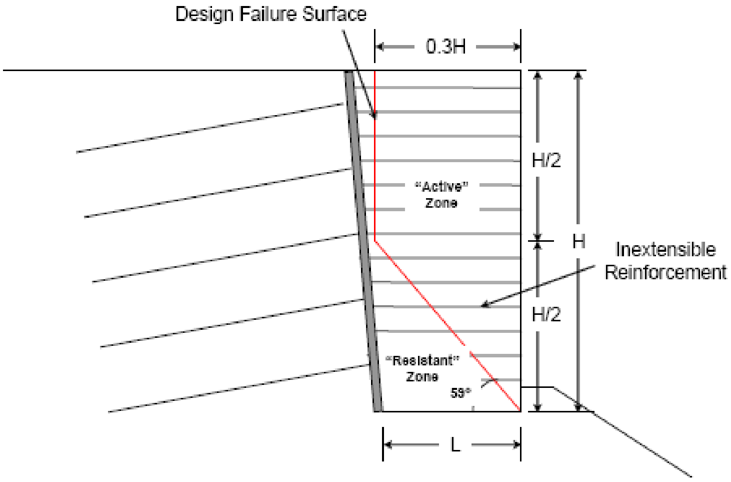


Figure 1: Sketch of a shored RSW (or SMSE) with inextensible reinforcements (after Berg *et al*., 2009).

The above concept of Shored RSWs, with ratios as low as 0.3H, was considered an attractive solution. However, this method was developed for low volume roads and not originally recommended in urban areas for roadway widening applications. The main reason is the relatively high risk for tension cracks at the interface between the existing wall and the new RSW under dynamic effects of traffic loading, referred to as a trenching mechanism. In addition, the design approach was mainly developed for static load conditions or in areas where the seismic horizontal accelerations at the foundation level are less than 0.05g.

In order to reduce the risk of traffic loading induced tension cracks between the new and existing walls and for seismic horizontal accelerations greater than 0.05g, an alternative shored RSW with cement stabilised backfill (CS-SRSW) was investigated and a new design procedure developed. The initial intent of the design was to use site-won crushed sandstone stabilised with cement as backfill material. Particular attention was given to the deformation modulus of the stabilised backfill material and stress conditions within the RSW that could promote cracking.

2 DESIGN PROCEDURES AND ANALYSIS METHOD

The use of cement stabilised soil walls is not a new approach in geotechnical engineering. For example, as part of the original Hills M2 Upgrade project, cement stabilised sandstone was used to form a gravity retaining wall up to 22 m high between Pennant Hills Road and Oakes Road (Chandler and Palmer, 1999). Another Australian example of the performance of a retaining wall with cement stabilised soil is presented by Ismail (2005). However, the key differentiator and innovation of the current application is perhaps the slenderness of the designed walls, with width to height ratios of less than 0.4, and the combination with soil reinforcement techniques. Several challenges, as described below had to be overcome before acceptance of this innovative design.

Perhaps, the first question to be addressed by the design approach is the assumed behaviour of the wall: flexible or rigid-monolithic? Conventional RSW are considered to be flexible, which would be even more pronounced at $L/H < 0.4$. However, the cement stabilisation will play a role in the deformational behaviour of the backfill, and, in fact, that was the main objective of the stabilisation, i.e. to address the "trenching mechanism" of the original SMSE concept.

As a starting point it was considered that the CS-SRSW could behave as a monolithic gravity wall due to the relatively high modulus of elasticity (E >1000 MPa) targeted for the stabilised fill even at low cement content (4% to 5%). This assumption was also based on similarities with the design of retaining walls with cement stabilized soil and RSW concrete panels as reported by Derek and Crockford (1991). In their design, the main objective of the reinforcement was to hold up the concrete panels, therefore enabling the use of shorter reinforcement length than typical RSW as it was not considered for internal stability. Derek and Crockford (1991) study included numerical analyses, physical modelling by centrifuge testing and a full scale of trial wall up to 7 m high and 200 m long.

Despite the assumption of a rigid-monolithic behaviour, cracking of the stabilised material was a concern during the design phase. In order to reduce the potential for cracking initiation under design loading conditions, the design procedure aimed to control the stresses within the stabilised soil mass to within the lower range of the elastic behaviour of the stabilised material. This was initially based on the concept of cracking initiation of intact rock samples in laboratory testing. In addition, according to DoT (1986), if a cemented material is subjected to repetitive (dynamic) loading within its elastic range and is not loaded beyond the stress at which microcracking begins, then the material will likely remain intact for an indefinite period. It is also stated in DoT (1986) that, based on laboratory tests on cement stabilised materials, microcracking apparently only initiates for stresses beyond approximately 35% of the unconfined compressive strength (UCS) of the material. Cracking due to drying shrinkage and thermal effects were also considered limited due to both low cement content and low water content for the stabilised material, with cement contents targeted at 4% to 5%.

However, it was also recognised that there could still be potential for cracking to occur in the long term, particularly if associated with material degradation and changes in moisture content and considering an intentional conservative approach. As a result, a second design approach was considered where the cement stabilised mass was assumed to be fully cracked, thus, behaving like a blocky medium with more similarities to a flexible RSW where the soil reinforcement plays a more significant role.

For both approaches discussed above, the following loading conditions were assumed: (a) live (traffic) load of 20 kPa acting on the wall; (b) horizontal seismic acceleration coefficient kh = 0.14; (c) vertical seismic acceleration coefficient kv = 0.07; and (d) maximum impact load I = 17 kN/m on the traffic barrier located on top of the CS-SRSW.

A minimum factor of safety (FS) of 2 under static loading and a FS of 1.2 under seismic loading were targeted for all mechanisms under analyses, except for bearing capacity where a minimum FS of 3.0 was targeted. In general, the proposed CS-SRSWs were to be constructed on a concrete platform founded on Class IV Sandstone (rock class as defined by Pells et al, 1998) or better.

2.1 CEMENT STABILISED BACKFILL SUBSTANCE PARAMETERS

During the design stage and before any laboratory test had been carried out, a cement content between 4% and 5% was assumed for the stabilised material. This value was based on the results reported by Chandler and Palmer (1999) which showed UCS values of 4.3 to 8.4 MPa for cored samples taken during construction of the cement stabilised wall of the original Hills M2 construction with a cement content of 4.5%. Chandler and Palmer (1999) also reported UCS values of 3 MPa for laboratory results on samples compacted at 98% of the standard maximum dry density within ±2% of the optimum moisture content.

