RETROFIT OF A STEEL-REINFORCED MSE WALL

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ABSTRACT
This paper describes the retrofit of an existing mechanically stabilized earth (MSE) wall where concerns existed that the connection between the steel reinforcing and precast concrete wall facing panels had been compromised. The paper describes the investigations undertaken, outlines design constraints and performance considerations, and discusses design and construction. The paper concludes by briefly discussing retrofit design and construction approaches used on three further MSE walls with similar concerns.
The field investigation program included constructing a test strip to characterize the shot rock MSE zone backfill and a field loading test to assess the stiffness of the backfill. After considering a range of approaches that included disassembling and rebuilding the wall, constructing a buttress wall, installing a soldier pile wall, grouting the backfill, and soil nailing, the retrofit finally adopted consisted of soil anchors which applied load through a load distribution concrete wall.

RÉSUMÉ
Cet article présente la mise à niveau d'un mur de terre armée pour lequel des doutes existaient quant à l'intégrité de la connexion entre les éléments d'acier encastrés dans le sol et le mur façade en blocs de béton préfabriqué. L'article décrit les travaux d'investigation, les critères de conception et les critères de performance ainsi que les défis de conception et de construction. L'article conclut par une discussion sur la conception et sur les méthodes de construction prévues pour la mise à niveau future de 3 murs de terre armée qui présente une problématique similaire.
Une réplique du mur a été réalisée dans le but de réaliser les essais de terrain. Les essais de terrain visaient à caractériser la rigidité du remblai en enrochement du mur de terre armée à l'aide d'essais de chargement. Après avoir considéré plusieurs alternatives dont : le démantèlement et la reconstruction du mur, la construction d'une butée de béton au pied du mur existant, l'installation d'un mur de palplanche, l'injection du remblai, l'utilisation de "Soil nail", la solution de mise à niveau retenue consiste en des "soil anchors" qui transfère la charge via un mur de béton fondé sur pieux vises.

1 INTRODUCTION

This paper describes the retrofit of an existing mechanically stabilized earth (MSE) wall where there were concerns that the connection between the steel reinforcing and precast concrete panels had been compromised. The retrofit was initiated following the collapse of several facing panels on an adjacent section of MSE wall supporting a bridge abutment which had been constructed using similar materials and construction methodology. After the facing panels on the adjacent wall collapsed, the MSE backfill directly behind the facing ravelled backwards by about 0.5 m to 0.9 m, and then reached equilibrium with only minor ravelling. Bridge abutment stability was not compromised.

The cause of the distress is outside the scope of this paper, which instead discusses the retrofit of MSE walls where issues at the connection between the steel reinforcing and the facing panel may exist.

2 EXISTING MSE WALL

2.1 Wall Details

Figures 1 and 2 show a partial view and a typical cross section of the MSE wall. The wall supports a two lane section of roadway ramp, it is about 300 m long and has a maximum of 8 m high. The precast concrete facing panels are nominally 1.8 m by 1.5 m in size, and the MSE wall steel reinforcing (length approximately 70% of wall height) is placed in layers spaced at about 0.76 m vertically apart. The steel reinforcing is not continuous, but placed in sheets with gaps of about 0.6 m between sheets (coverage ratio of 33%).

2.2 Foundation Conditions

The foundation soils consist of about 5 m of very stiff, medium to high plastic clay, overlying firm to stiff silty clay. The groundwater level is approximately 14 m below ground surface.

2.3 MSE Backfill

The MSE backfill consisted of owner-supplied crushed basalt shot rock from a stockpile adjacent to the work site. The backfill gradations (see Figure 3) were in accordance with BC Ministry of Transportation and Infrastructure Standard Specifications for Highway Construction (2009) Bridge End Fill (BEF), but tended to be on the coarse side of the specified grading envelope. The fines content (percent passing the 0.075 mm sieve) was variable and up to 8%. The percentage retained on the 19 mm sieve was typically between 40% and 70%. Because the fraction of material retained on the 19 mm sieve was greater than 30% compaction control could not be referenced to the familiar Standard Proctor moisture-density test (ASTM D698).

Figure 1. MSE wall prior to retrofit.