Based on the testing of different soil types, DoT (1986) demonstrated that for well graded sands and gravel UCS values above 3 MPa could in general be achieved with cement contents in the vicinity of 5% (Figure 2).

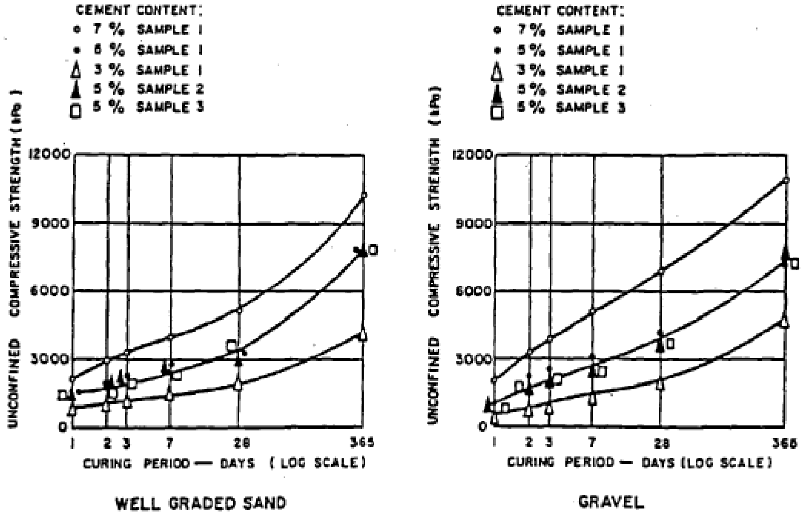


Figure 2: Strength variation with time for cement stabilised well graded sands and gravel (DoT, 1986).

The particle size distribution of the crushed sandstone samples from the Hills M2 Upgrade project indicated low fines content and gravel characteristics for all samples. As a result, an UCS value of 3 MPa was considered appropriate and achieveable for the cement stabilised material for the current design.

As discussed above, an important behaviour anticipated for the stabilised material was a relatively high stiffness. In the absence of test data, the Young's modulus of the cement stabilised material was estimated based on the UCS of the material according to AS 5100.5 – Bridge Design (Part 5: Concrete) by:

$$E = 0.043\rho^{1.5}\sqrt{f_{cm}} \qquad (1)$$

where ρ is the material density (kg/m3) and f_{cm} is the UCS (MPa) of the material

The estimated Young's modulus for the cement stabilised material was approximately 6600 MPa. Although this value is typical for Roller Compacted Concrete (RCC) and later field core samples gave similar moduli, this equation was considered to give somewhat high values. In addition, even if this estimate was assumed reasonable it only provides estimates for the substance modulus that does not take into account fractures or discontinuities so it would still have to be downgraded.

Indraratna (1990) stated that a "synthetic rock" will simulate real rock behaviour if the Poisson's ratio, friction angle and uniaxial strength ratio, σ_c/σ_t (i.e. compressive/tensile strength) are similar. As a result, it was assumed that the cement stabilised sandstone would present similar behaviour to that of a weathered sandstone rock. An alternative approach, based on rock mechanics correlations was then adopted (Deere, 1968):

$$E = MR \times UCS \qquad (2)$$

where MR is the modulus ratio, typically varying from 200 to 1000 and UCS is uniaxial compressive strength (MPa). A modulus ratio MR of 350, typical for sandstone, was adopted for the cement stabilised material which is somewhat lower than the value adopted by Chandler and Palmer (1999) for the existing Hills M2 cement stabilised wall. The adopted modulus ratio seems to yield consistent values with those obtained by Derek and Crockford (1991) of up to 875 MPa for a cement stabilised sand with 7% cement content.

The adopted geotechnical design parameters are presented in Table 1. Considering the same select fill material as that used in conventional RSWs, a minimum friction angle of 34º was assumed for the cement stabilised sandstone. The value of peak cohesion was then back-calculated from both friction angle and UCS values. A residual cohesion of 10% of the peak value was adopted to simulate a softening behaviour due to cracking. In addition, low bound values were also considered to assess the impact of potential mixing problems during construction and which, to some degree, gave strength parameters closer to the blocky medium approach. The adopted low bound parameters were similar to a sandstone Class IV type rock with a Geological Strength Index (GSI) of 45, if the fractures are taken into account in the failure criterion as an equivalent continuum using a Generalised Hoek-Brown material model (Marinos and Hoek, 2000).

Table 1: Design parameters adopted for the cement stabilised sandstone (monolithic approach).

Range	UCS (MPa)	E (MPa)	Poisson ratio	Friction angle (degrees)	Peak cohesion (MPa)	Residual cohesion (MPa)	Tensile strength (MPa)
Characteristic	3	1000	0.25	34	0.80	0.08	0.1
Low bound	1	350	0.25	34	0.25	0.08	0.1

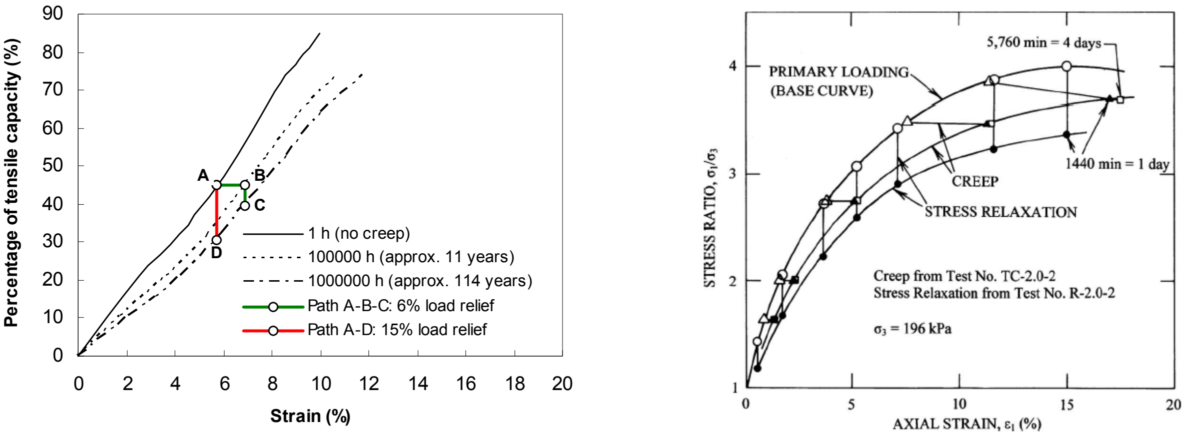
2.2 EFFECTS OF EXISTING RSW ON NEW WALL

As the main objective of the CS-SRSW design is to consider the stabilising effects of the existing walls with regard to reduction of lateral loads acting on the new wall. The design approach presented by Berg *et al.* (2009) assumes that no load is transferred from the existing shoring system to the new wall. To adopt such an assumption, the geotechnical capacity of the existing RSWs on the Hill M2 Upgrade was checked under their current loading (as no additional loads would be imposed by the new walls) for their "as-built" condition based on available designs drawings.

However, the polyester-polyethylene based geosynthetic soil reinforcement (Paraweb) of the existing RSW is known to exhibit some creep behaviour, and the movement restriction imposed by the new wall could result in the new wall being loaded by the existing wall if the Paraweb straps continue to creep over time.

Maccaferri (2009) presented a number of typical isochronous creep curves for the Paraweb reinforcement varying from 1 hour loading up to 120 years (temperature based extrapolation). The relevant curves for the design are presented in Figure 3a. As one would expect, the creep behaviour is dependent on the current level of applied load and larger creep extension is observed for loadings approaching the reinforcement tensile capacity.

As the existing RSWs were constructed some 13 years ago, it would be reasonable to expect that a large proportion of the wall movement due to creep effects would have already occurred, particularly given the logarithmic time scale creep behaviour. This is in fact observed in Figure 3a where the horizontal distance between the 1 h curve and the 11 years curve is, at any stress level, significantly larger than that between 11 years and 114 years. In addition, a compressible infill material was recommended at the interface between the existing RSW and the new wall, thus, negligible pressure would be expected to be transferred to the new wall. Nevertheless the effect of creep was further assessed and considered in the design.



(a) (b)

Figure 3: (a) Paraweb creep isochronous curves and load transfer approaches (modified from Maccaferri, 2009) (b) Time dependent behaviour - creep and stress relaxation - of a sand in triaxial compression (after Karimpour and Lade, 2010).

Detailed assessment of time-dependent behaviour associated with creep typically requires a reasonable modelling effort in geotechnical analyses. As a result, a simplified but conservative approach was adopted for the design. It is understood that creep is the development of time-dependent shear and/or volumetric strains that proceed at a rate controlled by the viscous-like resistance of the material structure. If a tensile load applied to the soil reinforcement is kept constant, the structure of the polyester-polyethylene material will likely rearrange which causes additional elongation, and wall deformation, for an unrestrained RSW face. In contrast, if the strain or elongation is kept constant, i.e. restrained from further displacement, at a particular stress, the rearrangement of the reinforcement structure promotes a decrease in the tensile load. This phenomenon is called stress relaxation. Both these time-dependent phenomena are also observed in granular materials. In sands these phenomena are associated with particle breakage and in clays with particle rearrengement. For example, Karimpour and Lade (2010) present an example of stress strain curves generated for both creep and stress relaxation behaviour of a sand under triaxial compression (Figure 3b). The sub-horizontal lines from the primary loading curve represent creep and the sub-vertical stress relaxation.

With the above mechanisms in mind, all geosynthetic reinforcement layers were conservatively assumed to be loaded to their design strength, independent of the actual mobilised tensile load, which corresponds to 45% of the ultimate capacity after all reduction factors are applied (installation damage, creep etc.). If the reinforcement is allowed to deform for approximately 11 years (time elapsed since construction of the original Hills M2 RSWs), the stress-strain state of the reinforcement would follow the path A-B as depicted in Figure 3a. Assuming that the new wall could behave in a fully rigid manner, i.e. not allowing lateral deformation or movement, any additional elongation of the reinforcement would be restricted, thus promoting the stress relaxation path B-C for the next 103 years in Figure 3a. This indicates that the new wall would have to sustain a load of approximately 6% of the ultimate capacity of the reinforcement, i.e. the difference in percentage of tensile capacity from B to C, without deforming. In theory, the new wall would also deform under these new loads, thus the path B-C would not be vertical but inclined downwards which would result in a lower load value being transferred. A similar assessment could be made if one assumes that no creep occurred in the first 11 years and the new wall is then positioned in front of the existing RSW. In this case a 15% load relief is estimated after

114 years, i.e. path A-D in Figure 3a. Given the uncertainties on creep behaviour, this higher load relief value was adopted, which generally resulted in approximately 20% of the active earth pressure acting on the existing wall face transferred to the back of the new wall.

2.3 GRAVITY WALL - MONOLITHIC APPROACH

Limit equilibrium analyses were adopted to assess the stability of the proposed CS-SRSW under traffic and impact loading as well as under a pseudo-static earthquake loading condition. The following conventional mechanisms were investigated:

- Sliding
- Overturning
- Bearing capacity
- Internal stability
- Eccentricity

It is important to note that, even for a conventional RSW the above mechanisms would be investigated. However, in the current monolithic approach the focus is on the behaviour of the wall without considering the effect of the reinforcement or at least only with a later mobilisation.

In order to prevent yielding of the cement stabilised material and consequent reduction in shear strength due to cracking, special attention was given to the eccentricity mechanism and its effect on concentration of stresses within the front part of the wall that could initiate cracking (Figure 4). Firstly, this was assessed using conventional limit-equilibrium methods (foundation type analyses) and limiting the maximum foundation stress, σ_{max}, to 30% of the UCS value of the intact stabilised material. Additional numerical analyses were later completed to further assess this mechanism.

In order to assess the internal stability of the CS-SRSW, internal failure planes ranging from the friction angle of the cement stabilised material to 85° from the horizontal plane were considered (Figure 4) ignoring the effect of the reinforcement. Assuming monolithic behaviour, it was confirmed that the cement stabilised wall would not require additional soil reinforcement. However soil reinforcement was included as previously discussed for the following reasons:

- To allow for a RSW construction method which uses the same type of face panels as the existing RSW where these panels also act as formwork for the wall.
- To provide temporary support at the face until the cement stabilised material achieves the required strength.
- For the blocky medium approach to be valid.