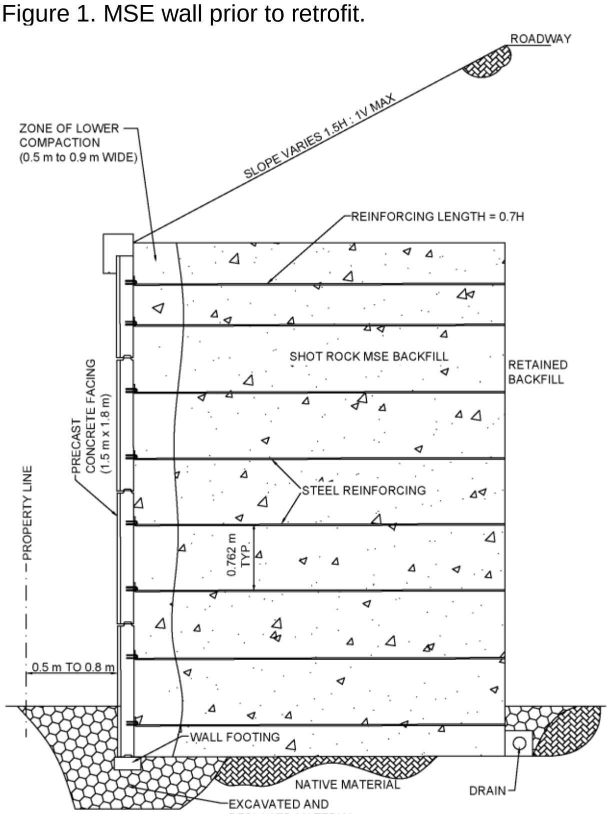

Figure 2. Typical section.

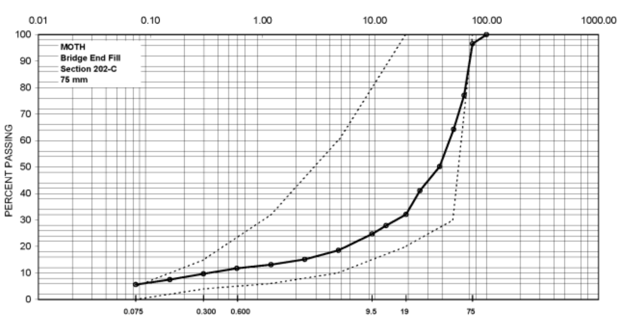

Figure 3. Typical MSE backfill gradation.

Backfill placement and compaction was undertaken generally as follows:

- The material was placed in 250 mm thick loose lifts.
- Each lift was compacted using a CAT CS-323C compactor (nominal 5 tonne static weight).
- A 0.9 m wide zone directly behind the facing was compacted (after the main MSE backfill had been compacted) using a 90 kg plate compactor to limit compaction loads on the facing panels.

3 DESIGN APPROACH

3.1 Constraints, Conditions and Requirements

The retrofit had to be designed to account for the following:

- The wall was located 0.5 m to 0.8 m from the property line so there was very limited space in front to accommodate permanent repairs (see Figure 2). However, a temporary construction easement to allow the retrofit to be constructed was negotiated.
- The mechanism of potential failure was not fully understood and there were differing opinions as to what had caused the collapse of the facing elements on the adjacent MSE wall supporting the bridge abutment. However, the potentially compromised connections were confined to the lowermost 0.6 m to 2.2 m of the wall and the upper connections were shown to be adequate.
- There was no evidence from visual observations or survey monitoring that, at the time the retrofit was undertaken, the wall was experiencing distress.
- At the time of the retrofit, the wall had been in place for about 18 months, so much of the potential settlement of the underlying native soils had occurred. This was confirmed by ongoing monitoring of the wall.
- The retrofit had to have 100-year design life.
- No highway traffic interruption was permitted during retrofit construction.

3.2 Previous Case Histories

A review of MSE wall case histories in which facing panels had bulged or collapsed without impacting the overall stability of the MSE structure was undertaken. These case histories showed that a range of retrofit options had been specified depending on the type of distress experienced as summarized below:

- *Corrosion of the steel connection from corrosive backfill:* Armour et al 2004 discussed a retrofit consisting of horizontal drilled and grouted cross-ties for back-to-back sections of MSE and anchored soldier piles at a bridge abutment.
- *Excessive differential settlement between the facing panels and the backfill* (Kim et al 2010, Neely and Tan 2011, Sankey et al 2011). In several of the case histories the facing panels were removed and re-set to relieve the stress on the steel reinforcement.
- *Poor quality backfill and inadequate backfill compaction* (Leonards et al 1994, Reith et al 2003, Thome et al 2005, Hossain et al 2012). Repairs

included a range of dismantling and rebuilding the wall, to stabilizing the wall using soil nails.