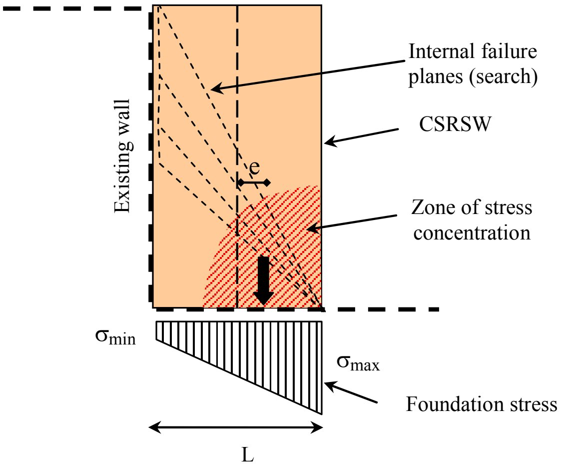


Figure 4: Internal stability analysis and effect of eccentricity on foundation loading and internal stress.

In order to improve the behaviour of the CS-RSW gravity block under earthquake loading, vertical pre-tensioned tie-rods were included in the design to provide additional overturning resistance and reduce stress concentration in the front of the wall due to eccentricity. These act mostly as passive reinforcement due to the low pretension value adopted to avoid cracking initiation at the top of the wall. Figure 5 presents the concept sketch of the proposed the CS-SRSW with 0.3< L/H < 0.4.

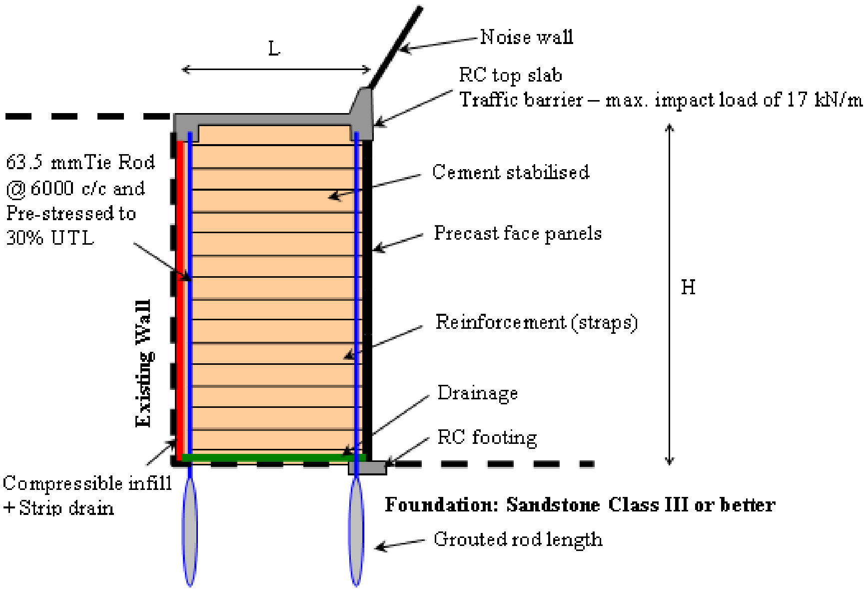


Figure 5: CS-SRSW concept.

2.4.2 Numerical model

In addition to the analytical limit equilibrium analyses briefly described above, numerical modelling using the commercial Finite Difference (FD) code FLAC2D was also carried out to assess the development of stresses within the cement stabilised block and the magnitude of displacements under the applied loading conditions.

The cement stabilised material and rock units were modelled as linear elastic-plastic materials. The rock unit follows a perfectly plastic Mohr-Coulomb failure criterion with friction angle ϕ = 35°, cohesion c = 250 MPa and Young's Modulus E = 1000 MPa. These parameters are equivalent to a sandstone Class IV type rock with a GSI = 45, i.e. where rock defects are taken into account in the failure criteria as an equivalent continuum. A strain-softening elastic-plastic model was used for the cement stabilised material to simulate potential cracking and consequent reduction in strength. The adopted parameters were presented in Table 1 above.

The tie rods were modelled in FLAC2D as cable elements with properties automatically "smeared" to account for the out-of-plane spacing (s_h = 6 m). The rod was assumed to be anchored in sandstone Class III / shale Class II or better material with a minimum grouted length L_b = 4 m and ultimate bond stress of 1000 kPa. A pre-tension of 500 kN was adopted. The top slab was modelled as elastic beam elements, structurally connected to the tie rods. A slab thickness of 0.3 m and Young's modulus E = 30 GPa were adopted.

Soil reinforcement straps were modelled using the FLAC2D strip element option, which is similar to a cable element. A friction coefficient μ = 0.5 was adopted for the reinforcement straps.

The cement stabilised block and foundation units were discretised in the numerical model as solid elements. The assumed effects of the existing RSW were modelled as a pressure applied onto both the rock foundation and to the rear of the new wall. Construction of the CS-SRSW was modelled in stages (layers of 1.0 m thickness were assumed for modelling purposes) to simulate the development of internal stresses during construction. Traffic load was modelled as a surcharge pressure applied to the top of the wall. Impact and earthquake loads were modelled as linear pseudo-static forces applied to the top and centroid of the wall, respectively. Under impact and earthquake loading conditions the new wall was assumed to behave independently from the existing wall as no tie connections are proposed even though the creep pressure was maintained.

In order to assess the factor of safety in the numerical analysis, the same modelling sequence was repeated with strength reduction factors (SRF) applied to the shear strength parameters of the stabilised mass. Overturning and eccentricity were identified as the critical failure mechanisms in the limit equilibrium analyses, mainly due to the point of application of impact and earthquake loads. As a result, only one case of the CS-SRSW was modelled with a limiting height H = 17 m and base width to height ratio of 0.35 (L/H = 0.35).