- *Backfill loss* (Galvan, Heere et a 2001). Repairs included pressure and compaction grouting, anchored soldier pile walls in areas of the most severe movement (Galvan), and shotcrete shoring (Heere et al 2001).
- *Poor drainage* (Thome et al 2005, Neely 2011, Lee et al 2013). Uncontrolled water inflow in to the backfill causing hydro-compaction of the backfill. In most cases surface drainage improvements were implemented.
- *Frost susceptible backfill* (Neely 2011). In this case history the collapsed panels were repaired by field-splicing replacement panels to the existing reinforcing strips and filling the void with non-frost susceptible backfill. However, after 20 years the panels again started bulging and required repairs.

3.3 Investigations

The available case histories indicate that distress is often related to backfill quality. Therefore, it was decided to characterise the compressibility and settlement potential of the shot rock backfill by constructing a control test strip followed by a full-scale loading test on a section of completed MSE wall.

The objective of the test strip (about 20 m long by 5 m wide) was to assess the expected settlement of compacted backfill by undertaking in situ density testing and settlement measurements under different levels of compaction to simulate the settlement due to traffic loading. The test used the same shot rock backfill and loose layer thickness, and the same compactor as was used in the original backfill compaction.

Two tests were conducted, the first using a 250 mm loose layer thickness, and the second a 150 mm loose layer thickness. The first test used a CAT CS-323-C compactor (5 tonne static mass). After each pass the in situ density was measured at several locations using a nuclear density meter, and after eight passes, the settlement magnitude was measured. After a further eight passes (cumulative total of 16 passes) settlement magnitude and density were again measured. Finally the layer was compacted using eight passes of a BOMAG BW213 DH-3 compactor (12.7 tonne static mass) and settlement and density again measured.

The results obtained are summarized in Table 1.

Loose Layer (mm)	Compactor Used	Cumulative No. of Passes	Increase in Dry Density (%)	Cumulative Settlement (mm)
250	5 tonne	8	11	60
	5 tonne	16	16	70
	12.7 tonne	24*	17.5	75
150	5 tonne	8	12	Not Measured
	5 tonne	16	18	

Table 1. Test strip results.

* 16 passes of 5 tonne, plus 8 passes of 12.7 tonne compactor

The test strip provided the following insights:

- Large rock fragments were mostly broken up by the compactor.
- The compacted material was very variable and areas of open-graded rock with voids were juxtaposed between areas of more well-graded material.
- An additional settlement of about 5 mm occurred under the 12.7 tonne roller was considered to provide an indication of the maximum likely settlement.

The field loading test consisted of piling standard Lock-Blocks™ three square (i.e. 4.5 m by 4.5 m on plan) to four blocks height (i.e. 3 m height) on the MSE backfill and measuring the settlement as each row of blocks were placed, and for 5 days following placement of the final row of blocks. The average contact pressure with four bocks amounted to 72 kPa. The footprint dimensions of the loaded area were selected to limit stress increments in the underlying native soils.

Although, the loading was applied statically and did not model repetitive traffic loading, it provided an indication of compressibility of the compacted fill. The average settlement measured over the duration of the test amounted to about 6 mm, and most of this was recoverable on removal of the lock blocks.

Based on assessment of the available data and test results it was concluded that the likely long-term internal settlement of the MSE backfill would not exceed about 0.5 to 1% of the backfill thickness. Tests conducted to measure the tolerable free rotation of the panel connection showed that the steel reinforcing could rotate freely. Therefore the 0.5% to 1% backfill settlement was considered unlikely to compromise the connection.

3.4 Conceptual Design

After considering a range of retrofit approaches ranging from doing nothing, disassembling and rebuilding the wall, constructing a new wall in front of the existing wall to act as a buttress, installing soldier piles, grouting the backfill to form a solid block, soil nailing, etc., the retrofit adopted consisted of soil anchors bearing against a load distribution wall constructed in front of the precast panels to distribute the load uniformly over the panels.

The limited space in front of the wall prevented the use of buttress type solutions, and the variable permeability of the backfill made grouting an uncertain option. Soil nails were disregarded because of concerns about the movement required to engage the soil nails and because it would be difficult to address corrosion concerns due to expected variable grout takes.

3.5 Detailed Design

Detailed design was undertaken for two typical sections with different heights. The lower section (about 5 m in height) included one row of reinforcing with potentially compromised connections, where one row of soil anchors would be installed. The higher section (about 8 m in height) included a 2.2 m height with three rows of potentially compromised connections, where two rows of soil anchors, 1.5 m spaced vertically, would be installed.

The anchor capacity was estimated using the design approach given in AASHTO 2002 and checked by CFEM 2006 methodology. The bond length of the anchors was estimated based on an ultimate bond stress of 150 kPa between the grout and shot rock MSE backfill (based on FHWA typical values for dense to very dense sand and gravel). The free length of the anchors was selected based on the geometry of the potential active wedge and considering FHWA recommendations for minimum free length.

The MSE wall geometry in conjunction with the above anchors was modeled in Slope/W software to assess the location of the critical slip surface as well as the global factor of safety. The design was based on the anchors carrying the full load of the wall (i.e. assuming that the steel reinforcing was not present).

The anchor design loads and lengths were then adjusted to achieve a minimum factor of safety of 1.5 for static conditions. This resulted in anchors spaced at 1.8 m horizontally with design loads varying from 100 to 130 kN.

Double corrosion protection (DCP) anchors of sizes #8 and #9 were used in the design and a drill hole diameter of 150 mm was taken for calculating the required bond length.

Installation of soil anchors had to consider the following:

- The shot rock backfill would be difficult for anchor installation and grout takes would be high.
- If the anchors were stressed, compression of the 0.9 m backfill zone directly behind the facing panel could cause cracking of the panel. Early in the design the structural engineer identified this risk.
- The anchor location would need to take account of the location of the steel reinforcing and the bending capacity of the facing panels.

In order to distribute the anchor loads evenly and reduce the risk of differential movement between the facing panels, a 280 mm thick concrete load distribution wall (1.5 m and 3 m in height) was designed for the entire length of the lower portion of the wall (see Figure 4). The structural engineer further required helical piles be installed under the load distribution concrete wall due to the concern that the vertical component of the anchor load plus the weight of the concrete wall would damage the facing panels and open the joints between adjacent panels.

Figure 5 shows the typical retrofit design used for the subject MSE wall.

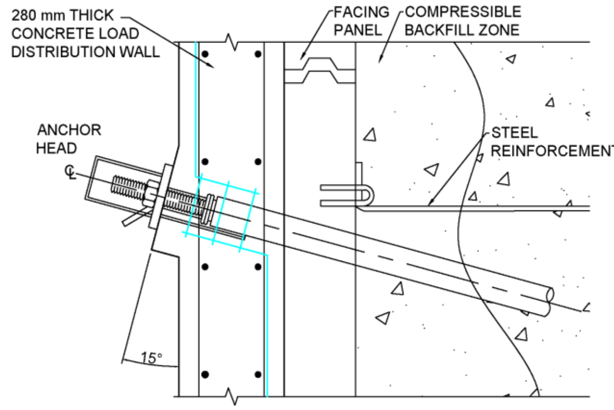

Figure 4. Typical anchor head and concrete blister section.

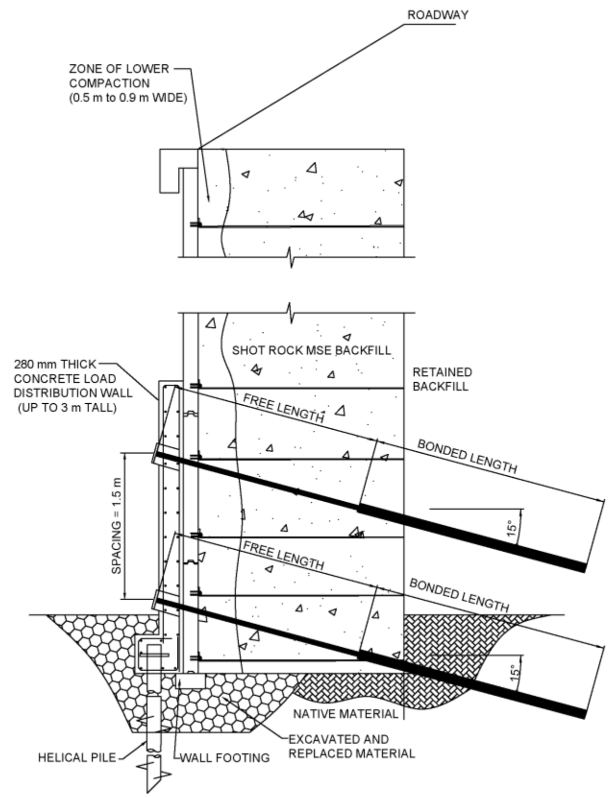

Figure 5. Typical retrofit design.