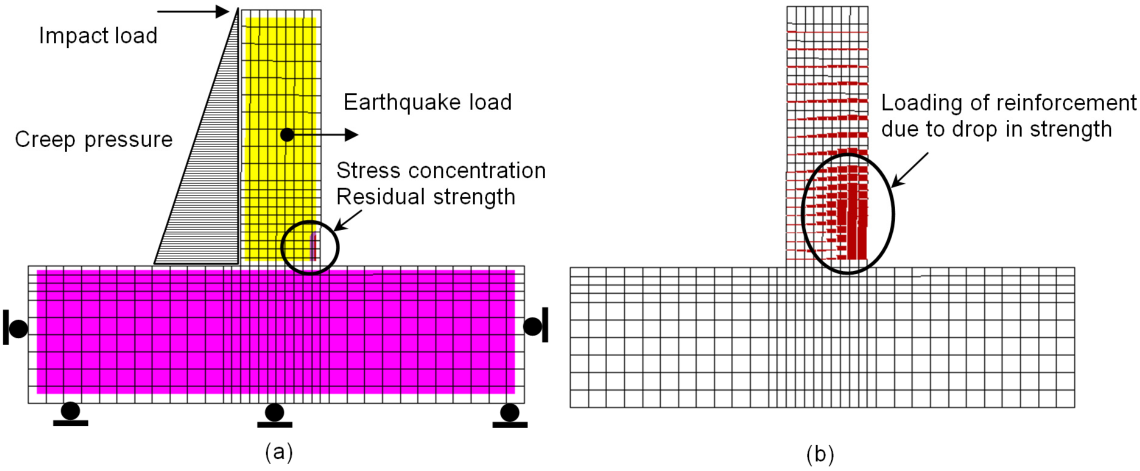


Figure 6: FLAC2D results for low bound case under impact loading: a) boundary conditions and material yielding b) reinforcement loads.

Selected FLAC2D output and results are presented in Figure 6 and Table 2, respectively. It can be noted that for the low bound case, the maximum principal stress, σ_1, within the CS-SRSW exceeds the material UCS value of 1 MPa which causes a reduction in strength of the cement stabilised material due to the strain-softening constitutive model adopted and the lower confinement near the wall boundaries. As a result, loads are transferred to the reinforcement which controls further propagation of material damage (yielding). In theory, the target UCS of the stabilised material was 3 MPa so this stress level would only represent intitiation of microcracking of the wall which confirms the benefit of having the soil reinforcement.

Table 2: FLAC2D results (H=17 m L/H=0.35 - monolithic approach).

Model stage	Low bound Properties				Characteristic Properties			
	Hor.$^{(1)}$ (mm)	Vert.$^{(1)}$ (mm)	σ_1 $^{(2)}$ (kPa)	σ_t $^{(2)}$ (kPa)	Hor.$^{(1)}$ (mm)	Vert.$^{(1)}$ (mm)	σ_1 $^{(2)}$ (kPa)	σ_t $^{(2)}$ (kPa)
Wall construction	0	1	500	20	0	1	520	0
Traffic load and existing wall pressure applied.	18	5	1000	0	11	2.5	1050	0
Impact load applied	25	6	1160	15	15	4	1200	25
Impact load removed and earthquake load applied	26	6	1140	33	16	4	1300	30

Notes: 1) Maximum cumulative horizontal and vertical displacement at the top of the wall at the end of the respective stage. 2) Maximum compressive and tensile stresses observed within CS-SRSW.

2.4 BLOCKY MEDIUM OR FLEXIBLE APPROACH - EXCEPTIONS TO RMS R57

The blocky medium approach was adopted as an alternative design check assuming the cement stabilised mass may fully crack. Under such a condition, the CS-SRSW behaves more akin to a conventional RSW. As a result, the method suggested by the FHWA design guidelines (Berg et al., 2009) for shored walls was considered appropriate. The main advantage of this design was that the stabilised blocky material will still have a higher value of Young's modulus and better interlocking of particles (i.e. blocks) than a conventional granular backfill, hence reducing the likelihood of trenching at the interface between the shoring system and the new wall. In addition, the tie rods will provide additional safety against seismic loading.

According to the FWHA guidelines (Berg et al., 2009), sliding, overturning and eccentricity are not considered valid failure modes for shored RSW. Lateral pressures acting on the RSW are self-induced as the shoring wall effectively reduces external loading, and these self-induced pressures would not realistically induce these modes of failure in walls designed in accordance with the guidelines. Analyses for sliding, overturning and eccentricity modes of failure, though conducted for traditional RSW, are not required for shored RSW design. Internal failure of a shored RSW is the primary failure mode and is addressed with appropriate backfill materials, suitable vertical spacing of reinforcement and adequate reinforcement strength and lengths.

For inextensible reinforcement cases, the critical failure surface has been assumed to be bilinear with the lower point passing through the toe of the wall (Figure 1). The FHWA guidelines state that this assumption is conservative compared to observations from centrifuge modelling.

Internal design differs from conventional RSW design with regard to pullout of the reinforcement noted as an exception to the RMS R57 Edition 2 Rev.1 (2007) design standard. Conventional RSW design requires that each layer of reinforcement resist pullout by extending beyond the estimated failure surface. In the case of a shored RSW system, only the lower reinforcement layers (i.e., those that extend into the resistant zone) are designed to resist the pullout force for the entire "active" RSW mass. As a result, the required pullout resistance of the reinforcement within the resistant zone is calculated as the pullout force derived using a slope stability or wedge approach considering the failure surface as shown in Figure 1. Therefore, the effect of the stabilisation is already taken into account by the material parameters of the backfill as presented below. The calculation of the pullout resistance in the resistant zone followed traditional design methods such as those outlined in RMS R57 ignoring any potential adhesion promoted by the cement stabilisation or additional interlocking in the case of steel ribbed reinforcement. However, the maximum tensile force with respect to rupture of the reinforcement requires an additional modification (exception) to the RMS R57 equation.

The above discussion was noted on the design drawings as exceptions to the RMS R57 standard, including the effect of the blocky behaviour in reducing the earth pressure applied onto the concrete face panels and reducing the maximum tensile force with respect to rupture of the reinforcement.

It is important to note that the effect of a higher pH environment on the durability of the steel reinforcement promoted by the cement stabilisation was a point of significant debate and further investigation is still required. For the current design, it was agreed that to achieve a 100 year design life a sacrificial corrosion thickness of 1.5 mm was considered appropriate on either side of the steel reinforcement when a certain rate of corrosion was assumed, in addition to a galvanising protection of 85 μm.