4 CONSTRUCTION

4.1 Construction Sequence

The sequence of the retrofit construction consisted of the following steps:

- Conduct pre-production helical pile and anchor testing to verify the pull-out resistance and ultimate bond stress.
- Install helical piles at nominal 1.8 and 3.6 m spacing (52 total).
- Construct load distribution concrete wall incorporating a PVC sleeve for later installation of anchors.
- Drill and install anchors – one or two rows at 1.5 m vertical and 1.8 m horizontal nominal spacing.
- Proof/creep/performance test anchors, and lock them off at 40 kN (which is 30% to 40% of the design load).
- Grout up anchor heads.

4.2 Verification Tests

Prior to installation of production anchors, and to verify the unit bond stress, two sacrificial test anchors were installed in the existing MSE wall backfill and tested.

These tests did not reach failure but did indicate that the ultimate bond stress was in excess of 150 kPa with maximum 6 to 7 mm displacement (see Figures 6 and 7) being recorded.

A vertical anchor installed in native soil to provide an indication of bond stress for lower row of anchors showed a design bond stress of greater than 75 kPa.

Figure 6. Pull-out test on a sacrificial anchor installed in the MSE backfill zone.

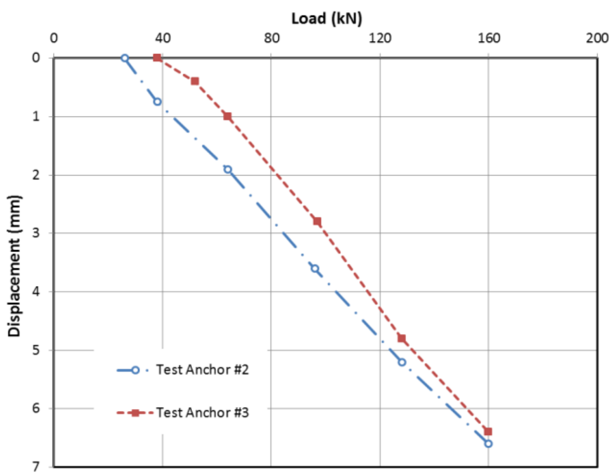

Figure 7. Load-deflection plot for bond stress verification anchors (133 mm diameter hole, 2.5 m bond length).

4.3 Helical Pile Installation and Testing

The piles were installed using an excavator-mounted hydraulic drive motor (Diggerdrive) with a torque rating of 16 kN.m. The piles consist of a lead section (2.1 m long with flights diameter of 254 mm, 305 mm and 355 mm) and SS175 extension rods with varying lengths of 0.9 m, 1.5 m and 2.1 m. Voids around the extension rod section were grouted using Basalite Microsil Anchor Grout through a 1.5 m long PVC pipe of 150 mm in diameter.

The minimum required pile length for the design load of 100 kN was found to be 8.8 m.

4.4 Production Anchor Installation and Testing

The anchor holes were 150 mm nominal diameter and drilled at a 15° inclination from horizontal (hole was cased to prevent the collapse), and the DCP anchors with centralizers every 3 m were placed in the hole and the bond length fully pressure grouted. Non-shrink Basalite Microsil anchor grout was used.

Figure 9 shows the ratio of actual/theoretical grout volume for bond length of the anchors. Typical section between Stations 1135 and 1185 consist of two anchors where the top row of anchors were installed first followed by the bottom row. As it can be seen in Figure 9, the bottom anchors took relatively less grout compared to the top anchors, which may be attributed to voids being partially grouted during installation of the top row of anchors.

In general, for sections consisting of two anchors the top row of anchors were installed first and the grout take within the bond length was around 3.5 to 4 times the net volume of the drill hole. The corresponding volume for the lower row anchors and for single anchor rows was generally between 1.5 to 2 times the theoretical volume.

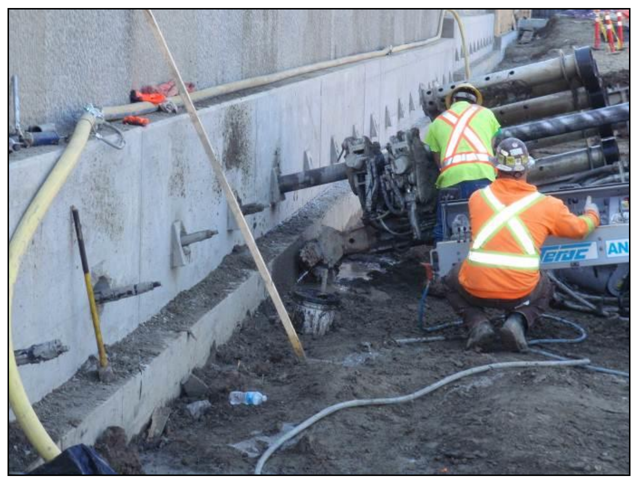

Figure 8. Drilling and soil anchor installation.

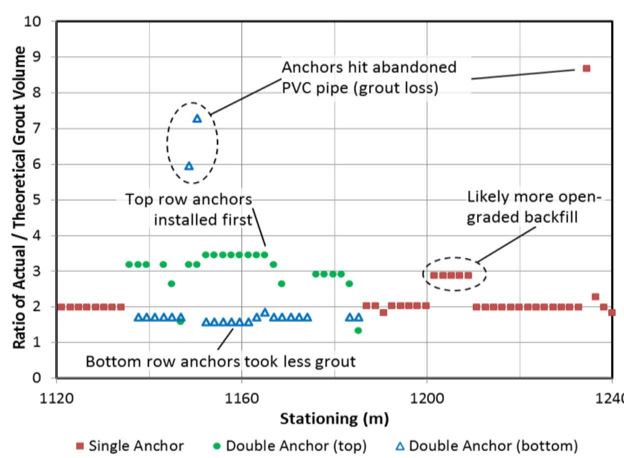

Figure 9. Ratio of actual/theoretical grout volume for the bond length of the anchors.

The only issue encountered was excessive grout loss into an abandoned 300 mm diameter PVC pipe which had not been identified prior to construction. Grout takes for three anchors intersecting this pipe increased to between 45 and 57 bags (compared to the expected 13 to 16 bags). These anchors were gravity grouted in two stages and the installation angle for the adjacent anchors was adjusted to avoid the abandoned pipe.

4.4.1 Proof Testing

Proof testing was completed on all anchors that did not undergo creep or performance testing. Proof testing consisted of incrementally increasing the load until the anchor was loaded to 133% of design load. This load was then held for 10 minutes and the movement of the anchor head was measured by two gauges. If movement was less than 1 mm during the 10 minute holding period, the anchor was considered to have passed the test. The total displacement of anchors under the maximum test load was generally between 5 to 8 mm (see Figure 10). The expected displacement based on the anchor configuration was between 5 to 13 mm.

Only one anchor failed the proof test. This anchor was post-grouted and successfully proof tested three days later.

4.4.2 Creep Testing

Performance testing was conducted by cyclically and incrementally loading and unloading the anchors to determine whether the anchor has sufficient resistance and if the apparent free length has been satisfactorily established. The magnitude of residual movement and rate of creep are shown in Figure 11.

5 WALL PERFORMANCE

Observations during construction indicate the retrofit design was practical and verifiable (by pull-out tests), and in general construction proceeded smoothly and work was completed within about 40 days.

To date the retrofit appears to be working well and no obvious issues have arisen.

On average the retrofit cost amounted to about $5,130/m^2 of exposed wall face.

A total of 20 points on the MSE wall were regularly surveyed during and shortly after construction to provide data regarding the magnitude of construction-induced wall movement. The maximum movement occurred within the early days of construction and was 13 mm. Most surveyed points were observed to move at most 10 mm in any direction.