2.4.1 Material parameters and assessment of equivalent face earth pressure

As discussed above, the CS-SRSW was assumed equivalent to a synthetic rock simulating the behaviour of a weathered sandstone rock. Consequently, if cracks (discontinuities) are included, the stabilised mass may be treated as an equivalent fractured rock mass.

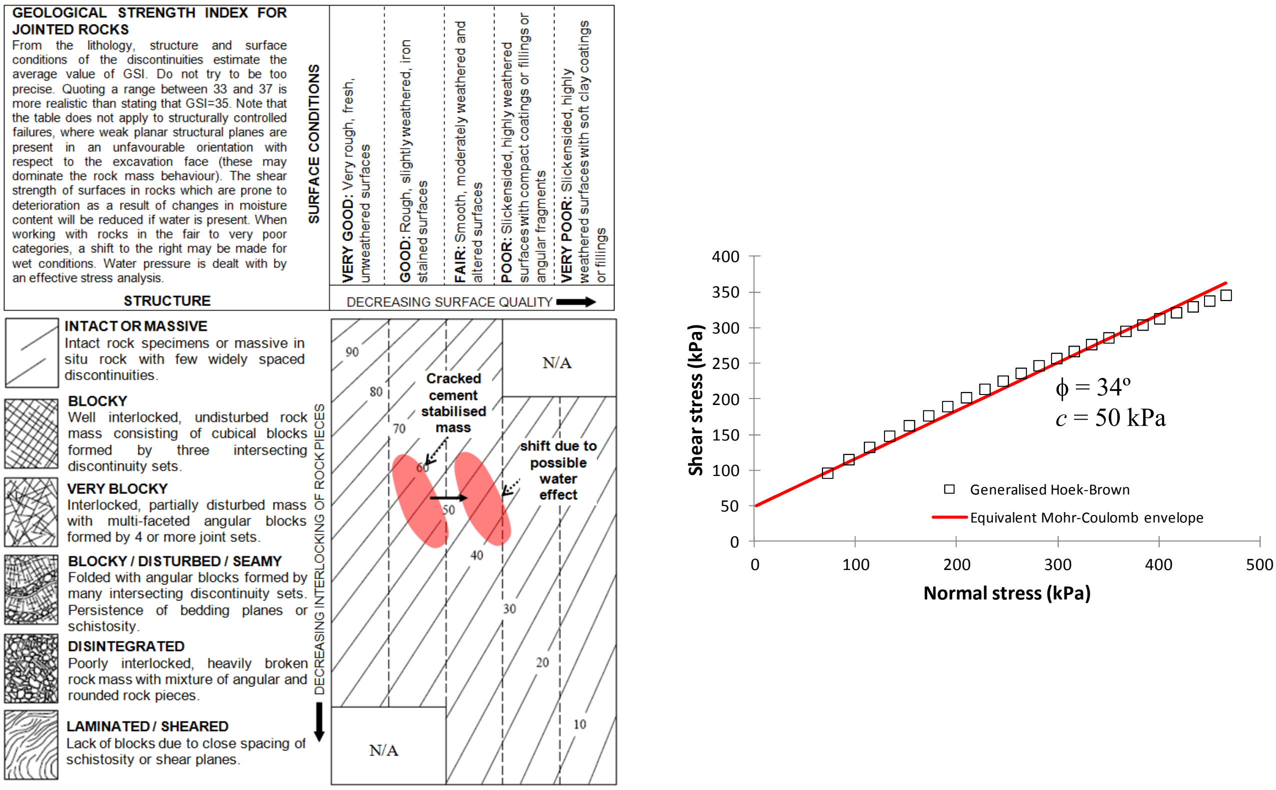
Cracking of the stabilised mass was conservatively assumed to be very intense resulting in closely spaced discontinuities (60 mm to 200 mm). Despite the intense cracking, the stabilised mass is assumed to be only partially disturbed and the resulting medium is equivalent to a very blocky rock mass.

It is important to note that continuous cracks that could structurally control the failure mechanism would only occur if failure planes develop. As a result, the initial cracks are unlikely to be persistent and the cracked stabilised mass may be represented by an equivalent pseudo-continuum where the discontinuities are accounted for through the material model for which the Generalised Hoek-Brown (GHB) failure criterion was adopted. The equivalent GSI of the cracked stabilised mass is shown in Figure 7a. Although drainage measures are recommended to reduce saturation of the stabilised mass, water effects inside the cracks are taken into account by modification to the GSI value as recommended by Marinos and Hoek (2000).

The adopted GHB parameters are: GSI = 45, σ_{ci} = 1 MPa (target design value of the cement stabilised sandstone with a material reduction factor of 3 applied), m_i = 17 (typical value for sedimentary sandstone type rocks), and a disturbance factor D = 0. It is important to note that the assumed rock mass parameters are consistent with the parameters proposed for sandstone Class IV (Bertuzzi and Pells, 2002) which according to the Pells' classification comprises weathered sandstones with UCS > 2 MPa, defect spacing > 60 mm and 10% of allowable seams (clay seams and/or poor quality crushed/sheared rock bands).

Since the design of RSW is more conveniently carried out with respect to shear and normal stresses, Mohr-Coulomb (MC) parameters were back-calculated from the GHB model to suit the expected range of confining stresses/normal stresses. The equivalent MC envelope is shown in Figure 7b which gives an equivalent friction angle ϕ = 34º and cohesion c = 50 kPa. The equivalent MC parameters are reasonable considering that for the cracked stabilised mass dilation is expected to occur promoting interlocking of the blocks and that cohesion is obtained because the discontinuities are not fully interconnected, not persistent nor oriented in the same direction.

If the modulus of the cracked stabilised mass is calculated according to the relationship with GSI as proposed by Hoek and Diederichs (2006) and the modulus of the intact mass is assumed to be E_i = 1000 MPa, a cracked modulus E_{rm} = 230 MPa is found which is somewhat similar in magnitude to that adopted for the monolithic approach low bound.



(a) (b)

Figure 7: (a) Geological strength index for jointed rock masses (modified from Marinos and Hoek, 2000) (b) Equivalent Mohr-Coulomb parameters for the cracked stabilised mass.