6 LESSONS LEARNED AND APPLICATIONS TO OTHER PROJECTS

Following completion of the MSE wall retrofit described above, three similar MSE walls supporting shallow bridge abutment footings supporting wildlife overpasses were subsequently retrofitted. Some of the lessons learned were applied and included the following:

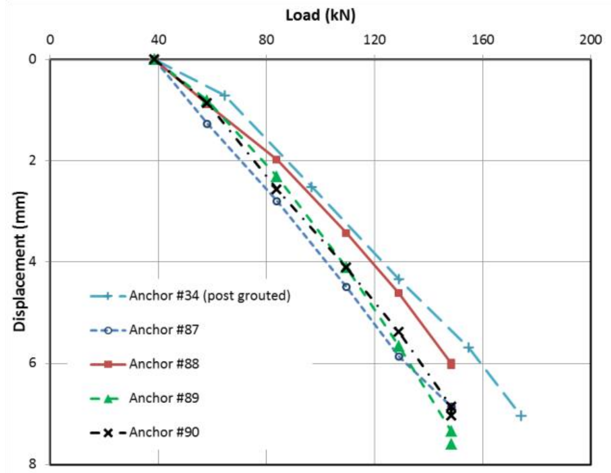

Figure 10. Load-displacement plot for proof tests.

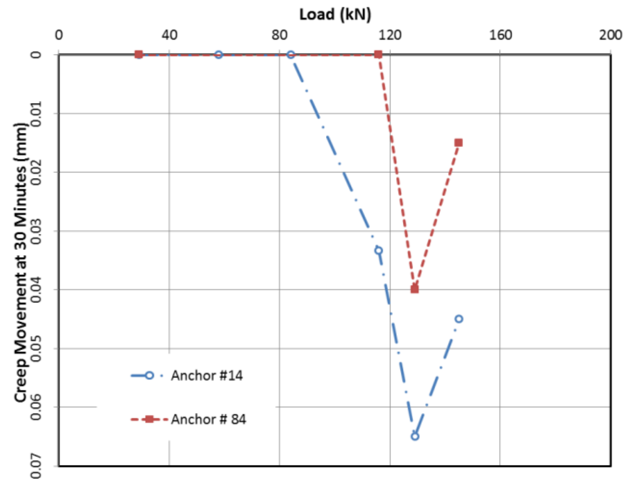

Figure 11. Creep movement versus load measured for two anchors.

- Anchors were installed at joints between panels to avoid drilling through the reinforcing steel. This again required a structural element (i.e. the concrete wall) to distribute the anchor loads evenly between adjacent panels.
- Anchors should be installed at 2° off horizontal to limit the vertical load component which reduced the potential for damage to the facing panels.
- Revise construction sequence to install cast-in-place concrete wall following anchor installation for improved constructability and aesthetics. This also minimizes delays in anchor installation and testing due to concrete strength concerns.
- Complete full survey of existing structures and backfill to ensure existing conditions are still in alignment with as-built information from original construction

In one of the walls retrofitted there was insufficient backfill behind the wall to allow the use of soil anchors. In this case deadman anchors were installed (see Figure 12) by constructing walls behind the abutment and then drilling anchors through the walls to intersect the deadman which were then subsequently buried.

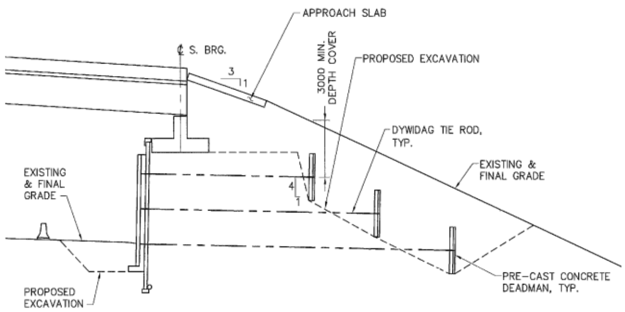

Figure 12. Typical detail of deadman installation for retrofit of a MSE abutment wall at wildlife crossing.

ACKNOWLEDGEMENTS

The writers acknowledge the permission of Ledcor CMI to publish this paper. We further acknowledge the contribution of the following to the case histories discussed: James Lowe (Ledcor CMI), Craig Smith (Allnorth Consultants), Dete Mordhurst and Jonathan Lu (All-Span Engineering & Construction), and Yadav Pathak, Mike Laws and Cevat Catana (Tetra Tech EBA). We appreciate input from Bob Patrick (Tetra Tech EBA) in the project and for reviewing the paper.

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