Using the above strength parameters, the maximum tensile force acting per metre width at the j^{th} layer of reinforcement due to loads acting on the face of the wall in the non-resistant zone (i.e. inside the failure zone in Figure 1), could then be calculated with a modification to the RMS R57 formula by:

$$T^*_{pj} = \left\{K^*_{1(Z_j)}\sigma^*_{vj} - 2c\sqrt{K^*_{1(Z_j)}}\right\}\left(S_{vj} + S_{vj+1}\right)0.5 \qquad (3)$$

where $K^*_{1(Zj)}$ is the earth pressure coefficient in accordance with RMS R57 but using the above friction angle, σ_{vj} is the vertical stress at the depth of the j^{th} layer, S_v is the vertical spacing of the reinforcement.

3 LABORATORY TESTING OF THE CEMENT STABILISED MATERIAL

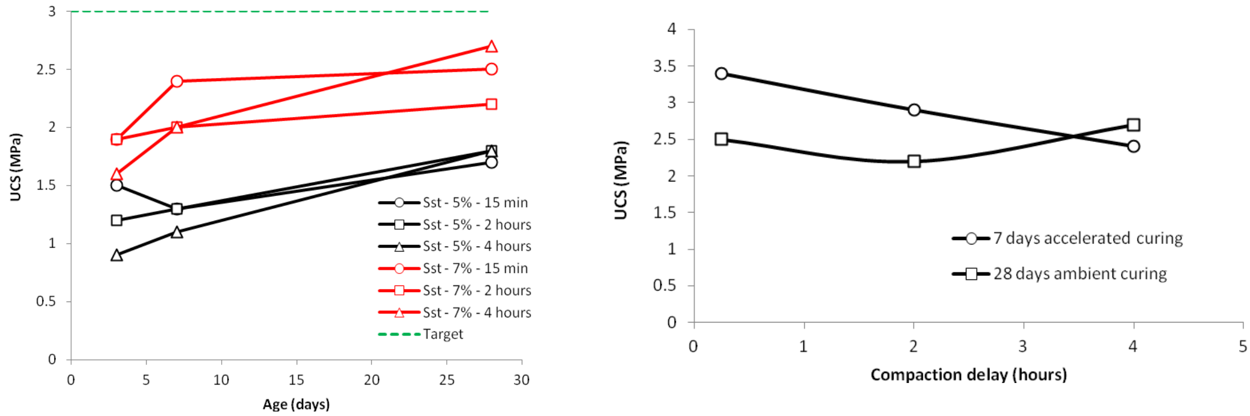
As discussed above, during the design phase and prior to any material testing, it was assumed that a minimum uniaxial compressive strength UCS = 3 MPa at 28 days could be achieved with a well graded sandy gravel (crushed sandstone) stabilised with 5% cement.

After several rounds of discussions between the design team, Transurban and RMS, it was agreed that the cement content would be increased to 7% to address potential mixing problems and the design strength of the blocky approach would be limited to an UCS = 1 MPa, after applying a reduction factor of 3 to the above targeted laboratory UCS strength. This reduction of 3 was requested by RMS with the view of possible saturation of the stabilised material, as at that time no test results were available. The effect of material saturation on strength of the stabilised material was later further investigated by triaxial testing.

In order to validate the design assumptions above, a number of laboratory tests were then carried out on the stabilised material, prior to construction.

2.5 UCS TESTING

Figure 8 presents the test results for crushed sandstone samples stabilised at different cement contents (5% and 7%) and different compaction delay times. All samples were soaked for a minimum period of 24 hours prior to testing. The compaction delay time was assessed as an important factor as no batching plant (pug mill) was allowed to be set up along the Hill M2 Upgrade project. Therefore, the stabilised material had to be mixed off site and transported. Due to traffic conditions, delays in compaction after mixing of in excess of 4 hours could occur and by that time the hydration process of the cement would be reasonably advanced. The delayed compaction would then break some of the already established “bonds” reducing overall future strength.



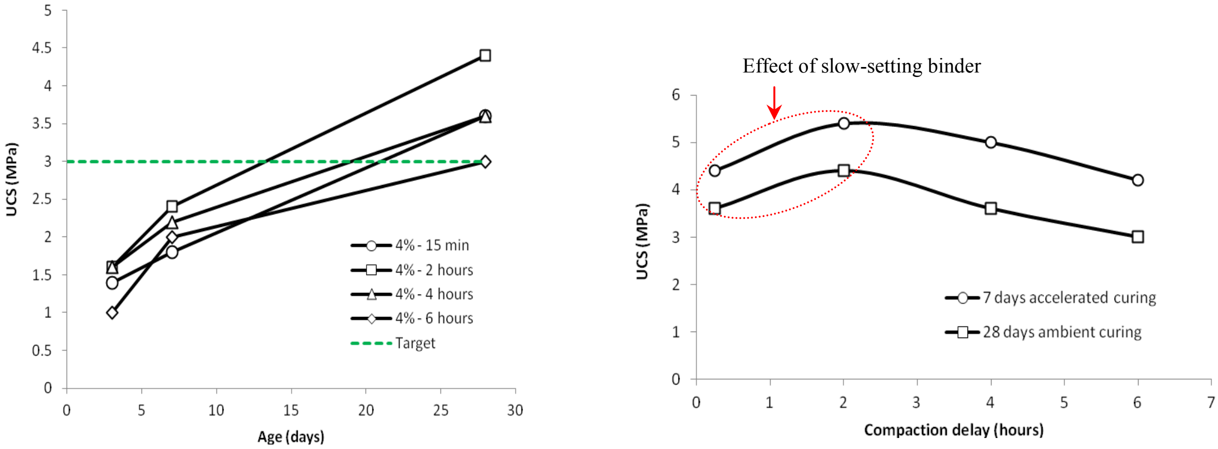
(a) (b)

Figure 8: UCS results for crushed sandstone (a) with different cement content, compaction delay time and age (b) 7% cement content with varying compaction delay time and curing.

As noted in Figure 8, the proposed site-won crushed sandstone did not achieve the target strength of 3 MPa even at 7% cement content and no delay in compaction (i.e.15 min). The maximum strength that could be assigned for such material would be approximately 2 MPa at 7% cement content. A likely cause of this lower strength was attributed to the grading of the crushed sandstone, possibly associated with further break down during compaction. The particle size distribution of the proposed material was observed to be gap (poorly) graded gravelly sand instead of the recommended well graded sandy gravel. It is interesting to note that some samples yielded higher UCS values at 7 days accelerated (oven) curing than those obtained at 28 days ambient curing. This may indicate a potential increase in strength for ages greater than 28 days.

Although a laboratory strength of 2 MPa could potentially be used if a lower material reduction factor could be proved acceptable and also considering that field samples could potentially have higher strength due larger particle size, it was decided that a material with better crushing and grading process control would be beneficial. It was decided to use a commercial material known to be well graded. A Dense Graded Base with maximum particle size of 20 mm (DGB20) from Boral blended with a slow setting binder (Stabilment) was chosen as it was also compliant with the RMS 3051 specification. This product is supplied by Boral as a Roller Compacted Concrete replacement which targets RMS R73 specification for Heavily Bound Pavement courses. The DGB20 consists of high strength basalts which present reduced micron dust when crushed.

Figure 9 presents the UCS results for the stabilised DGB, and shows that the target strength of 3 MPa is attained for all samples even with a delayed compaction of 6 hours. Similarly UCS values at 7 days accelerated curing were higher than those obtained at 28 days ambient curing. Figure 9b also highlights the benefit of the slow-setting binder in the delayed compaction.



(a) (b)

Figure 9: UCS results for DGB20 with 4% stabilment with different compaction delay time (a) at different ages (b) with different curing.

2.6 TRIAXIAL TESTING

Due to limits of the testing equipment used, only two samples of the stabilised sandstone were successfully tested. These samples were blended at 7% cement content with delayed compaction of 2 hours and tested after 7 days of accelerated curing under zero and 50 kPa confining stresses. The DGB20 samples were not tested.

Due to the limited number of successful samples, the most valid use of the triaxial results were perhaps the assessment of possible saturation and its effects on material strength as the samples were subjected to a water back pressure in the triaxial cell. After 3 days of backpressure up to 300 kPa, both samples had a pore pressure coefficient B = 0.93 which indicates a partial but possibly near saturation condition.

From the results, this partial saturation caused no significant drop in strength. For the unconfined sample, an axial stress of 3 MPa was observed, comparable to the 2.9 MPa shown in Figure 8b for the 7 days accelerated curing with 2 hours delay. Although the stabilised DGB20 material has not been tested for the effects of saturation, it was assumed that similar results could be expected considering that this material is better graded.

The triaxial test results indicate that the material reduction factor of 3 to account for saturation may have been too conservative. Even under a pressure equivalent to 30 m of water (300 kPa) for 3 days, the stabilised sandstone did not fully saturate, and at this partial saturation no significant drop in strength has been observed. The CS-SRSWs were not designed for such extreme condition, i.e. a 30 m water column, which is not expected to occur, particularly considering the double drainage system installed at the rear of the new wall: one vertical drain for the existing wall face and another for the new wall separated by a membrane. It is also important to note that this reduction factor was to be applied to the intact stabilised material only. The effect of water within cracks would be taken into account when converting the intact parameters (already reduced by the above factor) to the blocky medium parameters which has reduction factor due to water of approximately 1.3 with respect to compressive strength. Based on these testing results, the material reduction factor could be reduced, e.g. to 2, though to account for construction and mixing variations, it was kept at 3.

4 CONSTRUCTION

When the wall was nearly completed, cored samples were taken from the CS-SRSW for further testing of the *in situ* stabilised material. Care was taken with the location of the cores to reduce the risk of drilling through the steel reinforcement. Figure 10 presents a photo with the cored samples, indicating a good quality of the final material and its similarity to a rock or roller compacted concrete material. UCS testing with measurement of the Young's modulus was carried out on 9 samples taken at different depths. In general all these samples had ages in excess of 28 days but less than 90 days. The minimum UCS observed for those samples was 5.9 MPa, maximum of 12.3 MPa and an average of 8.3 MPa. The intact or substance Young's modulus varied between 10 GPa and 12 GPa which indicates that the predictions with Equation (1) would be acceptable or using the upper values of the modulus ratio in Equation (2). Nevertheless, the substance Young's modulus would still require to be downgraded to account for cracking and any discontinuities, and the relationship with GSI as before, cement stabilised mass modulus would still be in excess of 2 GPa, even higher than the characteristic value adopted for the rigid-monolithic approach. Figures 11 and 12 present some photos during the construction of the CS-SRSW.

Figure 10: Core samples of the stabilised DGB20.

(a) (b)

Figure 11: Construction of the CS-SRSW (a) Concrete platform completed (b) Initial layers of the stabilised DGB20 with details of the face panels and back double drainage system.

5 CONCLUSIONS

Site constraints precluded the use of conventional RSW at a number of locations throughout the Hills M2 Upgrade alignment where limited space was available for the extension of the existing relatively high retaining walls. An innovative design approach was adopted considering the stabilising effect of the existing RSW with regard to the reduction of lateral loads acting on the new walls, and targeting the safe design and construction of slender RSWs. Several challenges, as described above, had to be overcome before acceptance of this innovative design.

The concept of Shored RSW was adopted with improvements to the backfill behaviour in order to address some of the potential issues. Material testing on both laboratory and field samples confirmed the targeted behaviour of a stiffer backfill.

Figure 12: Aerial view of the CSRSW at its final height in June 2012.

It is important to note that RMS acceptance of CS-SRSW in the Hills M2 Upgrade project was to a very specific case, when pulling down the existing wall in order to build the new RSW was not an option. If such cases occur again, they will require similar investigation and deliberation before any decision is made, i.e. the previous RMS acceptance does not constitute a blanket acceptance of CS-SRSW for similar future cases. Likewise, the deviations from the RMS Specification R57 and R58 in this project are specific to this case and there should be no corresponding changes to RMS Specifications R57 and R58 as they were not intended to cover such situations.

6 ACKNOWLEDGEMENTS

Without the contribution and collaborative discussions of all members of the Hills M2 Upgrade project such innovative design would not be possible. As a result, the authors would like to acknowledge the significant contributions of Leighton Contractors, AECOM, Transurban, SKM and the RMS.

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